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Special Report No. 1

Frost Action In Roads And Airfields

A Review of the Literature

1765-1951

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by

A. W. JOHNSON

Engineer of Soils and Foundations

Highway Research Board

1952

Washington, D.C.

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FOREWORD

K. B. Woods, Chairman
Committee on Frost Heave and Frost Action in Soils

Frost action is not a simple phenomenon. The elements of climate, soil, ground water, pavement, and traffic are factors which influence the nature of frost action, governing the intensity of its occurrence or the degree in which it is detrimental. The thawing of the frozen ground and the marked reduction in bearing capacity following the thawing are each as much a part of the overall phenomenon as is ground freezing. In fact, any action associated with soil freezing or thawing which unduly stresses the road surface, base, or subgrade soil or changes their water content, porosity, structure, or ability to support loads, is essentially a part of frost action. Thus frost action includes the primary process of freezing as well as the secondary process of thawing and the effects of freezing and thawing of pavements and their foundations.

The Highway Research Board Project Committee on Frost Heave and Frost Action in Soils was reorganized in 1947. Its revised project assignment was to study the overall problem of frost action as related to highways. The complex nature of frost action brought out the need for a bibliography and review of published literature on the subject of frost heave and frost action in soil. As a result of recommendations made by the committee and the Department of Soils Investigation, the Highway Research Board assigned its engineer of soils and foundations, A. W. Johnson, to work with the committee and devote such spare time as was available to the project. An annotated bibliography was prepared and published as Highway Research Board Bibliography No. 3, "Bibliography on Frost Action in Soils," annotated in 1948. The demand for the bibliography required three printings.

Following the release of the annotated bibliography, Johnson continued his work with the development of a review of the literature on the subject. This book is an out-growth of several months of part-time effort on the part of the reviewer. The review complements the 36-paper "Symposium on Frost Heave and Frost Action in Soil" presented at the annual meeting of the Highway Research Board in January 1951.

The literature which pertains directly or indirectly to frost action in soils is voluminous. Articles on the subject are to be found in many publications here and in other countries. A review of that literature would involve a study of the many separate and related factors which influence frost action. A comprehensive review should include study of all literature pertaining to those various factors presented in English, as well as a number of foreign language articles. The reviewer has, insofar as is practical, included data from abstracts or available full translations of foreign language articles. However, lack of facilities for translation has made it necessary to omit many foreign language articles believed to have worth.

Time has not permitted a complete review. However, the review is intended to be comprehensive in scope of subject matter. Soil freezing is treated at considerable length. Some facets of the frost-action phenomenon which have no direct bearing on the design, construction, or maintenance of highways or airfields are covered. The intention of the reviewer here is to give breadth to the concept of

frost action, to aid in understanding the effects of soil freezing and thawing on foundations for pavement and structures. The review is not the normal digest type of review in which the reviewer would completely restate in his own words, the contents of the literature. Rather, it is an abstract type of review in which the reviewer has attempted to separate pertinent statements and tabulate and classify data and illustrations to make them available to the reader. In many instances the reviewer has chosen to quote directly from the authors writings to facilitate presentation of the ideas of the author. The abstract method was used because it greatly facilitated reviewing the literature and organizing the findings. The review summarizes the writings in chronological order and uses a form of referencing which permits the reader to know the year of publication of the writing. That method permits the reader to follow the development of knowledge and in some instances to observe the changing views of writers with the passing of time.

The problems associated with frost action in highways are as broad in scope as are the technological aspects of the phenomenon. They include proper road maintenance to hold frost damage to a minimum, regulation of loads during and following the frost melting period to prevent unnecessary damage to pavements, and proper design to prevent detrimental frost effects on new construction. The reviewer has purposely ommitted from this review much available information on the subject of regulation of loads because that phase of the frost problem is under study by another committee of the Highway Research Board, viz., Maintenance Project Committee No. 7: "Load Carrying Capacity of Roads as Affected by Frost Action."

PREFACE

The first step in attempting to solve a problem is to acquire as much of the available knowledge on the subject as is practical. That can be done by reviewing published reports and by talking with investigators actually working on the problem and with others who have had close working contact with similar problems for a long time. This review of frost action in soil was prepared to help investigators in that first step and thus prevent unnecessary duplication of work which has already been reported. Newly developed engineering information is not readily accepted and put to use unless all engineers keep pace with developments. The reviewer believes this presentation will also be useful to practicing engineers by not only acquainting them with design methods but also by broadening their concepts of the basic phenomenon of soil freezing. The degree of detail included is indicative of the dual purpose of the text.

The preparation of the review was a pleasant and rewarding task. However, it could not have been accomplished without the assistance of other staff members. Mrs. Dorothy Bright, Librarian, collected the various reports and edited the bibliography. Miss Donna McElhiney, secretary to the reviewer, typed the abstracts, the initial draft and the final manuscript for offset reproduction. Miss Evelyn Stoll prepared tracings of many of the charts.

A.W.J.

Washington, D. C.
February 1, 1952

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HISTORICAL INTRODUCTION

Ground freezing and heaving has, from the earliest times, been a subject of observation and discussion among country people. Early Swedish literature indicated that the people in Sweden understood that frost caused uplift of boulders in fields. According to Beskow this phenomenon was described in 1644 by Hiarne who said "...in fields where boulders have been picked out and the land cleared, more boulders will be generated...". The early literature also indicated familiarity with breaking of plant roots by heaving of the ground surface and contained descriptions of frost boils and ground hoar frost (needle ice). Heaving associated with freezing of the ground was mentioned by Runeberg in 1765, who found clear ice in the soil, and in 1854 by Volger.

During development of stage coach traffic in the 1700's both here and in other countries, it was observed that frost heaving caused damage to culverts. People at that time recognized that they did not fully understand the phenomenon. No doubt the ordinary difficulties associated with thawing of ice and snow, spring floods and softening of roads due to snow melt water were so great in the frost belt that they obscured the real effects of softening caused by soil freezing.

The coming of the automobile brought with it the necessity for preventing the softening and in some cases the complete break down of road surfaces during the spring thawing period. Then the technical problems associated with frost action attained real significance from a practical as well as economical point of view. The problem became an acute one prior to 1920 and in the early 1920's causing investigations to be undertaken in different countries.

The present concept that frost heaving is due to the growth of ice crystals into ice lenses perhaps owes its origin to Taber's study of the work of Lavelle, and Becker and Day on crystal growth. Lavelle observed crystal growth in saturated solutions in 1853. In 1905 Becker and Day found that growing crystals in a saturated solution of alum were capable of raising a kilogram weight "...several tenths of a millimeter". Taber, in a discussion of the work of Lavelle and Becker and Day in 1916 stated that "...the same argument applies to the formation of ice columns...". Two years later Taber showed experimentally that ice forming in clay soils lifted surface weights. He then concluded that heaving is caused by the growth of ice crystals into lenses or layers of ice. That, however, was only the beginning of Taber's extensive study of frost action in soils.

Taber's early statement brought forth considerable discussion in the published literature. New designs appeared, some bordering on the whimsical and others unsupported by either facts or experience. Many engineers did not comprehend Taber's concept and continued to hold to the older concept which had been expressed in 1765 by Runeberg that frost heaving was due to the volume change of the contained water changing to ice (approximately 9 percent volume increase). Others cited instances where the heaving was so great that volume change of water changing into ice could not possibly account for the heaving when the depth of frost penetration was considered. The comprehensive report of Taber's great work in 1929 not only proved the validity of his earlier conclusions but gave some indication of the range of conditions under which ice crystals would grow into ice lenses and cause heaving. He dispelled the old concept that heaving was due to the natural water-to-ice change in volume by producing growth of crystals and severe heaving with the use of benzine and nitrobenzine which solidify with a decrease in volume.

As early as 1897 Holmquist observed that in clay soils the soil between the lowest ice layers was often unfrozen indicating a lower freezing point after some water had been drawn out in the process of forming the overlying layer of ice. That observation was followed by extensive studies of the freezing point in soils during the period 1914 to 1923 by Bouyoucos who found marked differences in the freezing points in different soils and for a given soil at different moisture contents. It was the findings of Holmquist and Bouyoucos and other later investigators which made possible a plausible explanation of Tabers concept of water continuing to flow to and nourishing growing ice crystals until they develop into ice lenses.

Long before Johansson had confirmed his belief that water moved to the freezing zone, people were aware that heaving was associated with an increase in soil water content. Runeberg in 1765 had determined the moisture content of a chunk of frozen clay and found it contained almost four times as much moisture as the unfrozen dry clay layer and said "Is it not a wonder then that a clay layer can displace a layer that rests on it, when the water freezes?"

Since that time much effort was given toward obtaining a better understanding of water movements in soils. Some of the early concepts developed on analogy between capillary tubes and soil pores in which water rose or was held in the pores through tension in surface films. Several writers classified water into hygroscopic, capillary, and gravitational or free water. Briggs (1897), Bouyoucos (1921), Lebideff (1927) and Zunker (1933) are among those who held that forces responsible for water movements were analagous to those which move water in a capillary tube. Buckingham (1907) had a different view. He made his comparison with the flow of electricity through a conductor and assumed that water flowed in an analogous manner, the driving force for the water being the result of the difference in moisture content in the soil. He called that driving force the "capillary potential".

Those who studied the freezing and heaving of soils saw in the existing concepts no completely satisfactory explanation of the forces responsible for drawing the water into the freezing zone. Taber agreed that during the growth of an ice layer in clay, water is supplied to the crystals through small capillary passages but held that "...the upward flow of water should not be attributed to capillarity as there is no free surface or meniscus. The uplift is due to the cohesive forces in the water". Beskow's concept of the fundamental processes of freezing and heaving did not differ materially from those of Taber. He saw the practicability of recognizing water movement by what he called capillary suction, the rate of flow being dependent on the pressure difference and the size of the pore channels. He developed a capillarimeter to give a relative measure of the height of capillary rise in soils. His practical treatment of water movement as it is related to frost heaving is perhaps the most comprehensive available.

Some of the earliest field studies in the United States were measurements of soil moisture content, and soil heaving of subgrades by the Illinois Department of Highways in the Bates Experimental Road in the early 1920's. Also, they conducted load-bearing tests with repeated applications of load. Their tests indicated a marked reduction in load carrying capacity of subgrades during the frost melting period. The acute need for means for preventing severe differential heaving and the consequent frost boils brought about cooperative studies by the Bureau of Public Roads and the State highway departments of Michigan, Minnesota and Wisconsin. Those studies brought out the effectiveness of different depths and widths of granular bases on different soil textures, soil profile arrangements and soil water conditions.

Michigan's studies were started in the 1920's and included both laboratory and field investigations. They brought out the relation between soil type, soil profile characteristics, soil water conditions and heaving. The Michigan work reported in 1931 resulted in the statement of a new theory on frost heaving in which a fluctuating frost line was held to be productive of major ice segregation associated with severe differential uplift. The discussions which followed included the presentation by Casagrande of one of the earliest statements on grain size limits for heaving and non-heaving soils.

The study of frost action in Sweden closely paralleled the work in this country but was on a larger scale. In 1925 the Swedish Institute of Roads sponsored a conference on frost action in soils in which representatives from highway, railroad and other technical interests were present. The published proceedings of the conference summarized the practical knowledge of the frost heaving phenomenon and the preventive measures used up to that time. With that background of information and cooperation, Beskow and his co-workers attacked the problem. The results of his work were published 'piece-meal' as the work progressed and covered the various phases ranging from snow plowing, drainage, significance of geologic factors, supporting value during the frost melting period, the capillary properties of soil and how to measure it, measures to insulate and to isolate frost, the use of chloride salts and sulfite liquors, corrugation and counter measures, to the technical aspects of the freezing and heaving phenomena. The results of those fundamental researches were published in a single bulletin (1935) which remains one of the most comprehensive treatises on the mechanics of frost action in soils. It was translated into English by Osterberg in 1947. Many of the quotations herein attributed to Beskow (1935) are taken from Osterberg's excellent translation.

It is not surprising to find scientists interested in the more vivid physical aspects of the freezing and thawing phenomena as evidenced in the heaving and in the erupting frost boil. But it is notable that scientists in the early 1800's were also interested in soil temperatures associated with frost action. Forbes, in 1837, not only observed soil temperature to depths of over 25 ft. but did so with specially constructed thermometers 26 ft. in length capable of measuring temperatures to one-hundredth degree Fahrenheit. Some of the earliest measurements in North America were made in Canada by Callendar in 1895. Callendar observed seasonal changes in soil temperature; the effects of rainfall in increasing soil moisture and

in influencing the temperature and thermal conductivity of the soil; and the effect of snow cover as an insulator in protecting soil from freezing. Callendar applied those data in computing thermal diffusivity of the soil under the different conditions of moisture content and temperature of the different seasons. Pattens extensive laboratory studies (reported in 1909) on the flow of heat in soils first gave specific data on the marked effect of soil moisture content on the thermal properties of unfrozen soil. Electrical engineers, in their studies of heating of underground cables have added to the available knowledge on the thermal properties of soils.

Engineers assigned to construction and maintenance of roads have long known in qualitative terms, the nature of climate and soil state necessary for the occurrence of damaging frost action. However, it was not until 1930 that Sourwine, a highway engineer, found he could interrelate frost occurrence and highway frost damage so as to make use of existing climatological records as a means of determination of probable ground freezing occurrence. The interrelationship he worked out on the basis of freezing point in the soil, the air temperature and the duration of the cold period is one of the first developed. At about the same time A. Casagrande correlated depth of freezing and duration of cold in terms of cumulative degree days of below freezing temperature with observed depth of frost penetration. Water supply engineers quickly saw possibilities in Casagrande's interrelationship as means for predicting when frost will penetrate to the depth of water service line.

The early studies which were in a large degree fundamental in nature were followed by field studies of various designs to prevent or reduce the occurrence of major frost heaving. A symposium held at Purdue University in 1938 indicated that the problem of severe differential heaving at that time was well understood. Engineers could recognize, from soil survey data, areas where severe heaving might occur and would also prepare designs with reasonable assurance that they would be successful. Engineers were also aware of the marked reduction in bearing capacity which followed frost melting in areas of severe heaving and which expressed themselves in the form of frost boils and a subsequent breakup of road surfaces of the lighter types. However, as the wheel loads of commercial traffic became heavier and the number of commercial vehicles increased rapidly in the early 1940's it became evident that engineers had insufficient knowledge of frost phenomena to predict the effect which the heavier and more concentrated traffic would have on the great mileage of roads subject to a somewhat lighter form of frost action. Those more recent developments resulted in the extensive investigation of frost action by the Corps of Engineers. That work was begun in 1945 and constitutes the most comprehensive field and laboratory studies of frost action conducted in this country.

The perennially frozen ground (permafrost) of the North, estimated to cover one fifth of the earth's surface, has long been studied by Russian scientists. Only recently Americans have begun extensive studies of permafrost. The presence of permafrost in the United States in areas near the limits of glaciation is indicated by recent discoveries of fossil ice wedges and soil involutions and stone rings which are evidence of extreme frost activity during a past glacial era.

DEFINITION OF FROST ACTION

Many different terms have been used to describe the processes of soil freezing and heaving and thawing and their effects on pavements and structures. Some of them are: frost action, freeze damage, frost heave, frost boil, mud boil, and spring breakup. The term, frost, is defined as the act or process of freezing, that is, congealing of liquids with special reference to water. Since freezing is the fundamental phase of the overall phenomenon of freezing and thawing, the term frost action is used throughout this review to denote that phenomenon. The term freeze damage is a commonly used term but it has a more specific meaning in that it refers to the damage caused by freezing.

The phenomenon of frost action and its detrimental effects on highways and airfields, if considered in its entirety is not a simple one. The elements of climate, the soil, the soil water, the pavement and the traffic, individually and collectively, are factors determining the nature and degree of damage which may result from frost action. Any action resulting from freezing and thawing which alters the water content, porosity or structure of the soils or affects their capacity to support loads is essentially a part of frost action. Thus the term frost action is used in this review when referring to the primary process of freezing as well as the subsequent effects of thawing of pavement foundations.

Heave Damage to Pavements and Structures

The damage to roads caused by frost action can take place in two ways. The actual heave, if non-uniform may damage pavements permanently by fracture, or may leave a rough riding surface. The secondary effect of softening of the road bed which accompanies thawing of ice layers may bring about a reduction in load carrying capacity which will result in damage by loads which can be carried during the summer, fall and winter seasons without damage to the road.

Heaving itself would cause no damage if uniform. However, due to variations in soil composition, moisture content and depth to ground water, and other factors, severe heaving is seldom uniform in nature. It is the abrupt differential heave which causes the damage. Such heaving often fractures pavements. Cracks are usually transverse where they reflect changing ground conditions. Old rigid type pavements built without longitudinal center joints have often cracked longitudinally due to a greater uplift at the edges. Some more recent designs with longitudinal center joints have developed severe cracking at or about the quarter point as a result of heaving. Skelton /1940-12 found that "...the critical period for a concrete pavement due to frost action occurs when the subgrade and pavement are frozen solidly together..." as any tendency toward differential heaving causes the pavement to break. Another form of heave damage to rigid type pavements has occurred in Minnesota. Lang, /1937-8 described high joints in concrete pavements which in extreme cases measured as much as two inches higher at the joint than at the mid-point of the slab.

Flexible type pavements likewise fracture both longitudinally and transversely on differential uplift. Since severe differential heaving is usually of a local nature, heaves can be equally as severe in flexible type pavements as in rigid type pavements. The principal damage to flexible type pavements, however, is caused by traffic during the frost melting period when the load carrying capacity of the subgrade is low.

A common type of damage to roads, regardless of type is the lifting of the crown of the road. Such heaving has been attributed to incomplete snow removal, the snow along the edges of the pavement acting as an insulator and retarding the penetration of frost while cleared areas permitted deeper penetration and greater uplift; and also to difference in subgrade moisture under edges and mid portions of pavements causing differential uplift.

The early literature includes many photographs showing the nature of heaving of gravel type surfaces as well as flexible type and rigid type pavements. A few of those are reproduced here to illustrate the nature of severe heaving and cracking. It is worthy of note that recent literature presents no photographs of severe heaving and associated pavement cracking. This indicates that present designs are generally adequate to prevent occurrence of severe heaving.

Figure 1. Abrupt and hazardous frost heave of a concrete pavement. After Aaron, 1934-3. (Photo courtesy Bureau of Public Roads)

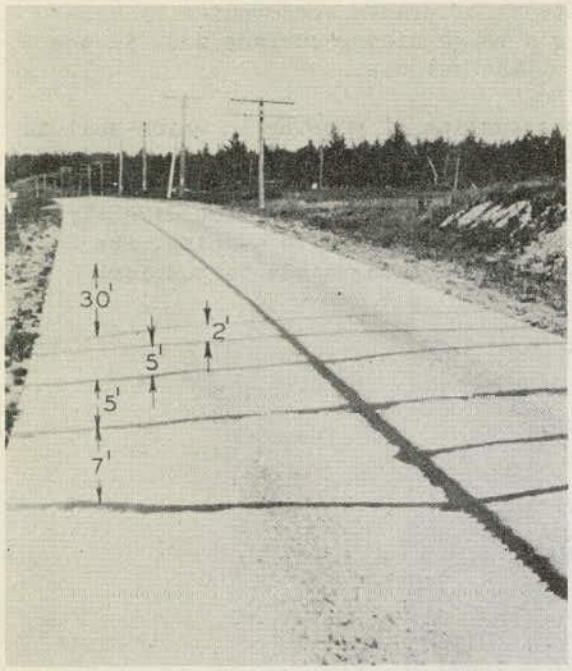


Figure 2. Cracking in concrete pavement due to frost heave. After Aaron, 1934-3. (Photo courtesy of Bureau of Public Roads)

Figure 3
Longitudinal cracking resulting from an abrupt change from a higher soil moisture under the edges to a lower one under the mid-portion of the pavement causing differential uplift.
(After Bleck 1948-43)

There are some published accounts of damage to structures by frost heaving. Few of those relate to roads. Damage to structures in this country has been limited largely to houses, retaining walls, culverts, manholes, inlets and similar small structures. Economically, the damage is of small consequence. Like many road ailments it is more annoying than costly. Severe differential heaving often occurs over pipe culverts, sometimes leaving a residual uplift which does not subside completely on thawing. The net result may be a bump in the road but it may also cause enough displacement of the pipe to cause sedimentation and reduction in capacity. It is not uncommon to find frost heaves where the road surface on each side of a culvert is lifted leaving a depression over the culvert. Possibility of damage to retaining walls due to lateral displacement is recognized by some practicing engineers, but the literature on the subject is meager.

Load Damage to Pavements Following Thawing

The effect of the softening of the roadbed due to thawing of the frozen soil and the consequent reduction in load carrying capacity of the soil causes more wide spread and greater damage than that caused by the uplift due to freezing. As thawing begins, the melting ice releases free water in the soil. The water is trapped in the thawed soil between the pavement above and the still frozen soil below unless a granular base course provides adequate space for its release. The weakened soil (or base course, if it is susceptible to frost damage), permits greater pavement deflections under traffic. These increased deflections disturb the soil and further reduce its load carrying capacity resulting in fracturing and if severe, in eventual failure of the pavement surface.

If the ice content is high and the soil is sufficiently 'worked' under traffic a free flowing mud is formed which is forced out at pavement edges or at breaks in the pavement. The appearance is much like a mud boil hence this secondary effect following thawing has in severe cases been termed 'frost boil'.

Early writings contain many descriptions and photographs of frost boils showing distress of light type surfaces approaching a condition of near or actual impassibility for vehicles. Present designs permit frost action to produce such vivid effects less often, although the recurrence of climatic conditions favorable for frost action and concentrated heavy traffic caused

their reappearance during the spring of 1948. The normal distress in flexible type pavements in the spring season is usually in the form of a close network of cracks accompanied by distortion of the riding surface. The distortion may produce a rough riding surface and, in the more severe cases, rutting followed by complete breaking of the surface.

The usual distress in rigid type pavements is an accentuation of pumping in which soil is ejected at pavement edges and joints, followed by cracking as the process continues. Pumping is prevalent when rigid type pavements which are subjected to heavy truck traffic lie directly on fine grained soil; and, when conditions of climate and soil moisture produce ice directly under and in contact with the slab. Although frost is not necessary to cause pumping, its presence often produces a soil condition which makes the road more susceptible to pumping.

Figure 4. An example of pavement pumping caused by excess water during the frost melting period in the spring. (After Bleck 1949-11)

Figure 5. Severe frost action resulting in deep rutting and breaking up of a bituminous surface in Indiana. (Photo courtesy of K. B. Woods, 1950)

Figure 6. Severe rutting and surface breakup of a surface treated mechanically stabilized secondary road in the Triassic "Red Bed" soil area in Virginia. (After Shelburne and Maner 1949-8)

Erosion and Sloughing of Slopes

The loosening effect of frost action on surface soils is well recognized by farmers and agronomists who anticipate its benefits in improving soil tilth and crop productivity and dread its potential damaging effects on some crops like winter wheat. Its action sometimes creates problems in maintaining highway slopes. Hursh /1948-7 reported that freezing and thawing is most severe on south facing slopes and has been known to cause "...erosion of one foot of soil in a single winter on a 1-1 slope clay bank". Winter heaving may leave the soil not only in a loose but also in a wet, sometimes almost fluid state in which it is easily eroded by heavy spring rains. Comparatively speaking, this effect is of minor importance yet it should be considered in some areas if slopes are to be protected. Means for protecting slopes are reviewed later.

Extent of Area in Which Frost Action Affects Highways

Until the middle 1930's frost action was recognized as a problem of economic importance only in areas where heaves and boils were evident. Those areas were limited largely to the northern tier of states. More recently, since frost action has been identified as the cause of reduced subgrade support in the spring, its occurrence has been recognized in all but the southernmost states. Shelburne, in his review of subgrade soil practices /1947-15 received reports from 28 states and the District of Columbia indicating that frost action was one of the subgrade problems encountered. During 1950 Shelburne /1951-46 again contacted State highway departments and found that only five southern states and California indicated that damage by freezing of road bases, sub-bases and subgrades was not a problem. The five southern states were: Louisiana, Mississippi, New Mexico, North Carolina and Oklahoma. Twenty-two states indicated it was a major problem.

Damage due to frost action in the northern states occurs every year, but differs in degree from year to year. Near the southern limits of ground freezing, some winters are marked by a total absence of frost action, others by severe spring breakup when ground moisture conditions are favorable for frost action. The time intervals between periods of damage increases progressively from North to South.

Cost of Repairing Roads Damaged by Frost Action

Numerous published accounts give estimates of road damage resulting from frost action. Reports usually have appeared in periodicals during spring seasons when damage was greater than normal. No summary has been found which shows both the area involved and the amount of damage for any period, and no attempt has been made in this review to summarize data, as any summary made from published accounts would be far from complete. The two reports summarized below serve to illustrate the estimated cost of repairing damages following two winters which were productive of severe frost action.

One of these reports resulted from an Engineering News Record /1928-4 poll of states concerning damage incurred during the winter of 1927-28. Eleven states replied. Seven of those gave figures on mileage and (or) cost of repairs. Those data are shown in Table 1.

TABLE 1

State	Mileage Affected	Cost of Repair
Maine		$500,000
Massachusetts	500	50,000
Connecticut	600	75,000
Rhode Island	10	
Wisconsin		50,000
Michigan	2890	373,000
Oregon	90	42,150
New York	Greater breakup of old thin macadams than ever before.	
New Jersey	Damage practically absent this year.	
Ohio	Very great damage 1926-27. Damage 1927-28 not more than half of 1926-27.	
Indiana	Bad, particularly on gravel roads.	

The winter of 1947-48 caused extensive and wide spread damage to roads. It was unique in that damage was severe as far south as Texas, Arkansas, Tennessee and Virginia. An example of estimated damage is that Kentucky /1948-2 reported that 1948 repairs to 924 miles of Kentucky's winter-damaged highways averaged $5,354 per mi. The total cost of resurfacing the roads broken up during the quick freeze and thaw period was estimated to cost $4,947,962. The account quotes the State Highway Engineer as follows:

> "It is estimated that an additional 1,000 miles will require the same costly treatment. The overall estimates show definite damage to nearly 5,000 miles of State highways. Almost 2,000 miles will require repair and resurfacing while repair will suffice for the remaining 3,000 miles."

Otis /1951-33 analyzed road repair costs in New Hampshire and arrived at figures which were chargeable to repairing road surfaces damaged by frost action. The following list of costs indicates the effect of frost damage on road upkeep for the period 1948-50.

Year	Primary System	Secondary System	State Total
1948	$265,000	$302,000	$567,000
1949	189,000	204,000	393,000
1950	168,000	188,000	356,000

He brought out that the winter of 1948 was a severe one and that the winters of 1949 and 1950 were relatively mild. Basing his cost on a mileage of 3,728 for the period January 1 to June 30, 1950, the average cost per mile for all types was about $95.

THE PHYSICAL PROCESSES OF SOIL FREEZING AND THAWING

The Structure of Frozen Soil

The building of layers or lenses of ice was mentioned in records of early observations. However, Kokkonen /1926-3 was one of the first to publish a detailed, well illustrated article describing the variation in structure due to freezing different types of soils. He classified the structure into massive, porous, and stratified frozen soils. He stated that both massive and stratified forms can occur in soils of low porosity independent of grain size when the water content is adequate and that there is a certain water content below which soil becomes massive and above which it becomes stratified. Beskow /1935-1 gave detail descriptions of both the massive and stratified types of frozen soil and also of hoar frost (described in Appendix 1) which forms on the ground surface.

Homogeneous Frozen Soil - Massive or homogeneous frozen soil is one in which water is frozen within the natural voids and there is no visible accumulation of ice lenses. This type of structure is associated with coarse grained soils, usually medium sands and coarser. Fine grained soils freeze without forming ice lenses when the water content is well below saturation, or when very quick freezing takes place.

Stratified Frozen Soil - Stratified frozen soil contains visible ice lenses which occupy spaces greater in size than the original voids causing heaving of the surface. It occurs in a wide range of structure formations. The ice may range from clear blue to a porous white type. Cavities may be only partly filled in which case the ice is usually white and porous in the form of hoar frost. Ice may form around objects such as stones or it may appear as clean or partly clean lenses separating homogeneous soil. The remaining moisture in the soil may be frozen in the pores (massive structure), or in fine clays cooled only slightly below freezing, it may be unfrozen leaving the clay between ice lenses soft and plastic /1935-1. The occurrence of soft, plastic, unfrozen soil between ice layers near the frost line was described by Taber /1930-9 and has been observed by many engineers in studies of frost.

Beskow /1947-12 summarizes the nature of stratified frozen soil as follows: "The fine grained soils in the frozen conditions are composed of layers of clean ice which are essentially parallel to each other and parallel to the surface. The character of these layers depends mainly on the fineness of the soil. In clays the layers are thick and widely spaced, built in uniform and distinct systems, being more distinct and thicker the finer the clay is. The

coarser the soil is, the less ice there will be both in thickness and in spacing of the layers. For example, in a silt we find very thin (a few tenths of a mm.), short (a few cms.), separate, parallel and well oriented layers a few millimeters apart, which gives the mass a streaked appearance. The coarser the silt gets, the finer and more distinct becomes the structure until finally at a certain range of grain size (0.06 to 0.1 mm.) it disappears entirely...".

"...aside from the regular structure...there exists...ice strata of another character in these soils. These strata consist of unusually plane beds parallel to the ground surface, the ice layers being especially tough and thick.... The explanation is that at certain levels the rate of freezing is less than normal and the zone of freezing remains constant. The layer has a chance to grow thicker than usual...".

The Michigan field investigations /1930-10 showed "...by far the most serious disturbances occurred in silts with ice banding very similar to that described in Taber's experiments /1930-9 for clay". Burton and Benkelman /1930-10 showed that in nature thick ice lenses occurred in soils of very fine sand and silt texture. Figure 7 shows an example of ice lenses formed in a lean clay soil.

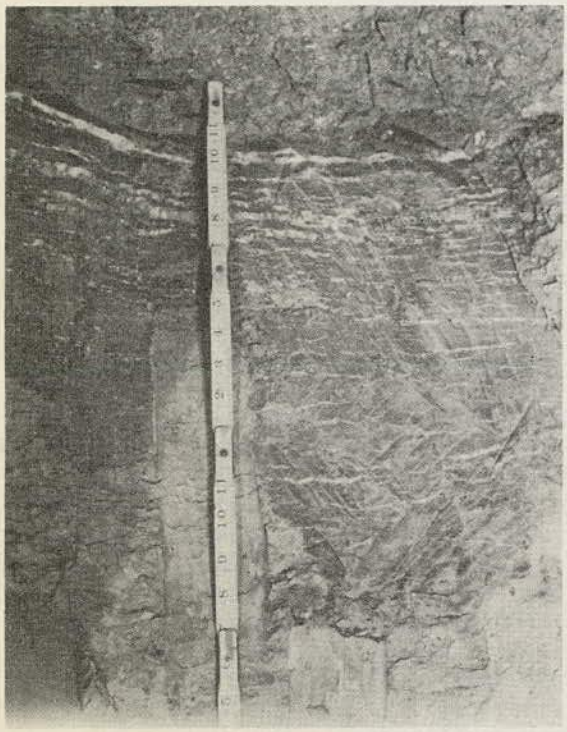


Figure 7. Ice Lens Formation in Lean Clay Subgrade. (After Shannon 1944-1)

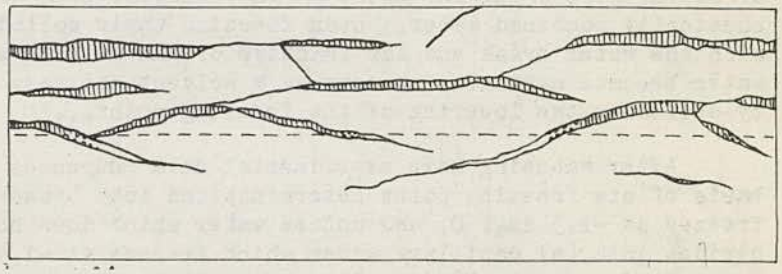


Figure 8. Schematic diagram illustrating ice crystallization in open fissures and progressive development of fissures below the frost line (dashed line is frost line). (After Beskow 1947-12)

Uneven soil textures may cause a variation in stratification. A small variation in the average grain size or in the grading can cause appreciable difference in pore volume and therefore in the water content at the beginning of freezing. Stratification of the soil has a great influence on the development of ice lenses. In sands, the occurrence of a very thin stratum of silt or clay may cause the formation of a thick ice layer. Any discontinuity in the form of a stone, crack or joint in structured soils may cause fissures to become filled with ice and widen the fissure. Beskow /1947-12 found that at the frost line the freezing of ice in fissures causes them to widen and work their way progressively downward. At the frost line the freezing stops but the open fissures extend slightly deeper as indicated in Figure 8.

Taber /1929-2 and /1930-9 found that "...in every experiment in which no additional water entered the system, the withdrawal of water from the lower part of the container to build up ice layers above caused shrinkage in volume, and, usually, the development of tensional cracks..."

The Freezing Point of Soils

Early observers /1897-1 found soft unfrozen clay between the deepest ice layers. Later observers found that a lower temperature was required to freeze the remaining water in clayey soils after the 'more-free' water had been moved upward and frozen during freezing of the overlying soil. Bouyoucos and McCool /1915-2 pointed out that the lowering of the freezing temperature in colloidal substances (gels) can amount to a few degree C. and studied the concentration of the soil water by means of its lowered freezing point. During the period 1916 to 1921 Bouyoucos made extensive studies /1916-3, /1916-5, /1917-1, /1917-4, /1920-1, /1921-3 of the

freezing points of different soils at different moisture contents. He used two methods: the freezing point method /1 and the dilatometer method /2. He concluded early /1916-3 that among the internal factors which influenced the freezing temperature were the water content of the soil and the concentration of the soil solution.

Bouyoucos found from his early studies /1915-2 that "...at low moisture content the lowering of the freezing point is extraordinarily high and very different for the various types of soil, being lowest in the sandy types and highest in the clay types; at high moisture content, however, the lowering of freezing point is relatively very low...with the exception of quartz sand the depression of the freezing point increases in a geometric progression as the percentage of moisture increases in arithmetic progression; in the case of quartz, however, the lowering of the freezing point increases directly as the moisture content decreases."

Later /1916-5 Bouyoucos found that the freezing points of the coarser soils (sands and light sandy loams) remained constant with successive freezings but for the finer soils the freezing point rose with successive freezings. In explanation he stated the hypothesis that "...the greatest portion of the water which the soils cause to become inactive or unavailable and thus lose its solvent action is due to the colloids which the soils contain. This inactive or unavailable water may exist in the colloids both as physically adsorbed water and as loosely chemically combined water. Upon freezing their colloids coagulate and the bonds uniteing them with the water break and the inactive or unavailable water becomes liberated. This liberated water becomes available or acts as a solvent and goes to dilute the original solution and thereby decreases the lowering of the freezing point..."

After amassing more experimental data Bouyoucos /1921-3 classified soil water on the basis of his freezing point determinations into broad groups as follows. Free water which freezes at -1.5 deg. C. and unfree water which does not function as a solvent and which he divides into (a) capillary water which freezes at -1.4 deg. C. and, (b) combined water which does not freeze at -78 deg. C.

Following his discovery of the effect of stirring /1923-3 he recognized the inadequacy of his previous hypotheses and explained further that "...upon freezing, the moisture in the small capillaries and that surrounding the particles as thick films accumulates in the larger capillaries of the soil by the force of crystallization. In other words, the water in the larger capillaries, upon freezing, draws upon itself by the force of crystallization the water from the finer or smaller capillaries and films around the soil particles, and grows at their expense. Thus the water in the large capillaries affects the freezing point depression differently from that in the small capillaries... This hypotheses assumes that the solution immediately around the soil particles and in the very fine capillary spaces is less concentrated than the mass of the solution. This assumption which accords with the results presented in this paper, holds that the force of crystallization tends to draw the moisture from the finer capillaries and from around the particles as films into the larger capillaries. It is readily seen that during freezing and thawing the dilute solution from the finer capillaries and the films from around the particles go to dilute the solution in the larger capillaries or the mass of the solution. The consequence is that the orginal freezing-point depression is diminished. When the soil mass is stirred the moisture is again redistributed and readjusted and the freezing point depression becomes as before."

/1 This method consists of observing temperatures at frequent time intervals by means of a thermometer inserted in a freezing sample, the beginning of freezing being accompanied by a sudden temperature drop during an otherwise gradual lowering of the temperature.

/2 The dilatometer method is based on the principle that water expands on freezing. A sample of moist soil is placed in a bulb to which is attached a graduated tube. The bulb is filled with kerosene or ligroin (a petroleum ether) and gradually cooled. The increase in the volume of the soil on freezing is taken as a measure of the quantity of frozen water.

The results of some of his work are shown graphically in Figure 9. Those results show that for a given soil, the greater the water content the more closely the freezing temperature approached 0 deg. C. the freezing point of free water. The sandy soils showed a small lowering while clay soils and highly organic soils showed a greater lowering of the freezing point with decrease in moisture content.

Jung /1932-4 found from his experiments that the freezing temperature of soil water depends upon the adsorption characteristics of the soil particles and published results of his experiments with a dilatometer to determine the freezing point of soils. Beskow /1935-1 pointed out inaccuracies in Jung's results due to errors in correcting for the volume change of the liquid and container and developed an improved apparatus and procedure for the freezing point method. Freezing point-water content relationships obtained by Beskow for different textured types of soils are shown in Figure 10.

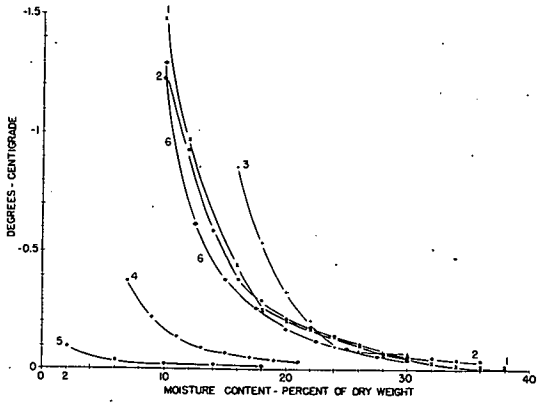


Figure 9. Freezing temperatures of different soils (curves 1 - 4 from tables 4 and 5, /1916-5. Curves 5 - 6 from tables 1 and 2 /1921-3. Types of soils tested are No. 1 humus loam, No. 2 and 6 clay loam. No. 3 silt loam, No. 4 sandy loam, and No. 5 a quartz sand. (After Bouyoucos)

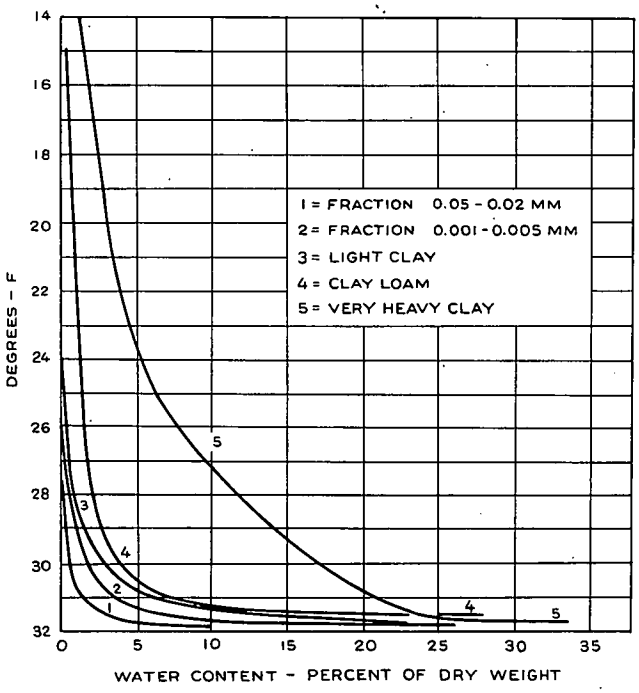


Figure 10. Relationship Between Freezing Temperature and Water Content for Different Soils. (After Beskow /1935-1)

Beskow /1935-1 in working with soil fractions found a straight line relationship between freezing point and grain size when soil is frozen in a saturated condition. Beskow explained /1947-12 that the lowering of the freezing point was due to the "adsorption power" of the soil particles as follows: "...The water molecules are grouped together in a skin around the soil particles. This skin consists of thousands of layers of molecules, the innermost layer being bound the strongest, the next, next strongest, etc." During freezing an "extra force" (lowering of freezing point) is needed to pull the water molecules from the skin and place them in the structure of the ice crystals. Zunker /1928-10 showed that the force of adsorption is practically directly proportional to the surface area of the particles down to a diameter of about 0.002 mm.: for smaller grain sizes there is a definite increase but it is inversely proportional to the specific surface of the soil. Thus the freest water which is essentially capillary water freezes first and as the temperature decreases more and more of the adsorbed water is drawn to the growing crystal.

Taber /1929-2 also found that all soil water did not freeze at temperatures several degrees below freezing. In discussing Bouyoucos' work he stated that Bouyoucos used small samples, employed rapid freezing with heat conducted in all directions and that those conditions were not conducive to ice segregation. Therefore, Taber modified his methods using a larger cylinder, meta-xylene, instead of ligroin, and a slower rate of freezing. In all Taber's tests he found that "...the change in volume indicated that for each 100 grams of dry clay about 6 grams of water failed to freeze". Taber concluded that "...the failure of part of the water to freeze is determined by pore space rather than size of particle..."

MacKintosh /1936-9, in a laboratory test showed that the 0 deg. C. line in a cylinder of clay soil undergoing freezing was about 2½ cm. below the limit of frozen soil. Later tests at Harvard University /1949-23 on soft clay showed that in one series the temperature at the bottom of the zone of ice lenses ranged from minus 1 to minus 2 deg. C. (30.2 to 28.4 deg. F.). The boundary temperatures in the second series ranged from 0.5 to 0.7 deg. C. (31.1 to 30.7 deg. F.).

Winterkorn /1943-15 held that Bouyoucos' /1921-3 classification of soil water based on freezing point determinations was arbitrary and not founded on knowledge of the properties of water as worked out by Tamman and Bridgeman. He discusses Tamman and Bridgeman's phase diagram for water (Figure 11) and concluded "(a) the melting point of water decreases with an increase in pressure only up to 2050 kg. per sq. cm. At all higher pressures, the melting point increases with increasing pressure, (b) the maximum expansion pressure obtainable in the freezing of water is 2050 kg. per sq. cm., and cooling below -22 C. does not increase the expansion pressure."

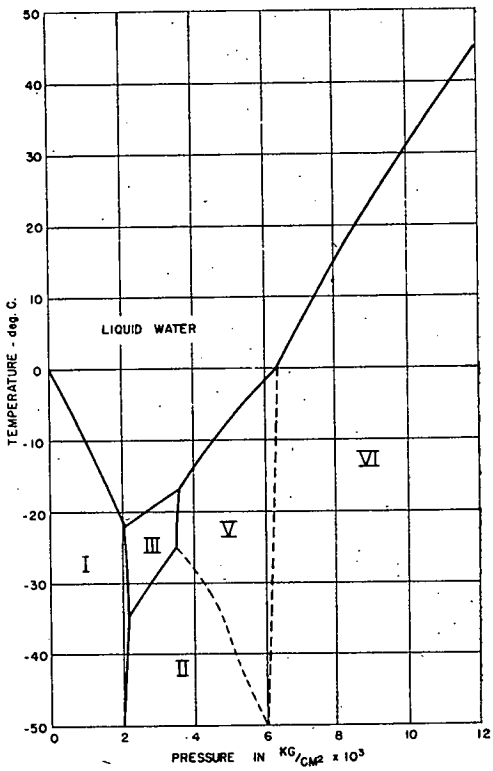


Figure 11. Phase diagram for one component system H_2O. (After Tammann and Bridgman)

To this Winterkorn adds, "The liquid state is an unstable condition for water below -22 deg. C. and if water does not freeze under expansion at this temperature, this is because it is already solidified or is being solidified as a result of external or internal pressures as ice III, V, or VI respectively".

Freundlich determined the adsorption pressure for alcohol and Florida fullers earth to be 20,000 to 25,000 kg. per sq. cm. The work of Winterkorn and Bauer showed the adsorption pressure of water to be at least equal to that found for alcohol.

Winterkorn holds that water in a soil capillary is most strongly held at the pore wall, while in the center of the pore it may be free or No. 1 water. Thus there exists a gradation "...from water which is already solid above 0 deg. C. (ice VI or V) through water with melting points decreasing to -22 deg. C. and thence through water with melting points increasing to 0 deg. C.". He computes that the decrease of pressure on normal water increases its melting point by 0.0075 deg. C. for each kg. per sq. cm. Then using a value of surface tension of 73 dynes per sq. cm., for a colloid radius of 10^{-5} cm. he computes the 'suction' to be of the order of 15 kg. per sq. cm. which will result in an increase of the freezing point of water by 0.113 deg. C. That suction is present immediately below the meniscus. Winterkorn further states that "This place then is the normal center of ice formation: between it and the solidly adsorbed water on the pore wall is a zone of liquid water possessing melting points down to -22 deg. C. (-7.6 deg. F.) and serving as a passageway for the conduction of water to the crystallization center".

Thus Winterkorn explains by means of the phase diagram for water what Taber and Beskow observed and deducted from general reasoning.

Data from laboratory freezing tests demonstrate that fine grained soils do not freeze at 0 deg. C. (32 deg. F.). An example may be cited. MacKintosh /1936-9 presented a sketch from tests underway at Harvard which showed the temperature gradient within a cylinder of clay soil undergoing freezing from the top downward. The sketch shows that the 0 deg. C. line is about 2½ cm. below the limit of frozen soil. The temperature at the frost line was about -3/4 deg. C.

Data from field studies apparently are too erratic to permit drawing conclusions based on the small range in temperatures at which soils freeze. Examination of data from the War Departments field studies on airfields /1947-2 shows reasonably close agreement between depth of penetration of the 32 deg. F. line and the observed depth of frost penetration. However where the two did not coincide the depth of frost penetration was greater than that of the 32-deg. F. line in more cases than it was less. Only at the Fargo and Pierre airfields was the 32-deg. F. line materially below the observed depth of frost. Where the frost occurred below the 32-deg. F. line the soils were usually lighter in texture than those found at Fargo and Pierre.

Summarizing, it may be seen that Bouyoucos found the depression of the freezing point related to the soluble electrolytes in the soil water, and that the freezing point is depressed in a geometric progression when the percentage of water is decreased in arithmetic progression. Later researchers believe that the freezing point of the soil is related to the "tightness" with which the soil water is adsorbed. Some authors have stated that if it were otherwise, how can almost instantaneous base exchange reactions be explained. Investigators agree that for a given soil moisture content below saturation, sandy soils freeze at higher temperatures than do clay soils.

The significance of the lower freezing point in clay soils is not that as subgrades they are, less apt to freeze, although from a practical standpoint that is true at low moisture contents. Rather, it is significant that movement of water to the zone of crystallization is made possible by a depressed freezing point. Without that depression, the formation of ice lenses and heaving in fine grained soils does not appear to be feasible.

Percent Freezeable Water in Soils

Bouyoucos /1917-1, /1917-4 tested more than 100 soils from 19 states to determine the percentage of water which froze at temperatures of -1.5, -4.0 and -78 deg. C. The soils represented a wide range of textural types. Examples of some of the data he obtained are shown in Table 2.

TABLE 2

Percent Freezeable Water in Soils (After Bouyoucos)

Soils	When Cooled to -1.5 deg. C. %	When Cooled to -4 deg. C. %	When Cooled to -4 and also to -78 deg. C. Twice %
Norfolk sand, Anne Arundel Co., Md.	1.39	1.39	
Plainfield fine sand, Wisconsin	2.94		
Miami silt loam, Kentucky	5.27		5.07
Miami silt loam, Delaware Co., Ind.	7.05	4.62	
Vernon clay loam, Archer Co., Texas	9.17	7.93	
Carrington silt loam, Wisconsin	11.36		9.85
Marshall silt loam, Kentucky	14.25		9.95
Carrington loam, Goodline Co., Minn.	16.22	8.22	
Black clay loam, Illinois	19.76		13.41
Houston clay, Franklin Co., Texas	21.88	12.50	

It has been mentioned that Bouyoucos /1917-4, /1921-3 classified soil water on the basis of the amount frozen at different temperatures. He classified water which freezes at -1.5 deg. C. as free water and, "...that which freezes at the super cooling of -4 deg. C. and also at the temperature of -78 deg. minus that at the above temperature is regarded as capillary-adsorbed water," while that which does not freeze at all at these temperatures is considered to be combined water. Applying these criteria to data given in Table 2 results in the percentages given in Table 3.

Wintermeyer /1925-1 conducted tests on about 150 soils to determine the freezing point and percent freezeable water in much the same manner as did Bouyoucos /1917-4 except that Wintermeyer used a different procedure for wetting the soil. On certain soils Wintermeyer found a relationship between percent freezeable water at -1.5 deg. C. and the dye adsorption number of the soil, the percentage of freezeable water increasing as the dye adsorption decreased. He also found that soils freeze at different rates. He found no relationship between mechanical analysis of the soil and percent freezeable water at -1.5 deg. C. He classified water content in the same manner as did Bouyoucos.

TABLE 3

Relative Amounts of Different Forms of Water Based on the Amount of Water Added (5 cc. to 25 g. of Soil) After Bouyoucos

	Free Water %	Capillary Absorbed Water %	Combined Water %
Quartz sand	100		
Plainfield fine sand, Wisc.	89		11
Miami silt loam, Kentucky	81	1	18
Miami silt loam, Delaware Co., Ind.	72	12	16
Vernon clay loam, Archer Co., Texas	74	6	20
Carrington silt loam, Wisconsin	61	5	34
Marshall silt loam, Kentucky	45	21	34
Carrington silt loam, Goodline Co., Minn.	32	38	30
Black clay loam, Illinois	34	30	36
Houston clay, Franklin Co., Texas	15	45	40

Ice Crystallization in Soil

Bouyoucos, Wintermeyer, Taber, Jung, and Beskow have shown that the freezing temperature of a portion of the soil water is less than that of free water and that the finer grained the soil, and the lower the soil moisture content, the lower the temperature at which soils will freeze. Thus the more nearly free water in the larger pore spaces freezes first.

If there exists free water in a large pore, or fissure, freezing will begin there, as it has a higher freezing point than the water in the smaller clay pores. Ice crystallization begins in the larger pores and fissures, the force of crystallization widens the opening, and the crystal continues to grow as the unfrozen soil water flows to the growing crystals. The coarser the soil particles the lesser the difference between size of fissures and other openings and the natural pore space and the lesser the difference in the freezing point of the water contained in the pores. Thus crystallization can occur as easily in the natural soil pores as in the larger openings.

In a fine silt or clay the particles are surrounded by concentric films of water. The binding force which holds these films decreases with increase in distance from the particle. These 'adsorption films' act partly to lower the freezing temperature of the water and also have a purely mechanical effect. Beskow /1947-12 states that "...if there is a small temperature depression underneath an ice crystal, the water molecules in the film nearest the ice crystal enter the crystal structure, that is, ice crystallization grows downward. This makes adsorption water film thinner, causing an increase in the negative pressure (pressure deficiency) causing water to flow to this surface, and the adsorption film swells until it reaches the same thickness as before. Actually, this whole process occurs continuously".

The Volume Change of Water on Freezing

Watson /1911-1 states that the density of ice at 0 deg. C. is 0.91674 and that of water at the same temperature is 0.99987. The increase in volume of one gram of water when it solidifies is 0.0907 cc. or slightly greater than 9 percent.

Theories on the Formation of Ice Lenses and Heaving

Some of the earliest recorded literature on frost action from Sweden showed that Runeberg /1765-1 believed that heaving was attributable to the volume increase of soil water on freezing. That opinion apparently was held widely until after 1900 when Johansson /1914-1 discovered that water flows to the freezing zone and two years later /1916-4 when Taber explained the cause of heaving.

It is difficult to learn the exact origin of presently held theories explaining the basic process of soil freezing and heaving. Experiments on growing crystals in saturated solutions were reported by Lavelle /1853-1 and confirmed by Lehmann /1888-1. Later Becker and Day /1905-1 found it practicable in a saturated solution of alum to "...grow clear crystals a centimeter in diameter which would raise a weight of one kilogram a distance of several tenths of a

millimeter". They found they could produce similar results by using a number of other salts. Abbe /1905-2 observed columns of ice exuding from the bark of a tree and, as did several others, described the formation of long slender columns of ice which formed on moist soil on a clear cool night.

Taber /1916-4 in a discussion on the growth of crystals reported by Becker and Day, the formation of ice columns by Abbe as well as his own experiments stated that "...all lead to the same conclusion: The essential condition for the formation of ice columns is that the water for growth of the ice column must be delivered through a small capillary opening at the base of the growing crystal which must not elsewhere come into contact with water."

The literature at the time of Taber's statement contained many apparent contradictions as to the nature and cause of frost heaving. Gilkey /1917-3 described the case where bridge piers 8½ ft. square and weighing 36,000 lb. were raised a maximum of 2 3/4 in. He argued that it is doubtful if frost penetrated more than one foot below the piers and that that depth of frost could not account for the heave. He believed the action to be similar to that of a hydraulic jack, the pier being analogous to the piston and the water or semi-fluid mud forced under the pier by pressure from the expanding strata nearer the surface, causing the piers to rise. Taber /1918-1 believed Gilkey's explanation inadequate and explained that the piers had greater specific gravity and greater weight per unit area and would expect "...the frozen crust of the earth to be forced up rather than the piers".

Taber /1918-1 placed metal weights on wet clays and sands and exposed them to freezing on cold nights. Where the weights rested on sand, the sand froze without perceptible raising of the weight. Where the weight rested on clay, ice formed, raising the weight. Taber stated that "...the lifting of the weights (on the clay) was due to the growth of ice crystals". Wycoff /1918-4 described up to 2 3/4 in. heaving of piers supporting columns and roof trusses. On excavating the soil he found ice layers and verified the findings of Taber. Norton /1918-5 in discussing Taber's article /1918-1 presented a sketch showing how walls of a cottage in Ontario were heaved, leaving the center at the original level. Since he found no ice between the sheathing and the ground, Norton believed that it demonstrated that the ground heaves or rises in winter because the water in the soil expands and not as Professor Taber suggested by the formation of ice crystals.

Bouyoucos and McCool /1928-5 explained that heaving of soils was due "...almost entirely to the accumulation or drawing of water at or near the surface upon freezing and to the building up of this water into capillary ice columns or long needle like crystals, or in some cases into solid ice". They stated further that "The principle...that underlies the heaving of pavements is also based on the natural tendency of water to move upwards and accumulate at the point of freezing; the movement and accumulation being impelled by the force of crystallization". They emphasized that heaving is not attributable solely to expansion of water upon freezing.

Taber had now completed his studies and in three reports /1929-2, /1930-7, /1930-9 gave the first comprehensive statements presented in this country on the process of soil freezing and heaving. His theory explaining the mechanics of heaving is based on the premise that (1) all soil water does not freeze at the same temperature which makes it possible for water to move to the growing crystal, (2) the growing crystal displaces material overlying it and thus develops ice lenses; and, (3) the frost line is relatively stationary during the growth of crystals forming the ice lens or stratum. The summary of Taber's work can perhaps be presented best by direct quotation of those portions which explain the mechanics of freezing and heaving.

He held that "During the growth of an ice layer in clay, water is supplied to the crystals through small capillary passages but the upward flow of water should not be attributed to capillarity, as there is no free surface or meniscus. The uplift is due to the cohesive forces in the water"..."Conditions existing during the growth of an ice layer in clay are particularly favorable for the uplift of water by molecular cohesion".

"A growing ice crystal is in contact with a thin film of water similar to the adsorbed layer that forms on other solids. As molecules are removed from the film and attached to the crystal, they are replaced by others from the surrounding water. When an ice crystal grows in a direction in which growth is opposed by a solid body such as a clay particle, the pressure is exerted through this film which separates them. ...After the available water has been exhausted, the film may be frozen, but it does not freeze easily. Cohesion is greater between the molecules in the film and between these molecules and the ice than it is between water molecules that are not close to ice crystals.

"Orientation and attachment of a water molecule to an ice crystal is accompanied by a slight repulsion, proportional to the change in volume, and this results in a slight displacement of the whole crystal relative to the neighboring solid. Crystallization may be accompanied by the conversion of dihydrol into trihydrol molecules, as advocated by some physicists, but this would not materially alter the process here outlined. As the new molecule is attached to the ice crystal, another molecule is drawn into the film by cohesion, and this is the direct cause of most of the displacement.

"The growth of ice layers in soils and the accompanying pressure effects are therefore attributed to molecular cohesion. The energy for the process is of course supplied by the removal of heat. Water is not only pulled into the film under an ice crystal with force sufficient to lift the overlying load but considerable force must be exerted in pulling it through dense clay to the growing crystal; hence in these experiments, water had been placed under tension sufficient to lift a column of water over 150 meters (492 ft.) in height".

"The growth of ice layers is stopped by lack of water supply, which may be due to rupturing of the upward moving filaments and films of water under tension or to the inability of molecules to enter films that are under pressure. Pressure tends to reduce the thickness of the nourishing film by expelling some of the molecules, and since the expulsive forces are increased the attractive forces must likewise be increased by lowering the temperature, else molecules can not enter the film. Pressure decreases molecular mobility in the film, retards crystal growth, and lowers the freezing point. The growth of ice crystals under pressure in open systems is possible because water occupying very small voids can be undercooled".

Taber /1929-2 gave the following explanation of why ice layers do not form in coarse grained soils: "The freezing isotherm /3 cannot advance as rapidly in water as in the minerals found in soils, for it is a poorer conductor of heat, has a higher specific heat, and also much heat has to be removed in converting water into ice. Hence, if ice crystals in growing downward come in contact with the top of a large soil particle and begin to surround it so that the temperature at the top of the particle drops below freezing, then the temperature of the bottom of the particle will reach the freezing point before the water with which it is in contact. Therefore, freezing takes place in part outward from the surface of large mineral particles that are in contact with water, and not merely through the downward growth of ice crystals as in pure water".

"When a growing ice crystal closely approaches a soil particle, the water separating them is gradually reduced to a very thin film and further growth of the crystal in this direction can take place only as molecules of water enter this film. If the soil particle is relatively small so that the molecules do not have far to travel through the film, and if growth is relatively slow, so that they have time to enter between the ice and the particle, then the growing crystal will exclude the particle; and if the particle is relatively large and if freezing is relatively rapid, the particle is gradually surrounded by the ice. To build up a layer of ice, which consists of many prismatic crystals, the capillaries supplying the water must be closely spaced".

Taber /1930-9 in referring to open systems believed that the total thickness of the ice layer equals the surface uplift and that the water content of the frozen clay between the ice layers is about the same as that of the clay below the frost line. He believed that under those conditions the freezing of the interstitial water was of little consequence.

The early studies in Michigan during 1926 and 1927 showed that expansion of water to ice could not account for frost heaves of the magnitude experienced. Burton and Benkelman /1930-10 stated that "Only a theory predicated on a natural moisture increase during the freezing period could possibly account for the excessive heaving which is followed in the spring by a super saturation of the soil on thawing". Extensive field and laboratory studies of frost action were made. Detailed studies were made of 156 locations where heaving occurred. The field experience showed that "...by far the most serious disturbances occurred in silts with an ice formation or banding very similar to that described in Taber's experiments with clay".

/3 Line of freezing temperature.

Michigan also conducted laboratory studies to measure heaving of different soils. Burton and Benkelman /1930-10 found that where silts were frozen in the laboratory, in contact with water at the base of the specimens, only minor heaving resulted, except when thawed and refrozen.

They showed (see Fig. 12) that two methods can be used to obtain "...a shifting of the frost line upward..." by either changing the freezing temperature from -20 deg. C. to 5 deg. C. (Case No. 1) or by changing the soil temperature at the bottom from +5 deg. C. to +20 deg. C. They indicated that the former represents what happens in nature. The laboratory tests were made using a constant rate of cooling on cores from known silt frost heaves. Under the conditions of freezing no frost heaving occurred.

Two series of specimens were placed in open ended tubes /1931-11. One series was frozen at a uniform rate of cooling (-2 deg. C. at top of specimen and + 2 deg. C. at bottom of specimen. Examination "...revealed the development of only a slight amount of expansion due to change in volume of the interstitial water being converted into ice." A second series was frozen under a constant cold room temperature. However, the temperature of the freezing cabinet was increased and decreased to provide vertical insothermal line movements. The authors gave the following account of the effect of a fluctuating frost line on the nature of ice formation and heaving:

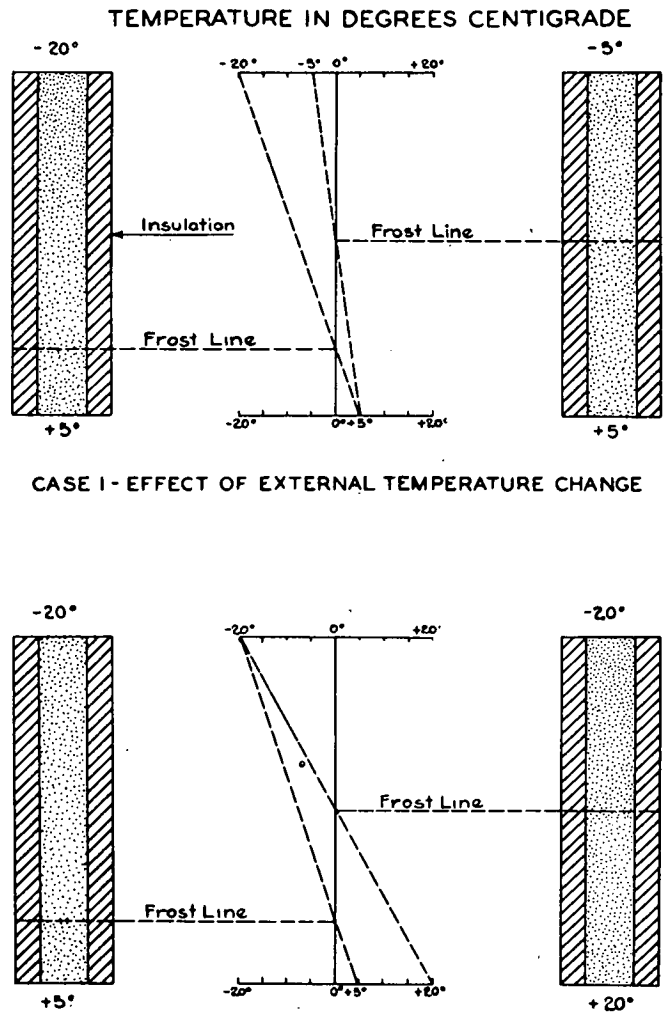


Figure 12. Theoretical Analysis of Temperature Movements in Soil Specimens (After Burton and Benkelman)

"On the downward movement of the isothermal line, a slight expansion of the specimen occurred. On the upward movement a break or void in the soil column became visible which followed the contact of the frozen and unfrozen soil upward. As the movement continued the frozen soil particles could be observed to detach themselves as the ice films surrounding them melted and settle through water present in the void coming to rest in a more compact state on the unfrozen soil beneath. Before the water void reached the top of the specimen a reduction of the temperature within the cabinet caused the isothermal line to again move downward. The water in the void space was then converted into ice and, as the frost line progressed below this level, a second increment of vertical expansion occurred due to another freezing of interstitial water. A second thaw or upward movement of the isothermal line resulted in the formation of another cumulative water void which was subsequently converted into an ice plate in the same manner as before. The total thickness of the ice plates introduced and the vertical expansion of the soil column were measured and found to be approximately equal in magnitude. In Figure 13 (1931-11) A represents unfrozen soil; B frozen soil slightly expanded due to freezing of interstitial water; C represents the cumulative water void after conversion into ice with an added increment of vertical expansion due to a second freezing of the soil beneath; E and F show the presence of a second ice plate developed in the same manner as the first". Similar tests were made on coarse sands. They behaved similarly to the silts. From the results obtained the authors concluded tentatively that excessive heaving may occur in soil of almost any grading or texture provided an adequate supply of water is present or available. The authors /1931-11 also stated a theory that a fluctuating frost line occurs in nature causing a series of events similar to those which were produced artifically in the laboratory tests. They held that the theory "...offered an explanation of the formation of ice plates of any size and number. All that is required is a soil saturated with water and a fluctuating frost line...".

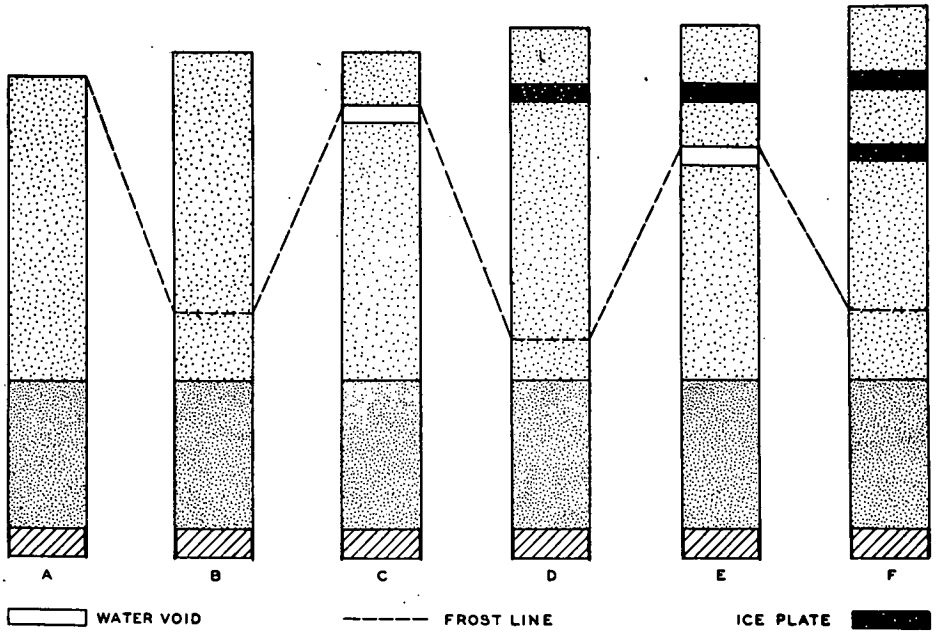


Figure 13. Ice Plates Formed by Controlled Isothermal Line Movements. (After Benkelman and Olmstead /1931-11)

The presentation of the theory by Benkelman and Olmstead was followed by some discussions. Watkins /1931-12 added that ice layers "...may also form in sands which are not saturated, due to the formation of an artifically high water table caused by the rise, condensation and freezing of moisture vapor. In stratified sands this vapor may cause ice layers between strata". Taber /1931-13 held to his original position that frost heave is due to growth of crystals and not to change in volume. To support his position that increase in volume is not a factor in frost heaving in open systems, he substituted for water other liquids which solidify with a decrease in volume. For example he was able to show heaving obtained by freezing a clay column which stood in a sand saturated with nitrobenzine. Casagrande /1931-14 raised the point that if a frozen sample in a cylinder is thawed from beneath, the frozen portion will adhere to the cylinder and cannot follow the subsidence of the thawing portion, while in nature the frozen layer does subside. He concluded in his summary of findings from M.I.T. and New Hampshire studies that no ice segregation was observed in sandy soil having less than one percent of material finer than 0.02 mm. even if the ground water was as high as the frost line.

Housel /1938-13 believed that the findings of Benkelman and Olmstead "...answered the obvious discrepancy between Taber's theory and the field experience in Michigan soils and provided a more adequate basis for identifying frost heave soils and locations. The theory of the fluctuating frost line does not necessarily supplant Taber's theory, however, as it is entirely possible that frost heaves may be produced under the conditions of a constant frost line and soils of high capillarity. Both theories have practical value and, within their specific limits of application should be useful in identifying objectionable field conditions".

Beskow's statements /1935-1, /1947-12 explaining the mechanics of frost heaving differed but little from those of Taber. He believed that in freezing a soil in which ice is forming "...the water next to the ice strata changes from a liquid to a solid state and has the same effect as a change into vapor by evaporation. At the frost line a drying out occurs, a squeezing together of the adsorption films, which spread further and further, causing a shrinkage. If the zone spreads to a place of contact with a free supply of water..., the water begins to flow upward from this point and the fundamental requirement for... a subsequent frost heave is fulfilled".

Soils having different grain sizes act differently. Beskow /1935-1 explained that in a saturated clay the amount of "mobilized" water is large, becoming larger, the finer the clay. In a heavy clay, the amount of mobile water can be so large and the permeability so small that the formation of ice lenses is limited largely to those formed by local flow from the soil adjacent to the ice band, and it cannot suck up water from the ground water table.

In silts and silty sands, the thickness of the adsorbed films of water is small. Thus the amount of water that can be set free by an increase in the capillary pressure is small though the permeability is great. Thus few thin ice bands develop in silts from the contained water alone. However, if the soil is in contact with groundwater, silts and silty sands, having relatively high permeability, cause considerable water to flow to the freezing zone causing large heaves. The phenomena described are indicated in Figure 14.

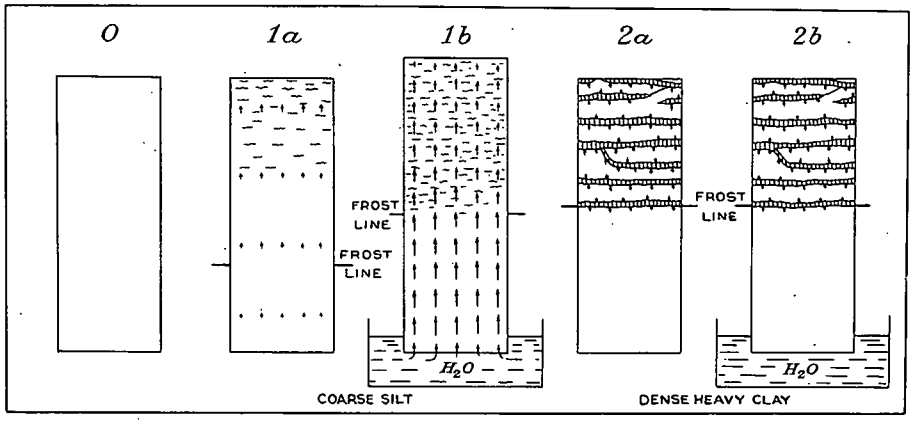


Figure 14. Diagram Illustrating the Difference by Freezing of (1) a Coarse Frost-heaving Soil (Silt), and (2) a Dense Soil, a Heavy Clay, When the Soil-Column is (b) or is not (a) in Contact With a Free Water Supply. In each case is the original water-content high (full capillary saturation). The differences are due to the following facts: No. 1 has a very slight water-surplus in itself, but a great permeability, thus being a good water-conductor; No. 2 has a great mobile water-surplus, but is very dense and therefore a bad water-conductor. Intermediate soils (fine silts, silty loams and loams) behave intermedially. (After Beskow)

The solid materials do not expand when cooled (in fact they contract). According to Beskow there must be a separation between the soil grains within the mass at certain places if ice lenses are to form. In order that growing ice crystals can expand they must push the soil particles ahead of them. This can happen only if new water molecules can come between the growing crystals and the particle surfaces they contact. Figure 15 indicates what happens at the frost line during the freezing of a fine grained soil compared to freezing of a coarse grained soil. Beskow explains the phenomenon as follows: "...The outer surfaces of the ice crystals do not rest directly against the particles but lie between the adsorption films. Assuming now that the ice crystals grow from below, displacing a layer of molecules taken from the adsorption films, these films become thinner and they in turn get back their thickness by attracting water molecules from the side...In a course soil the meniscii are very wide and thin. Free passage of water is thus small because it concerns the innermost, tightly bound molecules which are the least mobile. In a fine grained soil...the meniscii or films are relatively short and thick, and the flow of molecules can occur much more rapidly."

"In order to have frost heaving there must be a certain rate of diffusion or molecular mobility, in the films between the soil particles and the ice. In coarse soils this is small and for practical purposes is zero (in sands); no heaving occurs and the ice grows into the pores and surrounds the particles. On the other hand, in a fine grained soil such as a clay or a fine silt water molecules flow quickly through the adsorption films to the surface of the growing ice crystals, which pushes the soil particles ahead, or actually lifts itself and the overlying mass upwards". In this manner thick ice strata may be formed.

Road Abstracts, /1942-4 in a review of an article by Schmid attributes the following theory on frost heaving to him.

"Since the expansion of the soil caused by frost heaving is considerably greater than can be accounted for by the freezing alone, it must be due to air freed by the drop in temperature. Experiment has shown that in the absence of such air frost heaving does not occur. In soils containing colloids, frost causes alternate contraction and expansion of the colloids as the temperature falls and rises before and after the formation of ice. The place of the water absorbed in

swelling is taken by air released from the moisture. If the air is not sufficient to fill all the space, shrinkage of the soil occurs with every fresh formation of ice, but if the air is more than sufficient, expansion results. Thus the degree of frost heaving depends primarily on the air content of the soil or soil moisture, and only secondarily on the moisture capacity of the soil or on the ground-water".

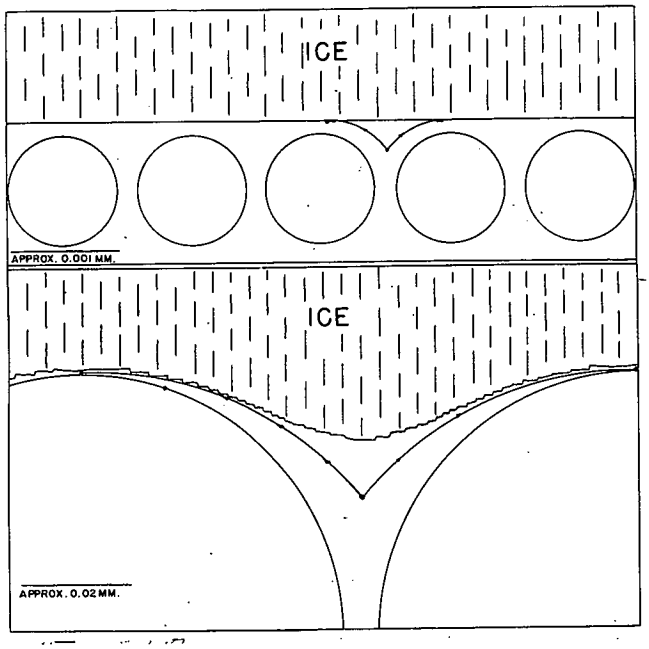


Figure 15. Schematic detail-picture of the frost-limit, or the contact between a growing ice-crystal and the underlying soil-particles (drawn as spheres), in two different types of soils; the upper one a clay, the lower one a coarse silt or fine sand. The points and the arrows show the maximum distances from pore mouths to ice-surface, that the water-molecules have to move in order to cause frost-heaving. (After Beskow)

Winn /1940-10 summarized the findings of most of the principal investigators on conditions necessary for frost action to occur briefly as shown in the following outline.

1. Destructive freezing associated with formation of segregated ice.
2. Total frost heave equal the sum of the total thicknesses of ice lenses.
3. Total frost heave in direct proportions to increase in total water content of frozen soil.
4. Soil must equal state of capillary saturation for ice segregation to take place.
5. A supply of water must be available either in soil or from some external source, i.e. - water table.
6. For normal conditions of temperature, the percentage of 0.02 mm. diameter of grains is critical.
7. One slow freeze will cause heaving. Thawing and refreezing increases severity but does not change basic action.
8. Cumulative curve of degree hours of freezing plotted against time is qualitative measure of increase of heave with time.
9. The following factors are all necessary for ice segregation and frost heave.
   a. Capillary saturation of the soil at beginning and during freezing process.
   b. A free supply of water from within or without.
   c. A minimum percentage of grains smaller than 0.02 mm. (3 to 10%).
   d. Gradual decrease in temperature of air above soil to below freezing temperature.

Heaving in Open Vs. Closed Systems

When a soil is said to freeze as an open system, it is assumed that water for ice segregation may be drawn from a mobile supply suspended in the soil and that the supply can be replenished from an adequate supply of ground water below or that surplus water can be expelled downwards as freezing progresses in coarse textured soils. A closed system infers that the water for ice segregation must come from that held within the soil and the soil does not have available an outside supply of water.

Beskow /1935-1 considered the possibility that marked heaving might occur from the supply of mobile water within soils without the presence of an adjacent water table, and conducted laboratory tests to determine the nature of heaving under those conditions. He used segmented glass tubes to minimize sidewall friction, and tested a fine silt soil (15 percent retained on No. 200 sieve, 52 percent silt, 23 percent clay) having a capillarity of 9.6 meters (31.5 ft.), freezing it under a load of 50 g. per sq. cm. (0.71 psi). He found that heaving is rapid at the start as the first water is drawn up to the frost line but the rate of heaving decreases until the full capillarity is reached after which heaving occurs at a constant rate until the mobile supply of water is frozen. The resulting heave under the conditions inferred is usually small. However, it is significant that freezing does result in a marked increase in the water content in the frozen zone causing a reduction in load carrying capacity on thawing.

The Corps of Engineers /1951-32 tested four cylindrical specimens 6 in. high of undisturbed Boston Blue clay, 100 percent saturated in a closed system. Heaving ranged from 8 to 14 percent. The lower portion of one of the specimens shrank from an initial of 4.27 in. to 4.04 in. That shows that ice segregation and heaving can occur in fine grained soils remote from a water table.

Beskow considered freezing of partly saturated soil as "...of some theoretical interest...". He made tests similar to those described above for a fine silt except they were made on partly saturated sands; one a fine sand having a capillarity of 54 cm. (21.3 in.); the other, a coarse sand with a capillarity of 10 cm. (3.9 in). Each was tested at about 10 percent moisture by weight. The fine sand was tested at three different rates of freezing and at different loadings. The results are shown in Table 4.

TABLE 4

Results of Freezing Tests on Moist Soil Samples (Not Saturated) (After Beskow 1947-12)

Type of Soil	No. 1 Fine Sand			No. 2 Coarse Sand
Test Number	1	2	3	4
a. Temperature Gradient, deg. C. per cm.	1.0-0.6	0.1-0.05	0.2	0.1-0.0
b. Freezing Time, ...hr.	2.5	6.5	6.0	2.17
c. Load . . . . . p.s.i.	0.36	a) 0.36 b) 0.71	a) 0.71 b) 1.56	0.36
d. Initial Water Content . . . .percent	10.7	10.9	10.2	9.8
e. Final Frost Depth. . . . . .mm.	11	75	38	29
f. Increase of Water in Upper Cm. of Frozen Soil. . . . . Percent by Weight	+ 9.2	+4.2	+8.6	-----
g. Total Heave. . . . . .mm.	0.80	0.25	0.13	0.12
h. Water Frozen in Sample (height)· ·mm.	3.17	11.21	8.35	4.4
i. Normal Expansion of Above in Freezing (0.1 h) . . . . . .mm.	0.32	1.12	0.84	0.44
j. Frost Heave Quotient . . . . . g:i.	2.5	0.21	0.16	0.27
k. Initial Height of Cylinder . . . .mm.	75	75	75	65
l. Initial Diameter of Cylinder . . .mm.	32.5	32.5	32.5	32.5

The results show an expansion, though relatively small, on freezing partly saturated sands, the expansion being affected noticeably by surface loading. For small loads (less than weight of 6 in. of P.C.C. pavement) the heave is several times that which may be attributable to the volume change of the contained water on freezing, ranging from 2.5 times for the lightest load (0.35 psi.) to 0.16 as much for the largest load (1.56 psi.). Increasing the rate of freezing decreased the heave.

From a practical standpoint the marked increase in water content of the frozen portion of the sand is worthy of note (Fig. 16).

Beskow considered the possibility that the relatively large moisture gain in the frozen portion was due to diffusion and applied the findings of Zunker /1930-17 in computing the maximum possible water. He then discarded the possibility that diffusion could cause marked increases in water content. He set forth the following hypothesis on flow to the freezing zone in moist sands:

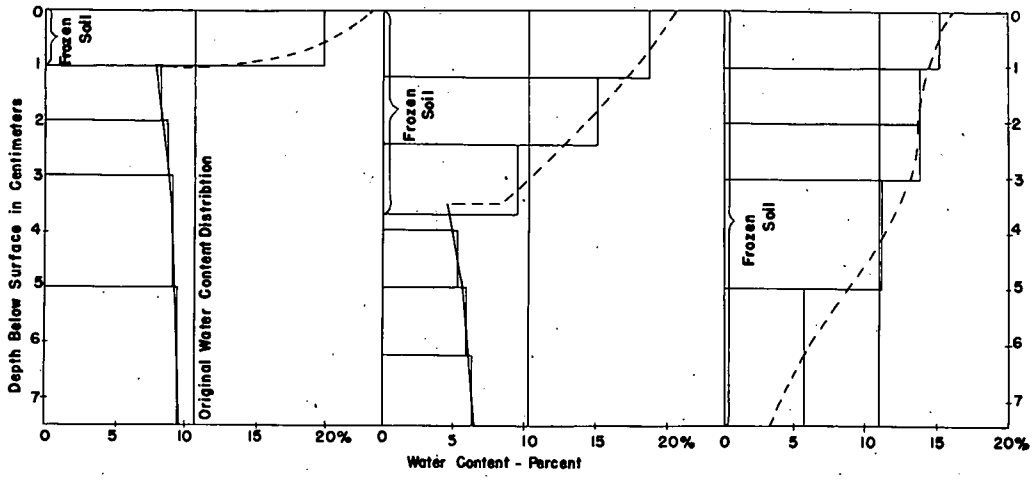


Figure 16. Distribution of Soil Moisture Content After Freezing Columns of Moist Sand - Columns correspond to test numbers 1, 2, and 3 in Table 5. (After Beskow)

"In freezing a moist sand, the adsorption-water membranes enclosing the surface of the particle also enclose (or try to enclose) the lower surface of the ice crystals in the frost line. The ice crystals grow in such a manner that the water molecules next to the ice enter the atom structure of the ice. This implies that the membranes or films which are in contact with the ice become thinner. But there is a tendency for the meniscii to have the same stress all over (i.e. the same thickness). The decrease in thickness at a point then causes a flow of water to that point...".

Taber /1929-2 froze a number of specimens of clay without access to free water. Because of the meager data available on the effect of freezing partly saturated soils, Taber's measurements of the soil moisture in the bottom portions of the cylinders before and after freezing are shown below:

Test Number	Percentage Water Before Freezing	Percentage Water After Freezing
1	29.0	20.8
2	25.8	19.9
3	24.2	18.3
4	23.2	18.3
5	19.9	16.8
6	16.8	12.3
7	9.5	7.7

It can be seen that there was a tendency for the water to become concentrated in the frozen upper part even in the soils with the lowest moisture content. Segregated ice occurred in 1, 2, 3 and 4 with maximum thickness of 3 cm. in 1 and smaller in 2, 3, and 4. Tests 5, 6, and 7 showed no visible evidence of segregation revealed by moisture determinations.

Taber and Beskow agreed that soils in nature seldom behave as absolutely closed systems. Taber mentioned the possibility that a closed system may exist when the water table is flat over large areas and nearly coincides with the surface. Of soil types only some mucks, gumbo soils and soils containing bentonite behave on freezing like closed systems because of their relative imperviousness.

Effect of Direction of Cooling

Taber, Beskow and others proved experimentally that ice crystals grow in a plane parallel to the direction of heat transfer. Taber /1930-9 inserted copper bars vertically into cylinders of clay. Their greater conductivity caused heat to flow to the bars and ice bands to form parallel to the bars. Beskow cooled large unconfined samples and they developed ice bands only parallel

to the surface. Under natural freezing conditions, heat is conducted away from growing ice crystals in one direction and they are in contact with water in an opposite direction; therefore, growth takes place in a single direction. This may be of some significance with regard to retaining walls where thrust may develop due to "lateral" heave.

Shrinkage of Soils on Freezing

The occurrence of shrinkage in soils, usually clays, during freezing has been observed by soils engineers in the Northern States. Very little has been recorded in the literature on shrinkage associated with soil freezing. Apparently, shrinkage may be due to contraction and cracking as frozen soil is further cooled; or to a drying out with water loss to the air or to adjacent soils undergoing freezing. Taber /1929-2, /1930-9 found that "...in every experiment in which no additional water entered the system, the withdrawal of water from the lower part of the container to build up layers above caused shrinkage in volume and, usually, the development of tensional cracks...". That shrinkage indicates the force with which the water is sucked out of the soil on freezing. Terzaghi /1925-2 showed that those forces are equal to the external pressures required to produce an equivalent change in volume, and may be very large. Hogentogler /1931-6 found that highly cohesive soils are apt to shrink on loss of water due to movement to the freezing zone. Minnesota /1945-8 reported that fills of dense "gumbo clay" (A-6) soils in the Red River Valley shrank 0.5 ft. after freezing began and swelled to their original profile after the spring thaw. The shrinkage caused "bumps" adjacent to bridges and culverts.

Illinois /1924-3 made careful measurements of soil movements at various depths under pavements and found that in every instance, the increase in elevation of the soil above the frost line was accompanied by a decrease in elevation of the underlying soil denoting a shrinkage as water was drawn from it to the freezing zone.

Transverse cracks or fissures which appear in shoulders and shoulder slopes and which are coincident with cracks in the pavement at intervals as small as about 50 ft. were observed in Wisconsin by Bleck /1949-11. Where that occurred, the pavement or base course or both froze solidly to the sub-base or soil to form an integral mass. Longitudinal fissures also occurred along the pavement edges concurrent with the transverse cracking.

Bleck explained that under some conditions of low temperatures, there occurs a dehydration of the soil induced by heat loss from the soil and that if that moisture is not replaced from below or from precipitation, a shrinkage occurs causing rupture at critical locations.

The Process of Thawing

The thawing of frozen ground takes place from both upper and lower surfaces of the frozen stratum. Thawing often is more rapid from the upper surface downward although that is by no means always the case as the reverse is sometimes true. For some reason, the literature contains little information on the physical process of soil melting and the subsequent changes in moisture distribution which takes place as the soil adjusts itself to an unfrozen environment.

Taber /1930-9 states that "...if the air temperature is well above zero deg. C. so that melting occurs from the surface, the water set free cannot escape downwards through the soil voids until thawing is practically complete". "If the air temperature remains barely below freezing for a long time a deeply frozen soil will thaw gradually from the bottom...and the water may then return to the underground...from which it was drawn. But since water is pulled to the surface during the freezing process by a force that is greatly in excess of the forces causing its downward movement in the soil, the water may not be removed as rapidly as melting takes place unless the latter is very slow".

Beskow /1935-1 is one of few authors presenting data on the rate of thawing. He found the rate of thawing from below to range from ¼ to 1 cm. per day. These values refer to ordinary flat ground and different values may be obtained under roads. He showed data on the rate of thawing from below from five different locations as follows:

Locality	Maximum Rate of Thawing From Below (cm. per day)
Sunderbyn	0.1
Trondelag No. 1	0.4
Trondelag No. 2	1.2
Trondelag No. 3	0.7
Trondelag No. 4	0.14

Bouyoucos /1913-1 mentions a case where the soil at 18-in. depth thawed before the soil at 12 in. Belcher /1940-14 stated that "Records of certain years illustrate that the majority of thawing occurs from below because of the fact that the air temperatures remain near the freezing point in the early spring...mild winters seem to have this characteristic of prolonged periods in the early spring when the mean temperature remains near 32 deg. F. This would cause a cycle of freezing and thawing on the road surface each day but little thawing below 6 in. The thawing then, occurs principally from beneath".

Several authors have written that damage to roads is most severe when the thaw is sudden and rapid from the upper surface downwards, causing the melt water to be released quickly. The water is unable to drain or diffuse through the soil because the underlying frozen soil is impervious. The large amount of water in the thawed soil causes a correspondingly large reduction in its load carrying capacity. Repeated loading causes manipulation which further reduces its ability to sustain loads. Under severe conditions it is ejected from under a pavement in the form of mud boils. Slow thawing from the lower surface of the frozen layer may permit the excess water to move downward with only small reduction in load carrying capacity.

Williams /1945-5 found that during melting the frost line takes the form of a trough through a cross section of the roadway. After a warm spell following a deep freeze, rain or snow turns to slush entering the base above the frost partition indicated in Figure 17. In the spring when the water thaws the trouble begins. Williams states that "Time and again it has been found that the drop of the frost line in the center of the roadway was 15 in. or more deep while on the road shoulder it would be nearer the surface".

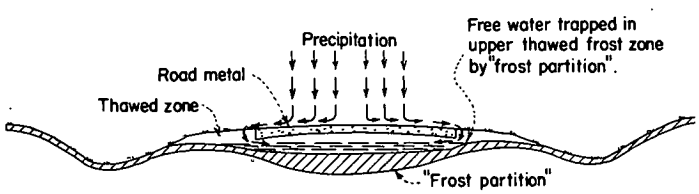


Figure 17. Cross section of road showing location and shape of "frost partition," and course of precipitation into upper thawed zone under road metal. Free water is trapped following early "deep freeze". (After Williams /1945-5)

The Corps of Engineers /1947-2, /1949-23 made numerous observations of depth of frost at various seasons and plotted frost profiles showing progress of melting from upper and lower surfaces. Thawing usually progressed faster from the upper surface. However it should be borne in mind that where determinations were made under paved surfaces, those surfaces were kept free of snow cover.

Settlement of Aggregates During Thawing

Loss of road metal in frost heave areas is often a subject of discussion among engineers. Taber /1930-9 mentions that soils when they become saturated on freezing and thawing permit objects with greater specific gravity to sink in them and states that "This probably explains the gradual settlement of gravel from under pavements". Burton and Benkelman /1930-10 observed settlement of aggregates in their freezing tests on 3 ft. high cylinders (soil had 8 percent clay, 60 percent silt and 32 percent sand) in which they inserted a gravel "cut-off" layer. Pieces of gravel from the cutoff layer settled a foot below their original position. They said "This serves as an explanation of loss of metal from gravel and macadam road surfaces which is quite common in Michigan".

Effect of Freezing and Thawing on Soil Structure

Several reports by agronomists and soil physicists are available in the literature on the effect which frost has on soil structure. This phase of frost action is in itself a special study. Therefore only two writings are reviewed here. Attention is given later to the effect of soil structure as a factor governing the magnitude, rate and nature of heaving.

Baver /1940-2 found alternate freezing and thawing a factor in developing soil structure causing a granulating action on soil clods that is usually more effective than drying and wetting. He cites the studies by Jung in which it was found that freezing may cause either aggregation or dispersion, the nature of the crystallization of the ice being the determining factor. With slow cooling the crystals form in tension free pore spaces which brings grains into closer contact causing aggregation. With rapid cooling, large numbers of small crystals are formed, breaking up the soil aggregates. The water content is as important as the rate of cooling. Aggregation increases under slow cooling with increase in moisture content up to about 50 percent of saturation. Gradner /1945-18 studied the effect of freezing and thawing on permeability of soils. Ice crystallization had a dehydrating and densifying effect in semi dispersed soils which developed a stable flaky structure in soils high in replaceable calcium

and an unstable flaky structure in soils high in replaceable sodium. Freezing and thawing did not restore permeability in sodium-saturated soils nor did replacement of sodium ions with calcium ions without freezing. However after calcium chloride was added, freezing did affect increase in permeability. Freezing and thawing aided in restoring permeability and structure in soils injured by sodium salts.

MAGNITUDE OF FROST HEAVE AND INCREASE IN SOIL MOISTURE

Writers have often stated that a large part of the damage associated with frost action is caused by traffic loadings during and immediately following the thawing period when the soil water content in the upper portion of the road is high. The increase in soil water is related directly to soil freezing. Investigators have shown that soil moisture moves in the direction of heat flow. Thus it is possible that some movement may take place as capillary or film flow or in the form of vapor flow without the presence of freezing. However, by nature that movement is generally small, while it is known that large water movements are associated with freezing. Because water movement is inherently a part of the frost action phenomenon it is mentioned throughout this review. It has already been mentioned under the subject "Heaving in Open Vs. Closed Systems". Because the magnitude of heave is related to magnitude of moisture increase they are considered under this common heading.

Magnitude of Heave

The literature contains a vast amount of observational data and some statistics on magnitude of heaves that occur under natural conditions. Information on magnitude of heave alone, without accompanying data on type of soil, proximity to water table, climatic conditions and the depth of frost penetration aids little in solving the problem but is of some general interest.

Burton and Benkelman /1930-10 (Michigan) stated that "The average height for all heaves occurring in silt was 6 in., in very fine sand and silt 5 in., in very fine sand 4 in.; in silty clay 5 in., and in sandy clay 3 in.". Lang /1930-4 reported heaves in Minnesota of as much as two or three feet." Beskow /1935-1 reported extensive observations of heaves in Sweden. He found depressions over culverts ranging from 6 to 16 in. in depth. Levels taken on railroads showed heaving up to 12 in. on "...normal moraine material in a relatively saturated state...". A survey of a 20-kilometer (12.4 mi.) section of a railroad in northern Sweden showed a magnitude of heave of 8 in. occurred over 3 percent, 4 in. over 15 percent and 2 in. over 25 percent of the total length. Beskow reported that heaving on Swedish roads ranged from less than 1 up to 21 in. Silt and clay sediments showed normal heave of 8 in. with maximum of 16 in. Moraines showed average heaves about half as great but showed the greatest variation. Lean clay on gravel or bedrock showed average range of heaves of 9 to 13 in.

Morton's /1938-10 and Gardner and Wright's /1939-5 observations in New Hampshire on the Granite State Park road over a period of several years showed that heaves of 6 to 7 in. were not uncommon. Ravn /1940-4 gave the maximum recorded heave in Denmark as 12 in. He found frost penetration of 32 in. can produce heaves up to 12 in. and penetration of 10 in. can cause heaves of 4 to 6 in. The Corps of Engineers /1947-2, /1949-23 found maximum heaves on airfields as follows:

Bangor, Maine	0.71 ft.
Presque Isle, Maine	0.54 ft.
Houlton, Maine	0.20 ft.
Madison, Wisconsin	0.18 ft.
Sioux Falls, S. D.	0.16 ft.
Fargo, N. D.	0.12 ft.
Watertown, S. D.	0.11 ft.
Pierre, S. D.	0.1 ft.
Bismark, N. D.	0.1 ft.

Maximum heaves at Pratt, Great Bend and Garden City, Kansas and Fairmont were 0.05 ft. or less.

Lang /1937-8 reported another form of heaving which occurred in Minnesota under "...high joints in concrete pavements" which in extreme cases measured as much as 2 in. higher at the joint than at the mid point of the slab. A later report gave differential movements at joints of the magnitude of 3/4 in. or more during the season 1932-33. The Illinois Division of Highways devised a frost movement indicator /1924-3 for measuring subgrade movements at various depths. A pit about 6 ft. deep was dug at the pavement edge. Metal plates were driven into

the pavement side of the excavation. The plates were connected rigidly to vertical rods which permitted reading of movements on an indicator board. Pipes surrounded the rods to permit free movement. Surface movements were recorded from precise level observations. An example of the nature of data recorded is shown in Figure 18. It may be seen that some shrinkage occurred in the soil immediately underlying the frozen layer. Additional data on heaving are shown in Table 5.

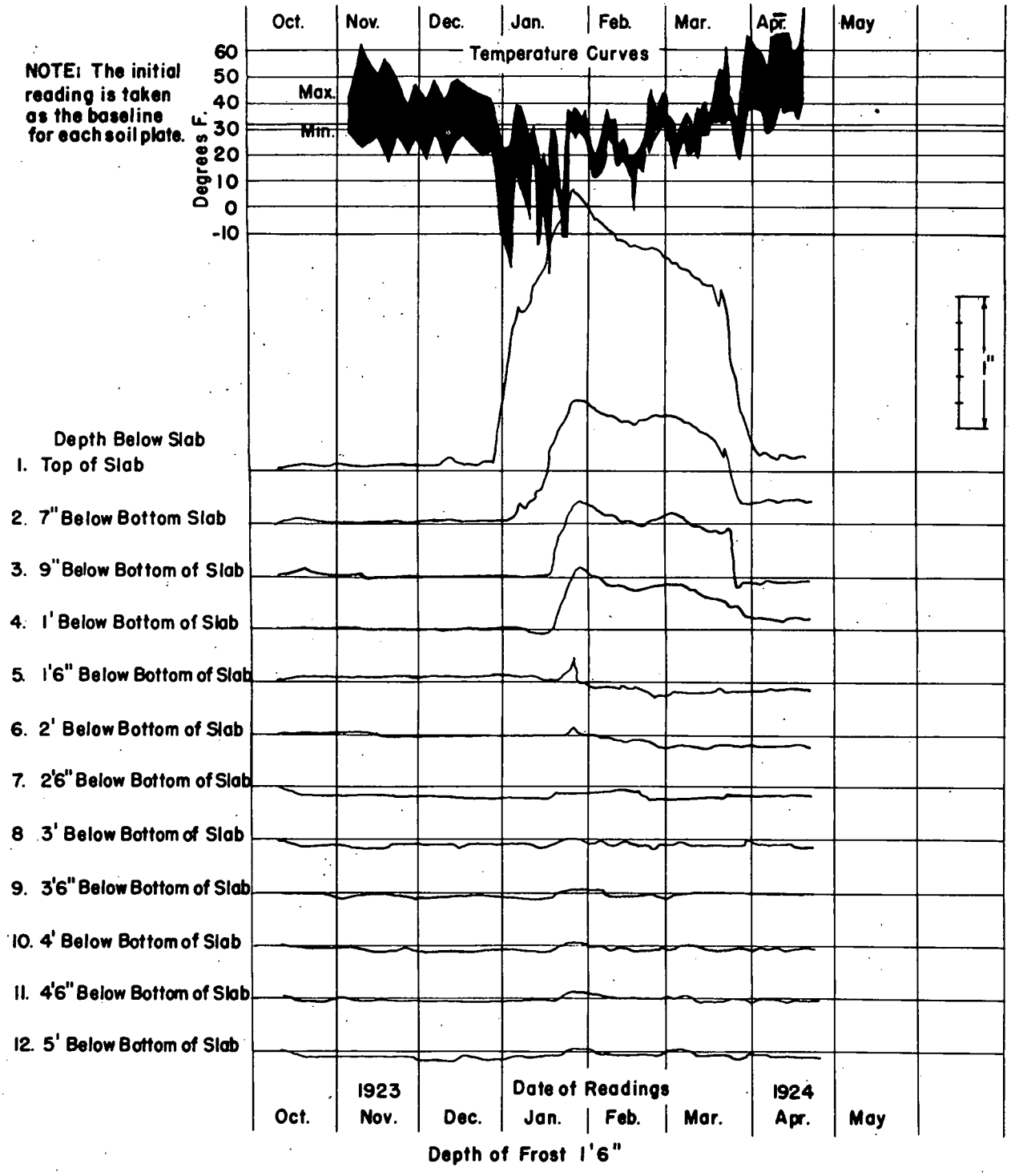


Figure 18. Frost Movement Indicator No. 1. Dist 1, Rt 5, Sect 8, Sta. 256 - 35 Plato Township, Kane County - Topsoil-Yellow Gray Silt Loam - Vertical (Movement) Scale 50 Small Div. - 1 (After Illinois Division of Highways)

Magnitude of Moisture Content Increase

Some of the earliest studies of highway subgrade moisture increase associated with soil freezing were those of the Illinois Division of Highways. The first of these were on the Bates Experimental Test Road /1923-5. Observations were made in all of the original 63 test sections for the period 1922-23 and for new sections from the fall of 1922 to the spring of 1923. The average weekly subgrade moisture contents of all sections (and all soil types) are shown in Figure 19 which indicates moisture contents were high in the winter and much lower in the summer. The considerable increase in moisture in the last month of readings coincided with the breakup of the pavement in the traffic tests. The results showed that the subgrade moisture content was influenced some by precipitation but perhaps more by the freezing of the subgrade and the maximum moisture content occurred at the time of thawing. Visual observations during thawing showed water on the top of the subgrade in some areas and the formation of a thin sheet of mud in some other areas.

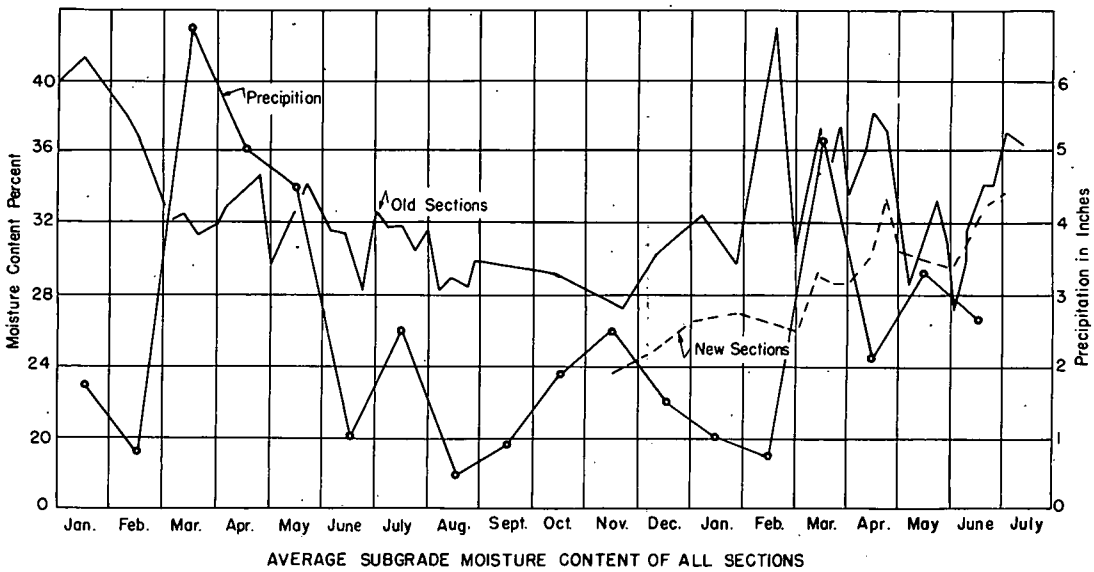


Figure 19. Average Weekly Moisture Content of All Sections
(After Illinois Division of Highways)

The Bates road tests were followed by studies /1924-3 of soil moisture in three southern and one central Illinois location. Altogether nine test installations were made to measure heave (with frost movement indicator described previously) and moisture content changes. The results of those studies are summarized in Table 5. It may be seen that in every instance the moisture content increased on freezing and decreased on thawing. Reference to Figure 18 shows that the upward movement of soil water was accompanied by a shrinkage of the underlying soil as water was drawn to the freezing zone.

Taber /1929-2 observed marked movements of water and accumulation in the freezing zone, and the accompanying decrease in soil moisture in the unfrozen soil (see Heaving in Open Vs. Closed Systems). His many photographs of ice accumulation in laboratory frozen specimens are worthy of study.

Beskow /1935-1 reported that the average increase in water content is about 20-30 percent by weight or 30-50 percent by volume. Where the thicknesses of ice layers are large (1 to 2 in.) they constitute a water excess of 100 to sometimes 300 percent of the original volume of the soil. He found that water transported upwards during freezing is divided unequally in the soil volume causing different density and thickness of ice layers. Usually where the rate of penetration of frost is small, ice layers are thick. He made it clear that the important consideration concerning magnitude of water increase for highways is not the total amount of water increase, but the relative thicknesses of the layers in which this increase is separated and the type of uneven distribution within the frozen soil. The reason for the importance of the distribution of the water is that it affects materially the bearing capacity of the soil on thawing, the reduction being related to the nature of the soil.

TABLE 5

Maximum Slab Movements, Soil Types, and Moisture Change Due to Frost Heave
(After Illinois /1924-3)

Indicator No.	County	Soil Type	Moisture Change Increase from %	Decrease to %	Decrease to %	Maximum Slab Movement (in.)	Approx. Depth of Frost (ft.)
1	Kane	Topsoil-yellow silt loam	22	40	20-22	2.09	1½
2	Kane	Topsoil-gray silt loam	20	35	20-22	0.78	2
3	Boone	Topsoil-black sandy loam	20	47	20-22	0.87	2
4	Whiteside	Topsoil-black mixed loam -subsoil drab clay	20	45	22-24	1.43	1½
5	Whiteside	Topsoil-yellow sandy loam	15	37	20 & decr.	0.49	1
6	Whiteside	Topsoil-brown sandy loam	21	44	30 & decr.	0.42	3/4
7	Sangamon	Topsoil-brown silt loam -subsoil yellow clay	no moisture data			0.63	3/4
8	Sangamon	6 in. clay subsoil, 8 in. black clay subsurface	no moisture data			0.35	3/4
9	Sangamon	Topsoil-gray silt loam	33	43	24 & decr.	1.92	1

Schaible /1941-1 observed moisture contents above and below the frost line as follows:

Type of Material	Moisture Content Below Frost Line %	Above Frost Line %
Weathered Clay - Slate	38	83
Weathered Clay - Slate	18	220 - 290
Loess - Loam	28.7	145

Shannon /1944-1 in his frost studies at Bangor, Maine reported that the water content in a silty clay subgrade increased from a normal value of about 25 to about 35 percent at the frost line below which it dropped immediately to about 24 percent. A typical example of increase in water content in a lean clay is shown in Figure 20.

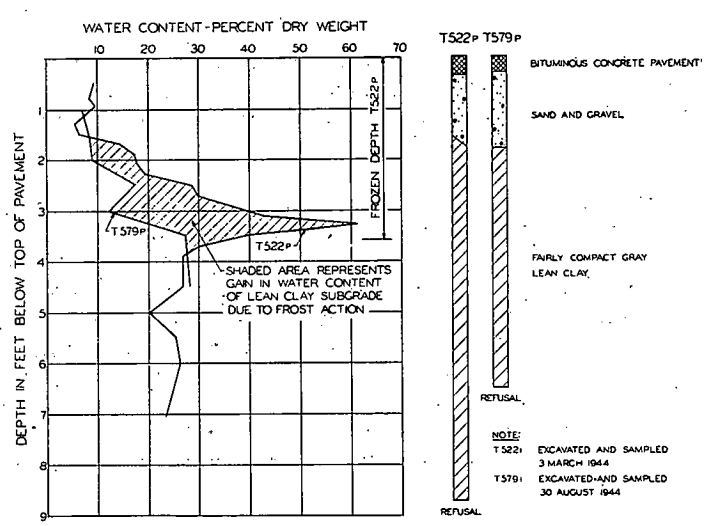


Figure 20. Typical Water Content Vs. Depth Profile
(After Shannon /1944-1)

Motl /1950-2 determined subgrade moisture contents when making plate bearing tests in Minnesota's study of "Load Carrying Capacity of Roads as Affected by Frost Action". He observed only minor changes in soil moisture content but did note a maximum at the time of frost melting. His results are indicated in Figure 21. Motl felt his findings were yet inconclusive but ventured that "...it does not appear that the small variations from the fall values are sufficient to account for all the loss of soil stability (an exception however, should be recognized in those cases where the moisture content in clay loam soils may be close to the maximum). It is suspected that frost action attacks the stability of a soil mass by altering its structure without necessarily changing the moisture content".

The Corps of Engineers extensive studies of airfields /1947-2, /1949-23 showed moisture gains in one or more subgrade soils in 14 of 15 airfields studied. There were instances when subgrade moisture contents were lower during the frost melting period than during the normal period but those occasions were in minority. Generally the moisture gain ranged from 1 to 4 percent (based on dry weight).

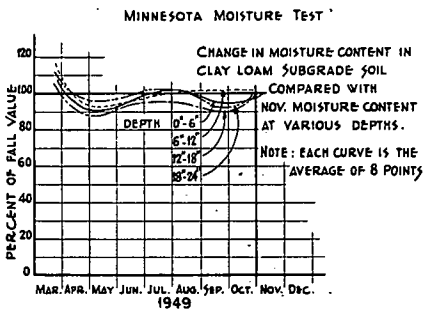


Figure 21.
Minnesota Moisture Test
(After Motl /1950-2)

Lund /1951-12 reported observations of soil moisture contents associated with spring road failures in Nebraska. He made direct comparison between "failed" and "good" condition of the pavement and obtained moisture contents at different depths in base and subgrade. Some of Lund's results are given in Table 6.

Several authors have indicated a relationship between measured heave and cumulative total thickness of ice lenses throughout the frozen depth. Shannon /1944-1 showed the comparisons given in Table 7.

It may be seen that in 3 of the locations for which total thickness of ice and heave are given, the thickness of ice exceeds the heave, indicating the possibility that shrinkage may have occurred in the soil from which the water was drawn.

TABLE 6

Relationship Between Percent Moisture Content and Road Condition
(After Lund)

DEPTH SAMPLED	Project A[a]		Project B[b]	
	Percentage Moisture		Percentage Moisture	
	In Failed Areas	In Good Areas	In Failed Areas	In Good Areas
Base Course	4.6	3.8	5.1	4.3
1 in. Below	18.5	17.2	19.0	17.0
6 in. Below	18.8	15.9	19.8	18.0
36 in. Below	19.7	17.3	20.4	17.5

a 10 borings in failed areas, 7 in good sections
b 27 borings in failed areas, 17 in good sections

TABLE 7

Comparisons Between Measured Total Ice Lens Thickness in Subgrade And Measured Heave of Pavement

Within or Adjoining Test Area	Cumulative Total Ice Lens Thickness At Test Pit	Total Heave of Pavement at Test Pit
	in.	in.
II	0.84	0
II	0.87	0.24
III	2.95	3.24
I	3.26	4.80
III	6.10	4.20

BEARING VALUE OF ICE AND FROZEN SOIL

Kersten and Cox /1951-49 conducted a series of penetration type bearing tests on soils in the unfrozen state. The test consisted of compacting soil in two 1-in. layers in a metal cylindrical mold having inside dimensions of 2 in. diameter and 4 7/16 inches high. The bearing value was obtained by loading a 3/4-in. diameter steel rod and recording loads at 0.1-in. penetrations.

The study was made on four soils. Grain size and plasticity data, and compaction test data for the four soils are shown in Table 8.

TABLE 8

Mechanical Analysis
(Particle Size-Millimeters Diameter)

Soil No.	Textural Class: U. S. Bur. of Chem. & Soils	Textural Class: Corps of Engrs.	Gravel: Over 2.00 %	Sand: 0.05 to 2.00 %	Silt: 0.05 to 0.005 %	Clay: Under 0.005 %	Liquid Limit	Plasticity Index	Modified: Opt. Moist. %	Modified: Max. Den. pcf.
P-4604	Med. Sand	SW	0.0	100.0	0.0	0.0		N.P.	12.2	119.0
P-4713	Sandy Loam	CL	0.4	53.6	27.5	18.5	24.6	9.3	9.0	127.5
P-4602	Silt Loam	ML	0.0	7.6	80.9	11.5	34.0	N.P.	15.5	110.0
U-4701	Clay	CH	0.0	9.2	37.5	53.3	77.0	53.5	19.8	107.1

Tests were conducted at a number of different moisture contents and densities and for temperatures above and below freezing for each soil.

The results showed that the density (dry wt. per cu. ft.) had a marked effect on the bearing value obtained in the tests. This is well illustrated by the values shown in Figure 22. which gives bearing value for the sand. The high density soil (curve 5) had a greater bearing

value than the low density soil (curve 2). The results also showed the effect of variation in moisture content when the density was held nearly constant. The densities of the soil represented by curves 4, 1, and 5 are within a range of 3 lb. per cu. ft. and have moisture contents of 9.00, 11.78 and 14.55 percent. In the frozen range, the bearing value shows an increase with increase in moisture content. A similar trend was found for the silt loam soil. The sandy loam and clay soils did not show the marked increases. It may be seen from Figure 22 that the freezing point for the sand is between 31 and 32 deg. F. and for the clay the average is about 29 deg. F.

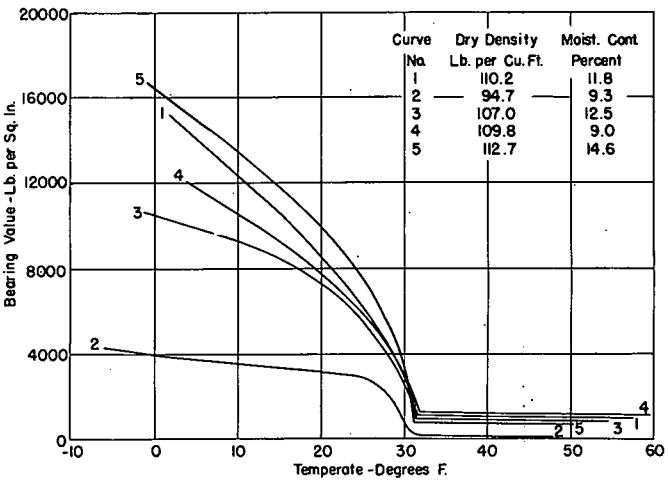


Figure 22. Summary of Bearing Value-Temperature Curves-Sand. (After Kersten)

Kersten and Cox /1951-49 also found that the bearing value of the frozen soil is dependent on the texture of the soil. That is illustrated in Figure 23 in which the bearing value-temperature curves for each soil at the moisture content density condition which most nearly approximates the optimum moisture content and maximum density of the modified AASHO compaction test. Bearing value curves for ice at different temperatures are also included. It may be seen that the order of soils from lowest to highest in bearing value is clay, silt loam, sandy loam and sand. That is the order of bearing value normally expected in those soils in the unfrozen condition.

Although the test method was an arbitrary one it did indicate the marked increase in bearing value with decrease in temperature for any initial condition of moisture content and density. It indicates that highways may have high load bearing capacities when the soil is frozen to an adequate depth.

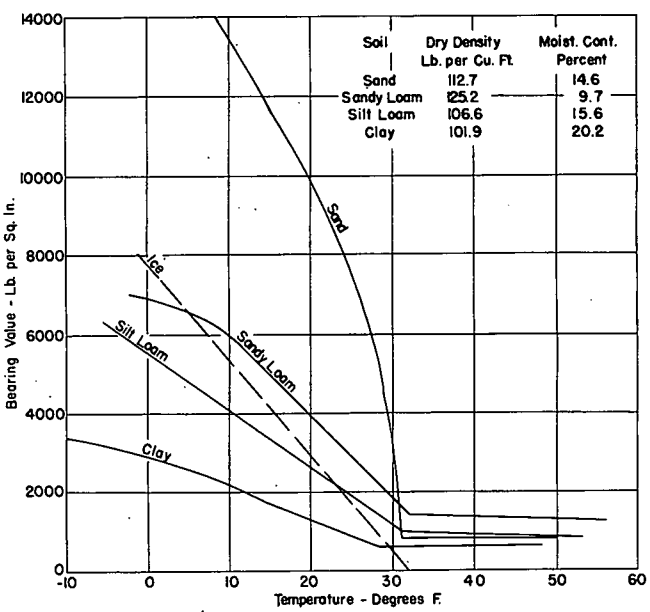


Figure 23. Bearing Value-Temperature Curves-Soil Approximately at Modified Optimum Moisture Content and Maximum Density. (After Kersten)

MAGNITUDE OF REDUCTION OF LOAD BEARING CAPACITY

Illinois /1924-4 cast a number of concrete slabs 3 ft. in diameter adjacent to the edge of the Bates Experimental Test Road for purpose of measuring slab reflections under repeated loadings. Several series of repeated loading tests were made, each with 10 applications of load, to determine change in supporting value with seasonal change in soil moisture content. Older /1924-4 reports the results as follows:

Figure 24 shows the results for Slab No. 3 of a series of tests which included ten applications of a load of 6 psi. repeated 6 times with intervening rest periods of 24 hours each. The loads were applied in each case at 15 minute intervals. The moisture content of the subgrade (26.8%) may be considered normal for this soil during the early fall months. The principal points of interest are: The permanent depression due to the first load applied after each period of rest; the recovery of elevation during rest periods; the resultant depression at the end of the third day; and the decreased permanent depressions on succeeding days. Figure 25 shows the effect of another series of load repetitions.

The excessive permanent depression recorded on February 20, 1922 may be accounted for partly by the increased moisture content, but probably was due chiefly to freezing and thawing of the soil. This may be inferred from the fact that excessive depressions were also recorded on March 8, at which time the moisture content was practically the same as on December 1. However, the soil had been frozen and thawed once or twice between February 20 and March 8. All other slabs used for similar observations behaved in the same manner but in varying degree.

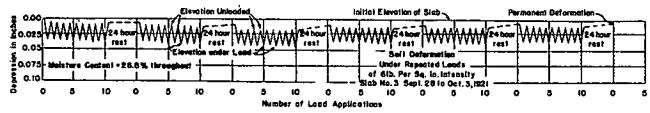


Figure 24. (After Older 1924-4)

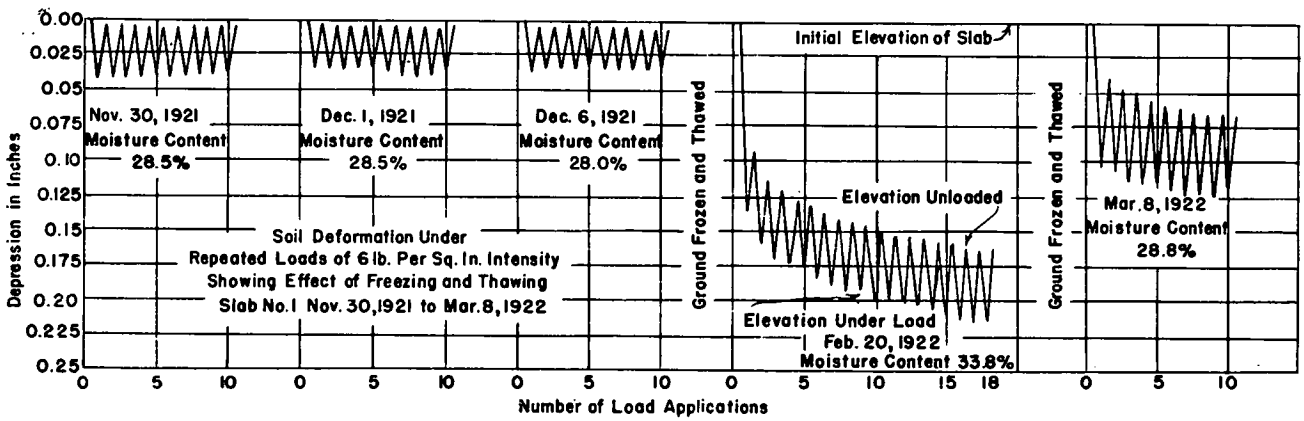


Figure 25. (After Older 1924-4)

Beskow /1935-1, /1947-12 found that the relative thickness of ice layers with respect to the thickness of the frozen soil and their depth below the surface are important factors in highways where reduction in load carrying capacity is the major consideration. If the water excess is high in the road it will cause softening and possibly frost boils on thawing, even if on soils which are otherwise stable. The deeper the water excess the less the reduction in load carrying capacity. Also, /1934-8 the reduction in load carrying capacity depends on the nature of the soil, a saturated silt needing only a small water increase to bring it to a liquid state.

Winterkorn and Fehrman /1944-11 made laboratory tests to determine the effect of freezing and thawing and wetting and drying on the density and California Bearing Ratio of five soils ranging from clay to sand. The results showed that bearing values: (1) after 4 days immersion were 3 to 11 percent lower than before immersion; (2) after freezing and thawing were 1 to 5 percent lower; (3) freezing and thawing plus immersion had a greater effect on bearing ratio; and (4) after wetting and drying and freezing and thawing the bearing ratio was pactically zero. The authors concluded that combined freezing and wetting have a detrimental effect on the "bearing power" of all soils within the textural ranges investigated.

The Missouri Soils Manual /1948-13 states that "Laboratory tests on granular soils have indicated a loss in bearing power up to 80 percent when the voids are increased only two percent above the voids present at maximum density". This is cited here to show how a slight reduction in density may affect bearing capacity.

Flexible Type Pavements

Shannon /1944-1 reported the results of studies by the Corps of Engineers at Dow Field, Bangor, Maine to determine reduction in bearing capacity of pavements due to frost action. The prevailing soil types at Dow Field were: (1) silty till containing about 50 to 80 percent sand and gravel, LL = 19 and P.I. = 2; and (2) a lean clay with 0 to 40 percent sand, LL = 29 to 33 and P.I. = 11 to 13. Frost depth was a maximum of about 4 ft. Plate bearing tests were made on the flexible type pavement in areas and adjacent to areas tested by traffic to measure reduction in supporting capacity of the pavement from the summer period to the frost melting period. Two testing procedures were used. One was a static loading which consisted of applying through a 30-in. diameter bearing plate a 5-psi. seating load for 10 min., unloading, zeroing the extensometers, then applying loads in five increments and observing plate movement until nearly complete deformation was reached. The total load applied was 60,000 lb. Seven tests were made in the frost melting period and seven in the summer. A summary of the results are shown in Table 9.

TABLE 9

Summary of Static Loading Plate Bearing Tests on Flexible Pavements, 30-in. Diameter-Plate

Test No.(a)	Located Adjoining or Within Test Area	Date of Test, 1944	Bituminous Concrete Pavement Thickness	Combined Thickness, Pavement and Base	Gross Load on Plate Which Produced Deformation in Inches of			Deflection Produced By Gross Load in Pounds of		
					0.05	0.10	0.20	10,000	20,000	40,000
			in.	in.	lb.	lb.	lb.	in.	in.	in.
20	I	12 April	4.6	21	2,000	3,000	6,000	0.40	0.80	1.41
42	I	28 August	3.7	22	13,000	25,000	45,000	0.03	0.08	0.17
21	I	13 April	4.1	21	8,000	15,000	25,000	0.06	0.15	0.34
39	I	26 August	4.1	21	19,000	30,000	48,000	0.025	0.06	0.15
22	II	15 April	3.4	47	17,000	26,000	50,000	0.04	0.08	0.15
34	II	23 August	3.4	47	20,000	39,000		0.025	0.05	0.105
23	II	17 April	3.4	39	15,000	36,000	65,000b	0.035	0.06	0.12
54	II	3 October	3.4	39	22,500	44,000	60,000	0.015	0.045	0.09
24	II	17 April	3.6	33	12,000	23,000	42,000	0.04	0.085	0.19
44	II	23 September	3.6	33	20,000	40,000	67,000b	0.025	0.05	0.10
26	I	20 April	3.4	22	8,000	16,000	40,000	0.06	0.12	0.20
35	I	24 August	3.4	22	25,000	48,000		0.015	0.04	0.08
32	III	1 May	4.0	26	14,000	27,000	48,000	0.03	0.07	0.16
37	III	25 August	4.0	26	8,000	17,000	39,000	0.06	0.115	0.205

a Tests are arranged in pairs with approximately 5 to 20 ft. distance between companion tests.
b Value extrapolated.

Shannon found that tests performed at adjoining locations during the two different periods showed nearly similar load deflection characteristics where the thickness of the flexible pavement and base was equal to or exceeded frost penetration depth. As the thickness of pavement and base were less than the depth of freezing, a given load produced a much greater deflection during the frost melting period. That is evident in Table 9. Repeated loadings showed that the increment deformation was greater during the frost melting period than during summer.

Traffic tests made at Truax (Madison, Wisconsin), Pierre (South Dakota) and Dow Field by the Corps of Engineers in 1944-1945 /1947-2 and the plate bearing tests made at Presque Isle (Maine), Dow, Pierre and Watertown, (S. D.) airfields and the C.B.R. tests made at all 15 airfields showed a reduction in the ability of pavements to support loads during the frost melting period. Although the traffic tests indicated a reduction, they were inadequate in number to provide quantitative data.

The plate bearing tests made at 19 locations showed a range in maximum ratio of normal to frost melting period load for 0.1-in. deflection (30 in. circular test plates) ranging from about 1.45 to about 2.7. Expressed as percentages, this means that the bearing value during the frost melting period ranged from 37 to 69 percent of the bearing capacity during the normal period with an average value of 55 percent. The data showed no distinct trend of increasing reduction in bearing value with increased depth of frost penetration.

The report states that "The best measure of the reduction is by a comparison of C.B.R. values during the frost melting period with those during the normal period as obtained from both traffic tests and in place C.B.R. Tests."

The C.B.R. values form the basis for the design curves shown later in this review under "Design Methods for Preventing Detrimental Frost Action". Motl /1947-13 reported the first of a series of tests by the Minnesota Department of Highways. His tests showed that the reduction in bearing was about 50 percent. The 1946-1947 tests were at 85 test points on 11 projects totalling about 100 miles in length. The work was continued during 1948, and 1949, and was extended to establish a working relationship between data obtained with the 12-in. diameter plate, the North Dakota Cone and the C.B.R. equipment.

Motl's Committee made a supplementary report /1950-2 of work done during 1948 and 1949 in measurement of reduction of bearing capacity during the spring season. The report covers studies in 8 states including Indiana, Iowa, Michigan, New Hampshire, New York, North Dakota, Ohio and Minnesota. Measurements in Iowa were made with the Iowa Subgrade Resistance Meter. Michigan used the Housel penetrometer, the North Dakota cone test and the Housel shear test. North Dakota used its cone bearing test; and others used plate bearing tests.

It is of interest to note that the pattern of reduction in bearing capacity is generally similar from state to state. This may be seen in Figure 26 showing results of plate bearing tests from New Hampshire, and Figure 27 showing results of plate bearing tests from New York. The results of the North Dakota cone tests in North Dakota show similar reduction in values as may be seen in Figure 28.

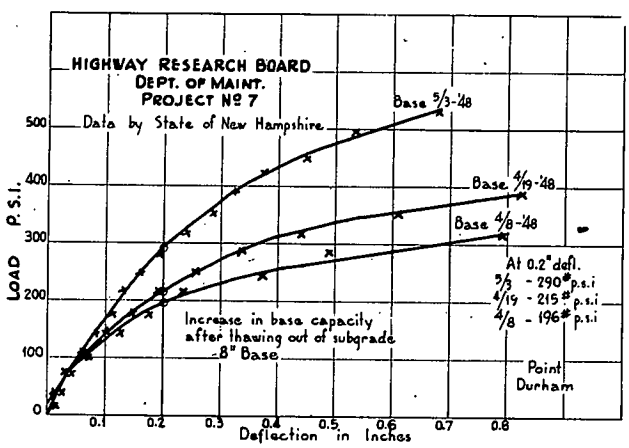


Figure 26. Increase in Base Capacity After Thawing Out of Subgrade - 8 in. Base-State of New Hampshire (After Motl)

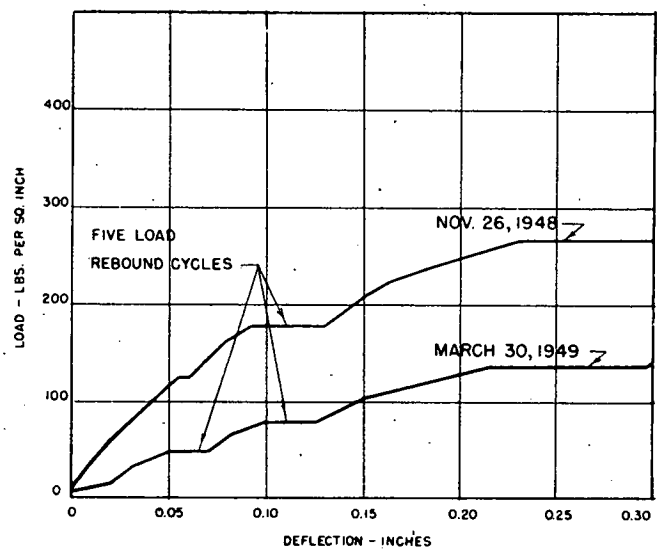


Figure 27. Load-Deflection Relationship of Curves for Fall and Spring, using 6 in. Diameter Plate on Subgrade - State of New York. (After Motl /1950-2)

Minnesota reported results covering a three year period /1950-2. Figure 29 shows that while the pattern is similar there is a marked difference in the percentage reduction in bearing capacity especially between the 1947 and the 1949 spring season values. The Minnesota studies included typical profiles and cross sections of the roadway during test periods. Heaves associated with reduction in bearing value were all relatively small ranging from about 0.02 to a maximum of about 0.09 ft. Observations were also made of moisture content changes. Those are reported in a preceding paragraph. A more recent report by Lawrence /1951-37 summarized Minnesota's work (see /1948-11 and /1949-12 for details of results and test procedures). He stated that "The average loss in bearing value for the 126 test points was 42 percent of the fall bearing value". The percentage reduction, within experimental error, was substantially the same for all soil types and flexible pavement designs. The average values for the various soil groups are given in the following:

B.P.R. Classification	Number of Test Points	Average Spring Bearing Value at 0.2-in. Defl. psi.	Average Loss %
A-2	16	205	40
A-4	9	222	46
A-4-7	14	145	45
A-7	71	96	42
A-6	13	85	40
A-5-7	3	62	41

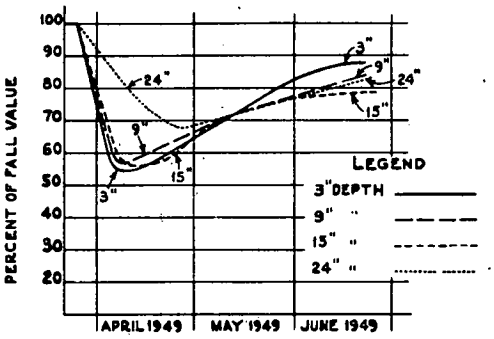


Figure 28. Loss of Strength at Depths of 3, 9, 15 and 24 in. below surface of Subgrade- Average of Ten Test Points - State of North Dakota (After Motl /1950-2)

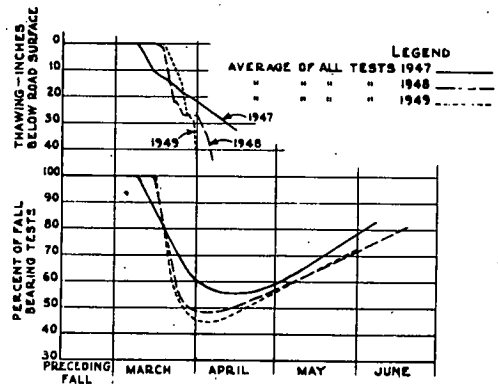


Figure 29. State of Minnesota - Dept. of Highways - Comparison of Results of 1947-1948 and 1949 Tests. (After Motl /1950-2)

The 1944-1945 studies of the Corps of Engineers /1947-2 were continued and "Addendum No. 1" was published /1949-23. A third report /1951-39 summarizes the results of in-place C.B.R. tests conducted on top of the base materials and (or) on top of the subgrade at a total of nine flexible paved test areas where tests were performed during the normal period in the fall of 1944 and in the frost melting period in the spring of 1945. Data from those tests summarized in Table 10 generally showed a reduction in bearing ratio in both frost susceptible base and subgrade materials.

Plate bearing tests with the plate placed directly on the bituminous concrete pavement were made at six airfields. Two types of tests were made. Static plate bearing tests consisted of loading a 30-in. diameter plate in five or six equal increments to a load in excess of the proportional limit or to the maximum permitted by the test equipment. Repeating load tests were made with similar equipment except a 24-in. diameter plate was used.

Repeating load tests were made with a 24-in. plate (except at Dow Field where a 19-in. plate was used in the 1943-1944 normal period) using a seating load of 3,500 lb. for 5 min. A load of 20,000 lb. was then applied rapidly and maintained for 10 min. and deformation readings were made after periods of 1/4, 1, 2 1/4, 6 1/4 and 10 min. The load was released and deformation readings read after a 5 min. period. Ten repetitions of the loading procedure were made.

The results of the static load tests (30-in. plate) are shown in Table 11. Column 11 represents average values for tests performed from September 1 to the start of the freezing period. Column 12 shows average loads at 0.1-in. deflection for the 2-week frost melting period when the strength was at a minimum. Column 13 shows the ratios between frost melting period and normal period static load test values.

The wheel load evaluations for both normal and frost melting periods and their ratios were determined for each test area included on Table 11. Normal period evaluations were based on in-place C.B.R. test results in the flexible pavement design curves for single wheel loads with 100 psi. tire pressure and "...frost melting period evaluations on frost condition design curves which are currently in use."

TABLE 10

In-Place Values of CBR for Nine Airfields
(After Corps of Engineers)

Site	Test Area	Material	Average CBR Normal Period	Average CBR Frost Melting Period
Otis Field, Camp Edwards, Mass.	A	ML Subgrade (GP & SP)	44	18
Truax Field Madison, Wisconsin	A	GF Subbase CL Subgrade	64 4	33 2
Truax Field, Madison, Wisconsin	B	GF Subbase CL Subgrade	41 5	31 3
Watertown Airfield, Watertown S. Dakota	A	GF-SF Base OL-CL Subgrade	37 12	27 8
Pierre Airfield, Pierre, S. Dakota	B	GF Base CL Subgrade	30 12	30 16
Sioux Falls, Sioux Falls, S. Dakota	A	GC Base CL Subbase CL-CH Subgrade	37 16 10	14 6 5
Fargo Municipal Airfield, N. Dakota	A	CL-SF Subbase OH-CH Subgrade	15 7	8 5
Casper Air Base, Casper, Wyoming	B	GW Base SF-CL Subgrade	58 27	28 21
Bismarck Airfield, Bismarck, N. Dakota	A	GF Base SF-ML Subgrade	24 24	21 18

The average value of ratio of frost melting to normal period static load for 0.1-in. plate deflection was 0.50. That value may be compared with the value of 0.29 which is the ratio of frost conditions to normal period wheel load evaluations for the same pavements. These ratios are shown in Figure 30 which also indicates the duration and magnitude of reduction in load carrying capacity of the pavement as determined by averaging static load test data from all test areas.

The Corps of Engineers held that the static load tests as performed by them do not give a correct measure of the relative traffic supporting capacity during the frost melting period because "...the gradually applied load used in static load tests allows escape of water, consolidation and build-up of resistance in the subgrade soils which influences to a great extent the frost melting period test results; and, the static load tests do not reflect the weakening due to subgrade remolding under repetitive loading such as the pavement is subjected to under traffic usage".

An example of the effect of repetitions of load is shown in the graph in Figure 31 which illustrates the marked effect of 10 pepetitions of load for each of the three test periods represented.

TABLE 11

SUMMARY OF STATIC LOAD TESTS ON SURFACE OF FLEXIBLE PAVEMENTS

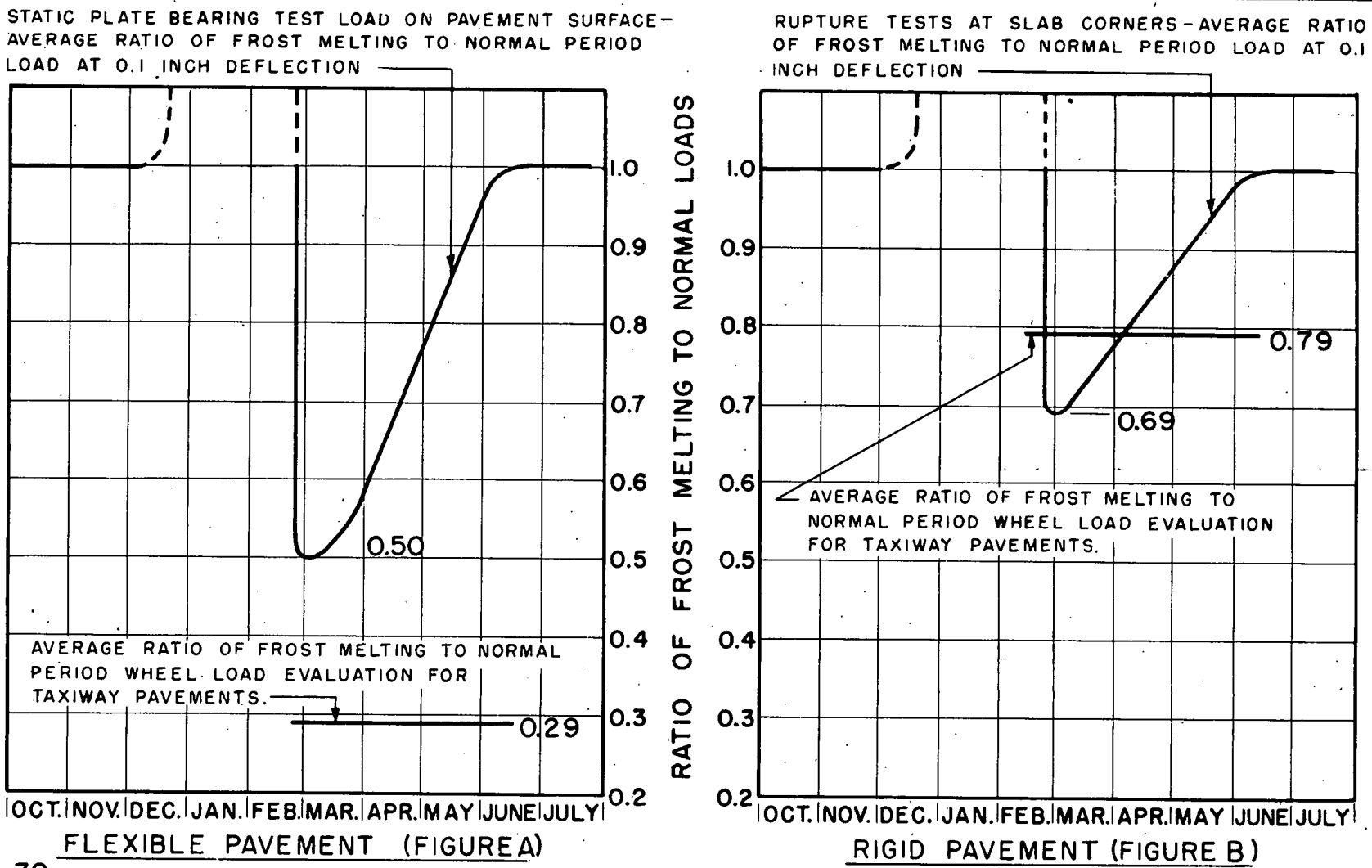
Site	Test Area	Pavement Thickness	Base			Subgrade			Frost Penetration	Static Load Tests Load at 0.1-in. Deflection			Design Wheel Load		
			Classification	Thickness	Finer than 0.02 mm.	Classification	Finer than 0.02 mm.	Normal CBR		Normal Period	Frost Melting Period	Ratio Loads FM/N /d	Normal Period/a	Frost Melting Period/b	Ratio Design FM/N
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
		in.		in.	%		%		ft.	lb.	lb.		lb.	lb.	
Dow	B	4.0	GW	27	3-6	CL	40-97	8	4.3	34,000(4)/c	17,000(3)/c	.50	80,000	27,000	.34
		3.5	"	40	"	"	"	"	4.3	41,500(2)	26,000(1)	.62	150,000	48,000	.32
	I&III	4.0	"	16	"	"	"	"	4.0	34,000(3)	15,500(2)	.42	33,000	11,000	.33
	C	4.0	"	43	"	"	"	"	4.7	44,000(4)	26,000(3)	.59	150,000	57,000	.38
		3.5	"	32	"	"	"	"	4.7	34,000(1)	17,000(1)	.50	106,000	33,000	.31
Presque Isle	C	4.5	Cr. Rock GW	3 24	- 0-6	GC	10-40	10	6.2	60,000(1)	32,000(4)	.53	108,000	27,000	.25
	D	4.0	Cr. Rock GW	3 24	- 0-6	GC	"	"	5.8	49,000(3)	24,000(9)	.49	108,000	28,000	.26
Truax	A	2.5	Cr. Rock GF	8 16	- 10-20	CI SF	60-80 7	5	3.9	36,000(2)	14,000(2)	.39	32,000	7,000	.22
	B	2.5	Cr. Rock GF	19 23	- 10-20	CL SF	60-80 7	4	4.9	51,500(5)	24,000(7)	.47	73,000	26,000	.36
Pierre	B	1.5	GF	12	6-14	CL	30-58	13	3.7	42,000(1)	17,500(7)	.37	27,000	5,500	.20
		6.0	GF	9	"	"	"	"	3.7	69,500(1)	38,000(8)	.55	35,000	6,500	.19
Sioux Falls	A	2.0	GC	10	6-12	CL Subbase	35-48	10	3.7	23,500(1)	12,500(3)	.53 Avg.=.50	14,000	4,000	.29 Avg.=.29

/a Design wheel loads for normal period were determined from the flexible pavement design curves (single wheel load, 100 psi. tire pressure) dated 13 June 1950.

/b Design wheel loads for frost conditions were determined from the Frost Design Curves dated 15 August 1950. The GF sub-base material at Truax was frost susceptible and was considered to control frost condition design.

/c Figures in parentheses indicate number of tests used in computing the average loads at 0.1-in. deflection.

/d N - Normal Period
FM - Frost Melting Period



FLEXIBLE PAVEMENT (FIGURE A)

RIGID PAVEMENT (FIGURE B)

FIG. 30 REDUCTION IN PAVEMENT SUPPORTING CAPACITY

NOTES:-

Curves based on static load tests from nine flexible paved test areas and from rupture tests on six rigid paved test areas at six airfields (Reference Tables 2 and 3).

Single wheel load evaluations (100psi tire pressure) for normal period based on airfield pavement design curves dated 13 June 1950 and evaluations for frost melting period using frost design curves dated 15 August 1950. (After Corps of Engineers)

It was observed from the trend of static plate bearing test data for three airfields that the duration of the period required for the pavement to return from approximately 50 percent to 80 percent of normal strength was approximately proportional to the depth of the frozen subgrade at the three sites (Presque Isle, Truax and Dow Airfields).

Traffic tests were conducted on five flexible paved test areas at the three airfields. Traffic was applied on the basis of a specified number of daily coverages during and after the frost melting period to simulate the continuous use of a pavement by aircraft. The criterion for failure was a condition where about 20 percent of the area was map cracked or when the flexing of the pavement reached one inch.

The traffic test data from all flexible paved test areas are summarized in Figure 32 which shows the percentage ratio of the actual pavement and base thickness to the Corps of Engineers frost condition design thickness for the specific test wheel load, number of traffic coverages and the pavement behaviour. It may be seen from Figure 32 that no pavement failures occurred where the actual pavement was greater than about 90 percent of the design requirement. This indicated a margin of safety which was believed justified to allow for heavy traffic in emergencies and to prevent excessive maintenance costs.

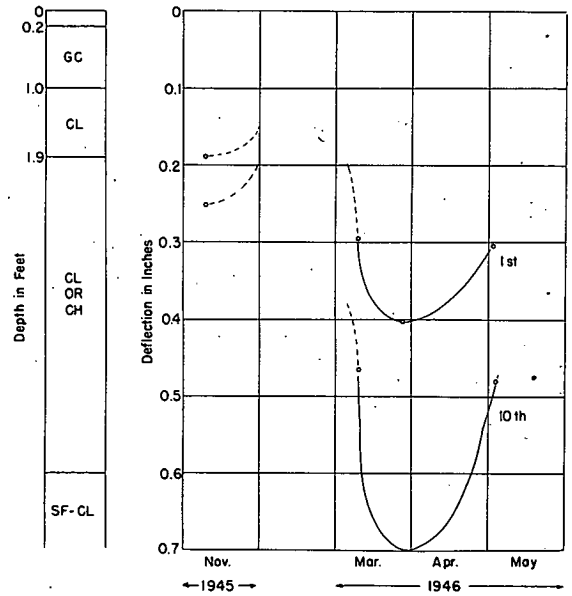


Figure 31. Results of Repeating Load Tests, Test Area A, Sioux Falls Airfield (Load 25,000 lb. - Plate 24" Diam.) (After Corps of Engineers)

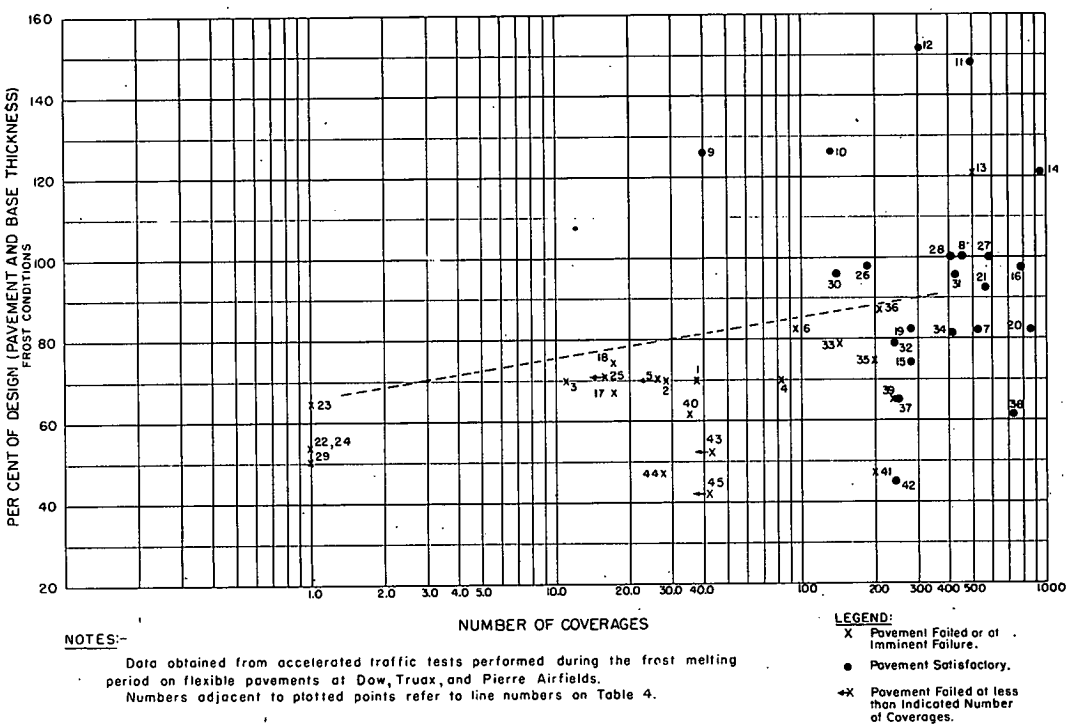


Figure 32
Percent of Design vs. Number of Coverages
Flexible Pavements
(After Corps of Engineers)

Rigid Type Pavements

The War Department /1947-2 traffic tests at Truax and Dow Airfields on Rigid Type Pavements showed a "definite reduction in pavement supporting capacity due to frost action". At Truax (Madison, Wisconsin) ice lenses occurred in the top 4 in. of the base and ice lenses adhered to the bottom of the pavement. Lenses also occurred in the subgrade at depths of 3.0 to 4.7 ft. The reduction in supporting value was reported to be "...due directly to the ice lens formation in the top four inches of the gravel base as the ice lens formation in the subgrade was at a depth which is considered too great to be effective under a 30,000 lb. wheel load." Pumping of water through the joints and cracks (during the frost melting period) carried out fines from the base. It was believed that the pumping weakened the base, causing a reduction in support where it occurred. It was also believed that a non-frost susceptible base /4 would not have caused pumping. Pumping occurred during the traffic tests at Pierre (South Dakota). The results indicate that pumping resulted from infiltration of surface water and not frost action.

The results of subgrade modulus tests obtained at Pierre, Presque Isle, and Dow Fields during the frost melting period are shown in Figure 33. Curve A represents the trend of the tests. The type of subgrade at all of these airfields falls into group 3. The results form the basis for the design curves which are shown later under the heading "Design Methods for Preventing Detrimental Frost Action". Comparison of Figure 33 and the design curves shown later indicates that there is not a close agreement between curve A and the curve designated "3". It is stated that the three design curves were purposely drawn to indicate conservation values for the subgrade modulus during the frost melting period.

The 1944-1945 investigation by the Corps of Engineers /1949-2 was continued through 1945-1947 /1949-23 by means of rupture tests and subgrade modulus tests conducted during normal and frost melting periods at seven airfields. In addition, a traffic test was made at Selfridge Field (Michigan). The results were analyzed and presented in summary form in a later report /1951-39

The rupture tests were made by loading a 24-in. diameter plate placed on the surface of the pavement at the corner of a slab at the intersection of a longitudinal construction joint and a transverse expansion joint. The results are summarized in Table 12. A comparison of loads causing 0.1-in. deflection during the normal and frost melting periods as given in columns 10 and 11 (Table 12) shows a consistent reduction in pavement strength during the frost melting period, the average ratio of frost melting to normal period load at 0.1-in. deflection for all test areas being 0.69.

Wheel load evaluations of the pavements at the locations of the rupture tests were made for both normal and frost melting periods using the latest Corps of Engineers design curves for single wheel loads with 100 psi. tire pressure. Those evaluations are shown in columns 15 and 17 of Table 12. The average ratio of the frost melting to normal period wheel load evaluations is 0.79 as shown on Table 12 as compared with the average ratio of weakening of 0.69 indicated by the pavement rupture tests. A comparison of the ratios of frost melting period to normal period rupture test loads and design wheel loads is presented in Figure 30. Although the ratios of weakening by both methods of approach are of the same order, the load at 0.1-in. deflection on the bearing plate was on the average four times greater than the design wheel load for the same period. Since the rupture tests were carried either to the maximum loading capacity of the equipment or to slab failure which was in all cases at deflection in excess of 0.1-in. the rupture loads in all instances were many times greater than the design wheel loads.

Consideration of the factors involved indicates that in these static loading tests on rigid pavements, as in the comparable tests on flexible pavements, many influences exist which tend to produce an incorrect ratio for the reduction in pavement supporting capacity, in relation to that determined by actual traffic testing. Nevertheless the agreement between the values 0.79 and 0.69 here obtained for rigid pavements is clearly better than that obtained for flexible pavements.

/4 The base material consisted of a sand-clay-gravel material (GF) with 10 to 20 percent retained in the No. 10 sieve, and with about 20 to 35 percent silt, about 8 to 11 percent clay. L.L. ranged from 19 to 30. P.I. ranged from 2 to 9.

TABLE 12

SUMMARY OF RUPTURE TESTS ON RIGID PAVEMENT SLAB CORNERS

Site	Test Area	Pavement Thickness	Base			Subgrade		Frost Pen.	Load at 0.1-in. Deflection			Flex. Strength of Conc.	Design Wheel Load				
			Classification	Thickness	Finer than 0.02 mm.	Classification	Finer than 0.02 mm.		Normal Period	Frost Melting Period	Ratio Loads FM/N /a		Normal Period		Melting Period		Ratio Design FM/N
													Sub. Mod.	Load /b	Sub. Mod.	Load /c	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
		in.		in.	%		%	ft.	lb.	lb.	-	psi.	psi.	lb.	psi.	lb.	
								1944 -	1945								
Dow	A	7.0	GW	13	3-6	CL	40-97	4.1	50,000(2) /d	28,000(2) /d	0.56	600	315	16,000	75	11,000	-
Truax	C	6.0	GF	40	10-20	CL	60-80	4.6	86,000(1)	55,000(5)	0.65	750	250	14,000	60	11,000	.79
								1945 -	1946								
Dow	F	7.0	GW	22.5	3-6	CL	40-97	4.5	40,000(1)	27,000(1)	0.67	600	315	16,000	120	12,000	.75
Presque Isle	A	8.0	GW	32	0-6	GC	10-40	5.8	60,000(1)	47,000(3)	0.78	690	340	27,000	170	21,000	.78
Truax	C	7.0	GF	39	10-20	CL	60-80	4.0	84,000(1)	48,000(1)	0.57	750	250	20,000	60	14,000	.70
	C	6.0	GF	53	10-20	CL	60-80	4.0	71,000(1)	43,000(2)	0.61	750	250	14,000	60	11,000	.79
Selfridge	A	11.0	GF	8	3	ML SF	34 20	2.9	85,000(1)	61,500(3)	0.72	735	165	50,000	55	39,000	.78
Pierre	A	7.0	GF	7	6-14	CL	30-58	4.0	44,000(2)	37,000(2)	0.84	700	120	15,000	55	13,000	.87
Sioux Falls	A	6.0	—	-	-	CH	55-83	3.5	35,000(2)	28,000(2)	0.80	675	48	9,000	30	8,000	.89
								1946 -	1947								
Dow	F	8.0	GW	19	3-6	CL	40-97	3.7	42,000(3)	28,500(2)	0.68	600	315	22,000	110	16,000	—
											Avg.=.69						Avg.=.79

/a N - Normal Period
FM - Frost Melting Period

/b Design wheel loads for normal period were determined from the Rigid Pavement Design Curves (single wheel load, 100 psi. tire pressure) dated 13 June 1950.

/c Design wheel loads for frost conditions were determined from the Rigid Pavement Design Curves (single wheel loads, 100 psi. tire pressure) dated 13 June 1950, using subgrade moduli obtained from Frost Design Curves dated 15 August 1950. The GF base material at Truax was frost susceptible and was considered to control the design.

/d Figures in parentheses indicate number of tests used in computing the average loads at 0.10 in. deflection.

Traffic tests were made on portland cement concrete pavements at Dow, Truax, Pierre and Selfridge airfields using wheel loads consistent with the evaluations of the specific test areas. The assumptions as to traffic frequency were the same as for flexible type pavements.

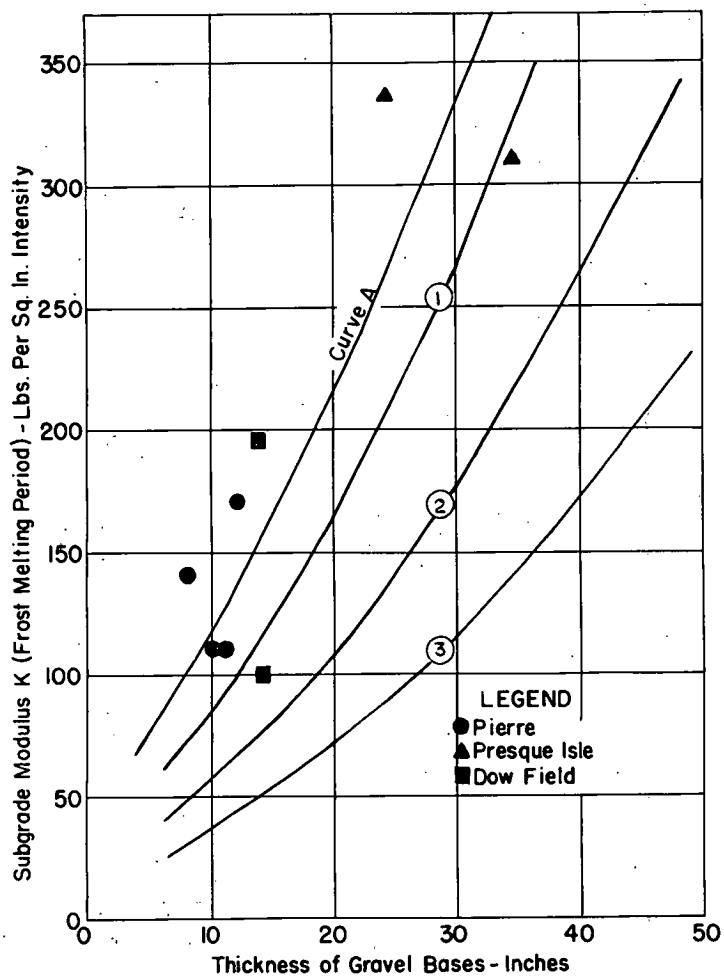


Figure 33. Summary of Foundation Modulus Tests Compared with Proposed Design Curves. Curves (1) (2) (3) are design curves of "Recommended Revisions to Engineering Manual". All subgrade soils are frost susceptible and fall in group (3). Subgrade Modulus Determined During Frost Melting Period.(After C. of Engrs.)

The results of the traffic tests are summarized in Figure 34 which shows the relationship between percentage ratio of the actual slab thickness to the frost condition design thickness for the traffic test wheel load, the number of traffic coverages and pavement behaviour. Design thicknesses were determined using values of subgrade moduli from the Corps of Engineers frost condition design curves dated August 15, 1950 and rigid pavement design curves dated June 13, 1950.

Although the data are limited, failure occurred in all test lanes except one (Pierre) when the existing pavement was less than 85 percent of that required by frost condition design criteria for traffic test wheel loads. It may be seen from Figure 34 that satisfactory pavement performance resulted when the slab thickness along the test lane was in excess of 90 percent of that required by the Corps of Engineers frost condition design criteria.

The Corps of Engineers conclude from their comparative studies of load supporting capacity on flexible and rigid type pavements that "The ratio of the safe wheel load during the period of maximum weakening due to frost action, to the safe wheel load during the normal period, is approximately 0.3 for flexible pavements and approximately 0.8 for rigid pavements.

Rogers and Nikola /1951-38 made measurements of the bearing capacity of 30 New Jersey soils under 4-by4-ft. concrete slabs. No effort was made to determine the supporting capacity of the slab on the 30 soils. The determinations were made by plungers through the slab and resting on the subgrade. The results of that investigation are reviewed in some detail under "Relationship Between Grain Size Distribution Characteristics and Frost Action".

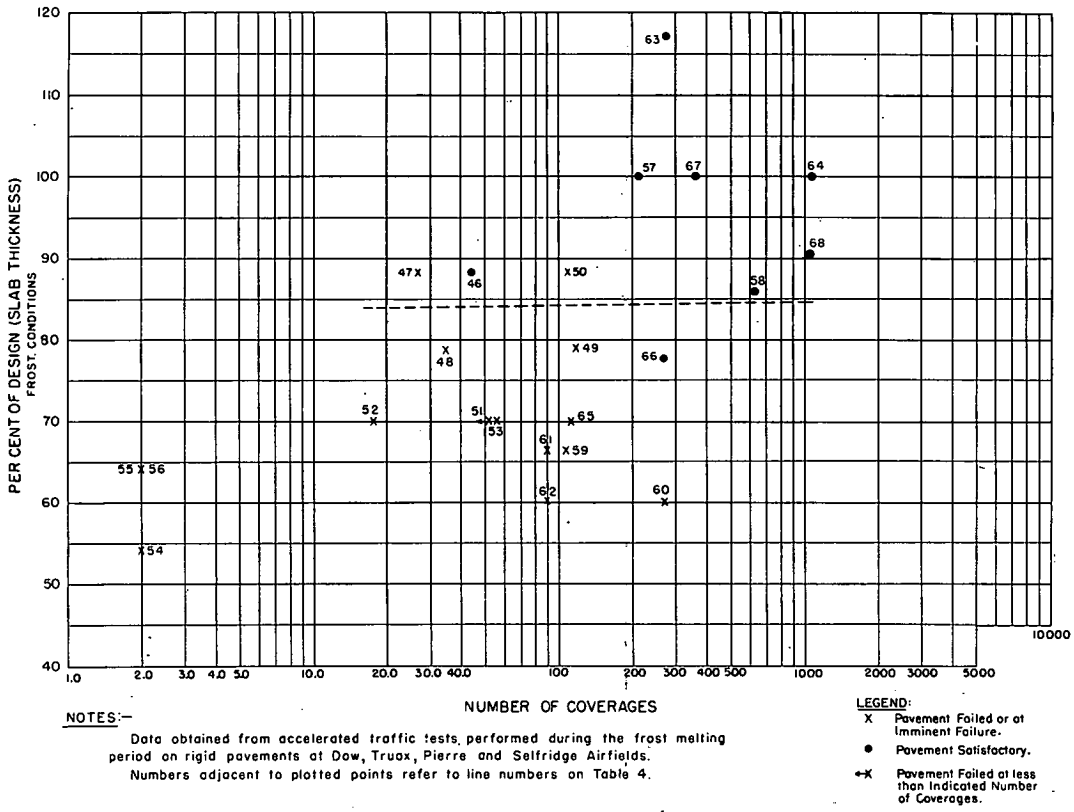


Figure 34
Percent of Design vs. Number of Coverages -
Rigid Pavements. (After Corps of Engineers)

FACTORS INFLUENCING THE MAGNITUDE, RATE AND NATURE OF FROST ACTION AND REDUCTION IN LOAD CARRYING CAPACITY

The nature of frost action and the consequent reduction in bearing capacity following thawing are influenced by many and diverse factors. Climate alone is of major importance. So also is the soil. The same may be said of the water conditions associated with the soil. There are other factors which if considered individually influence frost action in a lesser degree. Even those, if their effects are additive, are worthy of consideration by highway engineers.

There exists a close interrelationship between the various factors and their net effects. Frost action can occur only when the soil type, the associated water conditions and the temperatures are favorable for a sufficiently long period of time. This makes it difficult to break down and separate the factors and evaluate their net effects individually. However, an attempt is made to explain, insofar as is practical, the influence which each of the various elements which are responsible for frost action has on its magnitude on the rate of heaving, and on the reduction in load carrying capacity.

Influence of Climate

The qualitative influence of the various elements of climate in producing soil freezing is well known. Some efforts have been made to codify the available, climatic data and correlate the data with depth of freezing, height of heaving or severity of spring breakup (reduction in load carrying capacity of pavements).

The Corps of Engineers Investigation /1947-2 of frost action at 15 airfields showed that the degree of saturation beneath paved areas varied generally with climatic condition; the lower degree of saturation was found in the areas of low annual rainfall. They also found a reasonably close correlation between cumulative degree days of below freezing temperature and depth of frost penetration. This is discussed in detail later in the review.

Shelburne's /1951-46 questionnaire brought out several generalized statements indicating the effect of climate on degree of damage due to frost action in bases, sub-bases and subgrades. He asked the question "Is there correlation between degree of distress attributable to frost action and climatic conditions?" The following statements are indicative of the nature of the replies:

High precipitation plus low temperature causes extensive damage. Occurs about every 10 years.

Distress varies with amount of moisture available and cycles of freezing and thawing.

Repeated freezing and thawing causes severe damage.

Short periods of extreme cold alternating with above freezing temperatures cause most of damage.

Degree of distress is in proportion to depth of frost penetration and degree of snow removal.

A wet fall followed by severe winter and quick spring thaw causes most damage. A great number of cycles of freeze and thaw also plays an important part in causing frost damage.

Mid-winter thaws apt to increase frost action. Rapid spring thawing increases spring break-up.

Shelburne and Maner /1949-8 compared data on average mean values of temperatures and precipitation with data on spring breakup in Virginia. For a 30-yr. period, the winter of 1917-18 was the coldest with a mean temperature of 33 deg. F. The next most severe were the winters of 1935-36 and 1939-40 with a mean temperature of 37 deg. F. The winter of 1947-48 was fourth most severe with a temperature of 37.2 deg. F. In total precipitation the winter of 1936-37 was wettest with 15.5 in.; 1935-36 was second with 15 in. Those values compare with a 35-yr. normal for the 3-mo. period of 9.3 in.

The most severe spring breakup in recent years occurred in 1935-36. During which time an unusually low mean temperature (second lowest in 30 years) was combined with abnormally high precipitation (second wettest winter). It may be pointed out that the winter of 1947-48 had the most favorable combination of precipitation and temperature for a severe breakup since 1935-36. The two phases of the climate factor namely temperature and precipitation, appear to go hand in hand towards producing severe damage to roads if traffic does its part. A cold winter alone does not necessarily provide subgrade conditions for severe spring damage.

Bleck /1949-11 brought out that generally, the effect of climate may be considered under the broad aspects of humid, semiarid, cold, warm etc., but that within those broad climatic zones there are differentials created by immediate weather conditions which affect the amount of moisture in the soils which in turn is further affected by immediate temperatures. Those factors have large influence on soil movements and soil bearing capacity, both of which influence pavement performance. The range in values of annual precipitation in Wisconsin is cited by Bleck.

During the past 12 years in Wisconsin there have been three years in which the spring breakup was severe. The climatic conditions during those periods were generally similar. The three periods were 1935-1936, 1940-1941, and 1942-1943. Snowfall began early and considerable snow accumulated early. The early part of the winters were very cold, yet by late winter the most of the snow disappeared with very little runoff; thus supplying moisture to shoulders and subgrades where it froze and caused severe damage under traffic on thawing. Bleck constructed precipitation charts for the Wisconsin localities to illustrate the nature of precipitation cycles for those areas and held that consideration of climatic cycles is worthy of note because "... meterological conditions control and influence the quantity of moisture in the soil and the subsequent movements."

Freeze damage to highway base courses occurred in several areas in Texas during the winters of 1946-47 and 1947-48. During that period the Texas Highway Department /1948-46 conducted a state wide survey to determine the nature and extent of the damage. No evidence of frost boils in subgrades under concrete pavements were reported. The damage was limited largely to reduction in bearing capacity of flexible type bases, except that possibly some freezing of the subgrade

may have occurred in the Panhandle Area. The results of the survey made it possible to suggest a southern limit for design to prevent freeze damage to bases.

Rate of Freezing - Taber /1929-2, /1930-9 and Beskow /1935-1 found that the formation of ice layers is favored by a very slow lowering of the freezing isotherm; therefore the most favorable depths for ice formation are near the surface and near the maximum depth of frost penetration, which Beskow said produced difficult thawing effects. Beskow found that rapid freezing in early winter left the critical upper layers relatively barren of ice and therefore more stable during thawing. In border-line materials (in which very little segregation occurs) Taber found that ice layers form only near the top and bottom of the frozen part.

The rate of cooling is affected by the amount of water converted to ice. It is affected in a small degree by the texture of the soil and in a large degree by the water content of the soil.

Beskow's work /1928-9, /1929-6, /1935-1 resulted in the conclusion that the rate of frost heaving is, for practical purposes, independent of the rate of freezing. He held that this was valid only for relatively permeable soils for which the effect of rate of freezing "...is under all conditions, small and quite insignificant". An example of a frost heave-time curve obtained in the laboratory is given in Figure 35 which shows heaving progressing at a constant rate regardless of large variations in below freezing temperatures. His conclusions were based on the assumption that the rate of cooling is great enough to freeze the water as fast as it is sucked up. If the water can be frozen exactly as fast as it is sucked up, the frost line remains stationary. If water can be sucked up faster than it can freeze, then heaving is proportional to the rate of cooling. If the rate of freezing is greater than the rate at which water can be sucked up, then the rate of heave is independent of the rate of freezing. In other words the measure of frost heave is approximately a measure of the amount of water that can be sucked up. If freezing occurs rapidly, the water sucked up is frozen into many thin layers, if it occurs slowly the layers are thick. Thus in uniform soils, the water content (which is a measure of the ice stratification of the frozen soil) registers the rate of freezing and thick ice indicates slow and thin ice fast freezing.

Beskow /1935-1 stressed the importance of the effect of the climatic factor as it affects the rate of freezing because although rate of freezing did not influence the total heave, it did influence the thickness of the ice lenses, and thus the distribution of the water excess and the consequent reduction in bearing capacity.

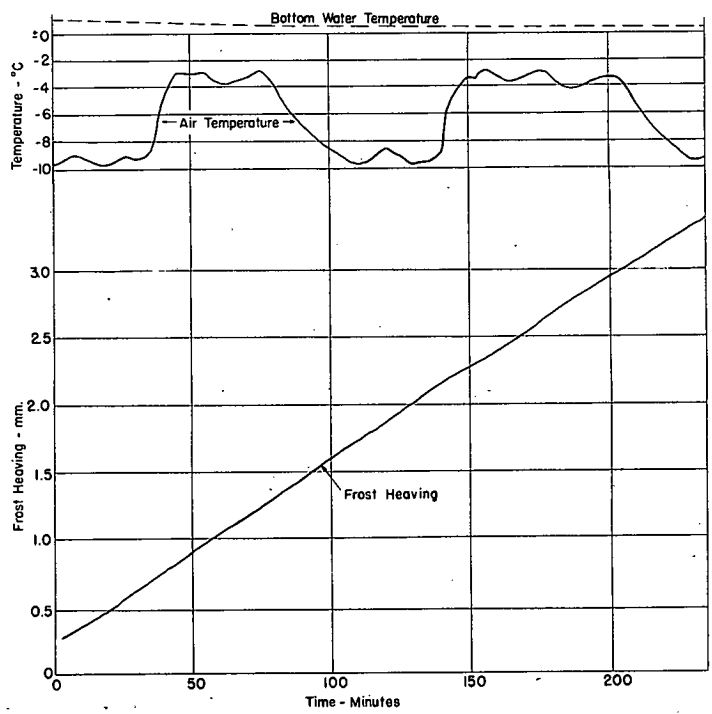


Figure 35. An example of the results of freezing tests illustrating the independence of frost-heaving on the rate of freezing. The soil used was a very fine silt (38 percent coarser than 0.02, 75 percent coarser than 0.006, 10 percent finer than 0.002) Pressure 410 gr. per sq. cm. (0.014 psi.). After Beskow 1935-1.

/5 When the rate of freezing is so small that the rate of heat conduction balances, the amount of water sucked up, the total amount of 'cold' is used in freezing the water and none is left over for further penetration of the frost line. Then the frost line becomes stationary, a single ice layer grows as the water continues to be sucked up.

Numerous writers have stated that one slow gradual decrease in temperature well into the freezing range is necessary and sufficient to cause ice segregation. Belcher /1940-14 presented data to show that "...the ideal condition of one slow gradual decrease in temperature well into the freezing range is not found at depths of less than 12 or 18 inches. Rationally it would seem that the ideal condition for ice lens formation would occur with an unvarying rate of descent of the temperature gradient with an infinitely small increment of change".

The above statements express, qualitatively, some of the relationships between temperature and frost action and emphasize that an important factor influencing the amount of heaving is the rate at which water can be drawn up to the freezing zone. The heat transmission characteristics of the soil and the manner in which they are influenced by the soil moisture content, the temperature differences in the soil and the relationship between duration of cold and penetration of cold all need to be considered if a fuller understanding of the causes of frost action are to be understood.

Effect of Thawing and Refreezing - There is complete agreement among investigators that frost action is usually more severe after a thaw than after a single freeze, and that there is greater increase in soil moisture content and reduction in load carrying capacity following several cycles of freezing and thawing. Some writers have gone so far as to state that heaving does not occur from a single freeze.

Harrison /1918-2 found that pavements do not heave during the first freeze. He concluded that such swelling as may attend the first freezing is general in nature and is of no consequence insofar as its effect on hard surface pavements is concerned. Taber /1929-2 found by experiment that the melting of segregated ice near the surface and prompt refreezing resulted in greater segregation than occurred on the first freezing. He found further /1930-9 that a sudden drop in temperature, after a spring thaw has left an excessive amount of water in the soil, is apt to be more destructive to pavements than the slow freezing of an entire winter. He states "The effects of refreezing after a thaw are also accentuated by the fact that the first freeze leaves the soil in a more or less loosened or expanded condition. Experiments have demonstrated that heaving is greater on unconsolidated clays than on those that are thoroughly consolidated." Burton and Benkelman /1930-10 found greater heaving on refreezing. Benkelman and Olmstead /1931-11 based their theory of frost heaving on the severe expansion which occurred on refreezing, especially when the frost line moved upwards and then again moved downward refreezing the thawed soil.

Depth of Frost Penetration - Taber /1929-2 pointed out that the amount of surface heaving is not proportional to the depth of freezing but, in nature, the depth of freezing is a limiting factor. Likewise Beskow /1935-1 found no direct proportion between heave and depth of frost. He recorded numerous measurements of heaving and depth of frost. He also brought out that the lowering of the line of freezing to zones of greater soil moisture (or nearer to the water table) provides easier access to water so it can be sucked up into the freezing zone. Therefore greater heaves are often associated with greater depths of frost penetration. The point brought out by Taber and Beskow is that it is not necessary to have great depth of frost to have great heave, if other conditions are favorable for heaving to occur. The influence of the various factors related to depth of frost penetration are discussed in more detail later in this review.

Effect of Duration and Intensity of Cold - Several investigators have found a reasonably close relationship between climate and both penetration of frost and magnitude of heave, when climate (that is the "quantity of cold") is expressed in terms of its combined duration and intensity. Some investigators have expressed "cold quantity" in terms of degree-hours of temperature while others have used degree-days as the measure of duration and intensity of cold. In either case, the "cold quantity" is the algebraic sum of degree-days or degree-hours, in which above freezing temperatures are considered negative values and below freezing temperatures are positive values. Because the "quantity of cold" is a large factor in influencing the depth of frost penetration, data showing relationship between duration and intensity of cold and magnitude of frost heave are reviewed later under the subject of "Penetration of Frost".

Influence of Pressure on Heaving

The pressure at the frost line, in the base course or subgrade soil in the case of pavements is made up of two components; the external load and the capillary pressure. The capillary pressure is a negative pressure (suction) but in effect it tends to compress the soil. Thus, capillary pressure may, insofar as its effect in tending to restrain heaving is concerned, be considered a positive value. Normally, capillary and load pressures act simultaneously and their forces, hence their effects are additive.

Load Pressure - Taber /1930-9 stated that "a relatively small surface load will entirely prevent frost heaving in an open system if the material is of such texture that only a little segregated ice forms under the most favorable condition." He found however, that small loads did not greatly affect the heaving of clay soils. In fact, he tested clay soils with different types of cover (one test with wood cover weighing 0.1 kg, a second with same weight wood cover plus a 3.1 kg. iron weight, and a third with only the iron weight) and found that the pressure had less influence on the amount of heave than did the thermal properties of the cover, the specimen covered with the iron weight showing the grestest ratio of uplift to depth of freezing. The amount of heave in open systems decreases with increase in pressure and the maximum load which may be lifted increases with decrease in particle size, but with much decrease in size of particle the material would become highly impermeable. The limit to the load which may be lifted by frost heaving in an open system is not due to the inability of the ice crystals to grow under higher pressure but " ..to the failure of the water supply". Taber supplied water under pressure greater than atmospheric pressure and found that under a load pressure of 14 kg per sq. cm. a total thickness of ice of 2 to 3 cm. was obtained while with water at atmospheric pressure no ice lenses were developed.

Ravn /1940-4 also found that surface loading influenced the extent of heaving, the compression hindering the movement of moisture. He found the height of heaving varied inversely as the square of the applied load.

The Corps of Engineers /1950-32 used surcharge loads of 0, 1, 2, and 3 psi. on a New Hampshire silt to determine the effect of loads (corresponding to 0 to 3 ft. of pavement and base) on heaving. The surcharge loads gave percentages of heave (in terms of depth of frozen soil before freezing) of 155, 138, 77 and 50 percent.

Beskow /1935-1 found that load pressure had a marked effect on heaving, especially in sands as has been mentioned under Heaving in Open vs Closed Systems.

Load and Capillary Pressure - Beskow /1935-1 conducted laboratory tests to determine how load and capillary pressures affected heaving. Some of his results are shown in Figure 36, 37, and 38.

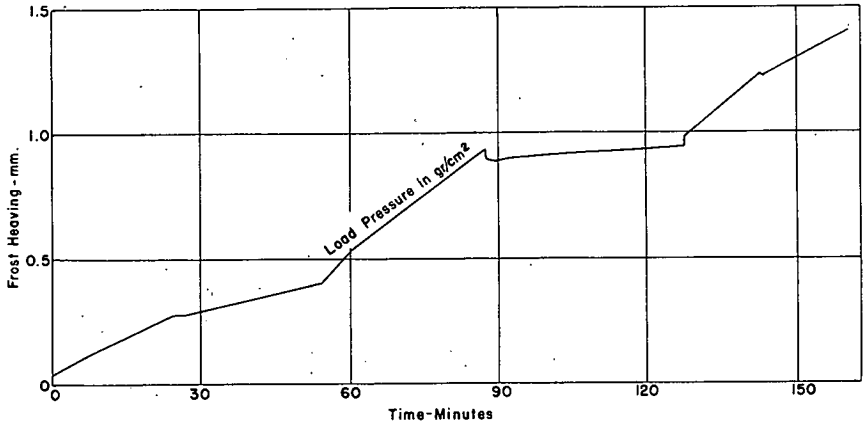


Figure 36. Diagram showing frost-heaving at different pressures (load or capillary). To every pressure corresponds a special inclination of the curve, i.e. a special frost-heaving rate. The frost-heaving per hour is measured on the straight parts of the curve, i.e. where the heaving rate has become constant, and by use of these values the diagrams of type of Figure 37 is drawn. Soil-specimen: pure fraction, 0.01 - 0.005 mm. particle size (equivalent diameter). (After Beskow /1935-1)

Figure 36 indicates the effect of pressure (surface load or capillary pressure) on the rate of heaving. Figure 37 shows the relationship between capillary pressure and rate of heave for various grain size groupings. Figure 38 shows curve of rate of heaving versus pressure on natural soils, the tests being made using load and capillary pressures independently to determine their relative effects on magnitude of rate of heave. Figure 37 shows the large effect which pressure has in restraining heaving in the coarser grained fractions, relatively low pressure being required to prevent heaving (for all practical purposes). He /1947-12 summarized his findings in part as follows:

"The rate of frost heaving for a given soil is inversely proportional to the square of the pressure (equals sum of load and capillary pressure), after the pressure exceeds a certain but not large value. The reason for this "initial pressure increment" is that when the pressure approaches zero, the theoretical rate of heave does not approach infinity but approaches a limiting (but quite large) value".

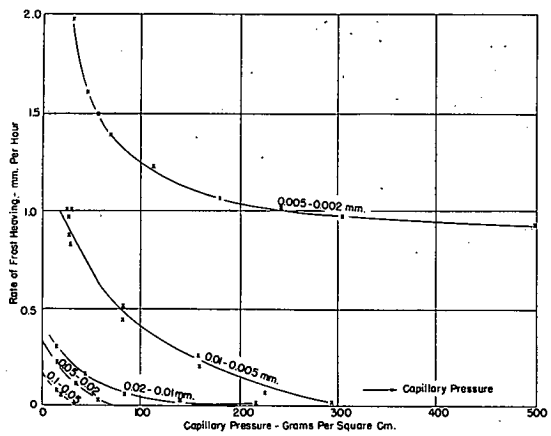


Figure 37. Diagram of frost heaving rate as a function of pressure, obtained from diagrams similar to Fig. 36. Here the pressure is mostly capillary pressure (the only load pressure is that of the metal plate and the dial indicator spindle, about 16 gr. per cm^2). Soils: Atterberg's pure fractions. (After Beskow /1935-1)

Force of Crystal Growth - Taber's early discussion of frost heaving brought out that the force exerted in heaving was due to the force of crystal growth. He cited the work of Becker and Day /1905-1 who had found it practicable, in a saturated solution of alum, to "...grow clear crystals a centimeter in diameter which would raise a weight of one kilogram a distance of several tenths of a millimeter...". Taber /1917-2 attributes the pressure phenomena during the growth of crystals"...to the molecular forces associated with the separation of solids from solution and the attraction and orientation of the physical molecules as they are brought into position on the surface of a growing crystal".

Pressure Developed During Heaving - The maximum pressure which is developed as heaving takes place on different types of soils and under different conditions of climate and soil water condition has long been a subject of observation and speculation. Gilkey /1917-3 cited the case of bridge piers 8 ft. square and weighing 31,000 lb. which heaved a maximum of about 2 3/4 in. Taber /1918-1 in one of his early experiments, placed metal weights on wet sands and wet clays and left the soils to freeze during cold nights. Where the weights rested on sand there was no perceptible raising of the weight. When resting on wet clay, ice formed, raising the weight. Wyckoff /1918-4, in a discussion of Taber's article cited the case of a brick wall

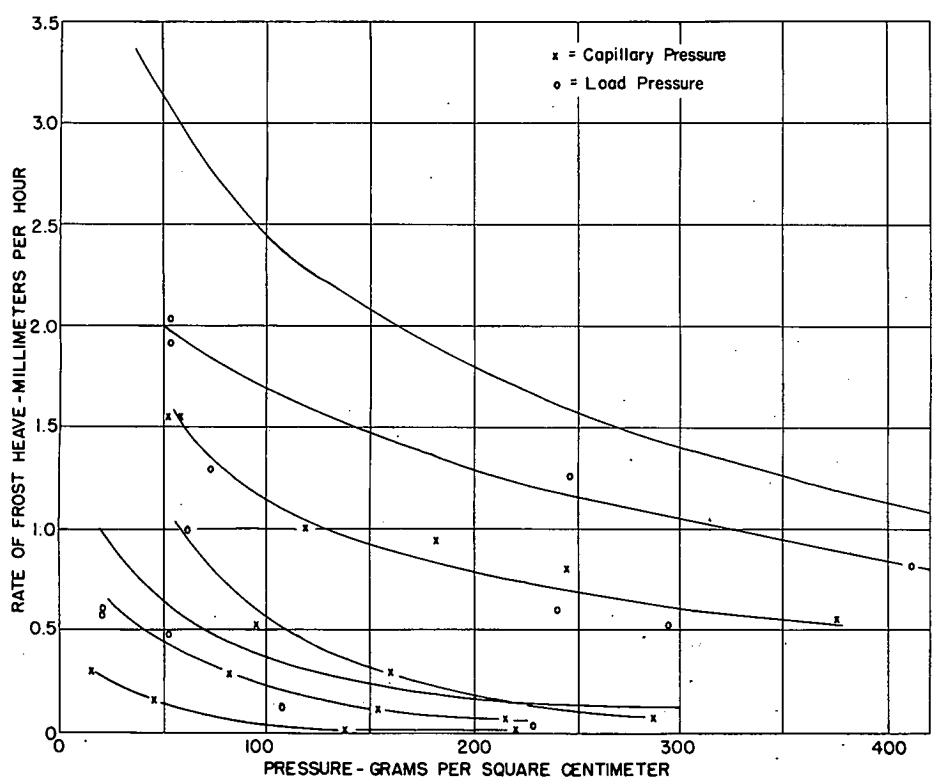


Figure 38. Diagram analogous to that of Figure 37, i.e. showing frost-heaving rate as a function of pressure for some natural soils, and the pure fraction 0.02 - 0.01 mm. (After Beskow /1935-1)

weighing 2,000 lb. per sq. ft. (14 psi.) which was raised 3/4 in. and several piers supporting columns and roof trusses were raised 1/2 to 2 3/4 in. Taber /1930-9 later conducted several experiments to determine the effect which surface loads have on ice segregation. In doing so he obtained some values which are indicative of pressures developed during heaving. He packed a clay tightly in a cylindrical paper container, placed the bottom on a sand saturated with water and froze it from the top down. He measured a maximum pressure of 7.31 kg. per sq. cm. (104 psi.). In another test with a cylinder of undisturbed clay surrounded by paraffin he obtained a pressure of 11.25 kg. per sq. cm. (160 psi.). The layers of segregated ice were few and had a maximum thickness of about one mm. Taber estimated that additional force (to that measured) was required to separate the clay and overcome resistance which would add to the pressure developed but he doubted if the pressure would exceed 15 kg. per sq. cm. (213 psi.) appreciably. He brought out that heave in open systems decreases with increase in pressure; also, that the limit of the load that may be lifted by frost heaving in an open system is not due "...to inability of ice crystals to grow under higher pressure, but to the failure of the water supply". He demonstrated this by supplying water under pressure, increasing the pressure developed and obtaining greater thickness of ice. Taber /1929-2 recognized that if a frozen cylinder "... has no lateral support, the pressures would be limited by the crushing strength of the material". It is of interest to determine how nearly the pressures obtained by Taber approach the compressive strength of ice. Tarr and von Engeln /1915-4 tested small blocks of pond ice at about 18 to 20 deg. F. and found a compressive strength of 1,000 psi. in the direction of the principal axis of the crystals and 350 psi. when loaded in a direction normal to the principal axis. Barnes /1928-1 found the compressive strength of St. Lawrence River ice at about the freezing point to be 363 psi. Taber /1929-2 indicated that in the presence of sufficient lateral support the compressive strength of the frozen material does not limit the load to which may be lifted.

The effect of pressure is for practical purposes, to give a small lowering of the freezing point, thus requiring a small lowering of the temperature if crystal growth is to continue. Bridgeman /1928-1 found that a pressure of 1,000 kg. per sq. cm. (14, 223 psi.) lowered the freezing point 8.8 deg. C.

Casagrande /1935-4 filled the lower portion of a container with saturated mica powder and the upper portion with Boston Blue clay at the liquid limit (42.4%). The container was sealed (frozen as a closed system) and the bottom kept at a temperature of + 3.5 deg. C. while the temperature at the top ranged from -1.0 to -7.5 deg. C. in the different tests. The water content of the clay between ice layers was reduced to 27.5 percent and in the clay beneath the ice to 29.6 percent. The mica was compressed more than one-fourth its volume and its water content was decreased from 75.3 to 41.7 percent. The clay between the ice layers was just as plastic as at temperatures above freezing, indicating that at -1.0 to -2.0 deg. C. none of the pore water in the clay itself froze. Casagrande conducted consolidation tests on the clay and mica used in the tests as a basis for estimating the pressures developed. The results are shown in Figure 39. Engineering News-Record, in reviewing Casagrande's work concluded that, the tests indicate that the crystallization pressures are not constant but increase approximately in direct proportion to the amount the temperature is below the freezing point.

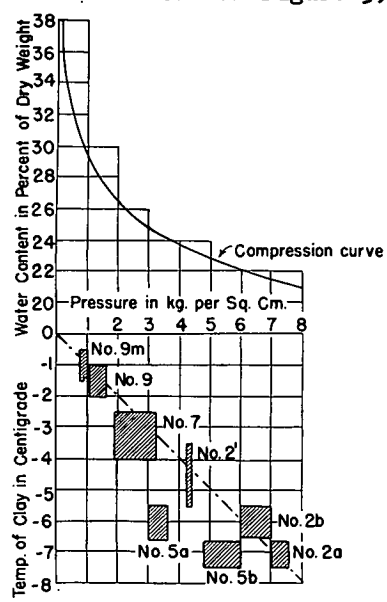


Figure 39. Results of consolidation test of clay to determine pressures required to reduce the water content to the values obtained in the freezing tests. (After Engineering News-Record)

Influence of Physical Characteristics and Properties of Soils on Frost Action

Several investigators have sought to develop relationships between soil characteristics or properties and intensity of frost action. Some of the authors who developed the theories reviewed in previous paragraphs showed that such relationships do exist. Others developed those relationships so they are useful in establishing relationships which aid in defining soils which are essentially frost heaving from those which are not susceptible to detrimental frost action.

Influence of Grain Size Distribution - Taber's /1929-2 experiments were among the earliest to bring out the effect of soil texture on frost action. In one experiment he inserted a vertical partition in a cylindrical container, filled one side with clay and the other with sand, allowed the materials to become saturated, then buried the cylinder in dry sand and froze it from the top downwards. The results he obtained are shown in Figure 40. He also conducted freezing tests on

different mixtures of a sedimentary clay and standard Ottawa sand. The mixtures contained 50, 40 and 30 percent clay by weight. Taber /1929-2 stated that "on freezing no segregated ice could be seen in mixtures containing less than 30 percent clay". The tests were made on compacted specimens placed in wet sand which were frozen from the top down.

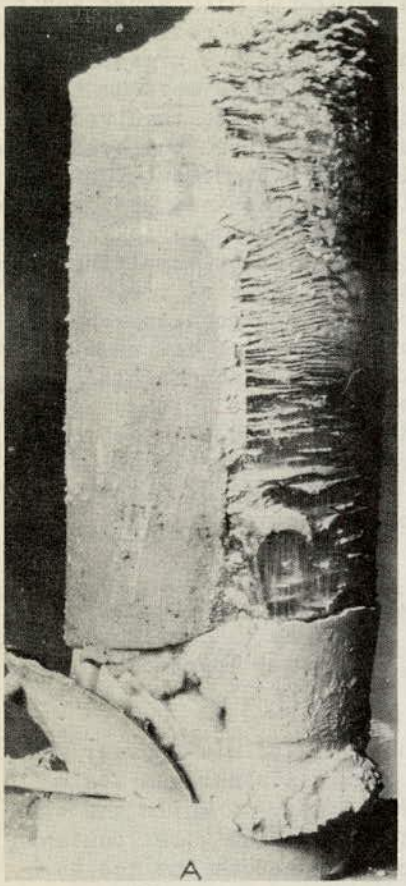


Figure 40. Frozen cylinder, half sand and half clay. Much segregated ice in clay but not in sand. (After Taber)

Further tests were made on fine sands, so fine that water would rise to a height of 20 cm. (7.9 in.) in them, but heaving did not occur. Taber ground quartz in a ball mill and screened it over a 200 mesh sieve (sieve opening 0.074 mm.). Freezing under the most favorable condition gave only the faintest evidence of segregation. Quartz dust with an average grain size between 0.006 and 0.001 mm. showed slightly greater evidence of segregation. He also used precipitated barium sulphate (av. grain size 0.002 mm.), lithopone (71 percent $BaSO_4$ and 29 percent ZnS giving a grain size of 0.00005 mm.) and Kadox (ZnO with grain size of 0.00025 mm.). The results of the tests showed that well defined ice layers formed readily in material having a grain size of 1 micron (0.001 mm.).

Taber /1930-9 also tested mixtures of bentonite and clay. He described his results in part as follows "on freezing from the top downward, tension is set up in the bentonite immediately below a growing ice layer. Since the stress is uniformly distributed and the material homogeneous, vertical cracks tend to develop, which in horizontal sections form a polygonal pattern. The cracks are gradually filled with clear ice, and as freezing progresses they advance downward, forming a columnar structure, which, combined with the normal horizontal ice layers results in the peculiar cellular structures...".

Burton and Benkelman /1930-10 found from the Michigan survey that 65 percent of heaves occurred in moraines. Sixty-five percent of the heaves in moraines occurred on three soils, and were about equally divided among the Bellefontaine, Coloma and Miami soils. Regarding texture Burton and Benkelman stated that, "Within the soil type itself the cause of heaving is due to definite associations of material favorable to this phenomenon. It should be further pointed out that from all indications to date, frost heaving due to ice segregation does not occur in material of a size greater than that in which capillary phenomena prevail". They made detailed analyses of 156 heaves. Of 94 heaves which occurred in fine grained soils, 5.3 percent developed in silts, 32 percent in very fine sand and silt, 4.3 percent in very fine sand, 30 percent in clay, 8 percent in silty clay, and about 17 percent in sandy clay. The average height of all heaves in silt was 6 in., in very fine sand and silt 5 in., in very fine sand 4 in., in silt clay 5 in., and in sandy clay 3 in.

Burton /1931-5 held that trouble from frost invariably occurred where silty or very fine sandy silt textures prevailed. Benkelman and Olmstead /1931-11 after examining about 200 locations in Northern Michigan found that "...heaves in excess of a few inches may occur in coarse sands or even in gravelly materials providing an excess of water is present, either from seepage over impervious strata or from a naturally high water table. ...In silts and very fine sands, however, even though drainage is provided, the amount of moisture either present by capillarity or drawn to the zone of freezing will be sufficient to fill voids created during a cycle of isothermal line fluctuations and cause excessive heaving.

As mentioned before, field observations have definitely established the fact that excessive frost heaving is not restricted to soils of any particular grading or characteristics. Heaving has been observed to occur in clays, silts, very fine sands, and in textures approaching the grading of gravels".

Casagrande /1931-13 described the behavior of eight, 3 ft. square slabs laid on two types of soil and exposed outdoors at the Massachusetts Institute of Technology during the winter of 1928-1929. The soils were placed to a depth of 3 ft. with provision for free vertical movement. The soils used were a sandy silt with a small clay content and a clean fine uniform sand. The soils came from New Hampshire where the silt had caused heaving and the sand had not. The results

of accurate measurements of all movements showing the heaving and frost penetration record for bituminous concrete and portland cement concrete pavements on the New Hampshire silt and the sand and the prevailing temperatures and depth of frost penetration are shown in Figure 41. He also presented data showing the record of soil conditions and frost heaving on 2,000 ft. of concrete pavement in New Hampshire during the winter of 1927-28. He drew the following conclusions from his studies:

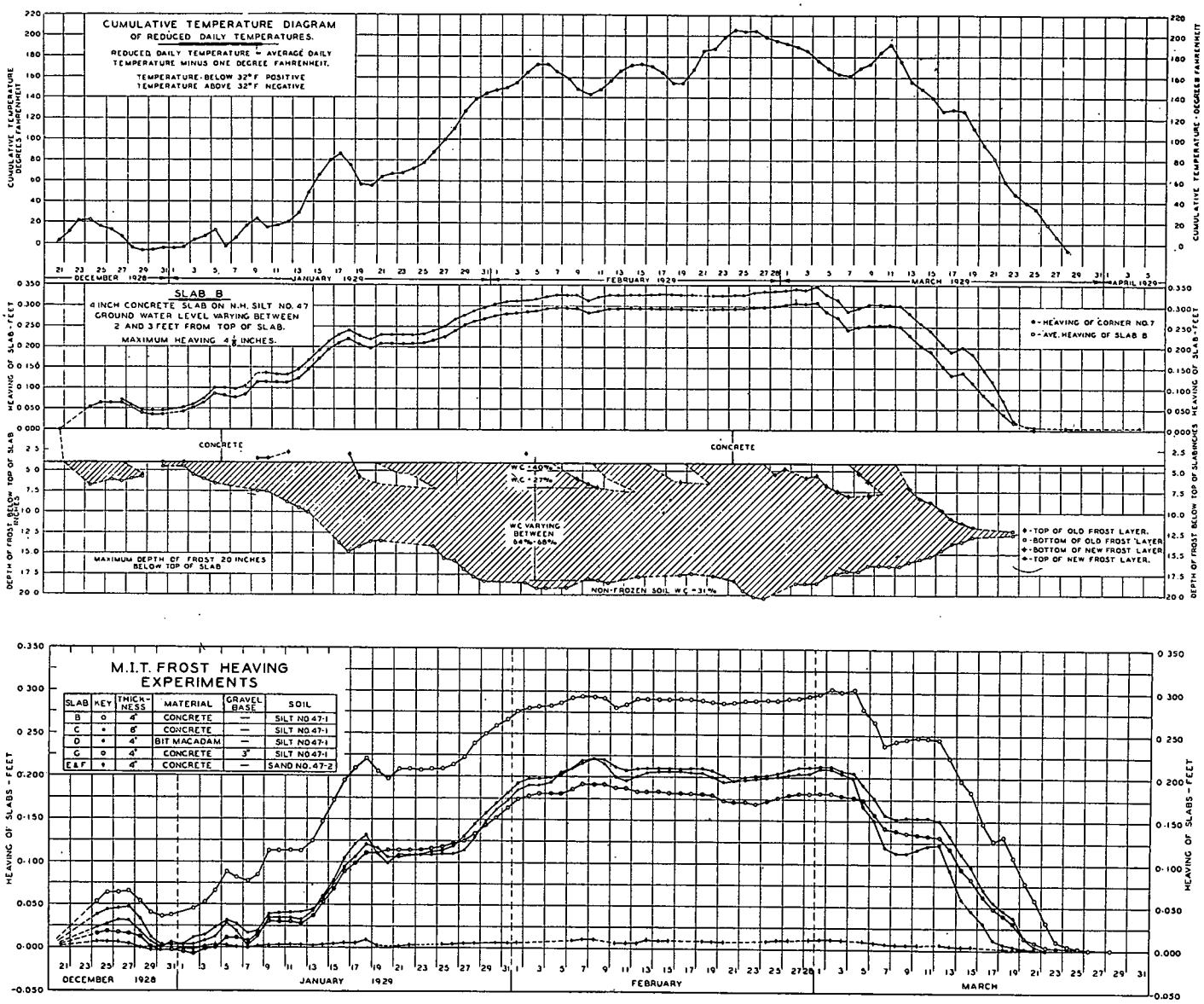


Figure 41
(After Casagrande 1931-13)

"Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soil containing more than three percent of grains smaller than 0.02 mm., and in very uniform soils containing more than 10 percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than one percent of grains smaller than 0.02 mm., even if the ground water level was as high as frost line."

In reviewing the work of Beskow /1935-1, /1947-12 it is important to bear in mind that he is one of the very few investigators who have made an effort to define what he meant by an "essentially non-frost heaving" soil. His definition of a zone of "essentially non-frost heaving" "...as that which has less than 3 to 4 cm. (approximately 1.2 to 1.6 in.) heave during a winter...".

Many engineers will agree that Beskow's definition was reasonable for that time. However, the present concept is that reduction in load carrying capacity during and following the frost melting period is also a significant phase of frost action. Soils near the border line separating

those which Beskow defines as essentially non-frost heaving from those which he defines as frost heaving may be dangerous in that they may produce critical reduction in load carrying capacity. This point should be kept in mind where evaluating Beskow's work in terms of conditions which obtain in the reader's locality. Beskow used two methods in his efforts to define grain size limits of heaving and non-frost heaving soils. One of these was laboratory testing of specimens made up of "pure fractions", the other by gathering samples of the coarsest frost heaving soils from different localities and determining their grain sizes.

His first efforts were devoted to a study of limiting sizes which are productive of homogeneous and stratified types of frozen soil structure. These studies were made on soil fractions. Some of the results are shown in Table 13. It may be seen from Table 13 that Beskow observed no ice banding in material having a minimum grain diameter of 0.1 mm. and "...small but visible banding" at the surface when the minimum grain diameter was 0.05 mm.

TABLE 13

RESULTS OF FREEZING TESTS USING SOIL FRACTIONS (After Beskow)

Sample Number	Soil Fraction		Change in Weight During Freezing Freezing Temperature = -2.0 deg. C.	Remarks
	Grain Size Limits	Capillarity K		
	mm.	cm.	g	
1	1 -0.5	11.3	-0.6	Non-ice banding
2	0.5 -0.2	25.5	-0.8	Non-ice banding
3	0.2 -0.1	41	-0.7	Non-ice banding
4	0.1 -0.05	99	+0.5	Small but visible banding at surface (to 7 mm. depth)
5	0.05 -0.02	186	+1.35	Sharp thin ice bands throughout the whole sample
6	0.02 -0.01	440	+3.05	Ditto, but more prominent
7	0.01 -0.005	820	+8.6	Extreme ice banding
8	0.005-0.002	1700	+18.05	Numerous and thick ice banding
9	0.002-0.001 (-0.0016-0.005)	7000	+15.4	Numerous and thick ice banding

Beskow recognized in his study of the effect of grain size on heaving that there could be a difference in the development of ice strata in the laboratory compared to natural freezing. By very fast freezing he could cause clays to freeze homogeneously. Also, in the laboratory it is possible to freeze a soil quickly and obtain considerable heaving "...without showing any ice strata under the most minute observation". He explains that the difference between such laboratory results is due primarily to the lower rate of freezing in nature. He concluded from his studies of grain size as related to structure of the frozen soil that in natural freezing "... as a rule, frost heaving soil is practically always ice stratified and we can say that the actual size between homogeneous and heterogeneous (stratified) frozen soil is the same as the critical size between frost-heaving and non-frost-heaving soil. Thus, insofar as soil fractions are concerned he concluded that "...above an average diameter of 0.1 mm. practically no stratification occurs...". He also attempted to evaluate the influence of grain size on heaving under different pressures as has been mentioned earlier in this review. (Very small heaving occurred for the fraction 0.1 - 0.05 mm. grain diameter and that heaving was highly dependent on pressure).

Beskow investigated the frost heaving properties of numerous natural frost heaving soils. Because of a difference in their frost heaving he summarized his findings into two groups depending on the origin of the soil; namely, sediments and moraine material.

Normal sediments (water deposited material) tested by Beskow are non-uniform in texture and resemble our own sediments. For those materials he found that the limiting grain size is a definite boundary. By choosing values consistent with the soils studied he concluded /1947-12

that "...this limit is at the point where 30 percent of the material is finer than 0.062 /6 mm. or 55 percent of the material is finer than 0.125 /6 mm. All soils coarser than this are definitely non-frost heaving.

"But the soils that lie just under this limiting value - the coarsest soil that can be frost heaving, or actually a medium coarse silt - are not under all circumstances frost-heaving, but only under certain conditions, such as a high ground water table and a very small load pressure. Such soils are only dangerous when located in wet slopes, where the load pressure is insignificant, and when in road beds with a very high ground water table (very poor drainage of an abnormally wet fall)."

Beskow found it more difficult to distinguish between frost heaving and non-frost heaving moraine soils in terms of grain size. The first difficulty concerns what consideration should be given the coarse fraction. He chose the arbitrary and conventional 2 mm. grain size (No. 10 sieve) to separate coarse and fine particles and neglected the coarser material.

From the results obtained Beskow constructed the limiting grain size curves for moraine materials shown in Figure 42. From the curves he states that "...non-frost-heaving moraines are those of which less than 22 percent of the material passes the 0.125 mm. sieve and less than 15 percent passes the 0.062 mm. sieve computed on the basis of percent weight of the material that passes the 2 mm. (No. 10) sieve". These values pertain to moraine materials having "normal" grading curves and also to mixtures of materials having approximately the same type of grading curve.

Contrary to the results obtained by Taber, Beskow found that as little as 2½ percent clay added to a sand causes a noticeable effect, 10 percent considerable, and 20 percent a very large effect in the frost heaving. He cautioned that for unevenly sorted mixtures such as disintegrated soils and certain soils found in roadbeds, the behavior is complicated and, the freezing properties cannot always be determined only on a basis of sieve analysis. He held that the more the colloidal material in such mixtures the greater the danger of heaving even though there may be very coarse soil in the mixture. He found further that coarse frost heaving soils have a permeability so great in comparison with their frost heaving rate that the "...pressure difference needed to suck up the water is nearly zero". If the grain size is assumed to be variable"...and the surface load and the distance to ground water to be constant, the rate of frost heaving varies inversely as the cube of the grain size. It is then obvious that the rate of heaving is very sensitive to variations in grain size so that a small increase in grain size causes a large decrease in heave. Doubling the grain size decreases the rate of heaving eight times...".

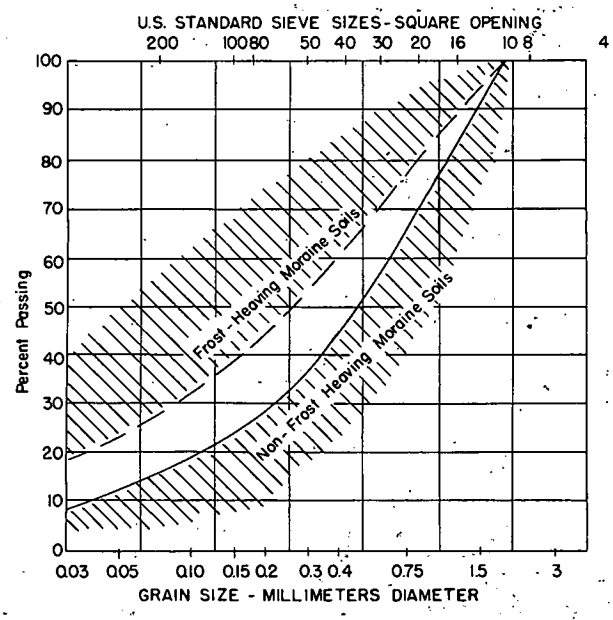


Figure 42. Limits between frost susceptible and non frost-susceptible mixtures of moraine materials or similar mixtures. (After Beskow)

It should be borne in mind that all of the above is dependent upon the definition of what constitutes an "essentially frost heaving" or a "non-frost heaving" soil.

In 1938 Beskow /1938-11 established the following grain size and capillarity limits for soils which are not subject to frost heaving.

/6 Note that AASHO M 92-42 sieve size openings for No. 120 sieve is 0.125 mm. and No. 230 is 0.062 mm.

	Passing the No. 200 Sieve	Capillarity
	%	in.
Well sorted sediments	Less than 40	Less than 40
Graded moraine soils (determined on passing the No. 10 sieve)	Less than 19	Less than 40

When figures exceed these limits the soils are classified as frost heaving.

Housel /1938-13 brought out that frost heaving may occur in a wide range of soil textures. He held that uniform clay soil caused little trouble from heaving but emphasized that concentration of water in clay soils is the primary source of spring break-up. He held that it is the intermediate textural range from fine sand to silt which presents the greatest difficulty.

Casagrande /1938-5 also explained that even heavy soils which do not lie within reach of ground water, but which have abosrbed large quantities of water can cause damage due to frost. He expanded somewhat on his previous /1931-14 statement on the effect of grain size, stating that "...In granular soils deleterious effects of frost may be expected, according to the Casagrande criterion when the proportions of grains smaller than 0.02 mm. in diameter amounts to at least 3 percent and in case of very uniform soils (U5) exceeds 10 percent. Soils which do not conform to these conditions, either exhibit only a very slight formation of ice or none at all. Highly fissile rock may likewise lead to localized damage by frost where the moisture conditions are unfavorable." He states that "The extent to which a soil is exposed to danger from frost can be determined with little experience by examining it with the naked eye and the hand. Admixtures of 3 to 10 percent smaller than 0.02 mm. diam. are found, for example, in impure sand or gravel and makes the hand dirty when rubbed." When dry the soil has only slight cohesion and can be recognized from the dust which forms. In border line cases a hydrometer test is made.

Riis /1948-25 states that when Casagrande's /1938-5 "heterogeneous number" is greater than 5, the soil should have not more than 3 percent by weight of 0.02 mm. material if it is to be non-frost-susceptible, but, if the number is less than 5 the -0.02 mm. fraction may be as great as 10 percent. He defined the "heterogeneous number" as the relation $d_{60} \div d_{10}$ where d_{60} denotes the grain size under which 60 percent of the material lies and d_{10} that under which 10 percent lies. Shannon /1944-1 in reporting the results of Corps of Engineers frost investigations at Dow Field, Bangor, Maine, verified Casagrande's grain size limits for non-frost susceptible soils.

Smith /1946-1 investigated gravel bases under sealed surfaces which suffered frost damage in New Zealand. His samplings included bases from failed areas and from areas where no distress was evident. He found that "...frost damaged gravels show a shortage in the grading on sizes 8 to 48, that is, in the coarser sands, and the voids resulting from this shortage are filled with clay with a plastic index usually of over ten". (A typical value was given as 18.5). It may be seen from the chart in Figure 43 that the summary of percentages retained between the 8, 14, 28 and 48 sieves are, for failed areas approximately half that for areas free from frost damage. Smith recommends treating "...all pit gravels with suspicion rejecting any with a binder fraction of over ten percent when placed". He also recommends placing the gravel in layers of not more than one-inch thickness to avoid segregation of material.

The Corps of Engineers /1947-2, /1949-23 presented data from several airfields giving the percent of material finer than 0.02 mm. grain size in granular bases and the occurrence of ice crystals and lenses in the bases. Those data are summarized in Table 14. It may be seen that no ice lenses formed on material containing up to 18 percent finer than 0.02 mm. The Western airfields showed no ice lens formation and showed crystal formation only at Sioux Falls where the percent finer than 0.02 mm. was 7 to 11. The bases, however, contained relatively low water contents. The reviewer cautions that the moisture content of bases in all of the Western fields may not have reached a maximum in the short time since the fields were constructed.

A survey by the Texas Highway Department /1948-46 of performance of road bases during the severe winters of 1946-47 and 1947-48 included data on sources of materials, hardness, absorption, and grain size distribution.

The Austin, Gober and Pecan Gap chalks of the upper Cretaceous are described as being highly absorptive (10 to 15 percent), susceptible to weathering and degradation under traffic. Typical test results indicate values of liquid limit of 30 to 43, plastic index 13 to 17, linear shrinkage 6 to 8 and Los Angeles abrasion of 55.

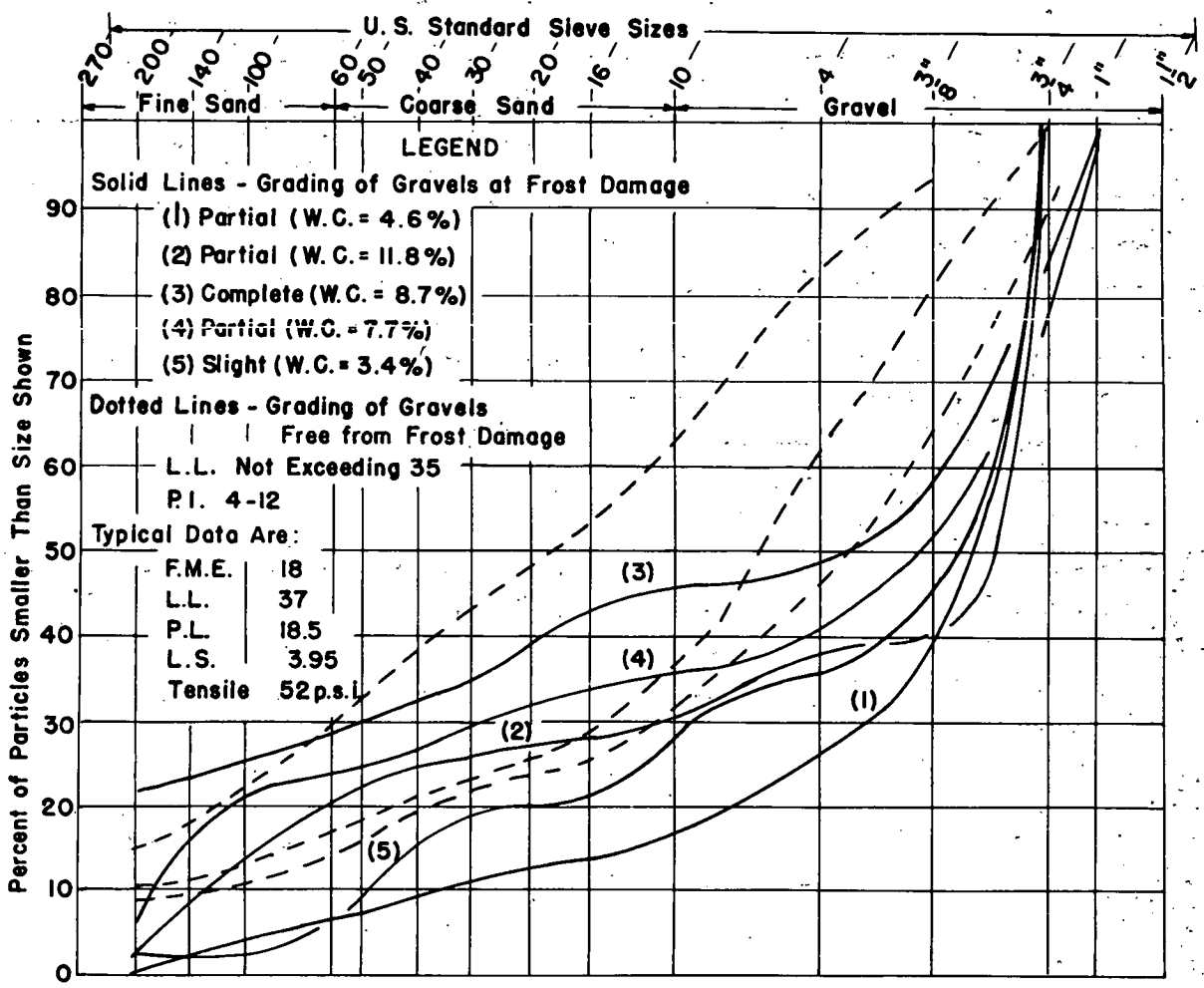


Figure 43
Mechanical Analysis of Gravels from Roads Damaged and Not Damaged by Frost. (After Smith)

The Ogallala formation, occurs as a broad alluvial outwash plain consisting of gravels, sands, clays, volcanic ash and caliche. The caliche ranges from hard to soft and is often porous. An investigation in 1941 reported that all freeze damage occurred in bases having plastic indexes over 15 and with more than 50 percent soil binder. Since then failure has occurred on somewhat better materials. Typical limits of grading and soil constants of Ogallala caliche are indicated in Figure 44.

Limestones of the lower Cretaceous formation outcrop over a large area in central and west Texas in arid to semi-arid, subhumid climatic belts and where the average frost penetration ranges up to a maximum of 6 in. Much of the freeze damage north and northwest of San Antonio is credited to the "pulverulent" Edwards limestone group of the Lower Cretaceous in this area. The material shows a Los Angeles abrasion value of 80, shows rapid failure in the soundness test; has a high liquid limit with respect to the plastic index (L.L. = 25 to 41, P.I. = 4 to 8); has a low shrinkage ratio (1.4 to 1.65) and, a high dust ratio. Figure 45 shows gradation of total base and soil binder of the pulverulent Edwards member.

The report indicates that "...gravel bases have shown a surprising resistance to severe freeze damage." Figure 46 shows the gradation and soil constants of three types of bases damaged by freezing in comparison with values of a good gravel base resistant to freeze damage.

TABLE 14

RELATIONSHIP BETWEEN PERCENT FINER THAN 0.02-mm. DIAMETER GRAIN SIZE AND THE OCCURRENCE OF ICE SEGREGATION IN BASES UNDER AIRFIELD PAVEMENTS (AFTER CORPS OF ENGINEERS /1947, /1949-23)

Airfield	Class and Thickness of Base	Percent Finer Than 0.02 mm.	Ice Segregation 1944-1945 Crystals	1944-1945 Lenses	1945-1946 Crystals	1945-1946 Lenses	1946-1947 Crystals	1946-1947 Lenses
	in.							
Presque Isle, Me	GW - 24-36	0-7	yes	no				
	22-38	0-7			yes	no		
Dow, Me.	GW - 15-63	3-7	yes	no				
	19-49	2-6	yes	no	yes	no	yes	no
Houlton, Me.	GW - 6	2-15	no	no				
Bedford, Mass.	GW - 13-19	3			no			
Truax, Wisc.	CR[a] - 8-20.5		yes	no	yes	no		
	GF - 36-60	9-20	no	numerous 0-1/16				
	GF - 25-36	8-20			yes	numerous 0-0.05		
Selfridge, Mich.	GF - 7-20	2-5			yes	no	yes	
Pierre, S. D.	GF - 6-16.5	6-13	no	no				
Casper, Wyo.	GW - 7-13	2-5	no	no				
Watertown, S. D.	GP - 8	4-15	no	no				
Fargo, N. D.	Soil Cement		no	no				
Bismark, N. D.	SC - 6-6.5	5-9	no	no				
Sioux Falls, S.D.	GC - 9.5	7-11	yes	no	no	no	no	no
Great Bend, Kan.	SW - 6	1-3	no	no				
Garden City, Kan.	SC - 10.5	7-8	no	no				
Pratt, Kan.	SF-CL - 0-12	24	no	no				

(a) Crushed Rock

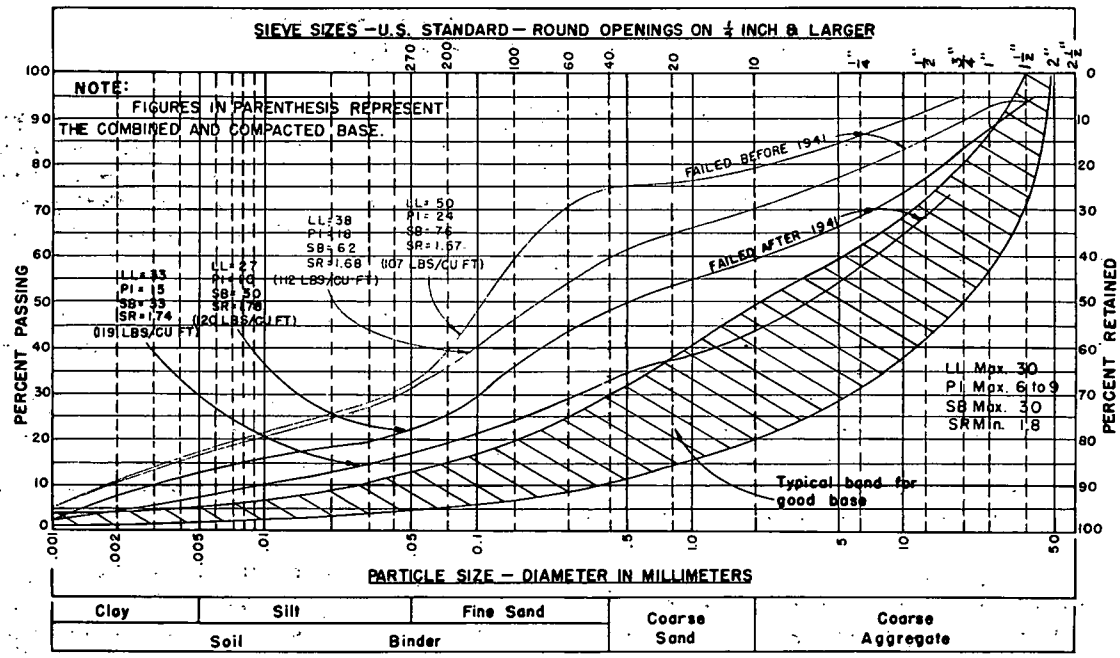


Figure 44
Cumulative Mechanical Analysis
Ogalalla Caliche
(After Carothers /1948-46)

England experienced one of the most severe winters in 100 years during 1946-47. Considerable freeze damage to road surfaces occurred both in the form of surface failures and heaving with accompanying softening of the subgrade during thawing. Croney /1949-24 and /1951-36 reported the silty soils to be worst offenders and showed from the results of investigations by the Road Research Laboratory that the particle size limits indicated in Figure 47 were the worst offenders. Certain chalks and limestones having saturation moisture contents of 20 to 30 percent were serious offenders. He brought out that where the chalk was broken up and redeposited in fill, the susceptibility to frost heave was increased, even the hardest chalks giving trouble.

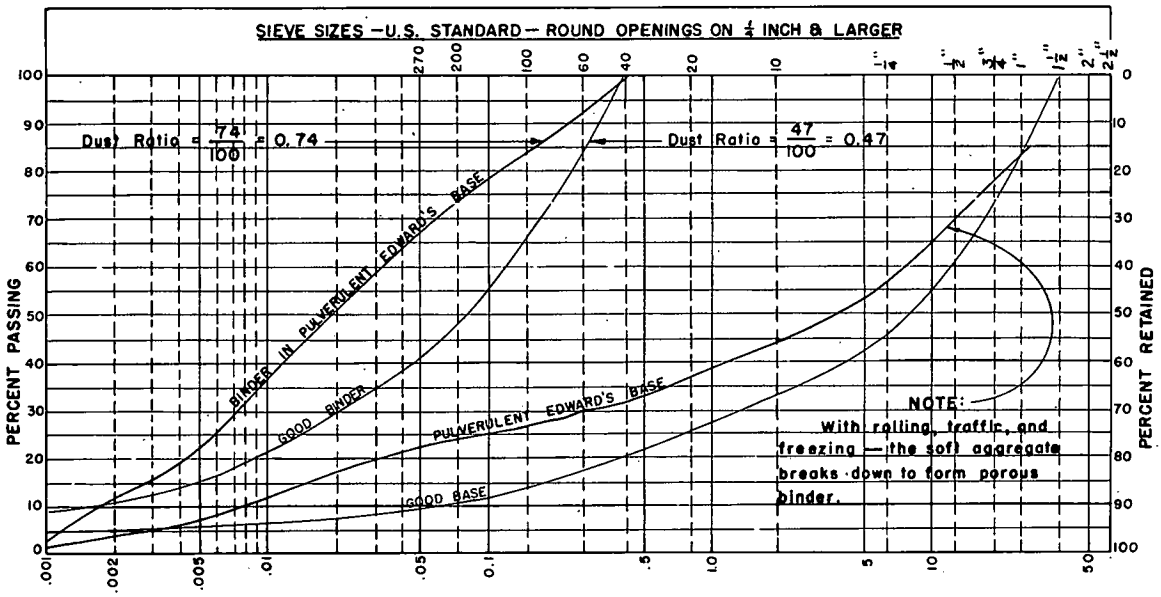


Figure 45
Cumulative Mechanical Analysis
Pulverulent Edwards
LL = 41, PI = 6, %SB = 32, SR = 1.43 X 62.5 = 89.4 #/Cu. Ft.
(After Carothers /1948-46)

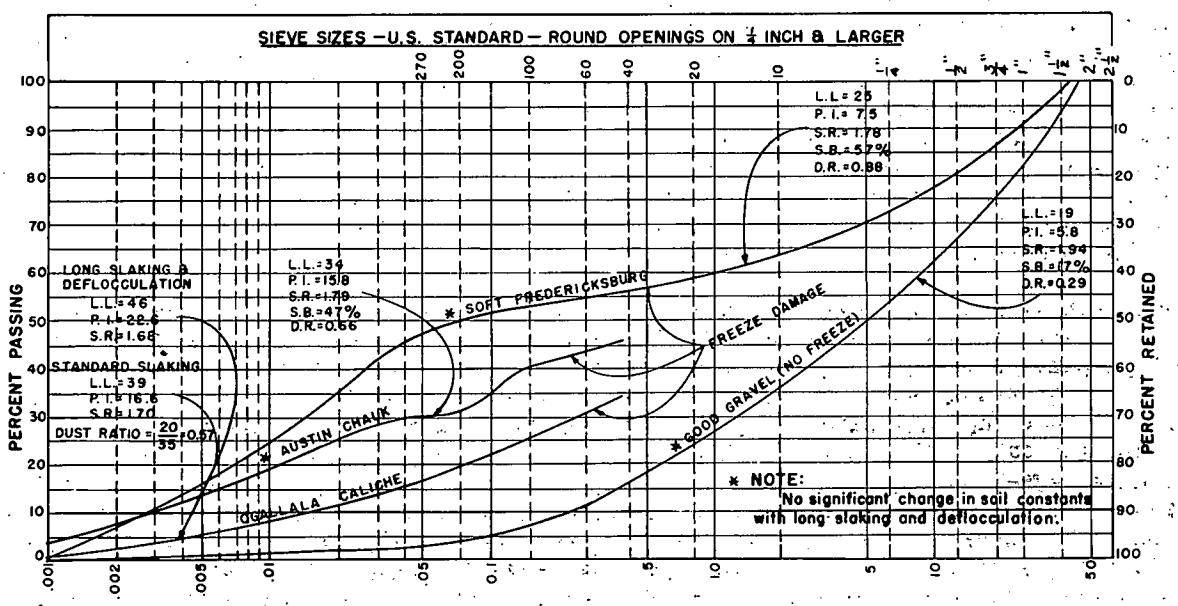


Figure 46
Cumulative Mechanical Analysis
Gradation of Freeze Damaged Bases
(After Carothers /1948-46)

The Corps of Engineers /1951-32 selected nine soils for laboratory study to determine the relationship between grain size distribution and frost heave and to check the validity of present criteria for frost susceptible soils. The soils ranged in texture from a well graded sandy gravel to a medium plastic clay. The laboratory study also included base and subgrade soils from 11 airfields in northern United States. Sources of the soil samples, their descriptions, and data on

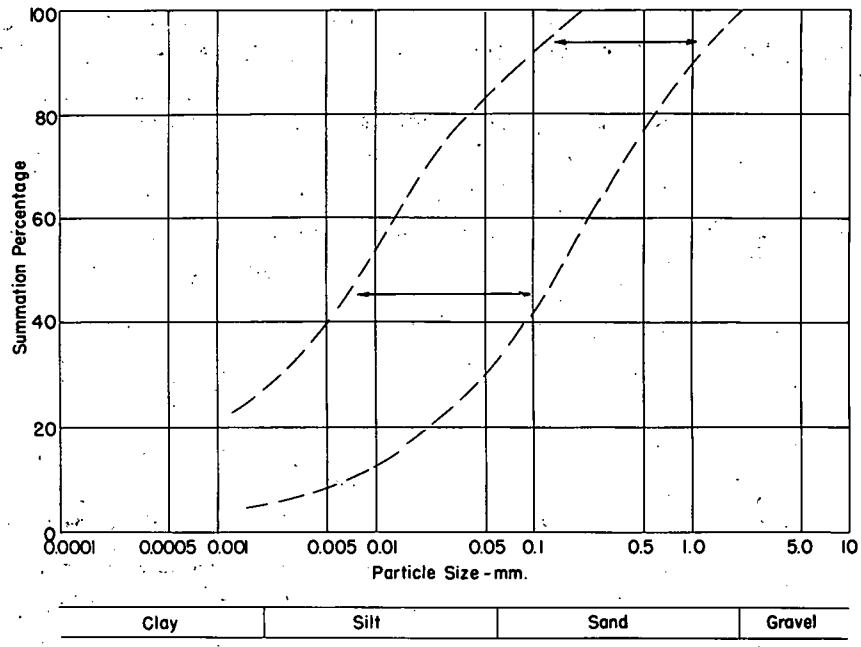


Figure 47
Particle-Size Limits Within Which Soils are Likely
To Be Frost-Susceptible (After Croney)
Based on failure investigations carried out in Southern England

grain size and density are given in Table 15. Soils with grain sizes smaller than 1/4 in. were compacted in 4.28-in. diameter steel cylinders. Soils having particles up to 2 in. in size were prepared in 5.91-in. diameter steel cylinders. Specimens were compacted to about 95 percent modified AASHO density or to Providence vibrated density (soil placed in cylinder, loaded with 1000 lb. or more and the cylinder vibrated with hammer blows on sides - maximum pressure of 4000 psi was used on some materials). After removal from the cylinder the specimens were coated with plastic material and a heavy coating of petrolatum and fitted smugly into 6-in. high open-end cardboard cylinders. Tests to date were all on specimens which were evacuated and saturated with de-aired water. They were frozen with the bottom of each cylinder in contact with free water. Most specimens were loaded with a surcharge of 0.5 psi. The rate of penetration of freezing temperatures was about 1/4 in. per day.

The initial results given in Table 15 may be appreciated better if illustrated graphically as in Figure 48. It may be seen that the blend of sandy-gravel and till (sample AGI-5 in Table 15) having 3 percent by weight of material finer than 0.02 mm. heaved about 11 percent while some other soils with nearly similar content of 0.02-mm. material heaved 2 percent or less. The results bring out that the nature, as well as the proportion of fines influenced heave. That is further brought out by the tests of blends of Manchester uniform fine sand with Boston glacial till (samples MFS-1 through MFS-4) when compared with the same sand blended with New Hampshire silt (MFS-5 through MFS-8). Only the minus No. 40 portion of the till was used for blending. The Corps of Engineers states that "In general, the test results available to date in this phase of the investigation indicate that the presently used criterion which states that well graded soils with less than three percent, by weight, finer than 0.02 mm. are not frost susceptible, has proven to be a useful rule but other factors, such as the character of the fines, must be considered in recognizing frost susceptible soils with accuracy or in predicting the intensity of ice segregation which may be expected.

Although the effect of boulders in soil on frost heave has been mentioned in the literature, little work has been done on the effect of size and percent stone within the range of "normal aggregate sizes". The presence of stones in a soil may reduce frost heave because of the increased rate of frost penetration due to the higher thermal conductivity of rock, the smaller amount of volumetric and latent heat in the soil mass and the reduced volume of frost susceptible material as well as the reduced overall permeability. The Corps of Engineers summarizes their results as follows:

COLD ROOM STUDIES OF FROST ACTION IN SOILS

Table 15

SUMMARY OF TEST DATA*

Type of Test	Sample Number	Material	Grain Size mm. - % Finer 2.0	0.42 (1)	.074 (2)	0.02	Per Cent Heave (3)	Dry Unit Weight, lbs./ft³	Void Ratio (4)	Permeability, k x 10⁻⁴ cm/sec.	Degree of Saturation at Start of Test	Atterberg Limits (5) L.L.	P.I.
Effect of Per Cent Finer Than 0.02 mm. (Open System)	LSG-1	Limestone AFB Sandy Gravel (3/4" max. size)	28	11	4	3	6.6	136.4	0.240	-	100	Non-Plastic	
	LSG-2		31	13	7	5.5	8.2	136.4	0.240	-	100	"	"
	LSG-3		29	14	9	6.5	18.1	136.4	0.240	-	96	22	8
	LSG-4		27	17	11.5	9	14.7	136.4	0.240	-	100	26	7
	AG1-1	Peabody Sandy Gravel Blended with New Hampshire Silt (1/2" max. size)	49	20	5	2	0.2	123.0	0.370	49.0	100	Non-Plastic	
	AG1-2		47	23	9	5	1.3	123.0	0.370	49.0	93	"	"
	AG1-3		45	24	12	7	2.3	123.0	0.371	54.0	87	"	"
	AG1-4		44	27	17	9	2.1	123.0	0.371	350.0	91	"	"
	AG2-5	Peabody Sandy Gravel Blended with New Hampshire Silt (1/4" max. size)	66	31	7	3	2.1	125.0	0.346	7.0	100	Non-Plastic	
	AG2-6		69	32	11	6	7.3	127.0	0.325	2.55	91	"	"
	AG2-7		72	35	13	7	5.8	127.0	0.325	2.05	97	"	"
	AG2-8		70	34	18	10	7.9	127.0	0.325	1.06	86	"	"
	AG1-5	Peabody Sandy Gravel Blended with East Boston Till (1/2" max. size)	51	23	6	3	10.9	133.0	0.268	0.85	100	Non-Plastic	
	AG1-6		47	23	8	5	10.8	133.0	0.270	0.40	94	"	"
	AG1-7		49	25	10	7	12.0	134.2	0.259	0.215	91	"	"
	AG1-8		50	27	13	9	11.0	134.2	0.260	0.138	89	16	3
	LS-1	Lowell Sand Blended with East Boston Till	100	72	6	3	0.9	104.0	0.611	-	96	Non-Plastic	
	LS-2		100	69	8	4	0.5	104.0	0.612	-	87	"	"
	LS-3		100	60	10	6	0.8	104.0	0.615	-	38	"	"
	LS-4		100	78	14.5	11	1.9	104.0	0.618	-	90	"	"
	LS-5	Lowell Sand Blended with East Boston Till	99	69	13	9.5	3.5	108.6	0.548	41.0	99	Non-Plastic	
	LS-6		99	70	19	14.5	6.9	109.5	0.539	10.8	100	"	"
	LS-7		97	75	24	18	21.1	109.5	0.543	4.9	100	"	"
	LS-8		97	77	31	26	25.3	108.5	0.561	2.8	100	"	"
	MFS-1	Manchester Fine Sand Blended with East Boston Till	100	99.5	11	3.5	8.8	107.0	0.564	7.2	100	Non-Plastic	
	MFS-2		100	99.5	17	6	11.9	107.0	0.565	3.05	100	"	"
	MFS-3		100	99.5	17	9.5	10.1	107.0	0.568	4.1	100	"	"
	MFS-4		100	99.5	20.5	11	16.9	107.0	0.570	2.5	100	"	"
	MFS-5	Manchester Fine Sand Blended with New Hampshire Silt	100	99.5	16	7	1.0	107.0	0.560	8.1	99	Non-Plastic	
	MFS-6		100	99.5	18	9	2.1	107.0	0.562	5.7	99	"	"
	MFS-7		100	99.5	22	11	2.7	107.0	0.562	4.25	99	"	"
	MFS-8		100	99.5	27	14	2.5	107.0	0.564	3.00	95	"	"
	MFS-9	Manchester Fine Sand Blended with New Hampshire Silt	100	99.5	29	16	6.6	114.0	0.465	1.47	100	Non-Plastic	
	MFS-10		100	99.5	34	18	9.6	114.0	0.465	0.74	96	"	"
	MFS-11		100	99.5	39	21	5.0	114.0	0.470	0.88	100	"	"
	MFS-12		100	99.5	45	26	16.0	114.0	0.470	0.36	97	"	"
	TD-1	Truax AFB Silty, Gravelly Sand (regraded to vary the fines 3/4" max. size)	81	68	7	2	3.7	125.0	0.348	-	96	Non-Plastic	
	TD-2		83	72	11	6	8.3	127.5	0.321	-	100	"	"
	TD-3		85	76	22	13	10.2	130.0	0.295	-	98	"	"
	TD-4		90	82	36	20	17.1	132.4	0.270	-	100	"	"
	TD-5	Truax AFB Silty Gravelly Sand (TD-5 is a mixture of two subgrade samples. TD-6 is a typical natural subgrade)	88	79	28	16	23.2	129.5	0.300	-	92	Non-Plastic	
	TD-6		88	78	35	21	28.2	128.8	0.315	-	94	14	2
Effect of Degree of Compaction (Open System)	NH-1	New Hampshire Silt	100	99	96	58	60.4	90.0	0.872	0.78	100	27	0
	NH-2		100	99	96	58	68.8	95.0	0.773	0.415	100	"	"
	NH-3		100	99	96	58	72.7	98.4	0.712	0.285	100	"	"
	NH-4		100	99	96	58	126.2	106.0	0.589	0.131	100	"	"
	EBT-1	East Boston Till (3/4" max. size)	80	72	56	44	109.1	110.0	0.565	0.13	100	23	7
	EBT-2		80	72	56	44	145.0	120.0	0.435	0.0046	100	"	"
	EBT-3		80	72	56	44	95.3	125.6	0.371	0.00093	100	"	"
	EBT-4		80	72	56	44	47.7	130.0	0.324	0.00028	100	"	"
	LF-1	Ladd Field, Alaska, Silt Subsoil	100	100	90	37	7.8	83.8	1.040	2.1	98	32	0
	LF-2		100	100	90	37	11.2	90.0	0.899	1.2	96	"	"
	LF-3		100	100	90	37	25.5	94.4	0.811	0.86	100	"	"
	LF-4		100	100	90	37	36.5	98.4	0.737	0.64	100	"	"
	LSG-5	Limestone AFB Sandy Gravel (3/4" max. size, graded to contain approximately 3% finer than 0.02 mm.)	28	9	3	2	13.9	123.0	0.374	-	91	Non-Plastic	
	LSG-6		30	10	4	3	8.3	130.0	0.300	-	98	"	"
	LSG-7		32	11	4	3	18.3	136.7	0.237	-	100	"	"
	LSG-8		33	12	5	4	14.6	136.7	0.237	-	97	"	"
	TD-7	Truax AFB Silty Gravelly Sand (3/4" max. size)	88	78	35	21	7.4	119.3	0.423	-	91	14	2
	TD-8		88	78	35	21	13.0	125.6	0.350	-	90	"	"
	TD-9		88	78	35	21	22.0	130.0	0.303	-	100	"	"
	TD-10		88	78	35	21	14.7	134.0	0.265	-	98	"	"
	MFS-13	Manchester Fine Sand Blended with East Boston Till	100	99.5	7	4	3.8	106.4	0.573	7.1	100	Non-Plastic	
	MFS-14		100	99.5	13	6	3.4	105.0	0.594	9.2	100	"	"
	MFS-15		100	99.5	14	6	5.7	110.0	0.520	2.08	100	"	"
	MFS-16		100	99.5	14	7	6.5	110.6	0.513	2.30	100	"	"
Effect of Size and Per Cent Stone (Open System)	LSG-9	Limestone Sandy Gravel (Graded to contain approx. 3% 0.02 mm. with max. sizes as shown) 2"	27	9	4	3	10.7	137.3	0.230	-	100	Non-Plastic	
	LSG-10	1"	27	9	4	3	14.0	135.4	0.248	-	93	"	"
	LSG-11	3/4"	32	12	5	4	13.5	135.2	0.250	-	100	"	"
	LSG-12	1/4"	40	15	7	4	22.3	133.7	0.264	-	95	"	"
	LSG-13	Limestone Sandy Gravel (Samples "scalped" to max. sizes as shown) 2"	24	9	6	5	33.1	134.5	0.256	-	100	24	6
	LSG-14	1"	31	12	7	6	32.[illegible]	133.9	0.263	-	96	"	"
	LSG-15	3/4"	37	14	9	7	17.4	135.0	0.254	-	92	"	"
	LSG-16	1/4"	55	23	15	13	56.7	133.9	0.265	-	99	"	"
	TD-11	Truax AFB Silty Gravelly Sand (Coarse aggregate added to give max. sizes as shown) 2"	48	42	18	10	3.8	130.0	0.310	-	92	14	2
	TD-12	1"	65	56	24	14	9.8	130.5	0.302	-	100	"	"
	TD-13	3/4"	80	69	30	17	10.8	129.2	0.313	-	85	"	"
	TD-14	1/4"	96	83	36	20	16.6	128.2	0.319	-	86	"	"
Ice Segregation in Saturated Clay in a Closed System	BC-1	Undisturbed Boston Blue Clay	100	100	100	84	9.5	83.6	1.030	-	100	43	22
	BC-2		100	100	100	84	11.6	83.3	1.037	-	100	"	"
	BC-5		100	100	100	84	8.1	79.4	1.138	-	100	"	"
	BC-6		100	100	100	84	14.3	79.1	1.146	-	100	"	"
Effect of Surcharge (Open System)	NH-13	New Hampshire Silt (Surcharge as shown, lbs/in²) 0	100	99	96	58	155.1	104.7	0.609	0.147	100	27	0
	NH-14	1	100	99	96	58	138.4	105.0	0.605	0.145	100	"	"
	NH-15	2	100	99	96	58	76.7	105.6	0.595	0.135	100	"	"
	NH-16	3	100	99	96	58	50.0	105.6	0.595	0.135	100	"	"

*This table presents data available up to 1 July 1950. Additional tests are in progress.

(1) U.S. Standard Sieve No. 40
(2) U.S. Standard Sieve No. 200
(3) Based on original height of frozen portion
(4) Ratio of volume of voids to volume of solids
(5) Tests made on material passing the U. S. Standard No. 40 Sieve

"Data from tests on four specimens of sandy gravel from Limestone, Maine (samples LSG-9 to LSG-12 inclusive) show decrease in heave with increase in maximum size stone from 1/4 in. to 2 in. in diameter, the percentages finer than 2.0 mm. ranging from 27 to 40 and the percentages finer than 0.02 mm. equal to 3 and 4. The data obtained from tests of four specimens of pit-run sandy gravel from Limestone, Maine, (samples LSG-13 through LSG-16), wherein the maximum stone sizes were decreased (scalped), each time allowing the percentage of fines to increase, were inconsistent. However, tests results on four specimens of Truax Drumlin material (samples TD-11 through TD-14) show progressive increase in percent heave with decrease in maximum size and percentage of stone".

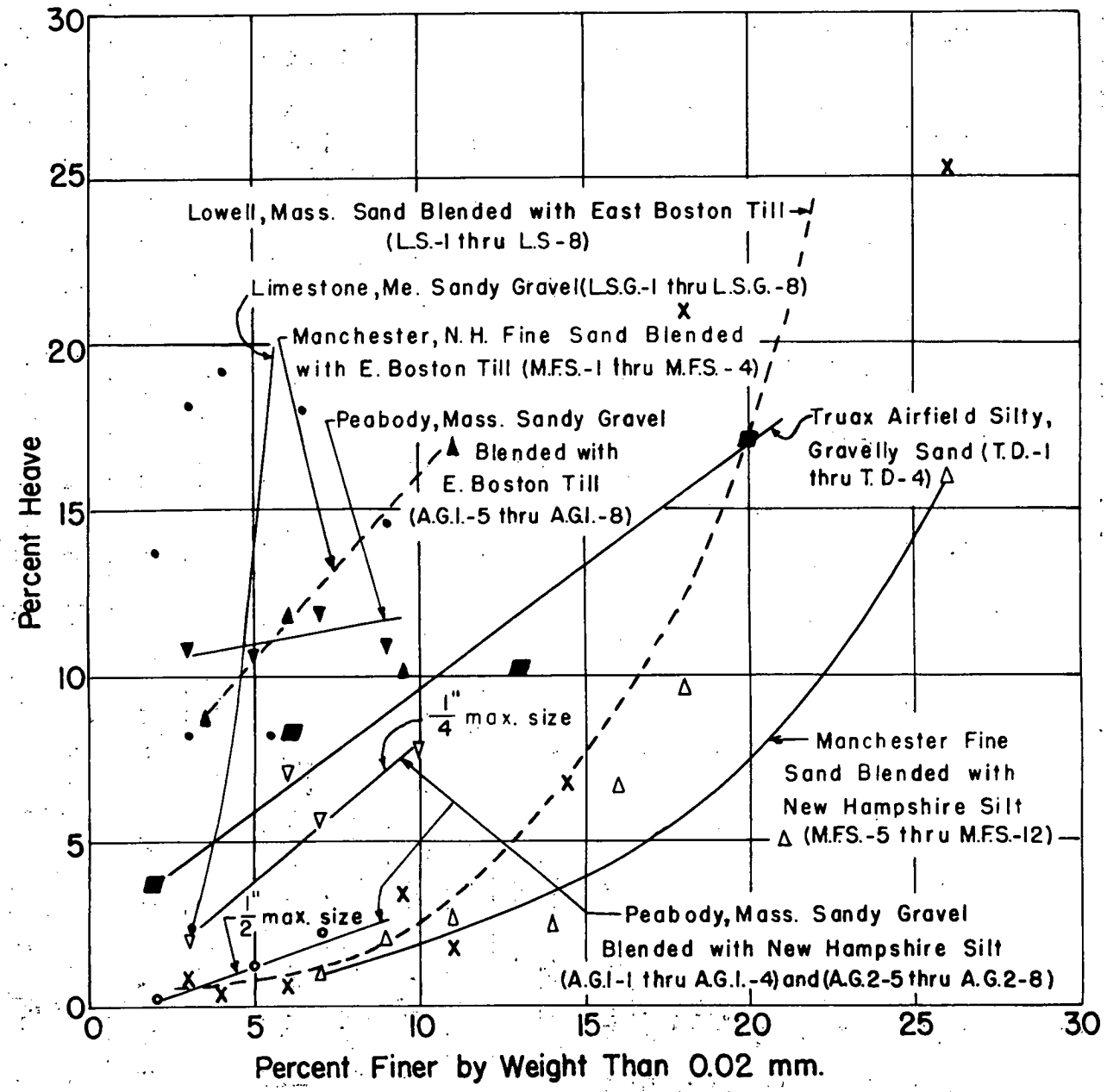


Figure 48
Effect of Percent Finer than 0.02 mm. Size on Percent Heave. (After Corps of Engineers)

Rogers and Nikola /1951-38 investigated 30 New Jersey soils to determine their reaction to weather conditions during the winter of 1949-1950. The 30 soils included derivatives from shale, basalt and gneiss; and transported soils included those of marine, lacustrine and glacial origin and are representative of the coastal plain and appalachian provinces of New Jersey. They represent soils covering about 3/4 of the State area. The results of soil tests are shown in Table 16.

TABLE 16

SUMMARY OF SOIL TEST DATA

Sample No.	Agronomic Name (as mapped 1917-27)	Test Results: Sieve Analysis, Cumulative Percent Passing					Hyd. Anal.		Physical, Minus #40		Proctor		HRB Designation	
		3/4	4	10	40	200	Silt Sizes %	Clay Sizes %	L.L. %	P.I. %	Max. Dens. p.c.f.	Opt. m.c. %	Sub-grade Group	Group Index
F-1	Penn	94	76	63	46	35	16	19	31	7	106	17	A-2-4	0
F-2	Wethersfield	94	86	82	64	43	19	23	32	16	119	13	A-6	3
F-3	Dunellen	100	98	95	76	27	—	—	16	0	120	12	A-2-4	0
F-4	Gloucester	100	90	86	79	56	31	21	25	6	109	16	A-4	4
F-5	Whippany	100	100	100	98	83	43	37	41	7	100	22	A-5	8
F-6	Sassafras	99	95	93	79	42	20	21	28	12	177	14	A-6	2
F-7	Sassafras	88	67	61	28	7	—	—	N.L.	N.P.	120	12	A-1-b	0
F-8	Sassafras	100	100	98	78	4	—	—	N.L.	N.P.	106	15	A-3	0
F-9	Hagerstown	100	99	98	92	83	40	34	43	20	101	20	A-7-6	13
F-10	Merrimac	100	90	77	41	11	—	—	N.L.	N.P.	125	9	A-1-b	0
F-11	Chester	89	74	70	55	46	26	16	33	11	109	18	A-6	2
F-12	Elkton	99	97	95	89	79	45	31	28	10	108	16	A-4	8
F-13	Montalto	63	49	36	23	12	6	5	32	9	114	17	A-2-4	0
F-14	Croton	97	80	73	68	64	23	27	41	21	100	21	A-7-6	15
F-15	Lansdale	99	87	85	69	55	21	32	41	15	95	26	A-7-6	6
F-16	Lakewood	100	100	100	73	1	—	—	N.L.	N.P.	102	15	A-3	0
F-17	Lakewood	100	99	98	64	3	—	—	N.L.	N.P.	106	14	A-3	0
F-18	Collington	100	100	100	80	26	10	15	32	8	105	23	A-2-4	0
F-19	Collington	96	91	87	69	39	12	18	48	14	97	27	A-7-5	2
F-20	Portsmouth	99	87	84	56	7	—	—	N.L.	N.P.	118	10	A-3	0
F-21	Holyoke	99	98	96	89	60	32	20	27	12	116	14	A-6	6
F-22	Washington	93	88	85	76	64	25	36	31	10	104	18	A-4	6
F-23	Dutchess	93	84	72	61	52	26	18	31	9	110	15	A-4	3
F-24	Dover	82	72	66	54	37	20	14	31	9	112	16	A-4	0
F-25	Sub-base Sand Hills	97	96	93	54	6	—	—	N.L.	N.P.	106	15	A-3	0
F-26	Sub-base Farrington	93	86	78	36	10	—	—	N.L.	N.P.	120	12	A-1-b	0
F-27	Sub-base Nixon	85	48	40	24	10	3	6	25	7	122	12	A-2-4	0
F-28	Sub-base Perrineville	94	78	71	41	2	—	—	N.L.	N.P.	108	16	A-1-b	0
F-29	Sub-base Bot. Jamesburg	87	35	26	12	3	—	—	N.L.	N.P.	123	10	A-1-a	0
F-30	Sub-base Top Jamesburg	89	66	48	17	4	—	—	N.L.	N.P.	119	13	A-1-a	0

The soils were compacted in 6-in. layers in a trench about 2 ft. deep and 9 ft. square on which was placed a 4-ft. square concrete slab 6 in. thick, which were provided with brass plugs at corners for purposes of taking elevations. Four 1½-in. diameter brass-lined tubes were molded in the slabs at the quarter points of each diagonal. Monel clad (1.3-in. diam.) steel plungers (sealed with grease) were placed through the holes to bear on the subgrade. The four plungers were loaded with weights to develop contact pressures of 25, 50, 75 and 100 psi. respectively. Figure 49 shows nine of the 30 installations. A record was made of maximum and minimum daily air temperatures as shown in Figure 50. A graphic record of penetration readings as indicated in Figure 51 was made for each of the 30 locations.

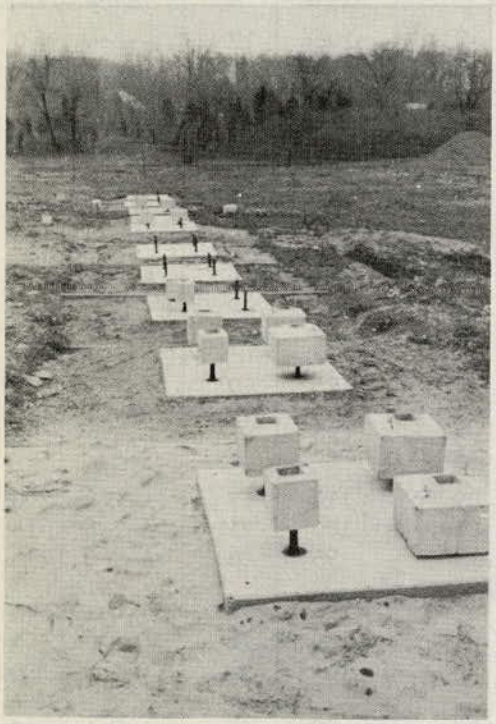


Figure 49. Arrangement of Field Installations (After Rogers and Nikola)

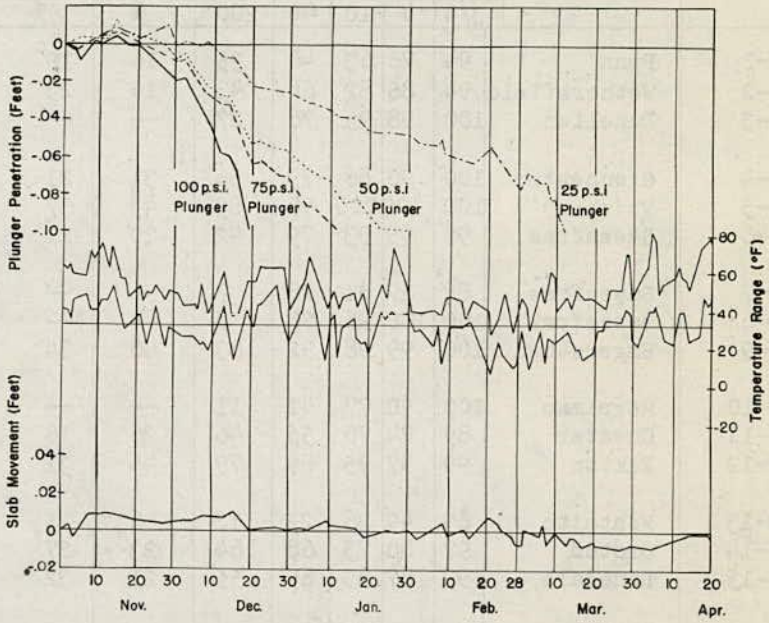


Figure 50. Typical Graphic Record Soil F - 10. (After Rogers and Nikola)

Figures 51, 52 and 53 show the duration of the weighted plunger application for the 30 installations. The length of operation of the plunger has been indicated by appropriate hatchures as shown in Figure 51. These three figures demonstrate the superiority of granular materials in resisting frost action and reduction in supporting value on thawing. Seven of the eleven materials classed as A-1-a, A-1-b, and A-3 continued to support one or more plungers for the duration of the test period.

The most erratic performances were those of the A-2-4 class because of the wide range of gradings in that class.

Although the New Jersey winter of 1949-1950 was a mild one, the authors felt they could draw the following conclusions from their work.

1. Soils and soil material in the A-3 class, consisting essentially of fine sands, when confined, show little loss of bearing power in the lower range of the contact pressures when subjected to freeze-thaw conditions. Little heaving occurs in these soils when subjected to freezing weather.

2. Silty materials, designated as A-4, appear to be the worst reactors; both in heaving characteristics and loss of bearing power.

3. Granular material, classified as A-1-a and A-1-b, show good support, especially for the lower contact pressures under conditions of freezing and thawing. Little heaving was measured when these soils were subjected to freezing weather.

4. Clayey soils and soil material exhibit varied reactions as to loss of bearing power under freeze-thaw conditions. In consideration of the relatively mild winter, these soils showed considerable heaving under freezing conditions. This study was continued through the winter (1950-1951), when subsurface temperatures were obtained and movements of plungers and slabs recorded.

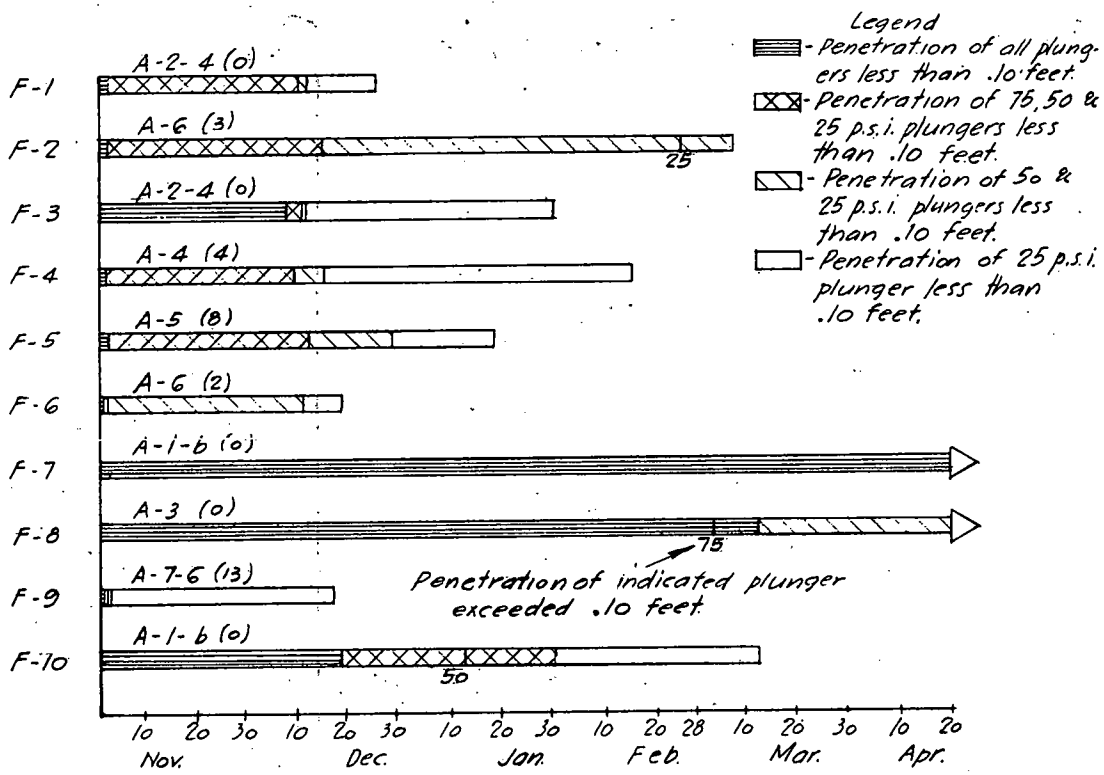


Figure 51. Duration of Weighted Plunger Operation - Soils F-1 thru F-10 (After Rogers and Nikola)

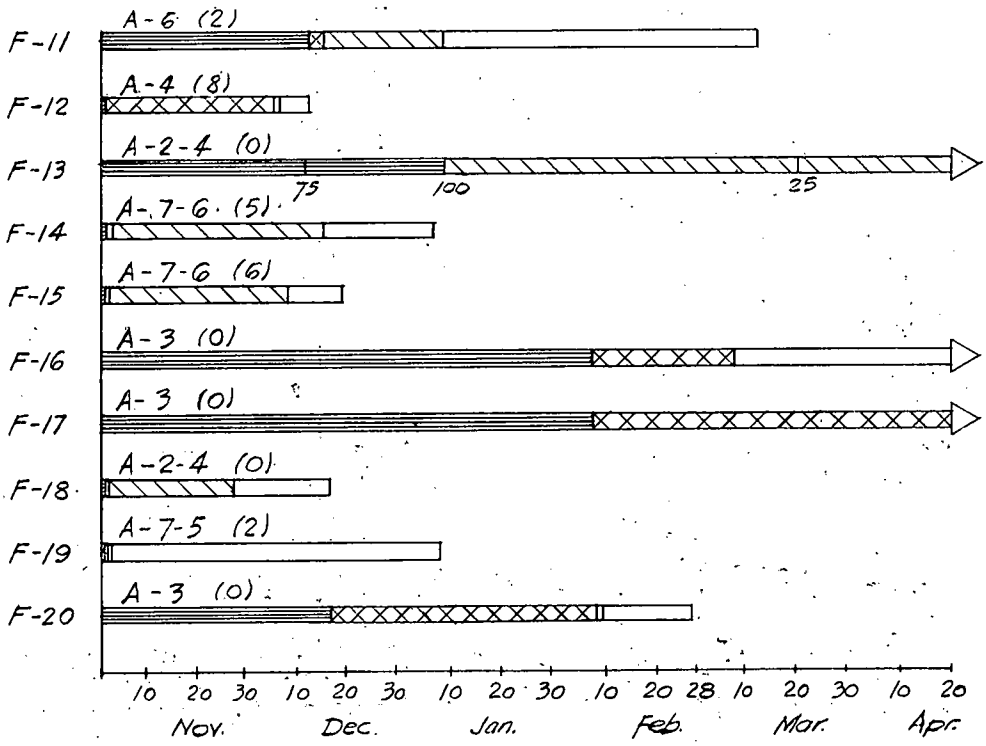


Figure 52. Duration of Weighted Plunger Operation - Soils F-11 thru F-20 (After Rogers and Nikola)

It may be seen that different investigators are in reasonably close agreement on limiting grain sizes for non-frost heaving soils. The reviewer has attempted to show graphically (Figure 54) the limits proposed by some investigators. Each of those investigators who has proposed limits has defined the nature of frost heave which occurs for the soil limits given.

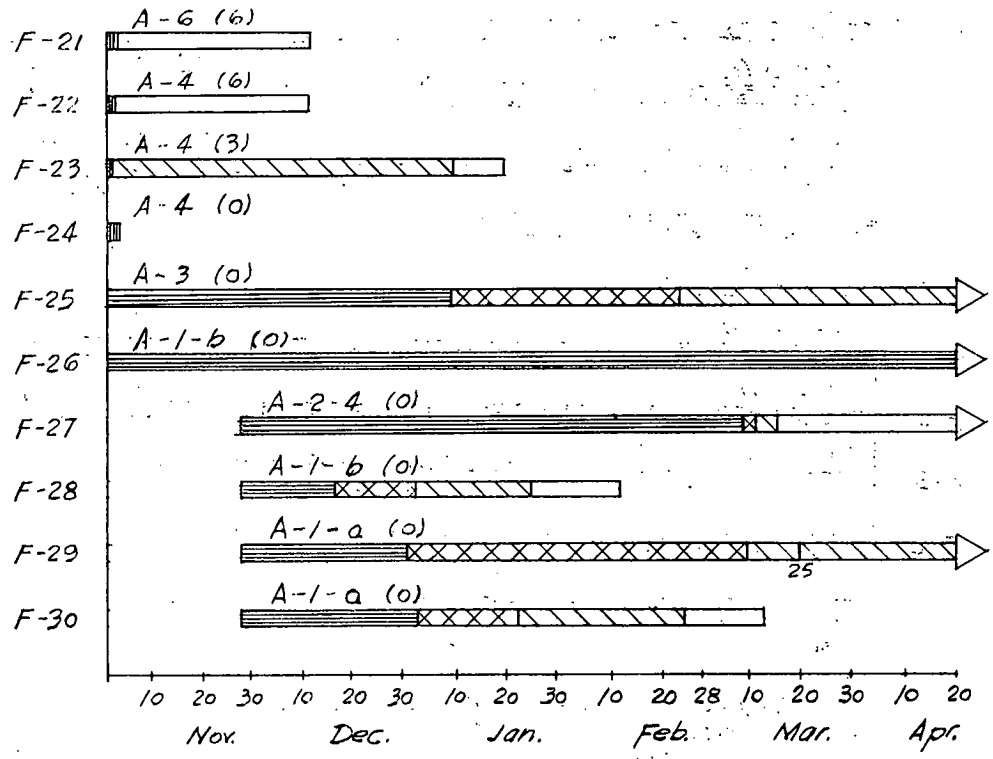


Figure 53. Duration of Weighted Plunger Operation - Soils F-21 thru F-30. (After Rogers and Nikola)

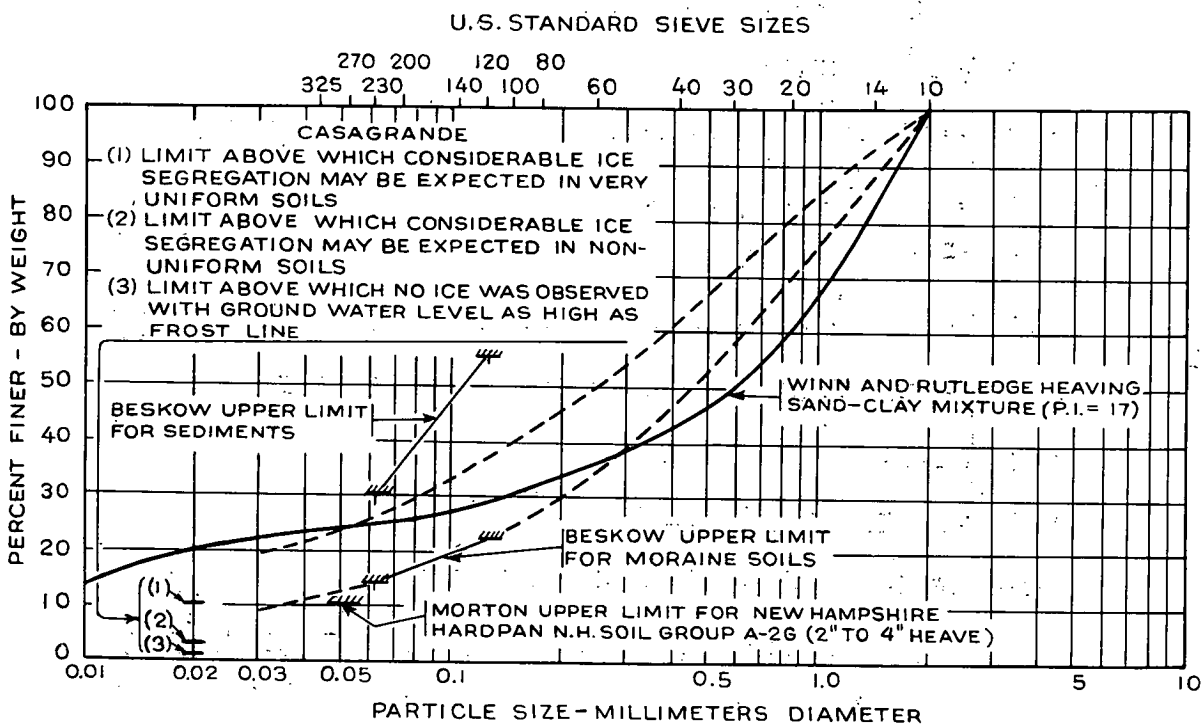


Figure 54. Comparison of Grain Size Limits for Frost-heaving and Non-frost-heaving Soils by Different Investigators

Beskow gives an average particle diameter of 0.1 mm. as the maximum size that will permit ice segregation under any conditions. A. Casagrande gives the critical size for ice segregation as 0.02 mm. for actual soils. Taber found only the faintest segregation in ground quartz having a grain size of 0.07 mm. Some investigators found that grading was also a factor. Casagrande found uniformity of grading important, allowing up to 10 percent finer than 0.02 mm. size if the soils were of a uniform grain size thus permitting better drainage. Smith found deficiencies in grading were a factor in stability on thawing. Also some investigators have pointed to the increase in soil water content which is associated with freezing, while others have shown that marked reduction in bearing capacity may accompany the moisture increase or the change in the state of the soil water.

Relationship Between Soil Classification Groups and Frost Action - A disturbed soil may be classified on the basis of one or more characteristics of its constituents, for example on the basis of its texture, that is grain size distribution. A soil may also be classified according to the physical properties of the soil mass, that is, its resistance to load deformation, its permeability etc., or its susceptibility to frost action. The basis for classification is usually the results of tests (liquid limit, plastic limit, etc.) which are indicative of physical properties. The soil may also be classified on the basis of the in-place characteristics of the soil profile as by the pedological method (sometimes called the agricultural method) or on the basis of the history of the soil deposit or parent material as by the geologic method.

With the exception of grain size distribution only limited emphasis has been placed on the relationship between soil classification and frost action. The relationship between soil texture and frost action have been brought out in the preceding review on "Influence of Grain Size Distribution". Hogentogler /1931-6 in listing performance characteristics of the "Uniform Subgrade Groups of the 1931 Public Roads Classification System", cites two groups as susceptible to frost action. They are:

"Group A-4...Likely to cause cracking in rigid pavement as a result of frost heaving, and failure in flexible pavements because of low supporting value.

Group A-5...similar to Group A-4...".

Morton /1936-5 found that the frost conditions in New Hampshire necessitated a revision in the grouping as established by the Bureau of Public Roads. He found need for further subdivision of the A-2 group on the basis of its frost action characteristics. The various groups and their frost heaving characteristics for New Hampshire were given as follows by Morton:

Group A-2G. Graded material that contains 10 percent or less, soil with a grain size smaller than 0.05 mm.; light frost action (2-4 in. heaving). Soils known locally as sand or gravel hardpans represent this group.

Group A-2F. Graded material that contains between 10 percent and 25 percent soil, with a grain size smaller than 0.05 mm. Frost action to the extent of 4-6 in. heaving. Soils known locally as silt hardpans represent this group.

Group A-2P. Graded material that contains over 25 percent soil with a grain size smaller than 0.05 mm. Excessive frost action (6-9 in. heaving). Soils known locally as clay hardpan or boulder clay represent this group.

Group A-3. Coarse materials only. Cohesionless sands and gravels. Free from frost action. Porous, will not support a ground-water table. Readily adjust themselves when used in deep fill sections. Excellent support for rigid pavements.

Group A-4. Silt without coarse material. High and rapid capillary action. Excessive frost action (8-14 in. heaving). Free water as a groundwater table not encountered. Underdrains constructed at standard depths (4-5 ft. below finished grade) are not effective in reducing frost action. Long period of readjustment necessary when used as fill in deep sections.

Group A-6. Clay without coarse material. Capillary action not as rapid as that encountered in silt group. 6-9 in. frost heaving. Free water as a groundwater table not encountered. Underdrains constructed at standard depths are ineffective in reducing frost action. Long period of readjustment necessary when used as fill in deep sections.

Group A-8. Muck and peat. Highly unstable. High organic and vegetable content. Capable of displacement or excessive settlement.

The Casagrande method of soil classification /1931-14, /1947-37, later called the airfield system has recognized qualitatively six degrees of potential frost action in soils. Those are: (1) none, (2) very slight, (3) slight, (4) medium, (5) high, and, (6) very high. The relative range of frost potential for each of the airfield soil groups is shown in Table 17.

The Civil Aeronautics Administration /1945-24 combined drainage condition with absence or presence of frost and set up four possible conditions which are applied to each of its 13 subgrade soil groups in determining pavement designs. The four conditions are; (1) no frost, good drainage, (2) severe frost, good drainage, (3) no frost, poor drainage, and, (4) severe frost, poor drainage. Because the C.A.A. Classification system is a part of its design method it is explained later under " Design Methods for Preventing Detrimental Frost Action", with regard to both flexible and rigid type pavements.

Livingston /1951-34 reports intensive frost action occurring in high mountain valleys of Colorado where the water table exists near the ground surface, winter temperatures are as low as -40 to -50 deg. F. with frost penetration as deep as 5 ft. and where surface drainage is poor due to the relatively flat grades (about 0.3 percent). The San Luis and Yampa Valley locations are exemplary. The type of soils which occur in the valley and in high mountain passes where intensive frost action takes place are indicated by the test values and soil grouping given in Table 18.

TABLE 18

SOIL TYPES SUBJECT TO INTENSIVE FROST ACTION IN COLORADO

Area	Soil Group	L.L.	P.I.	% Passing No. 200 Sieve	CBR
San Luis Valley	A 2-4 (0)	20-25	0-5	30-35	28-30
	A 4 (0)	27	8	36	29
	A 7-5 (20)	65	32	90	2
Yampa Valley	A 2-4 (0)	35	NP	24	42
	A 6 (12)	38	20	85	2
	A 7-6 (15)	45	20	98	2
Wolf Creek Pass	A 2-5 (0)	45	8	16	36
	A 2-7 (2)	55	29	30	10
	A 7-5 (15)	68	25	63	2

Rogers and Nikola /1951-38 investigated the effects of frost action on 30 New Jersey soils falling into the A-1-a, A-1-b, A-2-4, A-3, A-4, A-5, A-6, A-7-5, and A-7-6. The results of that investigation have been reviewed in some detail under "Influence of Grain Size Distribution".

Moisture Retention and Moisture Movement in Soil - There can be no detrimental frost action, as frost action is defined in this review, without movement of water to the zone of freezing. The process of water movement is fundamental to water gain and heaving on freezing, as well as to reduction in load carrying capacity following thawing of the frost. The process is not a simple one. Nor is there agreement on the nature of the process or the magnitude of the forces involved. It includes movement of water through a range from complete saturation to a condition of partial saturation where the quantity of moisture flow is not significant. Moisture flow is dependent on the nature of the soil, that is, its grain composition and shape and the grain size distribution, as they influence the amount and size and shape of soil pores. It is also dependent upon soil density and initial water content, and on soil structure because structure

TABLE 17

SOIL CLASSIFICATION FOR AIRFIELD PROJECTS

	MAJOR DIVISIONS	SOIL GROUPS & TYPICAL NAMES	GROUP SYMBOLS	PRINCIPAL CLASSIFICATION TESTS (ON DISTURBED SAMPLES)	POTENTIAL FROST ACTION
COARSE GRAINED SOILS	Gravel and Gravelly Soils	Well Graded Gravel & Gravel-Sand Mixtures, Little or No Fines	GW	Mechanical Analysis	None to Very Slight
		Well Graded Gravel-Sand-Clay Mixtures, Excellent Binder	GC	Mechanical Analysis Liquid & Plastic Limits on Binder	Medium
		Poorly Graded Gravel & Gravel-Sand Mixtures, Little or No Fines	GP	Mechanical Analysis	None to Very Slight
		Gravel with Fines, Very Silty Gravel, Clayey Gravel, Poorly Graded Gravel-Sand-Clay Mixtures	GF	Mechanical Analysis, Liquid & Plastic Limits on Binder if Applicable	Slight to Medium
	Sands and Sandy Soils	Well Graded Sands & Gravelly Sands, Little or No Fines	SW	Mechanical Analysis	None to Very Slight
		Well Graded Sand-Clay Mixtures, Excellent Binder	SC	Mechanical Analysis, Liquid & Plastic Limits on Binder	Medium
		Poorly Graded Sands, Little or No Fines	SP	Mechanical Analysis	None to Very Slight
		Sand with Fines, Very Silty Sands, Clayey Sands, Poorly Graded Sand-Clay Mixtures	SF	Mechanical Analysis Liquid & Plastic Limits on Binder if Applicable	Slight to High
FINE GRAINED SOILS Containing Little or No Coarse Grained Material	Fine Grained Soils Having Low to Medium Compressibility	Inorganic Silts & Very Fine Sands, Mo, Rock Flour, Silty or Clayey Fine Sands with Slight Plasticity	ML	Mechanical Analysis Liquid & Plastic Limits if Applicable	Medium to Very High
		Inorganic Clays of Low to Medium Plasticity, Sandy Clays, Silty Clays, Lean Clays	CL	Liquid & Plastic Limits	Medium to High
		Organic Silts & Organic Silt-Clays of Low Plasticity	OL	Liquid & Plastic Limits From Natural Condition & After Oven Drying	Medium to High
	Fine Grained Soils Having High Compressibility	Micaceous or Diatomaceous Fine Sandy & Silty Soils, Elastic Silts	MH	Mechanical Analysis Liquid & Plastic Limits if Applicable	Medium to Very High
		Inorganic Clays of High Plasticity, Fat Clays	CH	Liquid & Plastic Limits	Medium
		Organic Clays of Medium to High Plasticity	OH	Liquid & Plastic Limits From Natural Condition & After Oven Drying	Medium
	Fibrous Organic Soils with Very High Compressibility	Peat and Other Highly Organic Swamp Soils	Pt	Natural Condition	Slight

LEGEND FOR GROUP SYMBOLS

G - Gravel
S - Sand
M - Mo, Very Find Sand, Silt, Rock Flour

C - Clay
Pt- Peat
F - Fines, Matl $<$ 0.1mm
O - Organic

W - Well Graded
P - Poorly Graded
L - Low to Med. Compressibility
H - High Compressibility

controls the size and spacing of fissures in the soil. Other conditions being equal, capillary water tends to move in the direction of heat transfer, thus soil temperature and temperature differences are also influencing factors.

The writings on movement of moisture in soil, with special reference to capillary movement, stem from engineers, agronomists, physicists, hydrologists, and others. The writers have contributed many experimental and observational data on capillary movement as related to their particular fields of endeavor. They have also presented theories, some of which differ in their basic concept, of the nature of the forces which cause water to move in the soil capillaries. The total literature which bears on the subject is voluminous. It includes no single treatment which satisfactorily explains movement under conditions ranging from complete saturation to partial saturation. Limitations of time, and space in this review make impracticable an exhaustive treatment of this fundamental part of the overall frost phenomenon. The best that can be done is to review the writings of a few recognized authorities who have contributed to the subject with emphasis on those which make specific reference to frost action.

Capillarity - Some researchers classified soil moisture into three main types according to its characteristics, including the manner in which it moves in soil. The three main types of soil moisture were: hygroscopic or film water, capillary water, and free or gravitational water. They regarded capillary movement of water in soil as being analogous to movement in a capillary tube and considered that surface tension furnished the activating force for movement.

Many investigators have held the concept that surfaces of liquids exist in a state of tension, although they agree there is no visible skin or film on the surface. The concept of surface tension is apparently accepted by some as a physical reality. To others it is merely a concept that the surface resembles, in some of its attributes, a film or skin under tension. Thus it is an anology which explains the behavior of water in soil and which can be used as a tool to evaluate the magnitude of capillary forces (pressures), the height of capillary rise, and the rate of rise in soils of different textures if appropriate coefficients are determined. An explanation of the surface tension concept can be found in many text books on soil mechanics (see Terzaghi /1943-14, Hogentogler /1937-14, Taylor /1948-62).

In order to visualize surface tension and the magnitude of forces involved as applied to capillary rise, the problem is simplified by considering that the soil pores simulate capillary tubes. Then the effect of surface tension in limiting height of rise of water in capillary tubes is as follows:

The upward pull of the liquid in a cylindrical capillary tube is π dST cosα. The downward pull (which is equal to the upward pull) is $\frac{\pi}{4}$ d^2hDg, where

d - diameter of the tube in cm.
ST - surface tension of water in dynes per cm.
α - angle of contact between liquid and wall of tube.
h - height of capillary rise in cm.
D - density of the liquid.
g - acceleration of gravity in dynes.

Then

$$h = \frac{4\ ST \cos\alpha}{d\ D\ g}$$

The surface tension of water is 75.6 dynes at zero deg. C hence water at that temperature would rise to a height of:

$$h = \frac{4 \times 75.6 \times 1}{0.1 \times 1 \times 980} = 3.09 \text{ cm.}$$

in a tube 0.1 cm. in diameter. (since cosα and D are approximately equal to 1 at equilibrium).

Winterkorn and Eyring /1945-19 (also see Terzaghi /1943-14 and Taylor /1948-62) further illustrated the nature of the forces involved in capillary movement in soil. For purposes of simplification they assumed full saturation to the top of capillary rise. The following relationship was developed:

$$\frac{dz}{dt} = \frac{k}{n}\left(\frac{h - z}{z}\right)$$

in which

dz - distance differential in vertical direction
dt - time differential
k - coefficient of permeability of soil
n - porosity of soil
h - capillary potential
z - distance of capillary meniscus from ground water level

From this was obtained:

$$t = \frac{nh}{k}\left(\log \frac{h}{h-z} - \frac{z}{h}\right)$$

Winterkorn derived experimental data on a K-Putnam soil, and computed time-capillary rise values from which he constructed the values given in Figure 55.

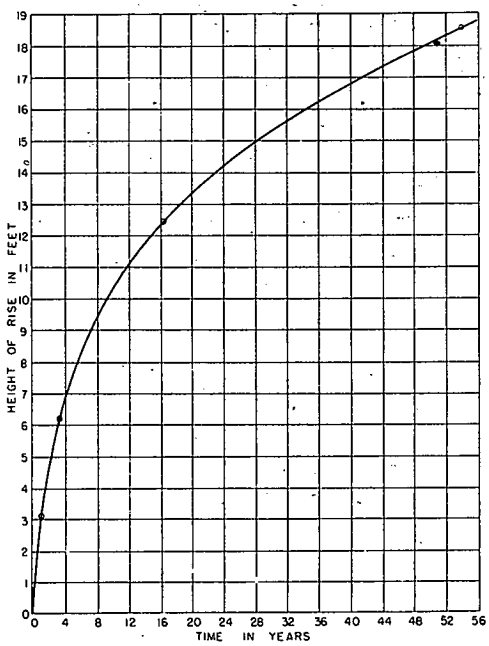


Figure 55. Rate of Capillary rise for K - Putnam soil (After Winterkorn and Eyring)

If the reader wishes to study the works of investigators who held to the capillary tube-surface tension concept, he is directed to the excellent contributions of Briggs /1897-2, Bouyoucos /1921-3, Lebedeff /1927-3 and Zunker /1933-8.

Concurrent with the development of the capillary tube-surface tension analogy, some investigators believed that the capacity which a soil had for retaining moisture could be expressed in terms of energy, that is, its ability to do work. Buckingham /1907-2 believed that the movement of moisture in soil was analogous in some respects, to the flow of electricity or to the flow of heat through a conductor. The analogy was based on the concept that in each case flow was from a region of higher to a region of lower potential, the potential being pressure. Buckingham called this pressure potential the "capillary potential". Stated simply it is merely a measure of the attraction of the soil for water. As actually measured it is the work required to pull a unit mass of water away from a unit mass of soil. Russell and Spangler /1941-12 presented the capillary potential point of view to highway engineers.

The capillary potential at any depth in a soil may be determined by measuring the pressure deficiency or the tension in the water at that point and dividing it by the unit weight of water. This pressure deficiency represents the amount by which the pressure in the soil water is less than atmospheric pressure. Russell and Spangler showed that the capillary potential in soils can be measured in situ by the use of tensiometers so long as the negative pressure does not exceed one atmosphere. Actually most flow of moisture occurs at pressures less than $\frac{1}{2}$ atmosphere.

Russell and Spangler brought out that properties of soil water other than negative pressure can be used to evaluate capillary potential. Those include curvature of air-water interfaces in the unsaturated soil, water vapor pressure, and depression of the freezing point. They used these and other methods to determine the relationship between soil moisture content and capillary potential for four Iowa soils for ranges of moisture content from saturation to oven dryness. Their results are illustrated in Figure 56, which supports the currently held view that there exist no distinct classes of soil moisture.

Housel /1951-50 was among those who did not accept surface tension as a reality. He devised a mechanism for describing capillary rise to replace "hypothetical surface tension" and proceeded to demonstrate the validity of that method in satisfying the laws of static equilibrium. He held that a substitute mechanism must recognize the following which have been established by experiment: (1) the product (rh) known as the capillary constant (sometimes

called "specific cohesion"), (2) "...the weight of any liquid raised above the free surface by capillarity is a measure of the forces of capillarity and these forces must exist in the system whereby static equilibrium can be established...and (3)...capillarity has its origin in the presence of a solid surface in contact with a liquid and, therefore, must be a boundary effect inseparable from the solid-liquid surface".

He illustrated the manner in which static equilibrium in capillary rise is achieved in Figure 57. Part (a) shows the pressures acting on the body of the liquid above the free liquid surface. For purposes of illustration he selected a size of capillary such that the meniscus is tangent to the free liquid surface at the center where the capillary rise would be zero. Part (b) shows "... the equivalent pressures on the body of the liquid consisting of the internal pressure of the liquid (P_{GL}) acting upward and undiminished over the full cross section and the projected pressure from the unbalanced molecular attraction on the liquid surface of the meniscus acting downward. Due to the directional characteristics of the molecular attraction at the liquid surface, the projected pressure on a horizontal plane is diminished to the vertical component of the inclined pressure which varies from maximum at the center to minimum at the solid boundary".

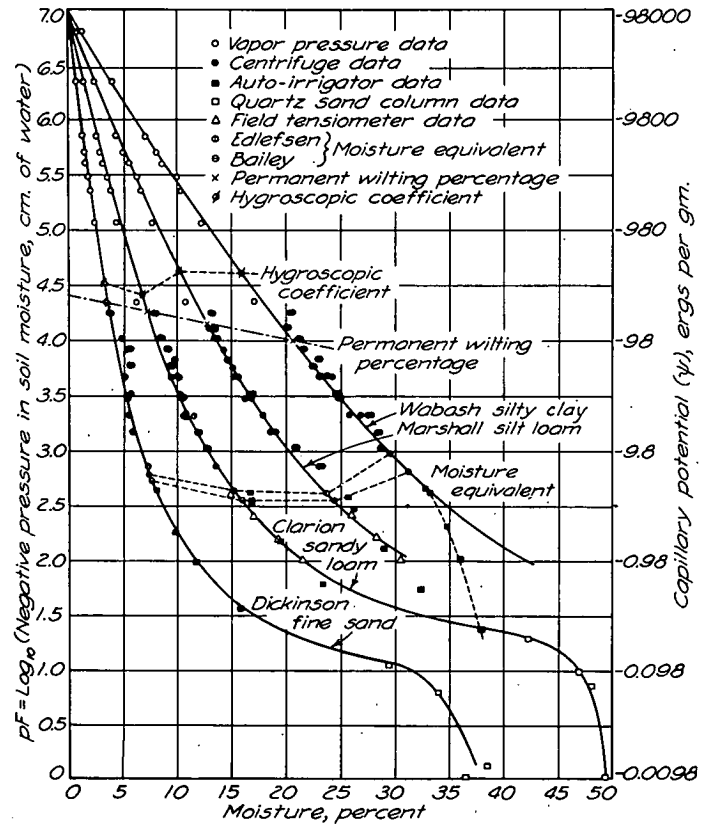


Figure 56. Soil Moisture Sorption Curves for Four Iowa Soils (After Russell and Spangler)

The meniscus is then curved upward in the vicinity of the solid boundary with vertical ordinates above the free liquid surface as prescribed in the following equation:

$$y = \frac{P_{GL}\ (1 - \sin \alpha)}{wg}$$

Where y is the distance to which water is elevated above the free liquid surface, g the acceleration of gravity and w the density of water. This ordinate becomes a maximum at the solid boundary. "The area under this curve above the free liquid surface shown cross-hatched in Figure 57 represents the ability of the system of forces originating in the presence of the solid boundary to sustain or support a volume of liquid in static equilibrium above the free liquid surface. In circular capillary tubes this area is a cross section of a volume of revolution representing the total volume of liquid sustained above the free liquid surface. If Figure 57 were considered as two plates of indefinite length perpendicular to the section shown, the area referred to would be directly equivalent to the volume of liquid above the free liquid surface per unit length of solid boundary.

"In such terms the capillary rise may be described as shown in part (c) of Figure 57 in which the volume of liquid supported by the internal pressure is represented as a boundary effect characteristic of the solid surface for the given liquid. It should be emphasized that the boundary effect, W_c if expressed as grams per centimeter or F_c if expressed in dynes per centimeter, is a function of the integrated volume of supported liquid which is a constant for any given combination of liquid and solid. It includes the integrated effect of both the internal pressure arising from the unbalanced molecular attraction P_{GL} and the contact angle θ in ..." the equation above.

"When this effect is applied to the closed boundary or circumference of a capillary tube the total weight raised above the free surface may be equated to an equivalent capillary pressure. Expressing this relation in force units leads to the following equation:

$$P_{GL}\pi r^2 = W_c g\ 2\pi r = F_c 2\pi r$$

$$P_{GL} = \frac{2F_c}{r}$$

"In capillary tubes of small diameter the capillary pressure created by the attraction of the solid walls of the tube may be equated to hydrostatic pressure wgh where the radius of curvature r is negligible with respect to the capillary rise (h).

$$P_{GL} = wgh = \frac{2W_c g}{r} = \frac{2F_c}{r}$$

$$rh = \frac{2W_c}{w} = \frac{2F_c}{wg}$$

taken as water having unit density, this boundary effect may be evaluated in grams per centimeter of perimeter W_c or in force units of dynes per centimeter F_c as is usually the case.

$$W_c = \frac{rhw}{2} = \frac{0.1488 \times 1}{2} = 0.0744 \text{ g.per cm.}$$

$$F_c = W_c g = \frac{rhwg}{2} = 0.0744 \times 980 = 72.8 \text{ dynes per cm.}$$

"With a mechanism described by which water may be supported above the free water level without use of the surface tension analogy, there remains only to express the final relationships in a formula for capillary rise, which follows directly from the equation."

$$h = \frac{2F_c}{rgw} = \frac{2W_c}{rw}$$

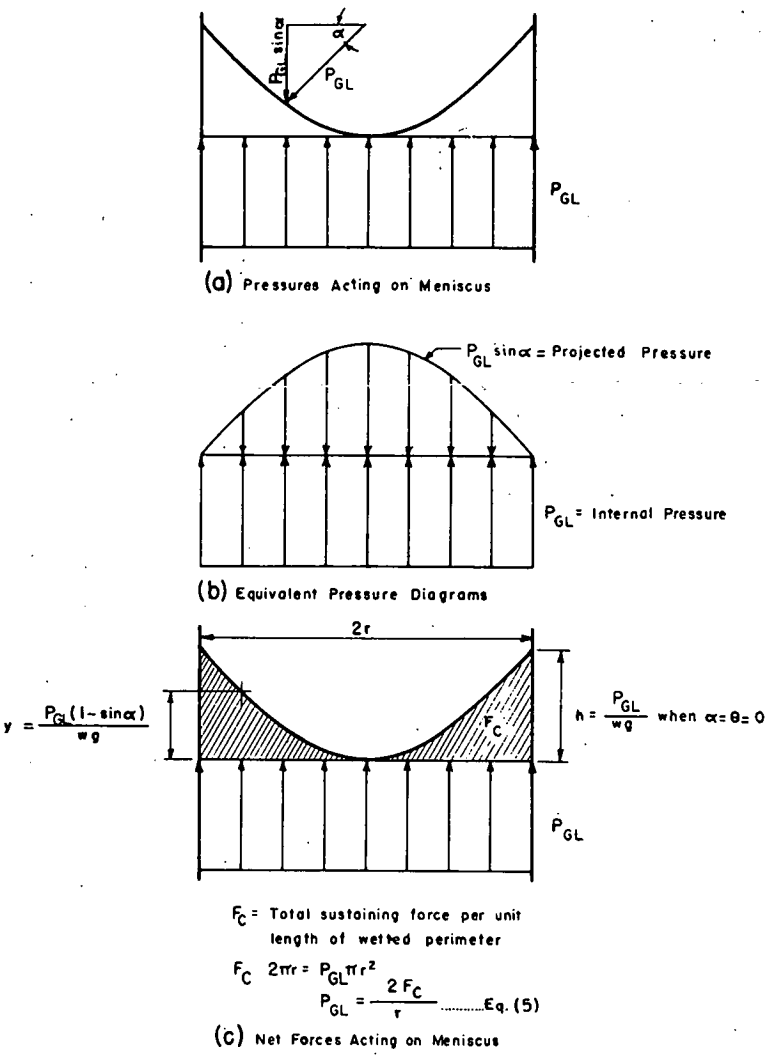


Figure 57. Static Equalibrium in Capillary Rise. (After Housel)

The above constitutes a brief and incomplete review of principal concepts of forces operative in bringing about so called "capillary" (antigravitational) flow of moisture in soil. It may now be of interest to present some experimental data which have been obtained from efforts to measure capillary rise characteristics in soils.

Briggs and Lapham /1902-1 included data on capillary rise in both moist and dry soils. They tested three sandy Michigan soils having the following grain size distribution:

Percentage of Grains Finer Than Sizes Shown

Soil No.	2.0 - 0.05 mm.	0.05 - 0.01 mm.	Smaller than 0.01 mm.
1	97.6	0.8	0.1
2	96.3	1.2	0.3
3	88.0	7.3	0.7

The results of the capillary rise tests, giving capillary rise in cm. are as follows:

Soil 1		Soil 2		Soil 3	
Dry	Moist	Dry	Moist	Dry	Moist
31.8	112.6	58.1	141.8	86.8	174.1

The authors gave no data on degree of saturation or relative rates of rise.

Taber /1929-2, /1930-9 found that the height to which water rises varies inversely as the diameter of the capillaries and that in fine sand may be 10 ft. or more. Housel /1938-13 summarized the work of Loughridge (1892-1894) and Hogentogler on the relationship between particle size and capillary rise. The results are shown in Figure 58. Curve A of Figure 58 shows the theoretical capillary rise in 24 hours computed by Hogentogler /1937-14. Curves B and C show maximum and 24 hr. rise observed by Loughridge.

Valle-Rodas /1944-16 investigated capillary rise in sands. He supplemented his capillary rise measurements with determinations of relative capillarity as determined with a Beskow type capillarimeter (described later). Valle-Rodas brought out some interesting relationships between water content at various heights above free water and capillarimeter value (shown as "passive capillary height"). Those comparisons are shown in Figure 59. Valle-Rodas showed that the moisture content at the capillarimeter value increased as the grain size decreased, except for very fine sand.

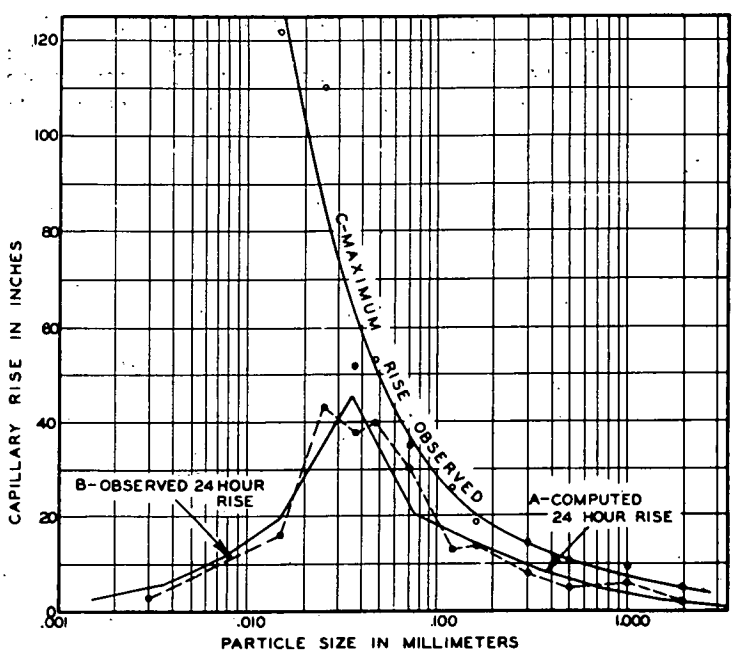


Figure 58. Capillary Rise in Various Textural Classes of Soil. (After Housel /1938-13)

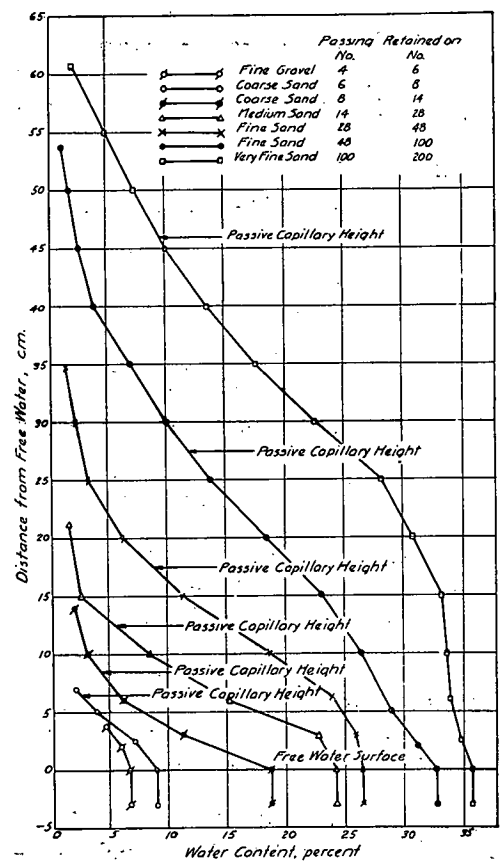


Figure 59. Distribution of Capillary Water (After Valle-Rodas)

The work of Lane and Washburn /1946-8 is of interest because they investigated the capillary rise characteristics of eight soils, observed both capillary rise and capillarimeter values (with a modified (larger) Beskow type capillarimeter), and because they controlled density in their test samples. The grain size distribution of the soils investigated are indicated in Figure 60. The soils tested represent median gradations of the corresponding grain size bandsof the Providence District Soil Classification (see Fahlquist and Kenerson on "Providence Soil Classification", discussion, Proceedings, A.S.C.E., October 1939). Capillary rise tests were made in 2-and 4-in. diameter open-end soil filled Lucite tubes. Tests were made on air dried samples compacted in layers. Tests on materials of classes 1, 2, and 3 extended for periods of 3 to 6 days. Tests on the finer grained materials of classes 4 to 8 extended over periods ranging from 70 to over 400 days. Results showing height of capillary rise as related to Hazen's effective size (the 10 percent size, d_{10}= the size where 10 percent passes and 90 percent is retained) are shown in Figure 61. It may be seen that there is a gradual increase in difference between capillary rise and capillarimeter values.

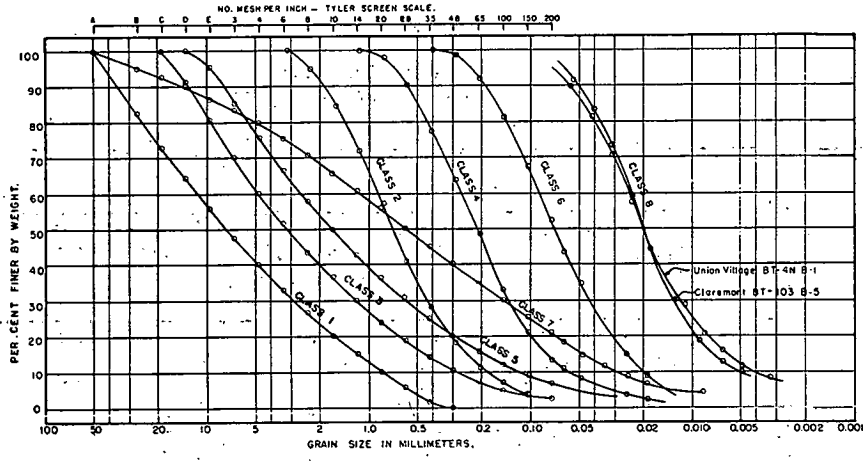


Figure 60. Gradation of Samples Tested
(After Lane and Washburn)

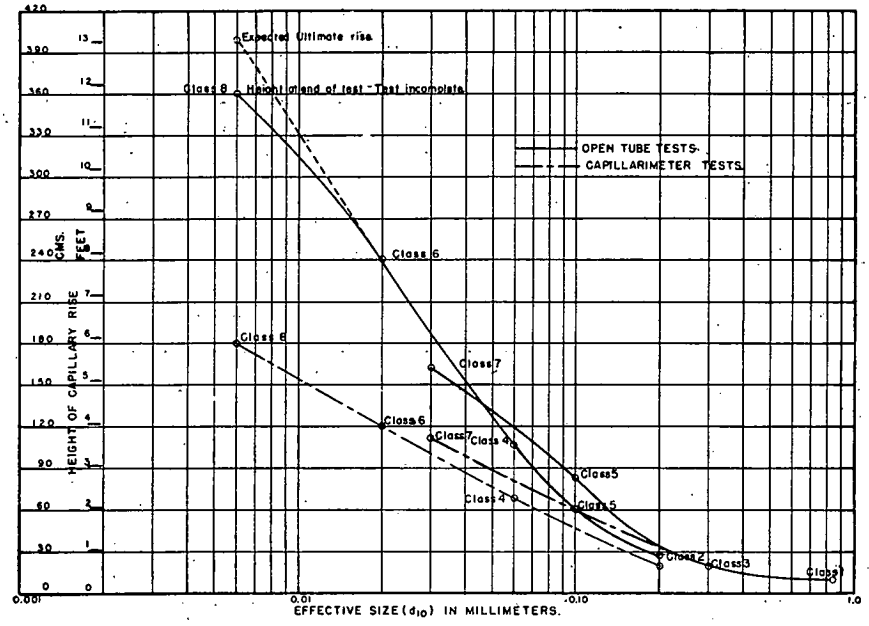


Figure 61. Capillary Rise - Effective Size Curves
(After Lane and Washburn)

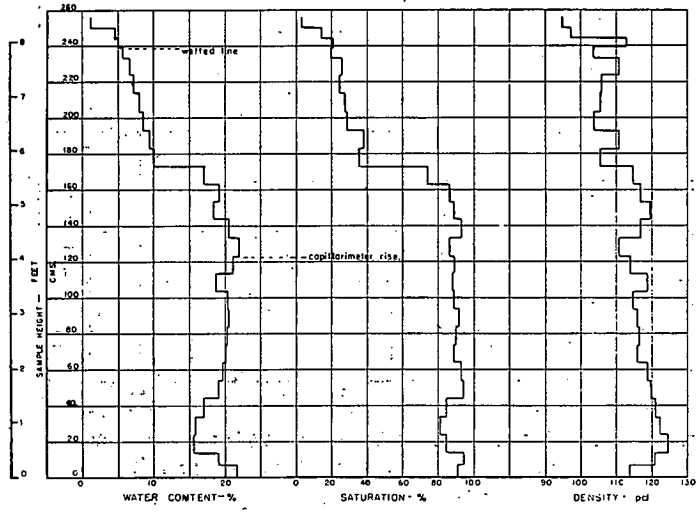


Figure 62. Water Content, Saturation, and Dry
Density at End of Test - Class 6
(After Lane & Washburn)

Complete test results including permeability values are shown in Table 19.

TABLE 19

CAPILLARITY TEST RESULTS
(After Lane and Washburn)

Sample Class No.	Specific Gravity	Effective Size-d_{10}	Capillarimeter Tests				Open Tube Tests			
			Void Ratio[a] e	Dry Density	Capillary Rise	Permeability[b] k X 10^4	Void Ratio[a] e	Dry Density	Capillary Rise	Permeability[b] k X 10^4
		mm.		pcf	cm.	cm. per sec.		pcf.	cm.	cm. per sec.
1	2.70	0.82	0.27	132.2	6.0	1100	0.27	132.2	5.4	1100
2	2.65	0.20	0.45	114.8	20.0	160	0.45	113.8	28.4	160
3	2.70	0.30	0.29	130.2	20.0	71	0.29	130.2	19.5	71
4	2.70	0.06	0.45	116.4	68.0	4.6	0.45	116.4	106.0	4.6
5	2.69	0.11	0.27	132.0	60.0	1.1	0.27	132.2	82.0	1.1
6	2.75	0.02	0.48	116.1	120.0	0.29	0.66	103.3	239.6	0.62
7	2.77	0.03	0.36	126.7	112.0	0.096	0.36	126.5	165.5	0.096
8	2.76	0.006	0.95	88.3	180.0	0.15	0.93	89.3	359.2	0.14

[a]Void ratio shown is overall value for entire sample.
[b]Permeability coefficient taken from k-e curves at overall void ratio shown.

Typical water content-saturation-dry density values for various depth increments in the capillary tube samples are shown for a silty fine sand in Figure 62 and for a sandy silt in Figure 63.

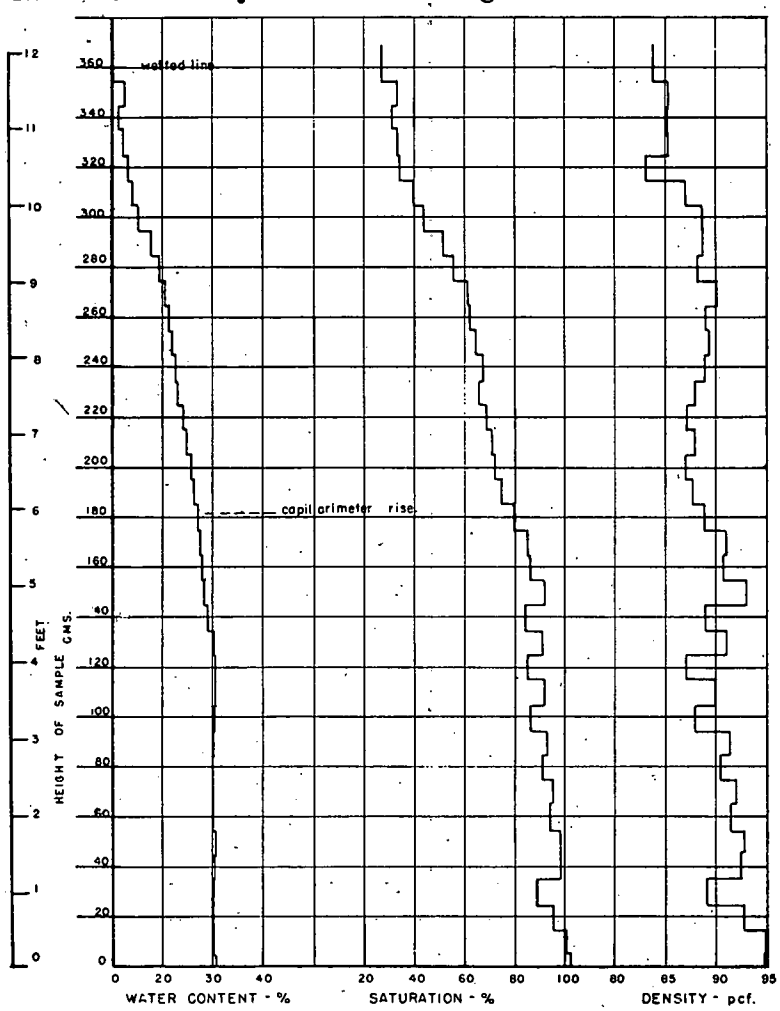


Figure 63. Water Content, Saturation, and Dry Density at End of Test-Class 8
(After Lane and Washburn)

Lane and Washburn considered that the capillarimeter value (which they termed the passive capillary rise) is controlled mainly by the large voids in the soil. They state "In these larger voids capillary tension is the least and, accordingly releases first, causing a break in the suspended water column. Contrastingly, the open tube method...determines the active capillary rise which is influenced by all the voids in the soil. Here the capillary tension is much greater as it is controlled by the smaller voids and accordingly causes a much higher rise." They found a marked reduction in percent saturation above a certain elevation for sandy soils and a relatively uniform decrease for the more silty soils (see Figures 62 and 63). They compared observed time-rise curves from tube tests with theoretical curves computed from the solution given by Terzaghi (see preceding equation). The solution is based on the assumption that complete saturation exists below a height z, and that e and k are at all times constant throughout the column. Figure 64 shows actual time vs rise for a class 4 sand. To the left is the computed curve (No. 1) calculated from the observed values of h_c, e, and k shown in Table 19. Neglecting small changes in e the theoretical curve can be matched with the observed curve at point $\frac{h_c}{2}$ by recomputing with an effective value of $k = 0.21 \times 10^{-4}$ cm. per sec.

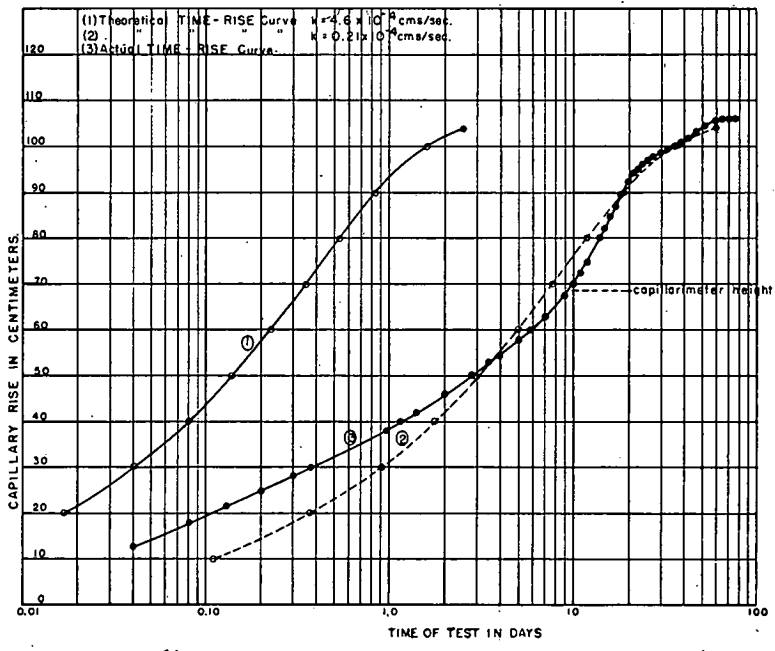


Figure 64. Capillary Rise-Time Curves-Class 4-
Actual and Theoretical Rise. (After Lane and Washburn)

Krynine /1946-20, conducted capillary rise tests on six gravels, one sand and two silts. His results agreed generally with those of Lane and Washburn. Krynine observed that preceding the line of visable wetting there was an accumulation of moisture attributed to condensation. The condensation phenomenon suggested to Krynine that water vapor precedes the boundary of visible wetting.

It is of interest to compare the work of Lane and Washburn with that of three early investigators of capillary rise, namely, Hazen /1893-1, Atterberg /1905-3 and Hilgard /1906-3. A comparison is shown in Figure 65 in which capillary rise is related to effective grain size. When it is considered that the work of Atterberg and of Hilgard was done on soil fractions, the results show reasonably close agreement. Lane and Washburn's capillarimeter values are also plotted for purposes of comparison.

Lambe /1950-13 held that much of the confusion which now exists concerning capillary rise is due to the oversimplification introduced by comparing capillary water in soils to that of water in a bundle of nonconnected capillary tubes possessing a single definite capillary head. He concluded from his investigations that several capillary heads are required for adequate representation. Figure 66 compares degree of saturation with height above elevation of free water for test IF, a drainage test on an initially saturated sample, and test VIA, a capillary rise test on an initially dry sample.

Lambe designates the distance from point A, on the drainage curve, to the free water surface as the maximum capillary head, h_{cx}, and the distance from point B which is the highest point at which complete saturation exists as saturation capillary head, h_{cs}. On the capillary rise test curve he designates the distance from the free water to the highest elevation to which capillary water rose (point C) the capillary rise, h_{cr}, and the distance to the highest elevation at which maximum degree of capillary saturation exists (point D) as h_{cn}. The value designated h_{ca} is a weighted average capillary head. The problem now is one of selecting the proper head to use in making computations. The value obtained with the Beskow capillarimeter corresponds roughly with the value h_{cs}, hence that value has practical use as one readily determined as will be shown later.

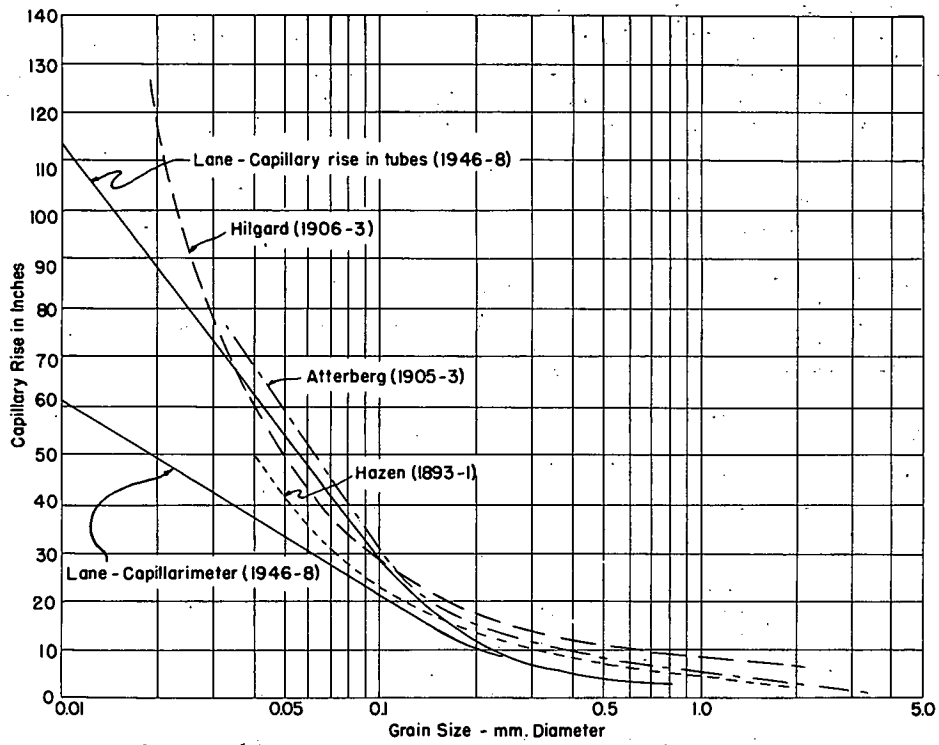


Figure 65. Diagram showing relation between capillary rise and effective grain size from experiments by Atterberg, Hazen, Hilgard and Lane. Atterbergs and Hilgards work pertain to soil fractions. Hazen's and Lane's work pertain to soils, the grain size being the "effective size" (Diameter in mm. of size where 10% passes and 90% is retained.)

Spangler and Pien /1951-23 showed that capillary potential of soil moisture (under isothermal and isobaric conditions) is dependent only on the height above a free water surface and is independent of texture, grain size, density, wetting angle, etc. However, the moisture content of soil at a given height is dependent upon all these factors. Therefore the moisture in different soils at the same height above a water table must adjust itself so that the capillary potential is a constant value corresponding to that height. Where capillary moisture in a stratified soil is in equilibrium with a water table, the moisture content just above and just below the interface between two soil strata may differ widely.

Spangler and Pien placed strata of soils in glass tubes having their lower ends in water to permit absorption. After quasi-equilibrium had been reached, moisture contents were determined at points throughout the height of the column. The moisture contents were plotted against height above water and compared with sorption curves for the soils (obtained by a soil tensiometer. Two soils were used. One was a glacial till having 7 percent gravel, 42 percent sand, 32 percent silt, 19 percent clay (0.2 percent colloid). The soil had a liquid limit = 23, P.I. = 8, Proctor density 121 p.c.f. and an optimum moisture content of 12 percent. The other was a loess soil having 19 percent sand, 71 percent silt, 10 percent clay (0.5 percent colloids). The soil had liquid limit = 28, P.I. = 5, Proctor density of 113 p.c.f. and an optimum moisture content = 13.5 percent.

The soils were placed in glass tubes in three layers and in different arrangement of layers in each tube as follows:

Layer	Tube A	Tube B
Top	21.5 cm. loess	20.3 cm. glacial till
Middle	30.5 cm. glacial till	31 cm. loess
Bottom	63.5 cm. loess	64 cm. glacial till

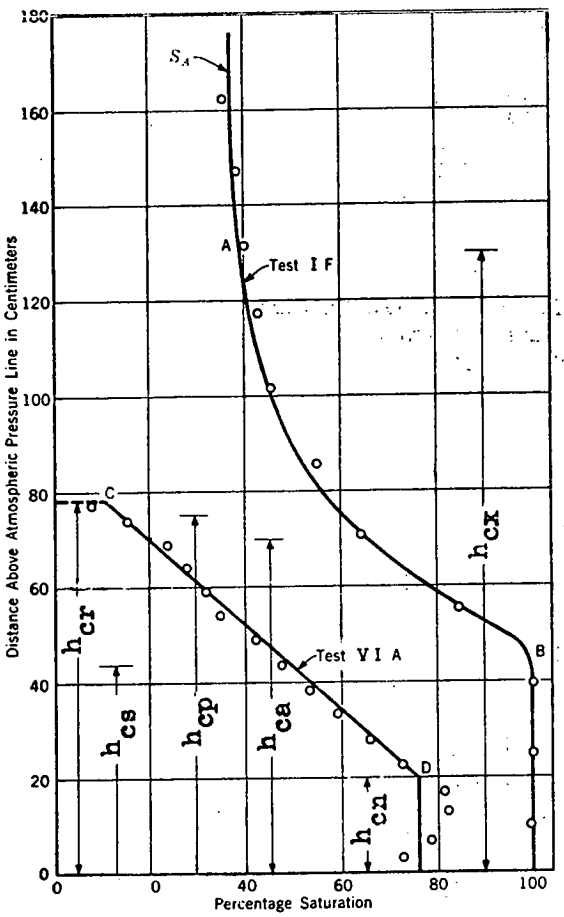


Figure 66. Range of Capillary Heads (After Lambe)

After the columns had been absorbing water for about 24 weeks moisture content determinations were made at intervals throughout the height of the columns.

Having obtained the sorption curves for the two types of soils used to make up the stratified columns in the tubes, Spangler and Pien constructed theoretical curves for soil moisture versus height above the water surface. The theoretical curves consist of segments of the soil sorption curves corresponding to the thickness of the soil layers in the experimental columns. They are shown in Figures 67 and 68 for columns A and B respectively. The actual measured values of moisture content at various heights throughout the columns are plotted to show the degree of coincidence between the actual and theoretical moisture contents.

Of the investigators of capillary movement of water in soils, Beskow's writings /1935-1 gives perhaps the most comprehensive treatment of both means for determination of the capillary properties of soils and the application of data on capillarity to the practical problem of identifying the frost susceptibility of soils and eliminating detrimental frost heaving. The reviewer feels that anyone who plans any investigation relative to frost action will want to read Osterberg's translation /1947-12 of Beskow's work especially pages 24, 25, 29, 74, and 77-106.

Beskow /1947-12 summarized the characteristics of flow of water in fine-grained soils as follows: "The elastic adsorption films represent a stable reservoir supply which, by causing a slight decrease in pressure at a point when evaporation occurs, causes a flow of water to that point by the tendency to equalize the pressure. This pressure change is transmitted inwards, and straightens out by straightening of the films, but occurs slowly, due to the dampening effect of the films. The water then flows slowly. The process of the rate of flow is then regulated by the porosity of the soil and the viscosity of the water.

"Now in a freezing soil in which ice is forming, the water next to the ice stratum changes from a liquid to a solid state and has the same effect as a change into vapor by evaporation. At the frost line a drying out occurs, a squeezing together of the adsorption films, which spreads farther and farther, causing a shrinkage. If now this zone spreads to a place of contact with a free supply of water...the water begins to flow upward from this point and the fundamental requirement for an appreciable volume increase and a subsequent frost heave is fulfilled."

Beskow's studies of the various factors which influence frost action led him to state that /1947-12 of the direct physical properties which might possibly be used to determine the frost heaving characteristics of a soil, capillarity has been found to be the most useful.

He found that control of the rate of flow of water by what he called "capillary suction" is dependent upon:

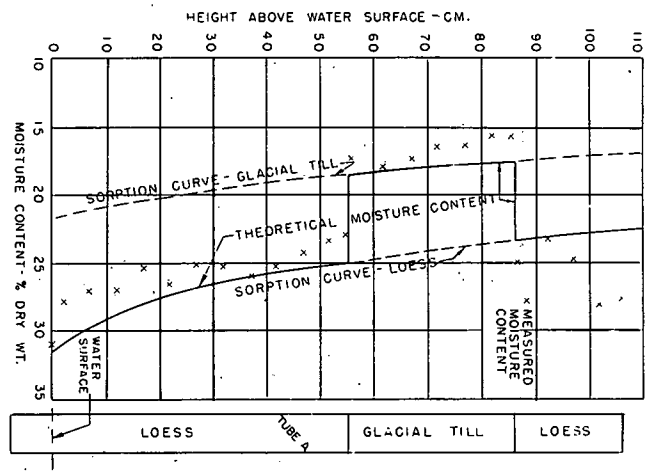


Figure 67. Sorption Curves for Two Types of Soils Showing Actual and Theoretical Moisture Contents. (After Spangler and Wei Te Pien)

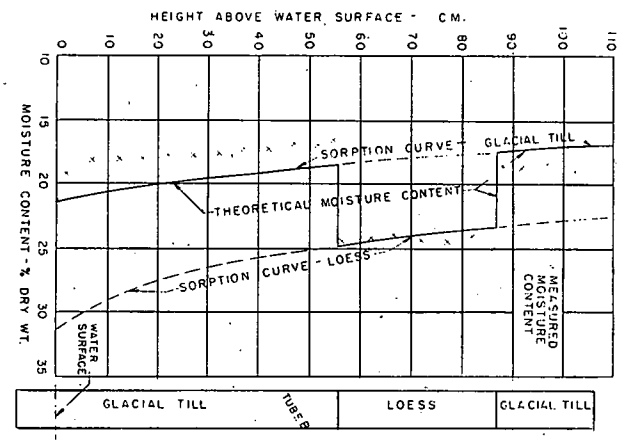


Figure 68. Sorption Curves for the Same Soils as in Figure 67 Except that there Relative Positions Have Been Reversed (After Spangler and Wei Te Pien)

1. The capillary pressure (k) at the frost line. The maximum value of k is the capillarity K of the soil.

2. The permeability of the soil (P).

3. The distance of the ground-water table (ℓ) (the ground water table is a condition where the relative capillary tension is zero).

The following formula for flow is given

$$Q = \frac{k - \ell}{\ell \times m} \; ; \; k \leqq K$$

where Q is the rate of flow of water (quantity per unit of time) = cu, cm, per sq. cm. per hr. = cm. per hr. and m is the specific resistance which is the reciprocal of the permeability or

$$m = \frac{1}{P}$$

In the above formula, the values of permeability and resistance (P and m) must be reduced to those of the actual occurring temperature due to change in viscosity. (Appropriate values are given by Beskow). Clays have such large capillarity that the effect of ground-water changes is small. When ℓ can be neglected compared to K, the formula becomes:

$$Q = \frac{K}{\ell \times m}$$

If K and m are inserted in the formula as functions of the grain size,

$$Q = c_k \times c_p \times \frac{d}{\ell}$$

Thus assuming that temperature and grading are constant, for a given value of ℓ for clays, the rate of suction is approximately directly proportional to the particle diameter.

Beskow developed a capillarimeter for determining the relative capillarity of soils. The capillarimeter, illustrated in Figure 69 served as a pattern for several capillarimeters built in this country. The results of tests by some of those have been given in preceding paragraphs. He established the emperical formula for capillarity (K) as a function of the particle diameter(d):

$$K = c_k \times \frac{1}{d}$$

where c_k is an exponent "...composed of the capillary constant and degree of packing". The value of the exponent c_k was determined as 0.60 when K is given in meters and d in millimeters.

He found the degree of packing (compaction) to be of much importance in making the determination of the capillarity by means of the capillarimeter.

He devised two methods /7 of packing and determined values of capillarity for two densities for each soil, one for a condition of loose packing (K_F) and the other for dense packing (K_M). The density of the natural soil fellin between the two extremes. The value for loose packing was found to be the "most suitable value of capillarity as a characteristic soil property" for natural sediments. He cautioned that the values of K_F and K_M are difficult to obtain on some materials, notably mixed granular material (road metal) and fairly pervious granular materials.

The results of his many tests, and correlation of capillarity with frost heave led him to state the following summary pertaining to capillarity:

"1. Soils with a capillarity of K_F less than one meter (coarse silts, sands and gravels) are under no circumstance frost heaving. For sediments this is defined as material of which less than 30 percent passes the 0.062 mm. sieve and less than 55 percent passes the 0.125 mm. sieve. For moraine, it is the material of which less than 15 percent passes the 0.062 and less than 22 percent passes the 0.125 sieve, all computed in percent of the material that passes the 2 mm. sieve.

"2. For small loads (and high ground water), soils with a capillarity of $K_F = \frac{1}{2}$ to $2\frac{1}{2}$ meters and $K_M = 1\frac{1}{4}$ to 4 meters may be dangerous (silt sediments: 30 - 50 percent less than 0.062 mm.) Such soils may cause bank slides even if they don't have any heave in roadways. For an extremely high ground water and slow freezing they may even be dangerous in the roadbed.

/7 The condition under loose packing is approximately equal to the liquid limit, and that for dense packing is obtained by beginning with a liquid consistency, removing water by vaccum and the sample tamped. The process is repeated until no further compaction is obtained.

"3. Soils with a capillarity of K_F greater than 2 meters and K_M greater than 3 meters (fine silts and finer sediments of which more than 50 percent is less than 0.062 mm) are under all circumstances frost heaving."

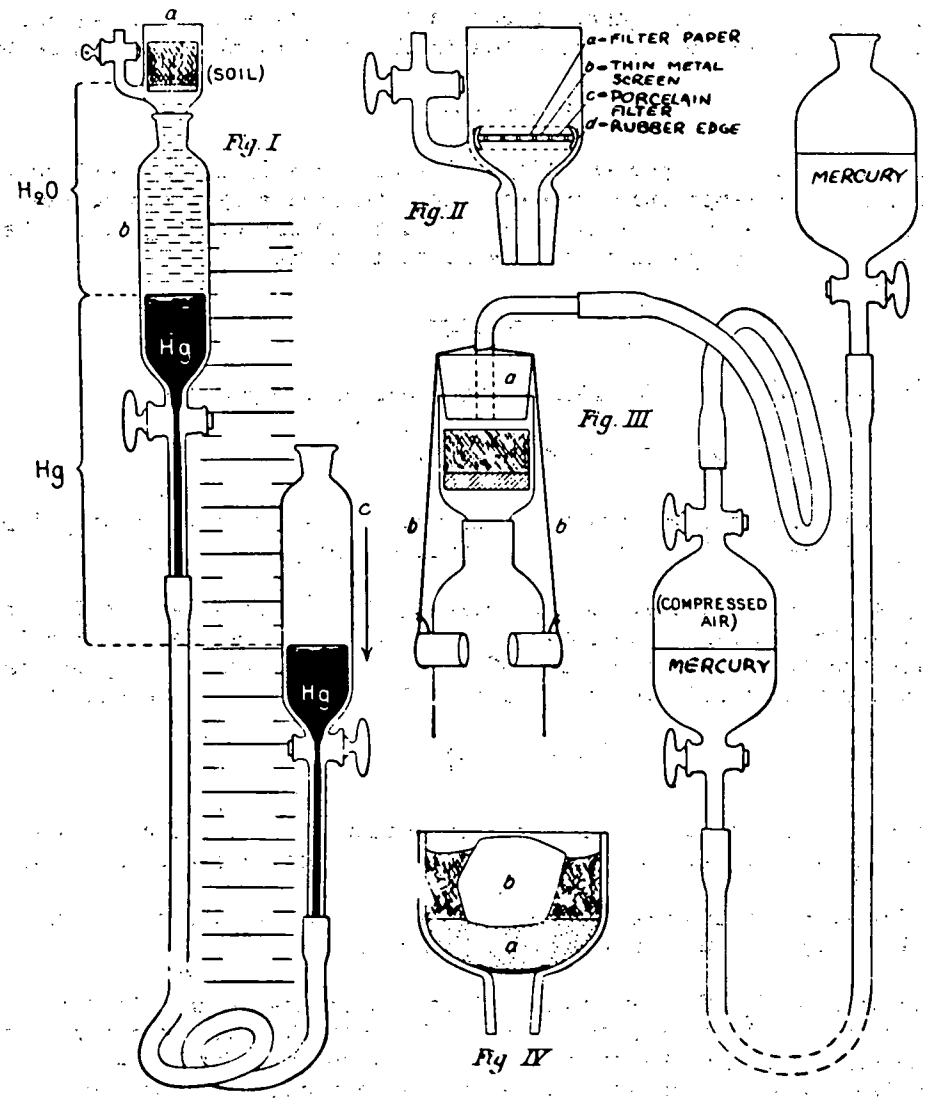


Figure 69

Diagram of Beskow's capillarimeter. Figure I is the normal apparatus, Figure II a detail of the specimen container, where a = filterpaper, b = metal screen, c = porcelain filter, d = rubber edge. In a later model, c and d were replaced by a perforated glass bottom welded to the container. Figure IV shows the arrangement for the determination of the capillarity for a piece of undisturbed natural soil, (a = sand bed, b = soil specimen, c = stiff clay). Figure III shows the arrangement with air pressure for determining capillarities greater than 9.5 meters.

The interrelationships between capillarity, grain size and hygroscopicity and frost heaving are given in Table 20.

TABLE 20

(After Beskow /1947-12)

Soil Group		Average Diameter	Amount Passing Sieve 0.062 mm	Amount Passing Sieve 0.125 mm	Capillarity K_F	Hygroscopocity W_h
		mm	%	%	m	
0. Non-frost-heaving under any circumstances	Sediment	0.1	<30	<55	<1	---
	Moraine	---	<15	<22		---
1a. Causing frost-heave only at surface and for very high ground water	Sediment	0.1-0.07	30-50	—	1-1 3/4	
	Moraine	---				
1b. Same, except effects whole road base for very high ground water	Sediment	0.08-0.05	15-25	22-36	1 1/4-2 1/2	—
	Moraine	—				
2. Normally frost-heaving and liable to frost boils for ground water depths 1½ m (moraine 1 m)	Sediment	<0.05	>50	—	2-20	<5 1-4
	Moraine	---	>25	>36		
3. Frost-heaving clays but not liable to boils	(Sediment)	---	--	—	20-?	5-(10?)
4. Non-frost-heaving stiff clays	(Sediment)	---	--	—	?	(> 10?)

In a later report /1938-11 Beskow reported the following capillary limits related to grain size in terms of material passing the No. 200 sieve. The values he gave are shown in Table 21.

TABLE 21

Capillarity and Grain Size Not Subject to Frost Heaving
(After Beskow /1938-11)

Type Soil	Passing No. 200 Sieve	Capillarity In Inches
Well Sorted Sediments	Less than 40	Less than 40
Graded Moraine Soils (Determined on material passing No. 10 sieve)	Less than 19	Less than 40

Beskow's comparative values of capillarity for the different grain-size fractions and for natural sediments given in Table 22 will be of interest to anyone making a study of capillarity. Additional data on capillarity are given later under "Permeability."

Beskow gave much consideration to the influence of fissures, such as occur in structured soils, on capillarity. He concluded that fissures having no contact between upper and lower surfaces are practically perfect insulators for water flow, and that water movement across a fissure can occur only at points of direct contact. Thus numerous fissures can lower the capacity of soil to suck up water to a small fraction of the normal flow. He also investigated the influence of stratification in soils on capillary movement.

In that study he applied his method to the computation of capillary flow in a soil consisting of layers having different grain sizes. He based his computation on the fact that in the distance between the ground-water table and the frost line, the negative capillary pressure can nowhere exceed the capillarity of the layer in which it exists; and the rate of flow is the

same at all levels and therefore the gradient of the capillary-pressure difference /8 in each layer is inversely proportional to the permeability of the layer. Beskow then starts from the bottom and sums up the resistances: so that for any level the resistance of the total column below that level is known. By knowing the capillarity of any layer the rate of upward "suction" can be computed. However the layer which gives the smallest rate of flow limits the entire flow.

TABLE 22

(After Beskow /1947-12)

Pure Fractions			Natural Sediments		
Grain Size Classification	Grain Size	Capillarity	Soil Type	Loose Capillarity K_F	Dense Capillarity K_M
	mm				
Coarse Sand	.2 -0.6	3 - 10 cm	Coarse Sand	3- 12 cm	4- 15 cm
Medium Sand	0.6 -0.2	10 - 30 cm	Medium Sand	10- 35 cm	12- 50 cm
Coarse Silt	0.2 -0.06	30 - 100 cm	Coarse Silt	30-200 cm	40-350 cm
Silt	0.06 -0.02	1 - 3 m	Silt	1.5- 5 m	2.5- 8 m
Fine Silt	0.02 -0.006	3 - 10 m	Fine Silt	4- 10 m	6- 12 m
Very Fine Silt	0.006-0.002	10 - 30 m	Lean Clay	8- 15 m	10- 18 m
Coarse Clay	0.002-0.0002	30 - 300 m	Medium Clay	14- ? m	15- ? m
Fine Clay	0.0002	300 m	. . . . .	. . . . .	. . . . .

Assume a series of layers lettered from below as a, b, c, d, etc., having respective capillarities of Ka, Kb, etc., and respective specific resistances of ma, mb, etc. If the soil in each layer has a similar grading then

$$\frac{m_a}{K_a^2} = \frac{m_b}{K_b^2} = \frac{m_c}{K_c^2} = C = \text{Constant}$$

Since for similar-grading characteristics the resistance is proportional to the square of the capillarity ($m - c \times K^2$). If the thicknesses of the layers is $\ell_a, \ell_b, \ell_c \ldots$" the absolute resistance of each layer Ma, Mb, Mc...is then

$$M_a = \ell_a \times m_a; \quad M_b = \ell_b \times m_b, \text{ etc.}$$

If successive computations are made using the upper surface of each layer as the surface to which the flow is going, the following formulas will give the maximum upward flow to the surface of layer a, b, c, etc.

$$Q_a = \frac{K_a - \ell_a}{m_a \times \ell_a} \quad ;$$

/8 Active capillary pressure gradient, i.e., the capillary pressure difference in cm. of water minus the height above the ground water table in cm. If the absolute pressure difference $\frac{dk}{d\ell}$ is n cm. of water per cm. of height (ℓ) we have $n - \ell = c \frac{1}{p}$ where P is the permeability and c is a constant.

$$Q_b = \frac{K_b - (\ell_a + \ell_b)}{m_a \times \ell_a + m_b \times \ell_b}$$

$$Q_c = \frac{K_c - (\ell_a + \ell_b + \ell_c)}{m_a \times \ell_a + m_b \times \ell_b + m_c \times \ell_c}$$

If in this manner layer is added on layer, it is found that the largest rate of flow Q can never be larger than any of the computed values of Q. Thus, for example, the maximum flow to layer e which can be called Q_{Ae}

$$Q_{Ae} \leqq Q_d, Q_c, Q_b, Q_a$$

The layer having the smallest rate of flow determines the possible rate of flow for all the overlying layers.

Figure 70 indicates schematically a sediment series a - f. The capillarity of each layer is laid out to scale on the left side. The curves for capillary pressure, K, (curves 1 - 4) are constructed for flow to various heights in the sediment. These are actually the pressure change per unit length $\left(\frac{dk}{d\ell} = g\right)$ which is inversely proportional to the specific resistance for each corresponding layer, which in turn is proportional to the square of the capillarity.

As an example of how this is computed, assume flow to the surface of layer d. Then the average gradient $G_d = K_d : L_d$. The total resistance M_d is then

$$M_d = \ell_a \times m_a + \ell_b \times m_b + \ell_c \times m_c + \ell_d \times m_d$$

where $\ell_a + \ell_b + \ell_c + \ell_d = L_d$ the average specific resistance is then $M_d : L_d$. If we call the gradient $\frac{dK}{d\ell}$ in layer a, g_a, in layer b, g_b etc. we get

$$\frac{g_a - 1}{G_d - 1} = \frac{m_a}{M_d : L_d} = \frac{m_a \cdot L_d}{M_d}; \quad g_a = \frac{m_a \cdot L_d (G_d - 1)}{M_d} + 1;$$

$$\frac{g_b - 1}{G_d - 1} = \frac{m_b \cdot L_d}{M_d}; \; g_b = \frac{m_b \cdot L_d (G_d - 1)}{M_d} + 1 \text{ etc.}$$

Thus for each layer, the gradient can be computed, and the capillary pressure curve constructed, as shown in Figure 70.

Vapor movement - The movement of soil moisture from one depth of soil to another or from the atmosphere into the soil has long been an object of consideration. Bouyoucos /1915-1 placed soil in brass tubes 8 in. long and 1½ in. in diameter and subjected the two ends to different temperatures 0 and 20 C. in one series and 0 and 40 C. in another series of tests. The duration of the tests was 8 hours. Soils were separated by a 1/4 in. air space. Bouyoucos concluded from his test that "the thermal movement of moisture due to distillation is practically negligible."

Lewis /1937-11 placed two pieces of 30-mesh screen 1 mm. apart at a predetermined point in a soil column (1, 2, 4, and 7 cm. from the open end) to determine the rate of upward flow in the form of water vapor across the small break in the column. He applied water at the lower ends of the columns at the rate of 11 and 44 mg. per hour. In presenting the results of his investigation, he stated: "These data seem to indicate that the moisture distilled across the break in the column and built up a moisture content, or capillary gradient, above that point. It appears to have required a difference of moisture content of 12 to 17 percent to force 8.5 to 12 mg. per hour of water in the vapor phase across an open space of about 1 mm. These rates are equivalent to 2.4 and 3.4 mg. per sq. cm. per hour or 0.69 and 0.97 inch in depth

(surface inches) per month". Lewis concluded that if this is true, the movement through the soil pores is negligible. Perhaps that is true for agronomic purposes, but the amounts given are not negligible when interpreted in terms of moisture increases for influencing frost action. The average moisture contents of the soil at different distances from the open end for each subgroup in the tests by Lewis are shown in Figure 71.

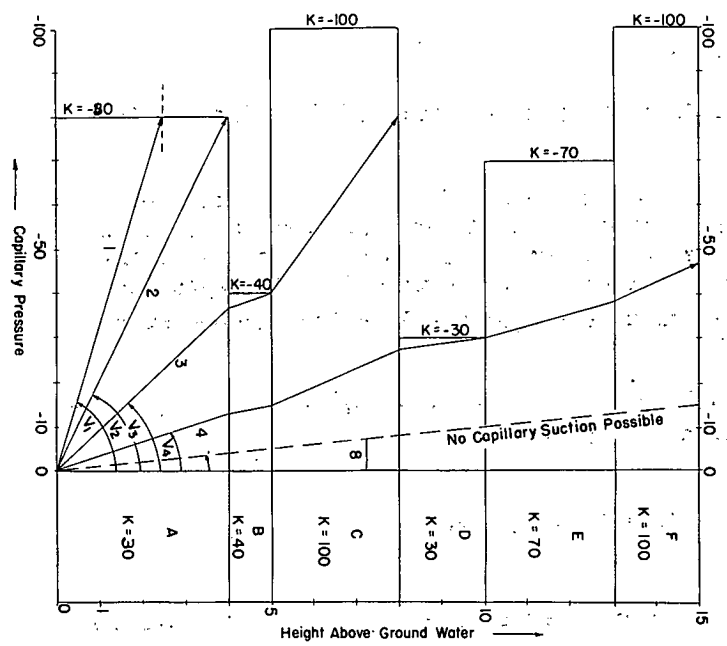


Figure 70. Schematic diagram, showing the effect of different layers on the capillary rise. Right side: the soil column, consisting of layers a to f, having capillarities shown. The capillarities are also shown graphically at the left as the maximum possible capillary pressure in the soil pores (negative pressure). The diagonal lines with arrow heads, 1, 2, 3, 4 represent the pore water pressures at different levels, giving the maximum possible rate of capillary rise for water flowing to the various levels shown by the corresponding arrow heads. The slope of these lines within each layer (v1 + v2 for the bottom layer) is a measure of the rate of capillary flow if the slope angle α is subtracted from it (the dashed line at a slope α is the pressure condition at which there is no flow). (After Beskow)

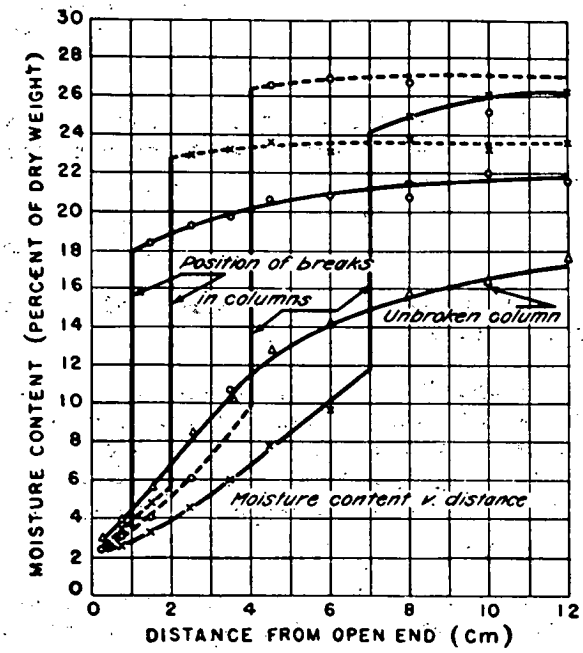


Figure 71. Average moisture content throughout length of five groups of four 6 inch cores of Willamette silt loam having 1-mm breaks at different distances from the open end. Water was added to all tubes at a rate of approximately 11 mg per hour. It was lost from the unbroken cores at an average rate of 10.3 mg per hour and at rates of 12.0, 10.2, 8.5, and 9.0 mg per hour from cores having breaks 1, 2, 4 and 7 cm, respectively, from the open ends. (After Lewis)

Beskow /1935-1 considered the possibilities of water movement by vapor. Vapor flow, that is flow by diffusion, is caused by pressure difference, the difference being in vapor pressure. There are two ways in which differences in vapor pressure can occur. One is due to temperature difference in which the upper surface would be cooler. He gives the average winter-temperature gradient at depths up to about 3 ft. as 0.02-0.04 C. per cm. Since air has a lower conductivity the temperature change in a crack or fissure would be larger. If the conductivity ratio is about 1:50 the temperature change across a fissure would be 1-2 deg. C. At temperatures near freezing the change in partial pressure of water vapor is about 0.35 mm. Hg per deg. C. This temperature difference would correspond to a vapor-pressure difference of 0.35 - 0.7 mm. Hg or 0.477 - 0.954 grams per sq. cm. per cm.

He takes the rate of diffusion of water vapor in air at a freezing temperature as 0.20 cm. sec. and computes that, for the conditions given above, the water moved by this method would correspond to a water layer of 6 to 13 hundredths of a mm. in thickness per day. Beskow, in considering water flow for heaving concludes "the diffusion due to temperature difference is so small that for water flow in frost heaving soils it is of no importance."

He brings out that a second possibility for a vapor-pressure difference is the difference in capillary tension in the upper and lower surfaces (of a fissure). He also concludes that no appreciable amount of water can be transported in this manner.

Numerous observers have noted high-moisture contents under pavements, especially during the cooler months, and have attributed the moisture contents to condensation of moisture moved in vapor form. The reviewer believes this phase deserves further study, inasmuch as the amounts which need to be moved to merely increase the moisture content of the soil (and not to furnish large quantities of water for formation of ice lenses and heaving as computed by Beskow) sufficiently to cause ice formation and reduction in bearing capacity on thawing were small.

Permeability - It has been shown that capillarity (both the height to which water will rise and the rate of water which will be moved) is related to the grain size and the degree of compaction. Beskow /1935-1 found that the resistance to flow (non-turbulent) of water through a capillary tube is inversely proportional to the fourth power of the diameter, and directly proportional to the permeability. He conducted permeability /9 (and capillarity) tests on Atterberg's fractions and also on several soils of different texture. The history of the natural soils insofar as associated water conditions and frost action are concerned was known. The results of Beskow's permeability tests are summarized in Table 23.

TABLE 23

(After Beskow 1947-12)

Permeability of Atterberg fractions

Grain Size	Permeability P $cm^3/hour \cdot \frac{cm}{cm} \cdot cm^2 = cm/hour$ (at) + 20° C.	Capillarity K	Real Particle Size d From Capillarity ($d = 0.06 \cdot \frac{1}{K}$)	$\frac{P}{d^2} = c_p$	Relative Permeability $P \cdot K^2$
mm		m			
0.2 -0.1	37.5	0.41	0.147	$1.73 \cdot 10^3$	6.3
0.1 -0.05	5.3	0.99	0.066	$1.21 \cdot 10^3$	5.2
0.05 -0.02	1.74	1.9	0.0322	$1.67 \cdot 10^3$	6.0
0.02 -0.01	0.46	4.4	0.0136	$2.48 \cdot 10^3$	8.9
0.01 -0.005	0.11	8.2	0.0073	$2.07 \cdot 10^3$	7.4
0.005-0.002	0.020	17	0.0035	$1.62 \cdot 10^3$	5.8
0.002-0.001	0.0012	70	0.00086	$1.64 \cdot 10^3$	5.9

From this work Beskow held that permeability is directly proportional to the square of the particle diameter, and for pure fractions, under a given degree of packing and temperature, the relationship is

$$P = 1800\ d^2$$

When d is given in mm, P in cu. cm of water per hour, the cross section in sq. cm, the head equals the thickness of the sample and the temperature is 20 C.

Table 24 contains Beskow's values of permeability for different grain size groups and natural sediments.

Lane and Washburn /1946-8 also presented data from capillarimeter, capillary rise and permeability tests under conditions of controlled initial compaction of the samples. Their data, has been given in summary tabular form under the subject of "Capillarity."

The War Department's Engineering Manual /1946-19 states that the coefficient of permeability

/9 Tests were made under constant hydrostatic pressure under constant temperature control (at 20.5 C.)

of sand and gravel courses graded between limits usually specified for stabilized material depends principally upon the percentage by weight of sizes passing the 200 mesh sieve. The manual presented the tabulation shown in Table 25 of approximate values of permeability for different percentages passing the 200 mesh sieve.

TABLE 24

Permeability (at 20° C.) and capillarity of various coarse materials (well sorted sediments). At temperatures near 0° C. permeability is reduced to 60 percent of the values given.

Pure Fractions				Natural Soils					
Type	Grain Size mm	Capillarity K	Permeability P cm/hr. (at + 20° C.)	Soil Type	Grain Size mm	Maximum Capillarity K_M	Permeability P, cm/hr. Normal Values	Permeability P, cm/hr. Limiting Values	Hygroscopic Capacity W_h
Coarse Gravel	20-6	<1 cm	$640 \cdot 10^3$-$58 \cdot 10^3$	Coarse Gravel . . (Non-Frost Heaving)	20-6	-	-	-	-
Fine Gravel .	6-2	1-3 cm	$58 \cdot 10^3$-$6.4 \cdot 10^3$	Fine Gravel (Non-Frost Heaving)	6-2	1-5 cm	15,000-1,000	25,000-150	-
Coarse Sand . .	2-0.6	3-10 cm	$6.4 \cdot 10^3$-$0.58 \cdot 10^3$	Coarse Sand (Non-Frost Heaving)	2-0.6	4-15 cm	1,500-70	2,500-15	-
Medium Sand .	0.6-0.2	10-30 cm	580-64	Medium Sand (Non-Frost Heaving)	0.6-0.2	12-50 cm	125-5	250-1.5	-
Coarse Silt .	0.2-0.06	30-100 cm	64-5.8	Coarse Silt (Non-Frost Heaving)	0.2-0.06	40-350 cm	10-0.3	25-0.1	-
Silt.	0.06-0.02	1-3 m	5.8-0.64	Silt . . (Frost Boils, Frost Heaving)	0.06-0.02	2.5-8 m	1-0.02	2-0.005	-
Fine Silt . .	0.02-0.006	3-10 m	0.64-0.058	Fine Silt. . (Frost Boils, Frost Heaving)	0.02-c:a 0.006	6-c:a 12 m	0.1-0.002	0.2-0.0005	-
Very Fine Silt	0.006-0.002	10-30 m	$58 \cdot 10^{-3}$-$6.4 \cdot 10^{-3}$	Lean Clay. . (Frost Boils, Frost Heaving)	-	10-c:a 18 m	0.005-0.005	0.01-0.0002	2-4
Coarse Clay .	0.002-0.0002	30-300 m	$6.4 \cdot 10^{-3}$-$0.064 \cdot 10^{-3}$	Medium Clay. . (Frost Boils, Frost Heaving)	-	c:a 15-?m	0.001-0.00005	0.002-0.00002	4-7
Fine Clay . .	<0.002	>300 m	$64 \cdot 10^{-6}$-$0.64 \cdot 10^{-6}$	Stiff Clay. . Very Stiff Clay	-	-	-	-	7-10 <10

TABLE 25

Average Coefficients of Permeability of Sand and Gravel Bases
(for estimating purposes only) (After War Dept. 1946-19)

Percent by Weight Passing No. 200 Sieve	Coefficient of Permeability (ft. per. min.)
3	10^{-1}
5	10^{-2}
10	10^{-3}
15	10^{-4}
25	10^{-5}

The coefficient of permeability of crushed rock and slag, each with many fines is generally greater than one ft. per. min.

The manual brought out that the value of the coefficient of a base in a horizontal direction may be 10-times greater than the average values given in Table 25, which are based on remolded samples. For uniformly graded sand bases the horizontal value may be 4 times as great, while very pervious materials may have equal permeability in all directions. The manual suggests that coefficients be determined by laboratory tests for final design.

Gardner /1945-18 found that frost action affects the development of soil structure and this influences permeability. He found that freezing after calcium chloride was added to replace sodium ions in sodium-saturated soils restored the permeability of those soils.

Permeability of Frozen Soil - Anderson, Fletcher and Edlefsen /1942-14 conducted some laboratory tests to determine the downward movement of water under gravitational force in frozen soils because of its importance in relation to soil erosion and runoff from water sheds. Tests were made on Yolo fine sandy loam and Capay clay. The soils, at various initial moisture

contents, were placed in glass tubes and frozen. The results, presented graphically in Figure 72, show that water penetrated the frozen soil when relatively dry. Little to no penetration occurred at the higher initial-moisture contents.

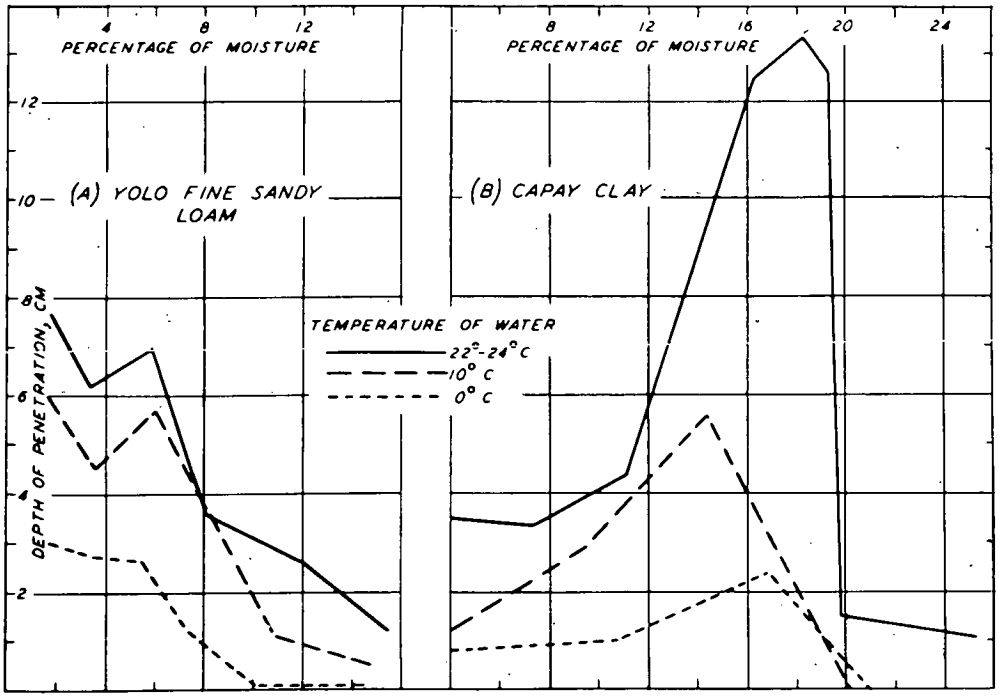


Figure 72. Permeability of Frozen Soil. (After Anderson, et. al.)

Relationship Between Capillarity, Permeability, and Grain-Size Characteristics - According to Beskow the resistance to flow through a capillary tube is inversely proportional to the fourth power of the diameter and directly proportional to the permeability. If the degree of packing and the grain shape are the same, the pore space is proportional to the square of the grain size and

$$P = c_p \times d^2 \quad \text{where}$$

d = average particle diameter, and
c_p is an exponent which takes into account degree of packing and water viscosity changes due to temperature changes.

He proved the validity of this formula by conducting permeability tests on pure fractions (see Table 23). In the right-hand column of Table 23 he indicated relative permeability as a product of P and K^2. Inasmuch as the values are approximately constant and show no systematic tendency to vary, he concluded the relationship was valid.

He also held that the product of the permeability and capillarity is a measure of the grading. The more "unsorted a soil is, the smaller is the relative permeability." This was his explanation of why very poorly sorted soils, especially normal moraines, have so little frost heave in spite of their high capillarity.

Hogentogler and Barber in discussing Russel and Spangler's work on "The Energy Concept of Soil Moisture" /1941-12 correlate the values of capillarity, permeability and average grain sizes obtained by test and shown in Table 26. (The capillary-rise data were obtained from capillarimeter tests, effective pore diameters were computed and values of coefficient of permeability were obtained from variable head permeameter test data.) The relationships obtained are shown graphically in Figures 73, 74 and 75.

Effect of Swelling Properties - Ducker /1942-11 cites Schmid's statement that high-swelling colloidal soils are dangerous for use in highways because of frost heaving. Ducker concludes that colloids which swell are not so frost sensitive as those which do not swell. "In other words, the amount of frost heave—decreases within the various groups of clay minerals as the capacity for swelling and the water retention capacity of the dispersed colloidal system increases". He held that the proportion of colloids which promotes swelling hinders the access of water to the freezing zone.

TABLE 26

FLOW CHARACTERISTICS OF DIFFERENT FRACTIONS OF RIVER SAND

Fraction		Mean grain diameter d[a]	Capillary Rise, h	Computed pore diameter p[b]	Coefficient of permeability k
Passing	Retained on				
Sieve No.	Sieve No.	Mm.	Inches	Mm.	Feet per day
10	20	1.183	2.48	0.472	1430
20	30	0.693	3.74	0.313	665
30	40	0.491	5.24	0.223	380
40	60	0.313	7.88	0.149	190
60	80	0.207	11.7	0.100	160
80	100	0.162	14.0	0.084	75
100	140	0.123	18.5	0.063	45
140	200	0.087	26.4	0.044	20
200	270	0.062	35.6	0.033	9

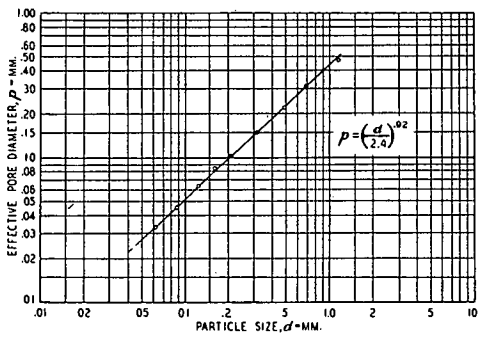


Figure 73
Relation Between Particle Size and Pore Diameter
(After Hogentogler and Barber)

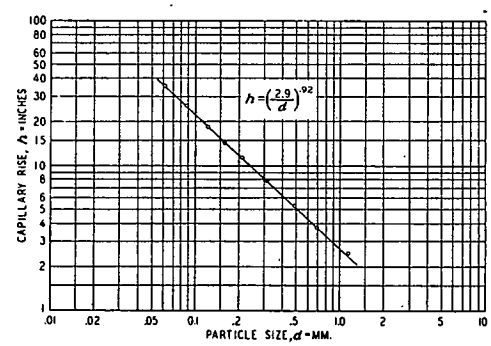


Figure 74
Relation Between Particle Size and Capillary Rise
(After Hogentogler and Barber)

Hygroscopicity /10 - Beskow /1935-1 /10 found the soil characteristic of hygroscopicity to be of little value in determining the upper grain-size limit of frost susceptible soil. He found that essentially frost-heaving soils had hygroscopic values up to 5, but since even the stiffer calys may be frost heaving he held that the ultimate limit of hygroscopicity beyond which damaging heave may not occur would be 10.

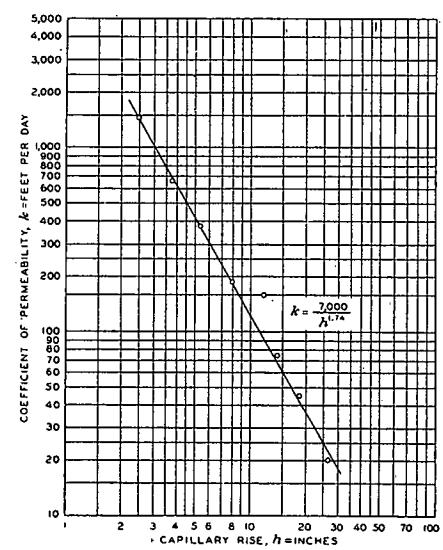


Figure 75
Relation Between Capillarity and Permeability
(After Hogentogler and Barber)

/10 Beskow did not define the soil characteristic hygroscopicity. As used in the United States it is the moisture content (percent of dry weight) which the soil retains when air dried (usually at room conditions of temperature and relative humidity).

Physico-Chemical Properties

The fact that water in fine-clay soils freezes only at several degrees below 32 F. was first explained as a super-cooling phenomenon. Later Beskow /1935-1, and others, explained that the freezing-point lowering was the result of the attachment of the water molecules by the adsorptive force of the soil particles and that a certain amount of energy (in the form of heat loss) is required to remove the water molecule from the adsorption film and build it into the growing ice crystal. Early investigators believed that the grain size alone was the principal factor in governing the manner of freezing.

Endell /1938-12, /1941-8 found from studies of the behavior of clay minerals that the nature of the clay mineral and its water adsorption characteristics influenced its behavior under freezing temperatures. Ducker /1931-15, /1939-1 and /1942-11 also found that the minerological content of clays, and hence the physico-chemical nature, influences the susceptibility to frost action.

Endell concluded from his studies that we may differentiate between:

(1) Cohesive soils, consisting chiefly of Kaolinite in which frost heave is very great;

(2) Cohesive soils with a Montmorillonite (bentonite) content of not more than 15 to 25 percent, which show more or less frost heaves;

(3) Cohesive soils with a Montmorillonite (bentonite) content greater than 15 to 25 percent in which frost heave does not take place.

However, while Endell concluded that Na- bentonite shows little frost heave (in a single test), on repeated freezing and thawing there develops a polygonal structure traversed by many fine cracks. Because the fine cracks develop in a few years, it is not safe in case of frost.

Ducker /1940-18 concluded that for highway construction, all clay soils are dangerous in case of frost, whether Kaolinite or Montmorillonite is their chief constituent, and that these soils should be provided with a frost protective layer.

Grim /1951-27 points out Taber's findings /1930-7 that very-fine colloid-size clay materials show little or no segregation of ice on freezing. He also brings out that a certain percentage of water does not freeze at moderately low temperatures (as was found by Bouyoucos /1936-24). These and other considerations suggest that water held in soil pores of all sizes and all kinds may not all have the same characteristics. Water directly adjacent to (absorbed to) a clay mineral surface is likely to be in a different physical state than water in the center of a large pore. Winterkorn /1943-15 summarized that difference as follows: Directly adjacent to the adsorbing soil solidly adsorbed water is to be found, the center of a pore space is occupied by ordinary water freezing at about 0 C. and between the ordinary water and the solidly adsorbed water is a zone of liquid water possessing a melting point down to -22 C. which serves as a passageway for the conduction of water to freezing centers.

Grim /1951-27 held that "the clay mineral composition is the major factor controlling the properties of most soils. The surface of the clay minerals provides the major adsorption surface in soils materials, and the adsorption characteristics towards water and various ions and organic molecules vary for the different clay minerals."

Montmorillonite - Grim /1951-27 preferred to begin his analysis of the relation of clay mineral compositions to frost action by consideration of a soil composed solely of montmorillonite because in it adsorption water penetrates between the individual molecular layers giving it a large adsorption surface (computed to be 800 sq. m. per gm.) and an enormous water adsorption capacity. He believed there is little doubt the water adsorbed on the surface of montomorillonite grains would consist of molecules in a definite pattern, and that, therefore, the water would not be mobile.

Montmorillonite has a high adsorption capacity for certain ions, and the character of the adsorbed ions governs, in a large degree, the thickness of water layers. When carrying sodium as the adsorbed ion, water can enter easily between the unit layers and build up thick layers of adsorbed water. Thus even with large quantities of water there would be no fluid water. Such clays are quite impervious, and on freezing there is little or no concentration of ice in

layers. However, when carrying calcium, magnesium, or hydrogen as the exchangeable ion, water contents greater than a relatively small percentage (about 40 percent of the dry clay) in comparison with sodium montmorillonite clay is fluid. Ice layers may develop on freezing if the moisture content is fairly high.

Kaolinite soils - Here the particles are 100 to 1000 times the size of montmorillonite particles and the surface area is relatively small. Even at relatively small moisture contents kaolinite soils would contain some fluid water. They are not impervious and should readily show a concentration of water in ice layers on freezing.

Halloysite soils - Should act similarly to kaolinite soils.

Illite soils - Many soil materials are composed primarily of the mica type of clay minerals illite and chlorite. Their adsorption characteristics are of the same order of magnitude as kaolinite soils but they immobilize slightly more water. They are not impervious and should show readily the concentration of water in ice layers on freezing. Illite and chlorite soils have exchange capacities 2 to 5 times that of kaolinite but only $\frac{1}{4}$ to $\frac{1}{2}$ that of montmorillonite. Some illite soils contain montmorillonite and hence take on some of the properties of montmorillonite.

Vermiculite and Attapulgite - Palygorskite soils - These soils are relatively rare and their properties are not well known.

Thermal Properties of Soils and Pavements

The effect of thermal properties of soils and pavements on frost action is a complicated one. The thermal properties, i.e. the rate at which temperature penetrates a material, may differ with differences in soil composition and grading, moisture content, and density and may differ with pavement type. The literature contains many instances where writers remark on the effect of thermal properties without citing data to give validity to their statements. Some of those general statements are obviously contradictory. Because consideration of the effect of thermal properties involves mathematics beyond that desired here, the reviewer has chosen to present here a simple brief of the subject and review it in more detail later.

Frost effects are the result of nature's effort to create temperature equilibrium, i.e., soil temperature = air T. In winter there is a flow of heat from the soil to the air, in summer from the air to the soil.

The amount of heat transference is governed by (1) the difference in soil and air temperatures and (2) the thermal conductivity of the soil and the pavement. For example, if the thermal conductivity of dry sand is 2.7 Btu., dry snow 0.75 Btu., and a solid plate of quartz 50 Btu., then a road surface with a 1-in. thickness of dry sand would transfer 2.7 Btu. per hr.; one covered with 1 in. of snow would transfer only 0.75 Btu. per hr.; but one covered with a plate of quartz would transfer 50 Btu. per hr. per sq. ft. per degree difference in temperature.

The total heat transfer required to change the temperature of a given mass of soil equals the product of the mass, the change in temperature, and the specific heat. The specific heat is the ratio of the quantity of heat required to raise the temperature of a unit mass of a substance to the quantity of heat required to raise an equal mass of water through the same temperatures.

To clarify the relationship between heat transference by a practical example, assume a 6-in. covering of quartz and sandstone (assume sandstone has a specific heat of 0.22 and quartz sand 0.19) and a 10 degree difference in temperature.

The quartz would transfer heat at the rate of

$$\frac{50 \text{ Btu.} \times 10 \text{ deg.}}{6 \text{ in.}} \quad \text{or 83 Btu. per hour}$$

and would be required to lose 50 lb. X 10 deg. X 0.19 or 95 Btu. to equal the air temperature. It would do this in $\frac{95}{83}$ or 1.2 hours. Sandstone, having a thermal conductivity of 16 Btu. would transfer heat at a rate of $\frac{16 \times 10}{6}$ or 26 Btu. per hour would be required to lose 50 lb. ($\frac{1}{2}$ cu. ft.) X 10 deg. X 0.22 or 110 Btu. to become equal to the air temperature and would do this in $\frac{110}{26}$ or 4.2 hours.

Smith /1946-1 obtained conductivity values for a number of different pit materials at water contents equal to the field moisture equivalents. (No mention is made of density). He applied constant heat to a surface of material which had been rammed into a wooden insulating box and read temperatures at various depths from the surface. From mean temperature differences for each zone (depth) and the known weights and specific heats of the materials, he determined the thermal conductivities K of the materials from the general formula:

$$Q = \frac{KA\,(T_1 - T_2)}{D}\,t$$

Where Q is quantity of heat transmitted in Btu:

A is area
T_1 and T_2 are mean temperatures at faces of zones
t is time in hours
D is depth of zone in inches

He found difference in conductivity with difference in moisture content, and accordingly tested two materials at several different moisture contents. His test results are shown in summary form in Table 27. He found the mineral matter of the gravel appeared to have a bearing on conductivity, as far as quartz is concerned. The gravels with the higher thermal values are mainly quartz in composition. The temperature rise of gravels was not directly proportional to thermal conductivity but was modified by the specific heat of the materials. He computed the time of frost penetration through an 8-in. layer of gravel roadway as follows:

TABLE 27

Source of Gravel	Percent Water	Specific Heat	Thermal Conductivity	Temp. rise in 1 hr. with 10° F. difference of temp. for 3" layer.
Mataura River Beach, Gore	F.M.E.	0.206	13.1	6.21°
Gordon's Pit	F.M.E.	0.236	14.2	4.20°
Woodend Pit	F.M.E.	0.204	15.0	4.60°
McKenzie's Pit	F.M.E.	0.252	20.3	6.75°
Awarua Pit	13.3	0.282	24.7	4.3°
Awarua Pit	10.3	0.261	23.3	4.80°
Awarua Pit	7.4	0.241	23.0	8.80°
Awarua Pit	4.6	0.221	21.4	4.99°
Oreti Beach Sand	0.0	0.177	3.74	1.57°
Oreti Beach Sand	5.0	0.210	6.55	2.62°
Oreti Beach Sand	15.0	0.284	8.18	2.26°
Oreti Beach Sand	30.0	0.340	10.5	2.2°

Assume a ground temperature of 40 F. and an air temperature of 25 F. Using the formula

$$T_m = \frac{(T_1 - T_2)}{2.3 \log (T_1/T_2)}$$

Where:

T_m = average temperature of the affected layer
T_1 = temperatures of the subgrade 8 in. below the surface = 40 F.
T_2 = the temperature of the air = 25 F.

Then:

$$T_m = \frac{(40 - 25)}{2.3 \log \left(\frac{40}{25}\right)} = 31.9 \text{ F.}$$

Table 27 shows Woodend gravel has a specific heat of 0.204 and a density of 105 pcf. The quantity of heat lost per sq. ft. of surface for this layer will be:

$$(40 - 31.9) \times \left(105 \times \frac{8}{12}\right) \times (0.204) = 115 \text{ Btu.}$$

The temperature at any depth below the surface within this 8-in. layer is given by the formula

$$D = B + Z \log T$$

Where D = depth below the surface in inches
T = corresponding temperature in deg. F.
B and Z are constants

at the surface D = 0 in.
T = 25 deg.

Substituting in the above equation

$$0 \text{ in.} = B + 1.398\ Z$$
$$B = -1.398\ Z$$

at 8-in. depth D = 8 in. T = 40 deg.

Again substituting, $8 = B + 1.602\ Z$ $Z = 39.2$ and $B = -54.9$.

For the depth corresponding to a temperature of 32.9 F.

$$D = -54.9 + 39.2 \times 1.504 = 4.2 \text{ in.}$$

and with a thermal conductivity of 15.0 from the general equation solving for time

$$t = \frac{Q\ D}{K\ A\ (T_1 - T_2)}$$

$$= \frac{115 \text{ Btu.} \times 4.2}{15 \times 1 \times (31.9 - 25)} = 4.67 \text{ hours}$$

Smith computed values for McKenzie's pit gravel (Table 27) and obtained a time of 4.74 hr. and also made computations for dry sand (H = 0.177 and K = 3.74) and obtained a value of 14.7 hr. He attempted to check these computed values by experiment. He placed a wooden box with a 7 in. of Woodend gravel having a temperature of 44 F. and at the end of 4½ hours the temperature had dropped 9.3 deg. Correcting for an 8-in. depth, the value becomes 8.1 F. which is equivalent to that calculated for the time of 4.67 hours.

The effect of an asphalt covering is to extend the time of freezing since thermal values of asphalt are less than those of dry gravels. Smith gives conductivities of 3.7 for " a dense honing mix" and 7 for a road oil seal with 3/4 to 1 in. chips.

Using a value of 5, a 1-in. seal, and the same conditions as the 8 in. of Woodend gravel, the time of frost penetration would be:

$$\frac{115 \times \frac{3.2 \text{ in.}}{15} + \frac{1 \text{ in.}}{5}}{6.90} = 6.9 \text{ hr.}$$

compared to the previous value of 4.67 hr. From these values Smith concludes the thickness of sealing has little practical significance as far as thermal values are concerned. He further compared the times of frost penetration for similar depths, temperature conditions, and surfacing and arrived at the following values for the gravels given in Table 27:

Gravel	Hr.
Mataura River	16.7
Awarua Pit	8.1
Woodend Pit	11.9
McKenzie Pit	10.8
Gordon Pit	17.9

Smith /1946-1 summarizes the results of his work on thermal properties of gravels and surfacing and concludes that "certain gravels and surfacing have greater or lesser rates of transmission of heat, but that this layer is relatively so thin that the few hours of protection gained by the use of the most suitable material is almost negligible when considered in terms of a long continued frost of perhaps 36 days. The principal protection against a deep

frost penetration is the ability of the earth to transmit heat to the surface rather than an efficient cover over the soil to provide thermal insulation."

Taber /1930-9 brought out that the composition of soils is a factor, and that the thermal properties depend on the specific heat and conductivities of the soil constituents. He stated, however, that "the soil minerals differ little from one another in their thermal properties but they differ greatly from water. Since minerals cool more rapidly than water, mineral particles, and especially large ones, favor the downward movement of the freezing isotherm in soils, and therefore tend to check the growth of layers of segregated ice. Other things being equal, soils with a high percentage of organic material might be expected to favor ice segregation, but from the tests so far made the difference appears to be slight." This is especially true when they contain sufficient colloidal material to make them relatively impervious. The following explanation may serve to show the effect which the thermal properties of the soil materials have on frost penetration. The large effect which water content has on penetration of freezing will be discussed later.

Influence of Soil Structure and Composition

Several investigators have observed that the structure of a frozen soil bears some relation to the structure of the soil before freezing. Beskow /1935-1 noted that ice crystallization occurred principally in open fissures which are progressively formed below the frost line. Riis /1948-25 believed the high frost susceptibility of chalk in Denmark to be due to the fine cracks which facilitate the passing of water.

Nonuniformity in texture also influences frost action. Beskow found that very small variations in average grain size or in grading may cause relatively large differences in pore volume and therefore can influence the water content prior to freezing. The differences in pore volume may influence the rate of water movement to the freezing zone and thus influence the amount of ice segregation. If the grain size gets larger with depth with the coarsest soil nearest groundwater level, the amount of flow is increased. The contrary is true if the grain size diminishes with depth. Also, if a very thin layer of silt or clay occurs in sands, a relatively thick layer can form, making it appear that a sand causes segregation. This is illustrated in the schematic diagram in Figure 76. He cites several cases where that occurred. In one instance a single ice layer 20 cm. (7.87 in.) thick formed at the boundary between a silt sediment overlying moraine material. The importance is most marked in varved soils (fairly regular alternating layers usually of silt and clay). Smith /1946-1 found frost difficulties when small lumps of clay occurred in gravel bases under sealed surfaces. The clay lumps swelled when frozen causing small domes 6 in. in diameter and ¼ in. high to appear on the surface. On thawing, the clay exuded and eventually caused the appearance of small pot-hole.

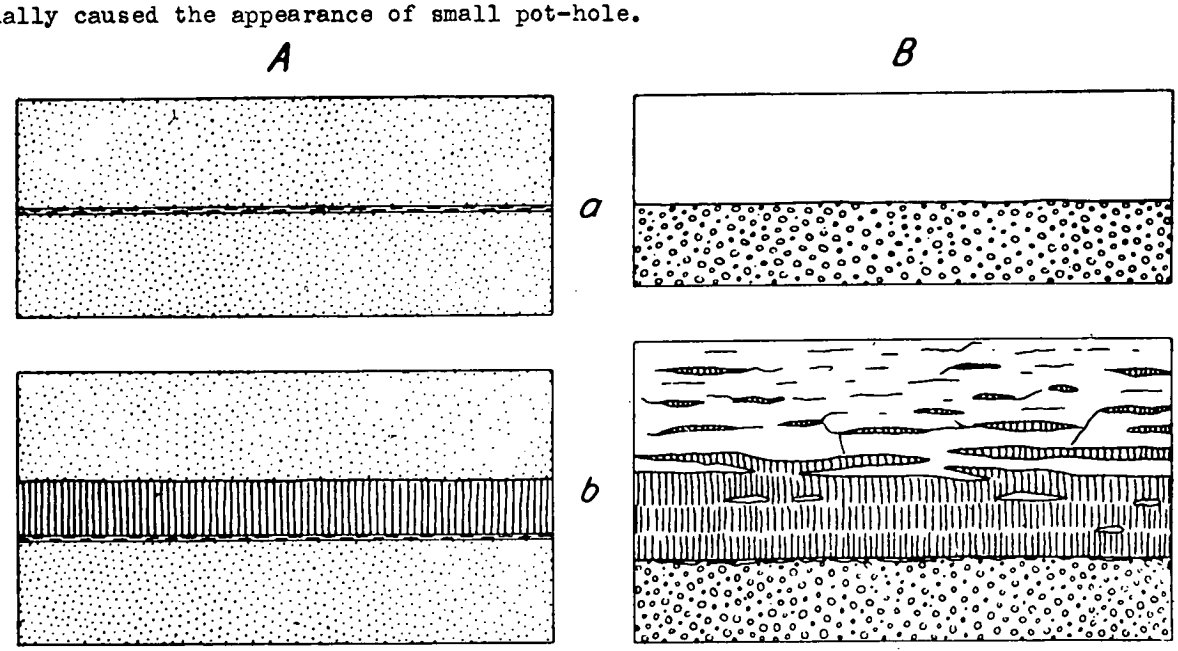


Figure 76. Diagram Illustrating the Origin of Particularly Thick Ice Layers, Above the Surface Between Different Soil Layers. (a) Unfrozen Soil. (b) Frozen Soil.

A. A Single Thick Layer of Ice Forming Over a Clay or Fine Silt Layer in Otherwise Non-Frost-Heaving Saturated Sand.

B. Very Heavy Ice Stratification At the Bottom of Clay or Clay Loam Layer Resting on Saturated Gravel or Sand. (After Beskow)

Effect of Soil State

It has been shown earlier that, unless water is fed under hydrostatic pressure, coarse grained soils are generally less susceptible to detrimental frost action. The susceptibility of fine-grain soils, i.e., silty and clayey soils, to frost action depends in a large degree on the water contained in the soil, the proximity to a ground water table, and in a lesser degree on the soil density.

Soil Water Conditions

Moisture Content - Some data on the influence of soil moisture content on the occurrence of frost action have been given under preceding sections of this review, particularly under "Heaving in Open Versus Closed Systems" and under the items of "Capillarity" and "Permeability." Regarding soil moisture, it is of interest to know (1) what minimum limiting moisture content will cause significant ice segregation and heaving in different soil types and (2) what minimum initial moisture content will cause sufficient moisture segregation at the frost line to result in significant reduction in load carrying capacity on thawing.

Unfortunately the literature does not adequately answer these questions. Taber's experiments with clay and Beskow's investigation of effect of moisture content in partly saturated sands have been reviewed earlier herein. Data by the Corps of Engineers /1947-2 are significant. They observed moisture contents during the normal (summer), freezing (winter) and frost melting (spring) periods for 15 airfields. The results showed that the segregation of ice in the form of crystals or lenses was negligible when the degree of saturation was below 65 percent. Few very-thin lenses were found in 4 of 36 locations where the normal-period (late summer and fall) degree of saturation was less than 65 percent on soils found frost susceptible during the investigation. The greater the degree of saturation the greater the magnitude and extent of frost action. Soils with natural moisture content below the plastic limit prior to freezing showed negligible segregation of ice in crystal or lens form.

Summarizing, it can be said that the information available to answer the two questions set forth above is limited and is only qualitative in nature. Further research is needed to provide additional reliable data on minimum limiting moisture contents which cause detrimental frost action.

Methods of Measuring Soil Moisture Content In Place - Because an accurate appraisal of frost susceptibility requires knowledge of the degree of saturation of soils and the effect of the degree of saturation on heaving and on reduction in bearing capacity on thawing, it is of interest to know of the various methods which are available for measurement of in-place soil-moisture content.

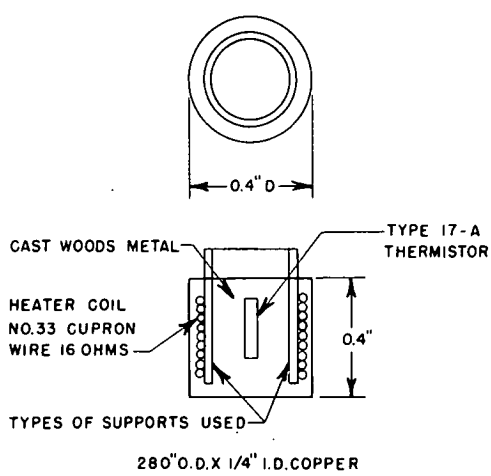


Figure 77.
Latest cell modification.
(After Aldous and Lawton 1951-19)

Belcher /1951-20 presents the results of preliminary laboratory and field experiments utilizing the ability of some radioactive materials to react with great sensitivity to moisture. The instrument developed consists basically of a source of fast neutrons which rebound with reduced velocity (becoming slow neutrons) upon striking the hydrogen nuclei of soil moisture, and a detector and counter of these slow neutrons deflected back to the source or its immediate vicinity. Calibration of neutron count with moisture content has met with marked success, although some difficulties are apparent. A closely allied technique of determining soil density through measure of gamma-ray scattering is described in the same paper.

Aldous and Lawton /1951-19 have described, in much detail, progress in the development of a suitable heat-diffusion type, moisture-and-temperature-measuring cell.

Many porous-block cells were tested. All were rejected and effort was concentrated on the development of a direct-contact cell. The latest modification in these studies is shown in Figure 77. Time-temperature rise characteristics of this cell are shown in Figure 78. The developers consider this modification unsatisfactory in

soils of moisture content above 15 percent by dry weight, and note difficulties of establishing and maintaining positive contact between cell and ambient soil. Modification continues.

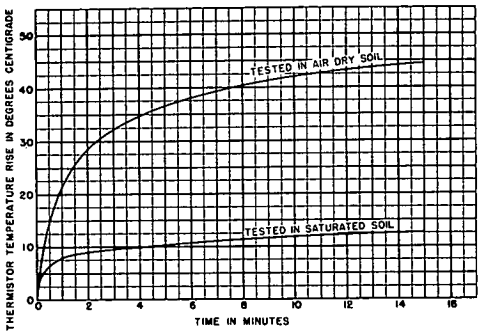


Figure 78. Time-temperature characteristics of above cell (Aldous and Lawton /1951-19)

The plaster of paris block electrical resistance method has been much used for agronomic purposes. A somewhat more recent development, utilizing similar principle, is the nylon electrical resistance method. Bouyoucos /1951-18 summarizes the advantages and disadvantages of the methods in their present stage of development. He lists the outstanding limitations of the plaster-of-Paris blocks as (1) short life under wet conditions, (2) lack of sensitivity at moisture contents approaching saturation, and (3) magnitude of influence by high salt-contents of the soil. As an illustration of the latter limitation, the blocks should be placed at least 5 in. below a concrete slab. The nylon unit has significant advantages as well as numerous limitations, among which can be numbered: (1) sensitivity to salt content, (2) tendency of air bubbles to adhere to the electrodes and nylon fabric wrapping, and (3) difficulty of achieving positive contact between unit and the soil.

Ground Water - A most useful, and important, and also perhaps one of the least understood (from a fundamental point of view) factor related to frost action is the occurrence of ground water. This review makes no pretense of covering the subject thoroughly. Rather, an effort is made to devote most of the review to that part of the literature which analyzes ground-water conditions as they are related to frost action.

The terms used to denote ground water are numerous and confusing. The terms ground water, phreatic water, vadose water, subsurface water, water table and free water are a few of the more common. Meinzer /1923-1 recognized that "part of the subsurface water which is in the zone of saturation is called 'ground water' or 'phreatic water.'" The subsurface water above the zone of saturation and in the zone of aeration he calls "suspended subsurface water" or "vadose water." A number of investigators have gone to much effort to classify soil water into such groups as hygroscopic, funicular, capillary, etc. It has been shown that there is no sharp definition between any classes of soil water.

Perhaps the best approach to defining ground water and understanding its behavior is on the basis of the pressure under which it exists. The pressure may be positive or negative, i.e., higher or lower than atmospheric. If greater, it can flow into a bore hole as free water. According to Beskow /1935-1 the ground water is the "water in a soil whose hydrostatic pressure is greater than atmospheric" and the ground-water surface is a pressure surface. Where it lies depends on the properties of the soil.

The capillary rise in a soil depends on the grain size of the soil, which in turn governs the size of the pores. If a dry column of sand soil is placed with its bottom end in water, the water will rise above the free water surface in an amount equal to the capillary rise. In very-coarse sands and gravels, which show little or no capillary rise, the capillary level and ground-water level will coincide, but as soon as the soil has some noticeable capillarity, the capillary level rises above the ground water level. Beskow cites some examples to explain the phenomena. Assume a tube to be filled with sand, inserted in a container of water, with the water level in the column to its full height and with the system is in hydrostatic equilibrium. The free water in the container represents the ground-water level. At this level in the tubes, the pressure in the pores is the same as atmospheric. Above that level the pressure is negative and is greatest at the meniscii of the capillary boundary and is equal to the total capillary pressure (capillary rise). The top of the capillary zone is a boundary between large and small water-contents. The ground-water table is in saturated soil and represents no increase in soil water-content. It is only a pressure boundary. Above that pressure boundary is the capillary zone which ranges in height between about 1.5 to 5 in. for coarse sand and 5 to 20 in. for fine to very-fine sand when full hydrostatic equilibrium is reached.

Fine-grained soils are seldom in hydrostatic equilibrium due to change in ground-water level or due to evaporation from the surface. This makes the determination of ground-water level in some fine-grained soils more complicated. Assume a tube of soil which is inserted

in water before a condition of equilibrium is reached (during capillary flow). At the bottom surface the full hydrostatic (positive) pressure is exerted. At the top of the capillary rise the full capillary (negative) pressure is exerted. The zero pressure (atmospheric) line which is the ground-water line lies between in a position such that "the distance to both surfaces is proportional to the capillary pressure (negative) and the pressure (positive) at the bottom of the sand column." For example, if the bottom of a tube of soil having a capillarity of 100 cm. is inserted 10 cm. below the free-water surface and the capillary surface is at the same level as the free water (10 cm. above the bottom of the tube), then the ground-water line is 1 cm. from the bottom of the tube. In other words, the positive pressure head at the time and for the conditions described is 1/10 of the capillary pressure $\left(\frac{10}{100}\right)$ and the ground water line will stand 1/10th of the distance up from the bottom to the height of the water outside of the tube. If a hole were bored through the soil, free water would begin flowing into the hole 1 cm. above the bottom surface.

Consider the case where the soil and ground-water conditions are as shown in Figure 79. If a bore hole is made to the depth of the gravel, the water rises rapidly into the bore hole until it reaches a height indicated as the ground-water level. This makes it appear that the ground-water surface is parallel to and near the ground surface as is indicated by the dotted line in the sketch, and that the danger from frost heave is the same all over. This is wrong. Actually, the ground-water surface is at some point in the soil above the gravel, as indicated in the figure and as explained above, and the deeper the hole is bored in the soil overlying the gravel, the higher the water will rise in the hole, reaching its maximum as the gravel is reached. The important distance, insofar as frost action is concerned, is not necessarily the depth to ground-water table but the depth to the source of water, which (in this case) is the depth to the gravel.

If the soil is a fissured clay, then the fissures will fill with water to the height it will rise in a bore hole and the ground-water level will be that shown by the dotted line in Figure 79.

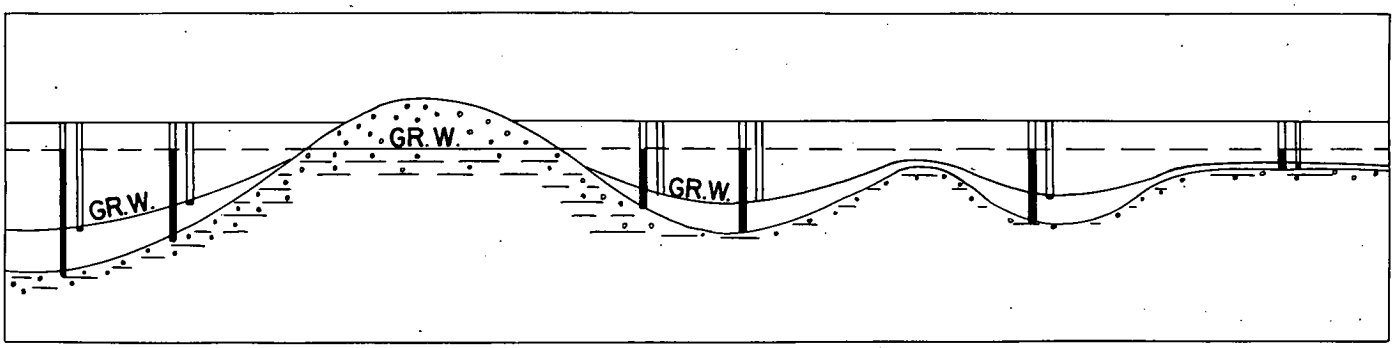


Figure 79. Schematic diagram illustrating position of ground water in nature: a cover of fine silt without fissures, resting on a highly permeable, water-conducting gravel ground. The ground-water level (Gr. W.) in such soils is only a pressure level without any practical importance. Decisive here is the distance to the coarse, water-supplying bottom layer; but also the water pressure at this bottom layer is of some importance, the greater the more permeable the sediment is (coarser, or, on the contrary, loamier and thus a little fissured).

If the soil were a real clay or loam, with a well developed fissure system, the conditions would be quite changed: then the broken line would represent the ground water level, below which the fissures, and certainly all the bores, would be water-filled.

(After Beskow /1935-1)

Beskow /1947-12, in considering ground water in soils, classifies soils into three main groups:

1. Gravels and sands - These soils are very permeable. The ground-water surface can be found easily by measuring the depth to the water surface in a bore hole.

2. Fine silts containing no clay - These soils have small permeability, small pores and no fissures. The ground-water level cannot be found practically and has no significance in determining its frost-heaving properties.

3. Clay - Clays are quite permeable due to fissures between soil aggregates. The ground-water table can be found as for sands. It is of great importance in determining the frost heaving properties.

The gravels and sands group include only the coarsest, heaving soils, the border-line coarse silts. However, the gravels and sands are important when they occur in strata in other soils. In fine silts without fissures the ground-water surface has little significance. Borings in fine silt, however, are very important in determining the depth to the permeable, water-bearing layer. The rise of water in the bore hole on reaching the permeable layer has little significance for the finer silts except that it determines whether or not there is an excess of water accessible to the upper layers. The coarser the silt the more significant is the rise of the water in the bore hole. The important thing to observe is the depth of the water supply.

The ground-water level in clays is critical, because the level occurs in the fissures much the same as in the pores in gravel. The quantity of water is not so large, being about 30-35 percent by volume in gravels and about 5 percent by volume in the fissures of the clay. Water rises rapidly in a hole bored through fissured clays. Beskow /1935-1 states that "the amount of water sucked up is a function of this very measurable ground-water depth, and the rate of frost heave is, for practical purposes, inversely proportional to this depth." He cautions that the fissure system in clays exists only in the upper part; the number and the size of the fissures decrease with depth and usually disappear almost entirely at depths of 9 to 10 ft. Thus, with increasing depth, the mobility of water decreases and the fissures affect the frost-heaving properties of clays less.

Summarizing, it can be said that soil types and ground-water conditions can be determined only through borings, and it is necessary to distinguish between two principal conditions of ground water:

1. Where free water rises in bore holes without encountering any change in soil composition or structure. This occurs in sands, coarse silts, fissured clays and some varved (stratified) materials having permeable, coarse layers. In those materials the level the water attains in the bore hole is the ground-water level which must be known if the frost heaving properties are to be known. He emphasized that it is the depth to ground water during freezing that is important.

2. Where free water breaks through and rises to a considerable height in a bore hole through fine silt (with practically no clay) when the hole reaches another coarser layer, Beskow states that "here the most important fact is where the water broke through and next the height to which it rose; but this level does not represent the ground-water surface." Here the true elevation of the ground-water level is almost impossible to determine practically and is not of major significance for frost heaving.

Silt lies somewhere in between these cases. When, on drilling through silt, a coarser layer is reached and water rises in the hole, it is good practice to drill another hole to 1/3 (then other hole to 1/2 and 3/4 of the depth) and wait 30 minutes or more to see if any water enters the holes in that time. If some water shows in the bottom of the hole in about a half hour, then it will be possible to determine the ground-water level and the level will have some significance.

Start

Effect of Depth to Ground Water on Heaving

Taber /1929-2 recognized that where the soil texture is uniform differential heaving will occur where there is a difference in the position of the water table relative to the surface. Casagrande /1931-14 presented data on heaving of and soil conditions (during 1927-28) along 2,000 feet of concrete pavement in New Hampshire but offered no discussion. The data are shown in Figure 80. It is interesting to note that the greatest heave occurred where the water table was nearest the surface. It has been mentioned previously that Beskow /1935-1 found a relationship between pressure (load and capillary pressure) and rate of heaving, stating that "the rate of frost heaving for a given soil is inversely proportional to the square of the pressure after the pressure exceeds a certain but not large value." It was mentioned that capillary and load pressure have a similar effect. The capillary pressure is directly related to the distance to the water table, increasing with increase in depth to the water table until the maximum capillary pressure (equal to maximum capillary rise) is realized.

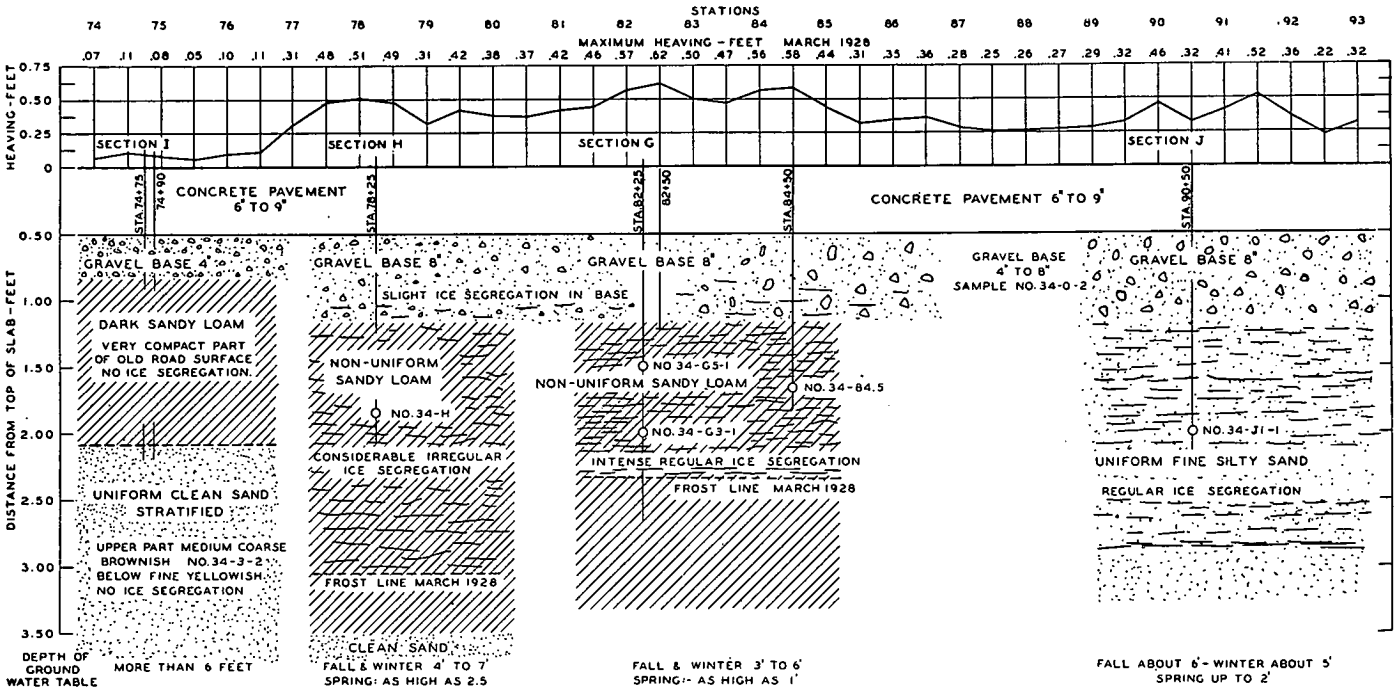


Figure 80. Relationship Between Heaving and Soil Type and Location of Water Table During the Winter Season (After Casagrande)

The relationship between rate of heaving and distance to ground-water is approximately a linear one, the rate being inversely proportional to the distance. That relationship is quite close for most soils, deviating for fine silts, unsorted silts, and silty-moraine material. Thus if the distance to ground-water is increased from 20 to 40 in. it will normally reduce frost heave by one half. Beskow /1935-1, /1938-11 presented several good examples from field data showing the effect of depth to ground water on heaving.

The application of knowledge concerning ground-water is of some value in subsurface drainage, covered later in this review. The Corps of Engineers investigations /1947-2 showed a relationship between depth to ground-water and frost action:

"Extensive to slight frost action occurred in frost susceptible soils where the water table is less than 12 ft. from the ground surface and where there is no stratum, such as a layer of clean sand, which will prevent the upward flow of water when freezing starts. Slight to no frost action occurred in frost susceptible soils where the water table is below 25 feet of where there is a stratum of clean sand above the water table which cuts off upward flow of water."

It was found that the degree of saturation varied generally with the depth to ground-water. The higher the ground-water table the greater the degree of saturation.

Density - The capillarity and permeability of a soil are closely related to and are influenced by the total porosity and by the sizes of the soil pores, which are dependent on grain-size distribution. The inter-relationships between porosity and frost action have been given previously in this review in terms of grain size. A relationship exists, however, between frost action and density (state of packing) for a given soil.

Taber /1930-9 stated, "Experiments have demonstrated that heaving is greater on unconsolidated clays than on those which are thoroughly consolidated." He also wrote, "A cylinder of undisturbed cretaceous clay will heave at the rate of 0.15 mm. per hr., while the same clay when pulverized and tightly packed will heave 0.8 mm. per hr." He gave no values on degree of compaction nor did he discuss the effect of soil structure on his results.

Beskow /1935-1 found the degree of packing affects the capillarity of soils and, since frost action depends on the rate at which water can be drawn to the freezing zone, affects heaving.

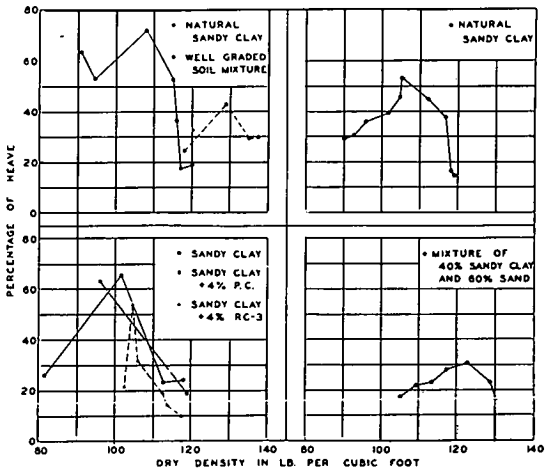


Figure 81. Effect of Density - Series 7, 8 and 9. (After Winn and Rutledge)

Winn and Rutledge /1940-7 made laboratory tests to determine the relation between frost heave and density on a natural, sandy clay /11 and on admixtures (q.v.) of the clay with other materials. The results are shown in Figure 81. Although the studies were limited in scope they showed a trend of increasing heave with increasing density until a maximum was reached. At greater densities heaving was less. The soil had the following test values: Sand 12 percent, Silt 50 percent, Clay 38 percent, L. L. = 46, P. I. = 27, A.A.S.H.O. Std. Density = 108 pcf.

The authors concluded, "The available data indicate that there is a critical density for sandy clay at which frost action occurs most readily when material is saturated. Below the critical density, frost action is directly proportional to density; above the critical density, frost action is inversely proportional to density. Increasing the density above the critical density increases the period of inactivity before heaving starts and decreases the rate of heaving and total heave in a manner similar to the addition of admixtures."

Lang /1940-11 molded "Proctor" specimens (4in. diam. and 4.6 in. high) of a clay loam, a silt loam, and two clays at several moisture contents; covered them with rubber jackets; and allowed them to absorb water. They were frozen 5 cycles with one end in water causing appreciable volume change. In reporting his results Lang questioned the value of compaction stating, "When one considers the possible disrupting effect of frost action there may be some question as to whether the cost is justified. Under favorable conditions for the formation of ice layers the densified soil may revert to its natural density."

Sloane /1949-10 conducted a laboratory study of the effect of freezing and thawing specimens of four soils which had been compacted to very high densities (to the maximum densities obtainable with the compaction equipment available). He concluded that if the soils are saturated and water is available "compaction to ultimate density would not prevent heaving and destruction of the soil structure but would only delay deterioration." Comparative studies on a heavy clay, compacted at A.A.S.H.C. Standard Compactive effort, showed that it produced "a very much greater heave" than did the very highly compacted specimens.

Bleck /1949-11, in discussing freezing of accumulated moisture in subgrades, states that "observations have been made where this moisture upon freezing had completely destroyed the density of so-called stabilized-aggregate-base courses compacted to measured densities of the order of 140 lb. per cu. ft."

Hansen /1943-12 observations concerning reduction in density due to frost action are quoted in part as follows: "From our knowledge of the properties of granular material which is likely to be used as subbase it is not believed that serious loss of density and increase in moisture is likely to occur in these materials."

The sizes of the capillary pore spaces was held by McDonald /1949-19 to be a major controlling factor in the susceptibility of a soil to frost action. He used the AASHO Standard Compaction Test to aid in identifying frost-susceptible granular-base materials. He modified the equipment by grooving the base of the mold to provide drainage during compaction and also observed penetration resistance readings and the behavior of materials at high water contents during compaction. He noted that some granular base materials when compacted, even at moisture contents above optimum, were firm and yielded but little under the impact of the rammer and gave a resounding blow when strucked. Other granular base materials gave a yielding and a cushioning effect under impact, and the blow of the rammer produced a "dull thud" with no rebound or the materials yielded readily and were deformed under the impact of the rammer. Drainage was recorded as none, slight, or large. McDonald found good correlation between spring-season softening (as indicated by distress in flexible surfaces) and the observations made during the compaction test as he performed it. He held that materials which are capable of holding water contents well in excess of optimum without drainage during the compaction test are likely to suffer instability during freezing and thawing. He concluded, "The relative loss in stability as measured by penetration resistance, from the optimum moisture content to the state of full water capacity is indicative of frost susceptible material when the stability loss exceeds 50

/11 Some textural classifications would term this soil a silty clay.

percent." Generally, base materials having more than 10 percent passing the No. 200 sieve are susceptible to loss of stability on freezing and thawing.

The recent work of the Corps of Engineers /1951-32 (see Influence of Grain-Size Distribution for Method of Preparing and Testing Specimens) included investigation of the influence of initial density on frost heave. The results obtained to date are shown in the second grouping in Table 15 (used previously) and in graphical form in Figure 82. It may be seen that for the New Hampshire silt and the Ladd Field, Alaska, subsoil, heaving increased with increase in the initial dry density. However, for the East Boston Till and Truax Drumlin materials, heaving increased with increase in density up to 120 pcf. and then decreased with further increase in density. Thus heave may either increase or decrease with increase in initial density, depending upon the degree of densification attained, and the effect of density on heave may be large or small depending on the factors involved.

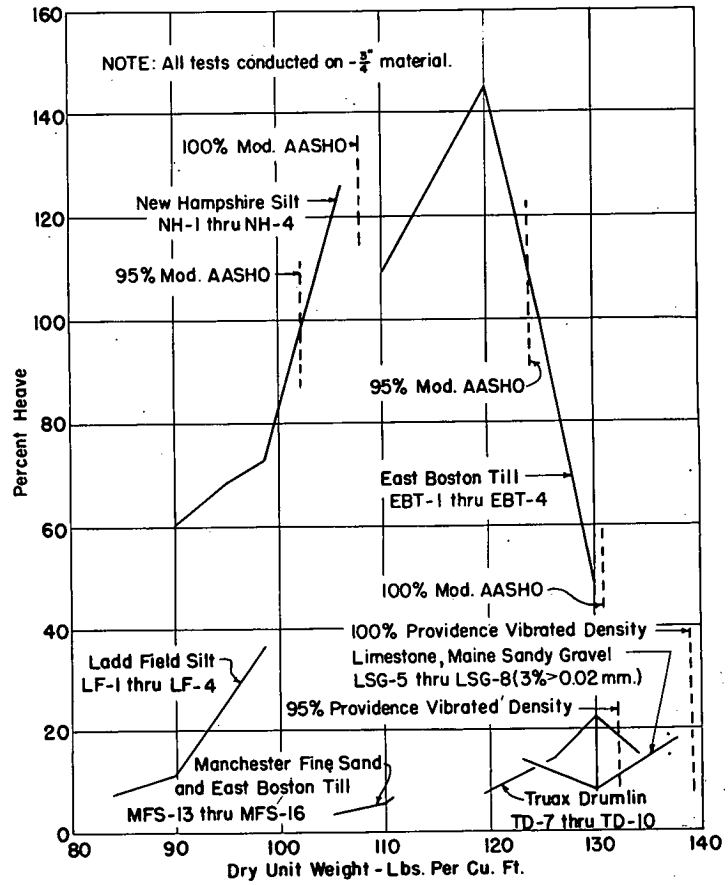


Figure 82. Effect of Degree of Compaction on Percent Heave. (After Corps of Engineers)

Degree of Saturation in Subgrades and Bases

Inasmuch as freezing, segregation of water, and reduction in bearing capacity can occur from the water contained in soils and without benefit of proximity to a source of free underground water (water table) it is essential to have an appreciation of existing subgrade moisture contents if one is to have a full appreciation of the potential frost susceptibility of subgrade soils.

The literature contains many writings which present data on subgrade soil moisture. The files of state highway departments contain a vast amount of data on subgrade moisture. Much of the available data is not classified according to the time of the year. Nevertheless, those data do have value in that they do represent existing subgrade moisture contents.

No effort is made here to make a comprehensive coverage of the literature, although soil moisture is one of the prime factors which influence frost action. Only a few of the important writings can be reviewed here.

Kersten's work /1944-14 was the first effort to collect, organize, and analyze data on subgrade soil moisture contents under flexible highway for a wide area. His efforts were devoted largely to the Midwest region. He grouped soils by textural classes (U. S. Bureau of Chemistry and Soils) and showed the difference in subgrade-soil moisture content for different soil types. His work may be summarized by expressing soil moisture content in terms of percent saturation and percent of optimum for some of the textural soil groups as shown in Table 28.

Table 28

Subgrade Moisture Contents Under Flexible Type Highways (After Kersten)

Textural Soil Type	Saturation Percent	Percent of Optimum
Sandy Loams	50-75	65-90
Loams	60-80	90-105
Clay Loams	70-85	95-105
Clays	85-95	105-115

That work was followed by a similar study of subgrade moisture conditions beneath airfield pavements /1945-12. The work on airfields did not alter materially the earlier findings except that the airfield study was nationwide and produced evidence that subgrade moisture contents in semi-arid regions were generally lower for all types of soils than in the more humid regions.

The work of Hicks /1948-52 was aimed at obtaining data on both moisture contents and densities of subgrades and soil bases in North Carolina as a basis for pavement design. His investigations were made at different seasons of the year to determine the yearly averages and ranges of soil moisture contents and densities under flexible pavements. He presented data not only on values of moisture content and density but also on the effect of compaction on soil moisture content. His findings are summarized in Tables 29, 30, 31, 32, and 33. It may be seen that the finer-grained soils showed a higher degree of saturation. A recent report from Norway (Riise /1951-22) also showed higher moisture contents in the finer-grained soils.

TABLE 29

Average Moisture Contents Found in the Subgrade Groups

H.R.B. Subgrade Group	Standard Optimum	Plastic Limit	Saturated
	%	%	%
A-1-b	82.5	36.4	69.0
A-2-4	75.5	43.7	62.9
A-2-6	104.3	62.3	85.3
A-4	106.1	65.0	82.6
A-5	114.7	54.0	89.8
A-6	109.1	75.2	85.4
A-7-5	118.9	68.2	91.2
A-7-6	109.4	70.9	90.9

TABLE 30

Average Relative Road Densities

	Standard Density	Modified Density
	%	%
Bases	100.5	96.5
Granular Subgrades	101.2	96.7
Silt-Clay Subgrades	96.8	88.8

TABLE 31
Bases
Influence of Compaction
On Road Moisture

Relative Densities Under 100 Percent

Average Relative Density	Standard Optimum	Plastic Limit	Saturated
%	%	%	%
98.5	75.0	43.8	60.3

Relative Densities 100 Percent and Above

Average Relative Density	Standard Optimum	Plastic Limit	Saturated
%	%	%	%
101.1	73.1	40.6	61.1

TABLE 32
Subgrade (Silt-Clay Material)
Influence of Compaction on Road Moisture

Relative Densities Under 100 Percent

Average Relative Density	Standard Optimum	Plastic Limit	Saturated
%	%	%	%
95.7	117.0	68.9	89.2

Relative Densities 100 Percent and Above

Average Relative Density	Standard Optimum	Plastic Limit	Saturated
%	%	%	%
100.9	99.4	64.4	89.0

TABLE 33
Subgrade (Granular Materials)
Influence of Compaction on Road Moisture

Relative Densities Under 100 Percent

Average Relative Density	Standard Optimum	Plastic Limit	Saturated
%	%	%	%
98.1	85.1	46.9	67.2

Relative Densities 100 Percent and Above

Average Relative Density	Standard Optimum	Plastic Limit	Saturated
%	%	%	%
103.9	70.5	42.3	61.8

During 1944 the Corps of Engineers began a study of soil moisture contents under 8 airfields located in New Mexico, Texas, and the Mississippi Valley. The purpose of the study was to obtain data on subgrade moisture contents for preparation of test specimens for setting up designs. The sites were selected in rainfall regions of less than 15, 15 to 35, and more than 35 inches annual precipitation as shown in the tabulation below.

Site	Nearest Town	Rainfall Region
Kirtland Air Force Base	Albuquerque, N. Mex.	Less than 15 in.
Santa Fe Municipal Airport	Santa Fe, N. Mex.	"
Clovis Air Force Base	Clovis, N. Mex.	"
Lubbock Municipal Airport	Lubbock, Texas	15 to 35 in.
Goodfellow Air Force Base	San Angelo, Texas	"
Bergstrom Air Force Base	Austin, Texas	"
Keesler Air Force Base	Biloxi, Mississippi	More than 35 in.
Memphis Municipal Airport	Memphis, Tennessee	"

The sites in each region were also selected to give a range of subgrade materials, and test locations were designated at the center of each runway, between an edge and the center, at the edge, and on the shoulder. In addition, tests were to be made on locations near an artificial crack-cut in the pavement. All moisture values were determined by direct sampling. The observations were begun in the fall of 1945 in New Mexico and in the spring of 1947 in the other locations. In addition to the moisture contents in place densities and California Bearing Ratio values were obtained at the time of the samplings. The results to date were reported by Redus and Foster /1951-24.

The authors report that, although the observations have not been extended as long as is desired, the following trends are indicated.

a. Edge versus center - Gravel base courses with plastic fines, GC, and Clay subgrades, CL and CH, generally showed higher values of moisture content and percent saturation at the edge than at the center. The other base courses and subgrades did not show consistent trends.

b. Pavement versus shoulder - Although exceptions occurred, the maximum moisture content and percent saturation were generally higher under the pavement than in the shoulder.

c. Crack - The California Bearing Ratio in the base course and subgrade generally increased with distance from the crack while the moisture content generally decreased.

d. Time effects - A study of the changes in condition with time revealed a seasonal variation in moisture content in both base and subgrade except in the sand subgrade at Keesler field.

Figures 83 and 84 show the frequency distribution curves (in percent of total observations) of the moisture contents and degrees of saturation for the base courses and subgrades at each of the fields. The plots are grouped according to type of material and arranged in order of increasing rainfall from left to right.

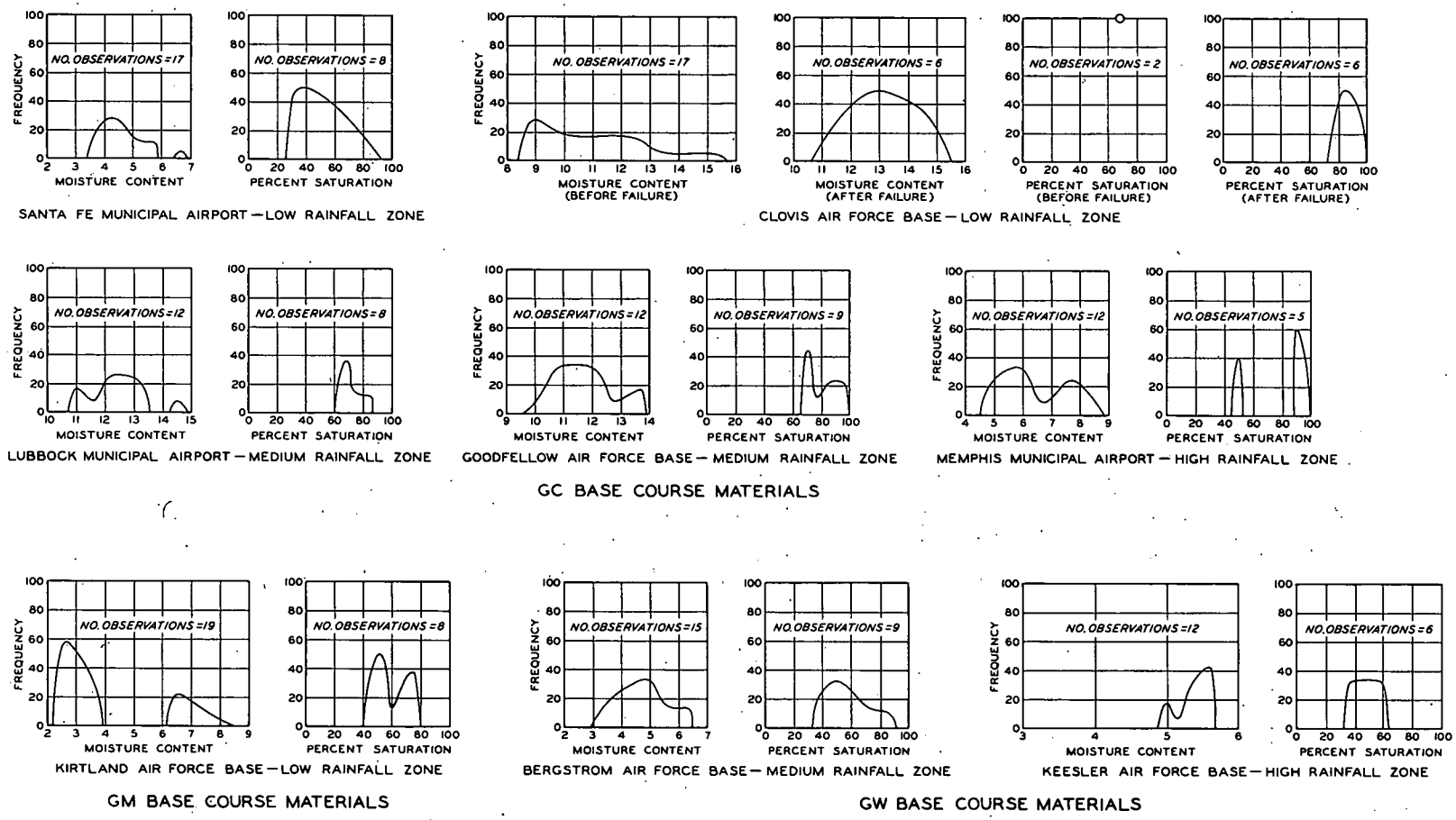


FIG. 83

FREQUENCY DISTRIBUTION
MOISTURE CONTENTS AND PERCENTS SATURATION
BASE COURSE MATERIALS

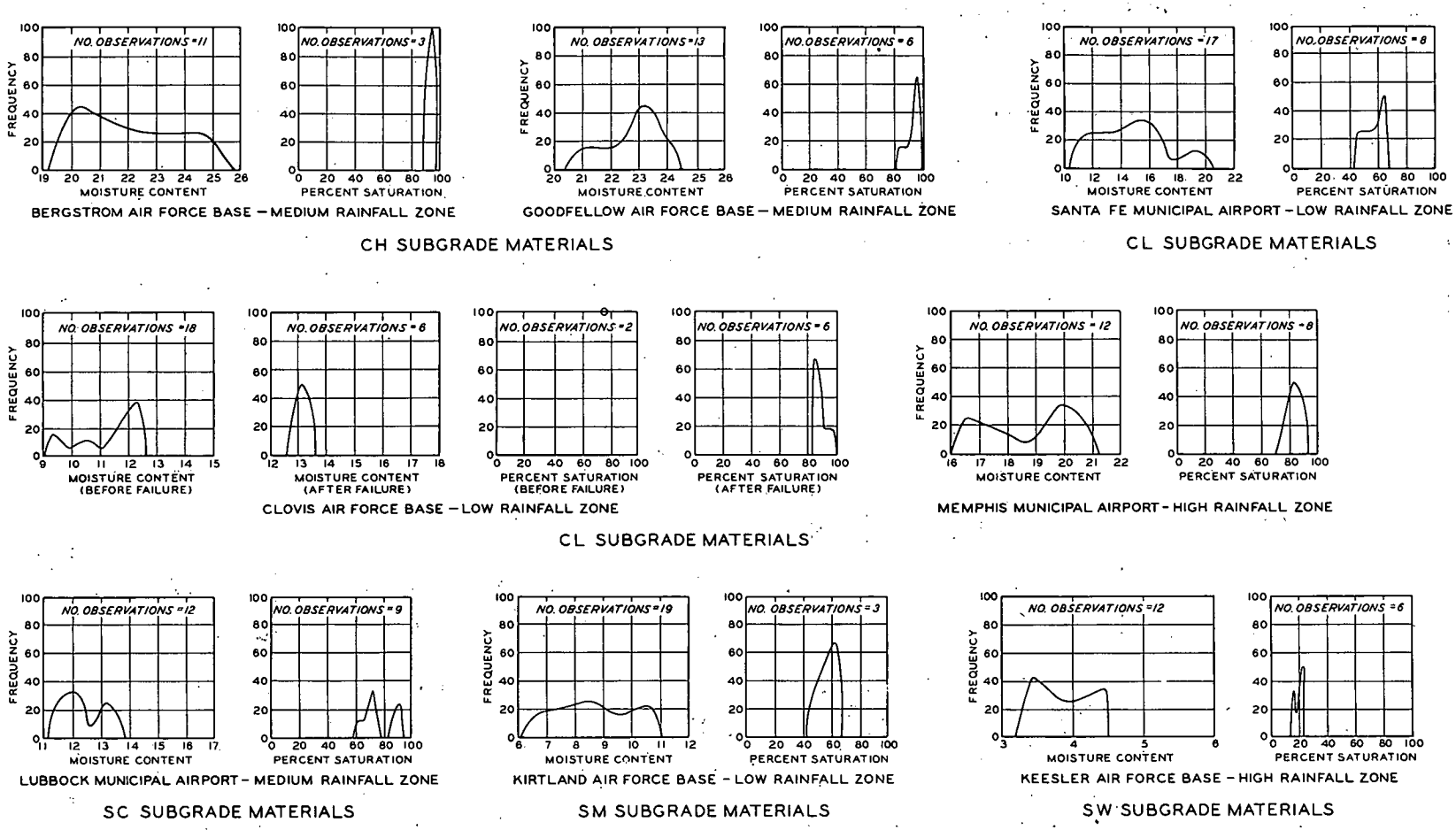


FIG. 84

FREQUENCY DISTRIBUTION
MOISTURE CONTENTS AND PERCENTS SATURATION
SUBGRADE MATERIALS

The values of moisture content and percent saturation shown in Figures 83 and 84 are summarized in Table 34 and were read from the plots at the points where the curve crosses the 20-percent-frequency line on the high side of the peak.

TABLE 34

SUMMARY OF MOISTURE CONTENTS AND DEGREES OF SATURATION

Material	Name of Field	Rainfall Zone	Moisture Content	Percent Saturation
	Base Course			
Clay gravel, GC	Santa Fe	Low	4.8	78
	Clovis (before failure)	Low	9.8	68
	Clovis (after failure)	Low	15.1	98
	Lubbock	Medium	13.2	71
	Goodfellow	Medium	12.5	98
	Memphis	High	8.1	98
Silty gravel, GM	Kirtland	Low	3.8	79
Sand gravel, GW	Bergstrom	Medium	5.3	65
	Keesler	High	5.7	61
	Subgrade			
Fat clay, CH	Bergstrom	Medium	25.0	97
	Goodfellow	Medium	24.1	99
Lean clay, CL	Santa Fe	Low	16.8	65
	Clovis (before failure)	Low	12.5	75
	Clovis (after failure)	Low	13.5	91
	Memphis	High	20.8	91
Clayey sand, SC	Lubbock	Medium	13.5	92
Silty sand, SM	Kirtland	Low	10.8	67
Sand, SW	Keesler	High	4.5	22

NOTE: Eighty per cent of the observations of moisture content and per cent saturation were less than the values shown in these columns.

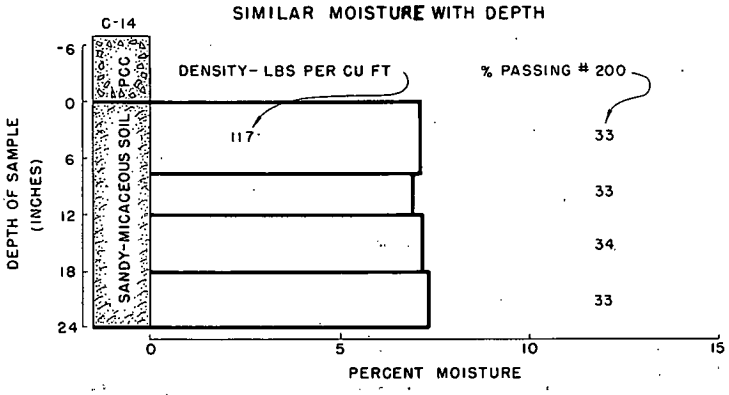


Figure 85. Comparatively uniform moisture distribution under a pavement that had been down 13 years at the time the samples were taken. The soil is a very uniform sand-micaceous silt and the moisture content of approximately 7% represents 44% of saturation. (After Hveem)

There was a general tendency for the degree of saturation to vary according to rainfall and for the finer-grained clayey soils to exist at higher degrees of saturation than the coarser-grained soils low in fines, as found by Kersten.

Hveem /1951-21 reported in summary form results of an extensive investigation of subgrade moisture contents begun in 1944 by the California Division of Highways. The sampling was done through holes bored with a diamond bit through concrete pavements. He illustrated by charts five different types of moisture distribution. Those are shown in Figures 85 to 89.

Figures 85 to 88 represent borings where the soil was of the same character for the entire depth. Figure 89 represents a condition of non-uniform soil type.

Hveem reported a trend of increasing saturation of subgrade soils with increasing age of the overlying pavement. Similar values were also reported by Lancaster in his discussion of Kersten's work /1944-14. The relationship is shown by Figure 90. Hveem also showed a relationship between surface area and percent moisture, the moisture content showing a trend of increasing with increase in surface area. That relationship is illustrated in Figure 91.

A review of reports on subgrade moisture contents show various means of presenting data by comparison with results of other soil tests as a means of interpreting their significance. The most common method is by expressing the moisture content as a percentage of full saturation. Other means include percent of liquid limit, percent of plastic limit, and percent of optimum moisture content (based on AASHO compaction test). A recent report by Riise /1951-22 expresses soil moisture content as a percent of capillary saturation obtained in a capillary absorption test.

Summarizing, it may be said that the degree in which a subgrade soil or base course in service is saturated depends upon:

The nature of the soil (texture and composition),
The nature of the subgrade soil profile,
Its porosity (degree of densification),
Climatic conditions (including seasonal effects),
The age of the pavement,
The type and condition of the pavement, and in a degree on -
The nature of the traffic.

Those items need to be taken into consideration in any evaluation of the frost susceptibility of any soil for any given locality and the conditions under which it is to be used.

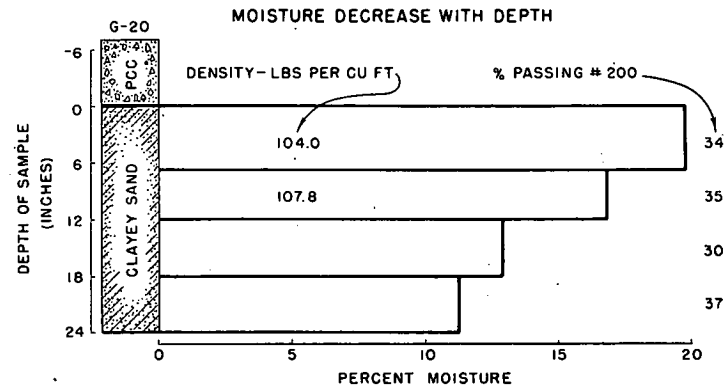


Figure 86. A number of samples were drilled where the moisture content was greatest in the upper layer, while the composition of the soil is virtually identical throughout the depth sampled. This is typical of this type of moisture distribution. This pavement had been in place for 13 years. (After Hveem)

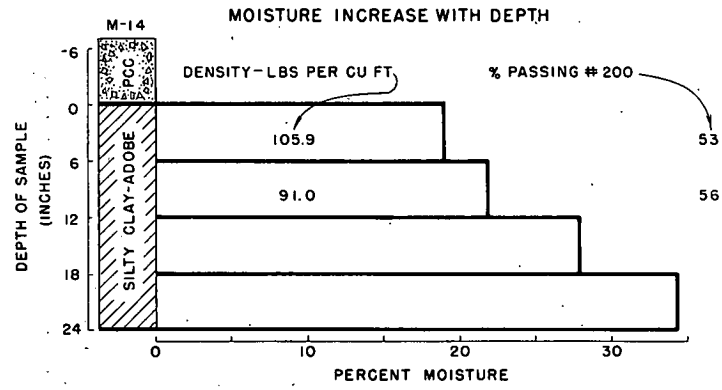


Figure 87. Moisture distribution in reverse of Figure 86. In this case, the moisture content increases consistently with increasing depth below the surface. The soils are very heavy silty clay type containing more than 50% passing No. 200. The pavement was 11 years old when the samples were taken. (After Hveem)

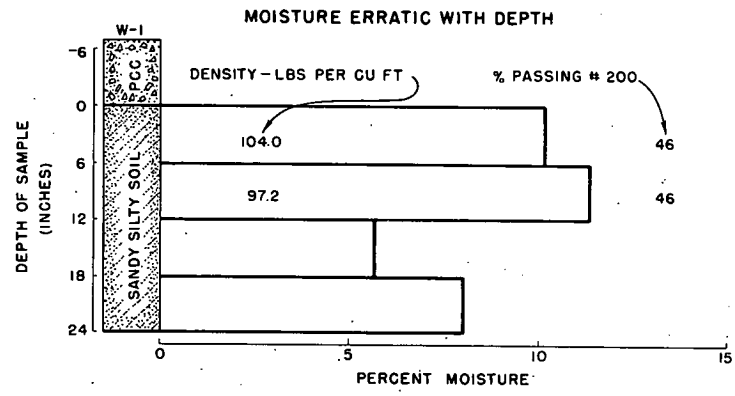


Figure 88. Boring taken through the pavement into sandy, silty soil where the moisture distribution is somewhat erratic. No definite pattern evident. The pavement had been down 15 years. (After Hveem)

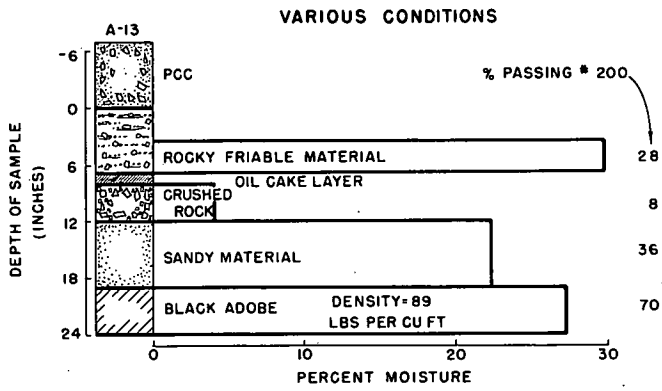


Figure 89. This figure is included to show the marked differences in the moisture content where each layer of the soil is of a different type composition. This particular boring was taken in a location where the first six inch layer beneath the pavement consists of a lightweight porous, granular material, presumably of volcanic tufa. The second layer represents an old road surface composed of an oil mixed surface on a crushed rock base. The third is a layer of sandy material imported as a subbase under the original bituminous surface, and the final layer is a local black adobe soil. (After Hveem)

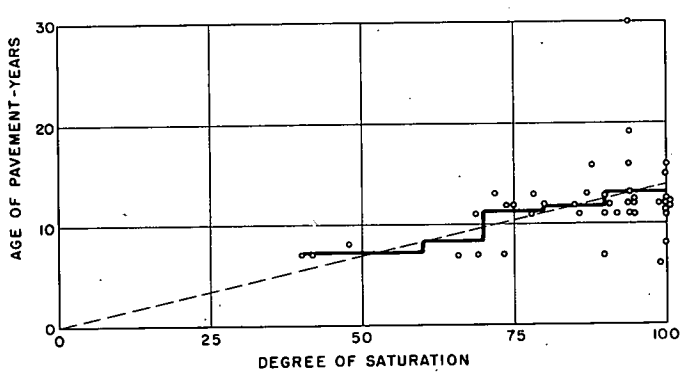


Figure 90. This chart illustrates the degree of saturation compared to the age of the pavement. These points seem to indicate that the rate at which moisture accumulates may be quite slow.

While the data are far from being conclusive because all pavement samples were at least five years old when samples were taken, It is undoubtedly true that in some cases interlying soils may have been saturated at the time of construction. Nevertheless, there appears to be a trend indicating that it may require a period of more than ten years for complete saturation to develop. (After Hveem)

Effect of Natural Formations

Geologic Formations - Burton and Benkelman /1930-10 made a detailed study of 156 heaved areas in Michigan during the period 1925-1930. They classified the occurrence of frost heaves according to land forms as follows:

Land Form	Heave Occurrance Percent
Moraine	65
Shallow outwash	15
Till plain	12
Lake bed	4
Deep outwash	4

Of the 65 percent of heaves occurring in moraines, 65 percent of those were almost equally divided between soils of the Bellefontaine, Coloma and Miami series. A detailed analysis of the cause and effects of the 156 heaves examined in the field showed that 94 occurred in a layer or pocket of fine textured material such as silt, very fine sand and silt, silty clay, clay, or sandy clay surrounded by coarser better drained material; 22 occurred in a pocket of silt of very fine sand and silt surrounded by clay; 30 occurred in medium to fairly coarse sand and six occurred in moderately coarse textures. The remaining four of the 156 heaves occurred where the subgrade soil appeared uniform with respect to character and moisture content.

The most severe occurrence of frost action in Wisconsin is in the flat phase of the Colby silt loam in the central part of the state. Bleck /1949-11 reports that the soils are wind deposited on old drift of granitic origin or on granite bedrock. The region also includes contact with the water bearing sandstones and shales of the lower Cambrian formation. Heaving also occurs in some sandy loam outwash soils where they form thin mantles over Cambrian formations. Differential heaving is not common to the driftless area although most of the soils are high in silt.

Otis /1951-33 reported a survey of 85 New Hampshire till deposits to determine the nature of the materials they contained and discussed their susceptibility to frost action. Average gradings of upper till contained 15 percent gravel, 23 percent coarse sand (2.0 - 0.25 mm.). 30 percent fine sand (0.25 - 0.005 mm.), 20 percent silt (0.05 - 0.005 mm.) and 12 percent clay. The material showed an average value of 40 percent passing the No. 200 sieve. He stated that the tills are frost susceptible, but because they occurred in large masses, differential heaving was not a major problem, although in specific instances heaves were up to 8 in.

Shelburne and Maner /1949-8 found a relationship between geologic origin of soils and their behavior during the "spring breakup" in 1948. They divided the soils in the Culpepper district of Virginia (on the basis of parent material) into five general soil areas and rated the performance of various types of road construction for those areas. Outstanding as a poor performer in the district are soils derived from Triassic "red beds". This was true for roads classed as primary as well as for the secondary type. An example of the relative performance of primary roads on the five different soil areas during the 1948 spring breakup is shown in Figure 92. The authors concluded that "the study showed the importance of the general soil area as a major variable in road performance."

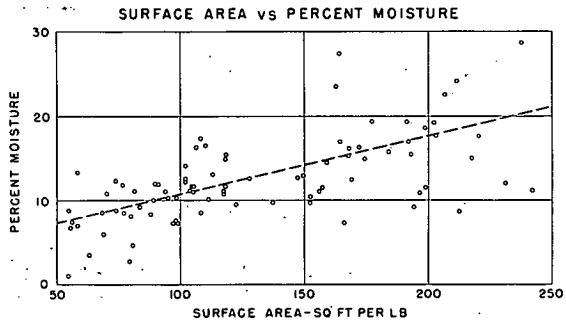


Figure 91. This chart illustrates the relationship between the calculated surface area of the soil particles and the amount of moisture. This relationship apparently represents the only discernable trend between characteristics of the soil and the moisture present. While the relationships indicated are not sharp or precise, it is also true that the values shown for surface area equivalents are only rough approximations and may be considerably in error for the finely divided clays or soils containing an appreciable amount of colloidal sizes.

The indication that there is some relationship between particle surface area and the moisture accumulates largely as a result of condensation from water vapor. (After Hveem)

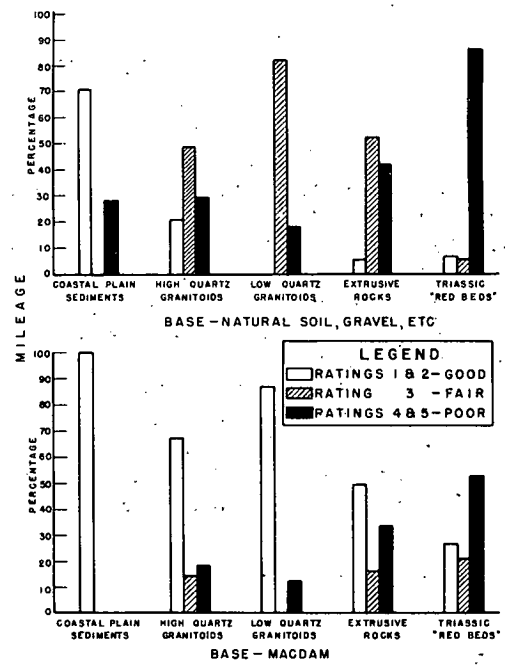


Figure 92. Primary Road Performance by Base Types. (After Shelburne and Maner)

Ledge Rock - Paradis /1934-4, Lang /1935-3, and Bennett /1940-13 found that the occurrence of ledge rock near the surface sometimes created water conditions productive of frost heaving. Paradis explained that the irregular surface of ledge rock often formed dams for underground water. He suggested that such wet basins should be drained and that "probably the best method of drainage in those cases is to crack the rock by blasting below the frost level in the ditches." Lang and Bennett found that excavation overbreak often forms pockets in the rock surface in which water accumulates and saturates the soil backfill, producing differential heaving. Otis /1951-33 reported that frost heaves are common in ledge excavation in New Hampshire.

The Soil Profile - The literature describes and illustrates many instances where the nature of the soil profile is related to the degree of frost damage which has occurred. A major portion of the profiles described are from areas of glacial deposits and many of them are examples of non-uniform materials below the weathered horizons. Although experienced engineers have little difficulty recognizing soil profiles and associated water conditions capable of producing damaging differential heaving, it is believed to be worthwhile to reproduce a number of sketches showing typical profiles associated with heaving.

Watkins and Aaron /1931-9 found that "very fine sands, silt-loams, and silty clay-loams having little or no apparent soil structure to be the greatest sources of frost boils when these materials are found associated with water carriers or where they act as water carriers themselves." They also found frost boils in New Hampshire to be associated mainly with stratified silty clay-loams and very fine sand-loams that had retarded percolation or "had seepage characteristics" and carried water into the subgrade where it froze. Aaron's report /1934-3 of the cooperative survey made by the highway departments of Michigan, Minnesota, and Wisconsin and the Bureau of Public Roads summarized the typical soil profiles in which detrimental frost heave has occurred in those states. Typical profiles associated with frost action studied during that survey are shown in Figure 93. Following are brief descriptions of the profiles shown in Figure 93.

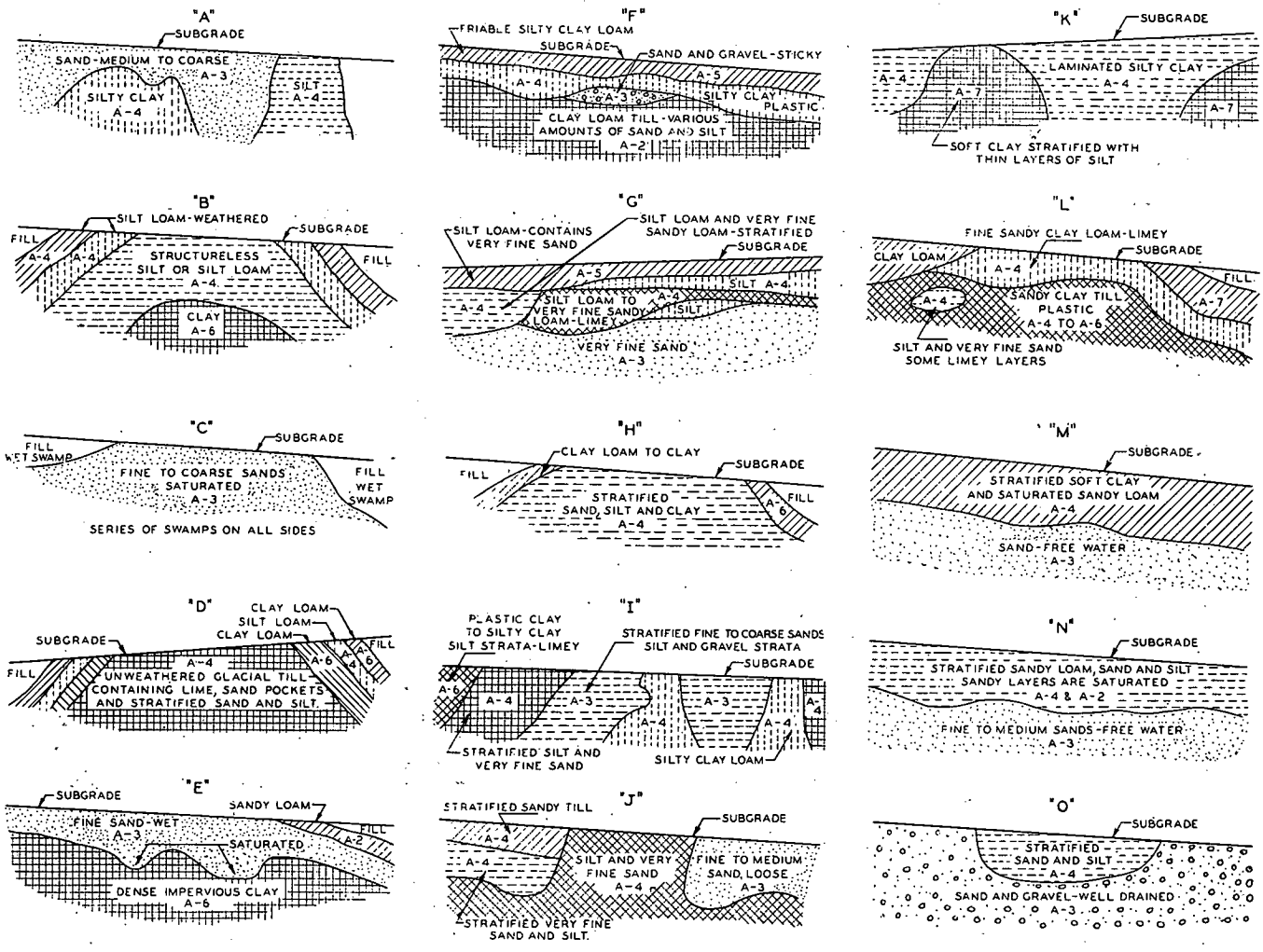


Figure 93
Soil Profiles in Which Detrimental Frost Heave has been Observed
(After Watkins and Aaron /1931-9)

A. Pockets of silt and silty clay varying in shape and depth occur within a deposit of porous sandy soils. The heaving in the silt pocket is excessive while that in the surrounding sandy soils is negligible.

B. The frost heave is confined to that portion of the road resting on the unweathered, or slightly weathered, structureless silt. The weathered upper layers of the profile are granular and apparently do not suffer detrimental frost heave. The weathered silt loams and structureless silt are A-4 soils while the underlying clays are A-6 soils. The silts contain high percentages of very fine sand and have high water-holding capacities.

C. An example of detrimental frost heave on sands of the A-3 group ranging from coarse to fine in texture, and having a water table close to the surface.

D. The relation between frost heave and the various layers in the profile in extremely-variable glacial materials is similar to that in B. The heaving occurs in an unweathered till containing lime, sand pockets, and stratified sand and silt. Considerable water collects in the sand pockets. The till is an A-4 soil, but it includes strata of A-3 soil, which is especially productive of differential heaving.

E. A soil profile where heaving occurs as a result of depressions in the surface of an impervious clay.

F. and G. A surface layer of unstable A-5 soil underlain by silts and very fine sand (A-4 soils) which vary in thickness.

H, I and J. Stratified silt and very fine sand which produce the greatest and most dangerous heaving.

K. Laminated silty clay in which very thin layers of silt assist in keeping the clay very soft and wet.

L. The majority of clay loam soils similar to those indicated are subject to detrimental frost heave only when wet and poorly drained, the sandy clay being more subject to heaving especially when it carries appreciable amounts of disseminated lime. No serious damage has occurred on lime-free soils of the A-7 group.

(Reviewers Note: The soil groups given above are for the Bureau of Public Roads Classification system in use in 1934).

Lund /1951-42 presented detail soil tests (see Table 35) from three different locations in Nebraska to illustrate the diversity of materials in which major heaves occur if water accumulates in a pervious soil underlain by a less pervious one.

Table 35

SITUATIONS SUBJECT TO FROST HEAVE

	Example No. 1		Example No. 2		Example No. 3	
	Upper Stratum (Top-Soil)	Lower Stratum (Pierre Shale)	Upper Stratum (Fine Sand)	Lower Stratum (Pierre Shale)	Upper Stratum (Peorian Loess)	Lower Stratum (Glacial Till)
Liquid Limit	45	60	NP	60	35	45
Plastic Limit	20	30	NP	30	15	20
Percent Sand	15	15	98	15	15	20
Percent Silt	55	45	1	45	65	55
Percent Clay	30	40	1	40	20	25
Percent Passing #40	100	100	70	100	100	100
Percent Passing #200	90	90	5	90	90	90

Example No. 2 represents a region where pervious fine sands overlie the impervious Pierre shale over large areas. In those areas free water fills the pores in the lower 2 to 7 ft. of the sand. Frost heave results if the subgrade of a highway is constructed within about 3 ft. of the water table. Similarly, an excess of water occurs in the lower stratum of Peorian loess at its contact with till. Design practices to prevent detrimental frost action in those areas are described later.

Many other Writers have presented data on and drawings of soil profiles which produce major frost heaves. Among them are Mullis /1930-8, Burton and Benkelman /1930-10 and Clark /1935-6.

Pedological Soil Types - Belcher, Gregg, and Woods /1943-5 brought out that much work had been done to correlate frost heaving with pedological types of soil. Their bulletin on Indiana soils describes matter of the various soil series, shows test data for the various horizons, and cites problems associated with the soils. In their description of the Conover series, they cite the occasional occurrence of frost heaving where cut sections intersect the weathered profile creating an abrupt transition from one soil layer to another. The Michigan Field Manual of Engineering /1946-16 has for many years shown in its design tables estimated lineal feet of frost excavation necessary for each 1,000 ft. of cut section through any particular soil series. The Missouri Highway Department Soil Manual /1948-13 gives a description of the profile of each soil series encountered in Missouri and cites the soil series (for example the Marshall, Shelby, and Putnam series) which are susceptible to damaging frost action. The occurrence of frost in the flat phase of the Colby series of Wisconsin (Black /1949-11) has been mentioned. The Wisconsin Design Standards /1949-20 also bring out the effectiveness of the pedological classification of soils in the evaluation.

Effect of Grade Position

Cut and Fill Sections - Several writers have observed that frost action occurred more frequently and more intensely in cut sections than on fills. That has been given some recognition under the subject "Soil Profile." Burton and Benkelman /1930-10 studied 500 heaves in Michigan and found that 76 percent occurred in cut sections, 10 percent on fills, and 14 percent on transitions from cut to fill. Of 141 heaves in cut sections given special study, 80 percent were located in cuts 4 or more feet below the original ground-line elevation.

Effect of Pavement and Shoulder Design and Condition on Heaving

The principal findings of the literature on Design to Prevent Damage Due to Frost Action are presented later in this review. It is intended here to present only those parts of the literature which make specific reference to the effect of design and maintenance condition of pavements in bringing about or alleviating conditions which cause frost damage. It is difficult to prevent some overlapping between this and the section presented later, hence the reader is asked to study the section on design in conjunction with this section.

Observations on the Illinois Bates Experimental Test Road /1922-5 showed that the type or thickness of a pavement had little or no effect on the amount of heaving, a 9-in. pavement heaving the same as a 4-in., although the thicker pavement responded less quickly to the first freezing. The literature has brought out that abrupt changes in design thickness of subbases and bases for pavements and in depth of granular backfill at culverts has resulted in differential heaving. Bennett /1940-13 observed that an abrupt change from a 24-in. to a 15-in. gravel base on a level grade may cause unequal heave. He also brought out that culverts are usually lower in relative elevation than the adjoining road during the winter.

Observations by several investigators have shown that soil moisture under pavements differs from that under the adjacent shoulder and that the moisture difference is responsible for differential heaving. The Corps of Engineers studies /1947-2 showed that frost heaving at four test areas was maximum at the edge and decreased toward the center with some test areas showing a slight settlement during the winter. Data indicated that "this type of heaving appears to be caused by a flow of water from adjoining turfed area into the subgrade beneath the pavement. Greater heaving at edges than at center of pavements occurred only at test areas of bituminous concrete pavements without subsurface drains at pavement edges." H. W. Smith /1946-1 in his study of sealed surfaces in New Zealand found that the "most common and by far the most disastrous type of freeze damage" occurs near the outside edge of the traffic lane, due to freezing and thawing of moisture entering at the edges and through cracks in the surface. He associated the major damage with bases constructed with this trench type of design. C. W. Smith /1948-28 reported that the condition of the surface of flexible pavements had an influence on the freeze damage in the Amarillo area in Texas. Cracks in the surface formed during dry-weather shrinkage permitted entrance of surface water into the subgrade and caused reduction in bearing capacity on freezing and thawing.

Lang /1937-8, /1940-11 and Allen /1945-8 presented data showing that leaking joints in concrete pavements in Minnesota resulted in higher subgrade moisture and greater heaving at the joints than under midsections of the pavement. Minnesota conducted an experiment to verify field observations of heaving at high joints where the joints leaked. The test section was on a new pavement on A-6 soil (P.I. = 32) having a moisture content of 18 percent prior to freezing. Five joints were open and water was fed into them continuously from November 10 to 20, 1932. Moisture increased to 29 percent for the 0-3-in. depth and to 23 percent for 3- to 12-in. depth below pavement. The record of movement in the watered and unwatered sealed joints and open joints is shown in Figure 94.

The Corps of Engineers investigation /1947-2 gave evidence that surface water infiltrating through joints into the base and subgrade prior to freezing "augmented to a slight degree the available water for frost action. At three of these test areas the heaving of the cement concrete pavement was more uniform compared to adjacent bituminous-paved areas and the settlement which occurred at three bituminous-concrete-paved test areas did not occur in the three cement-concrete test areas."

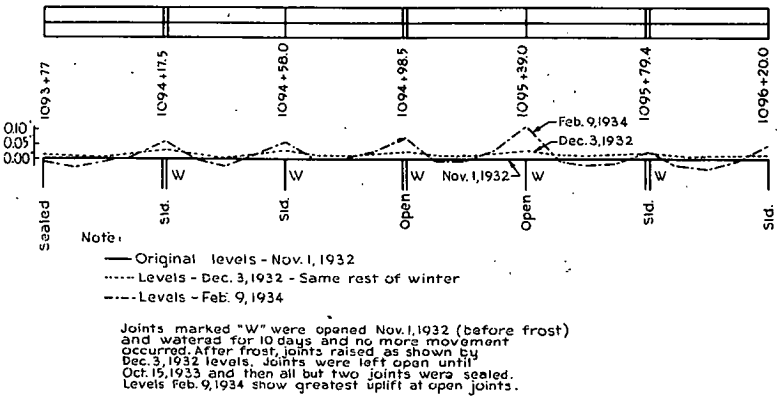


Figure 94. Watered Joint Test Section S.P. 3-53-2 Near Brandon, Minnesota, 1932 Construction (After Lang /1940-11)

Influence of Admixtures on Frost Action

Several investigators have sought to lessen the susceptibility of soils to damaging frost action by the use of admixtures. Some of the admixtures have enjoyed wide usage in the "stabilization" of soils to serve as base courses under wearing surfaces and need no introduction here. For purposes of presenting the findings in the literature the reviewer has chosen to place admixtures into three groups: (1) materials which dissolve in the soil water; (2) materials added to form a stronger binding medium than soil water; and (3) materials which cement the soil grains together.

Soluble Admixtures - Some of the early work of Bouyoucos /1913-1 suggested the possibilities of admixing dissolved substances when he restated the axiom that solutions of greater densities have lower freezing points, higher boiling points, lower vapor pressures, and higher viscosities than solutions of lower concentrations. Beskow /1935-1 held that the rate of heaving depends on the thickness of adsorption films at the interfaces between growing ice crystals and the underlying soil particles. Increased pressure squeezes the films together and decreases the rate of heave. Decreasing the pressure has the opposite effect. From that reasoning he believed that admixing dissolved substances which decrease the thickness of films should decrease the heave.

Bouyoucos /1913-1 first admixed various percentages of normal solutions of potassium chloride (KCl) and of ammonium chloride (NH_4Cl) to sand and observed their effect on the lowering of temperatures. He then admixed 750 cc of 0.5-, 1.0-, and 1.5-normal solutions of each of five salts with 6,855-gram samples of quartz sand placed in wooden boxes and recorded the freezing points of the different samples. His results are shown in Table 36.

Table 36
Freezing Points of Solutions in Sand
(After Bouyoucos /1913-1)

Solution	Freezing Point Deg. F.
Water	31.8
Sodium chloride (0.5 N. NaCl)	28.8
Sodium chloride (1.0 N. NaCl)	25.4
Sodium Chloride (1.5 N. NaCl)	22.2
Potassium chloride (0.5 N. KCl)	29.0
Potassium chloride (1.0 N. KCl)	25.6
Potassium chloride (1.5 N. KCl)	23.0
Potassium carbonate (1.0 N. K_2CO_3)	28.
Potassium carbonate (1.0 N. KN_3)	26.
Calcium chloride (1.0 N. $CaCl_2$)	27.
Amonium carbonate (1.0N.$(NH_4)_2CO_3$)	28.2

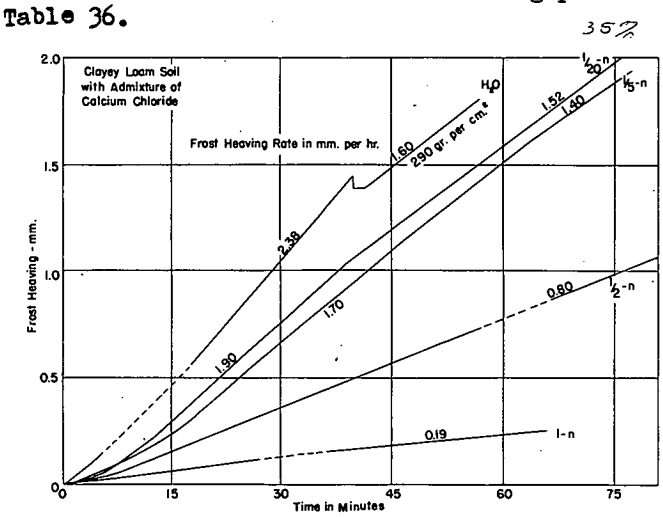


Figure 95. Frost-heaving curves of clayey loam H 28, with pure water and $CaCl_2$-solution in different concentrations. Concentration is given in normality: I-normal $CaCl_2$ = 55.50 gr per 1,000 cm^3, or about 5½ percent; 1/10 n thus represents a concentration of about 0.55 percent salt. (After Beskow)

Later /1917-4 Bouyoucos added 5 cc. of normal solutions of three salts, (NaCl, $CaCl_2$, and KCl) to 25-gram samples of air-dried fine sandy-loam, clay-loam, and Wisconsin Superior clay, and determined the percent of water which failed to freeze with and without admixtures. The water which failed to freeze was increased from 16 (no admix) to 62 percent (NaCl) for the sandy loam, from 29 to 60 for the clay loam and 38 to 58 for the clay. Values of water which failed to freeze with admixes of KCl and $CaCl_2$ were only slightly less than for NaCl.

Calcium Chloride - Bouyoucos work given in part in Table 36 shows that soils treated with a normal solution of calcium chloride had a reduced freezing point and that the admixture also reduced the percentage of water which froze at a given freezing temperature. Beskow /1935-1 made laboratory freezing tests on soils treated with calcium-chloride solutions ranging in concentrations from 1/20 to normal. His test results presented graphically in Figure 95 show that the effectiveness of calcium chloride is small at small concentrations but increases very rapidly with increase in concentration. The laboratory researches of Winn /1938-9 and Winn and Rutledge /1940-7 included studies of the effectiveness of calcium chloride in reducing frost action. Their results can be summarized briefly as follows:

(1) That calcium chloride was more effective in reducing frost action when used with well-graded soil mixtures than when used with natural silty clay /13.

(2) That calcium chloride (natural silty clay /13 plus 2 and 4 percent and graded soil mixtures /12 plus 1, 2, 3, and 4 percent) provided good resistance to frost action primarily because of freezing-point lowering.

(3) That silty clay /13 plus 3, 4 and 6 percent calcium chloride and graded soil mixtures /12 plus 1, 2 and 4 percent calcium chloride showed no frost damage, irrespective of initial moisture content.

Slate /1942-1 continued work on calcium chloride by Winn and Rutledge broadening the scope. Slate divided his studies into three parts, consisting of (1) field studies of application of chloride through pockets in the road, (2) laboratory model studies on migration, and (3) laboratory freezing tests on admixtures to soils.

Field studies of application through pockets consisted of digging three holes, each 2-ft. square and 6-in. deep and spaced 12 ft. apart, on the center line of the 20-ft. road, tamping 66 lb. of commercial-flake chloride in hole No. 1, tamping 33 lb. in hole No. 3 and placing 66 lb. (in a saturated solution) in hole No. 2. Soil was tamped in and the holes sealed with RC-3 liquid asphalt.

Slate concluded from his studies that the chloride will migrate laterally if placed in pockets but migration is slow for silty clay soils under pavements. Example of data on the extent of migration at the end of 12 months for each of the three test holes are shown in Figures 96 and 97.

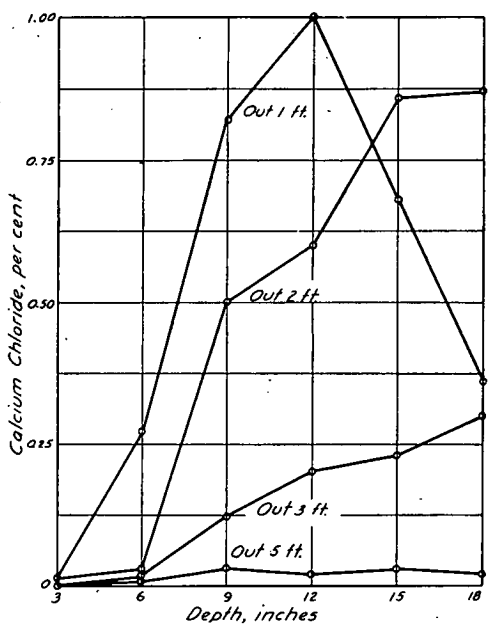


Figure 96. Extent of Migration from Hole No. 1 after 12 Months. (After Slate)

Laboratory model studies on migration consisted of two series, one to simulate field placement in pockets (no surfacing was used) and the other of mixing the chloride and soil /14 to a depth of 2 in. below the surfacing. The experiment permitted the water table to rise to within 12 in. of the surface.

The ground water had an effect on migration Simulated rain (series one) resulted in carrying

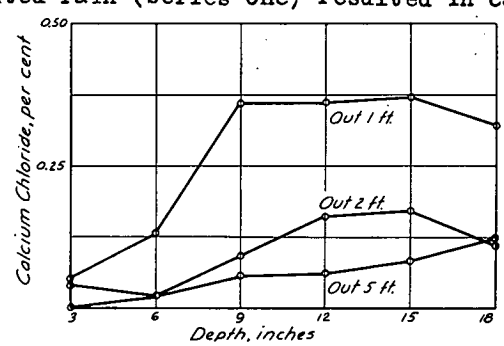


Figure 97. Extent of Migration from Hole No. 2 after 12 Months. (After Slate)

the chloride down with the moisture. Contrary, evaporation carried the chloride upward through the soil. There was a tendency for diffusion to cause the chloride to migrate to the region of least concentration in the soils.

Laboratory freezing tests were made on prepared specimens containing 0, 0.5, 1, 2, 3 and percent of calcium chloride and water (total liquid content = 20 percent of dry weight of soil)

/12 Graded soil mixtures were 11.5 - 88.5 percent (also 16.5 - 83.5) nonplastic gravel silty clay mixes and 20 - 80 percent (also 40 - 60) nonplastic sand-silty clay mixes respectively.

/13 The silty clay had an LL = 46, P.I. = 27.

/14 The soil contained 15 percent sand, 68½ percent silt, 16½ percent clay, the LL = 22.5, PI = 3.2 and sp. gr. = 2.72.

to the above silt soil. The specimens were compacted in a 7-in. high by $3\frac{1}{2}$-in. diameter tube (14 blows-per-layer, 3 layers $5\frac{1}{2}$-lb. hammer). The specimens were frozen from the top downward with the bottom in contact with water. Relative heights of heave were recorded.

The "Pennsylvania Road Builder" /1942-12 states that calcium chloride increases density, thus reducing the water voids, and gives anti-freeze properties to the soil moisture. It suggests 0.5 percent calcium chloride in soil aggregate mixes, 1 percent in clay soils and 2 percent in silty-clay soils affords effective protection to -10°F. It also suggests that treatment should be made effective to an average depth of 18 in. below the surface in the northern half of the U. S. It does not presume it is practical to completely prevent subgrade freezing, but only to such depth which will "minimize and equalize the movement." The article gave no data to validify its statements.

Nebraska /1949-11 used 0.5 percent calcium chloride in the construction of a stabilized soil base and constructed another stabilized soil base without calcium-chloride admix (See Highway Research Board Proceedings, Vol. 18, Pt. 2 Page 231, 1938). After comparing the maintenance records and performance it was concluded /1949-11 that the addition of chloride "did not improve the ability of the base courses to resist those influences which cause softening and failure". Additional data on the effectiveness of calcium chloride are given in later sections of this review.

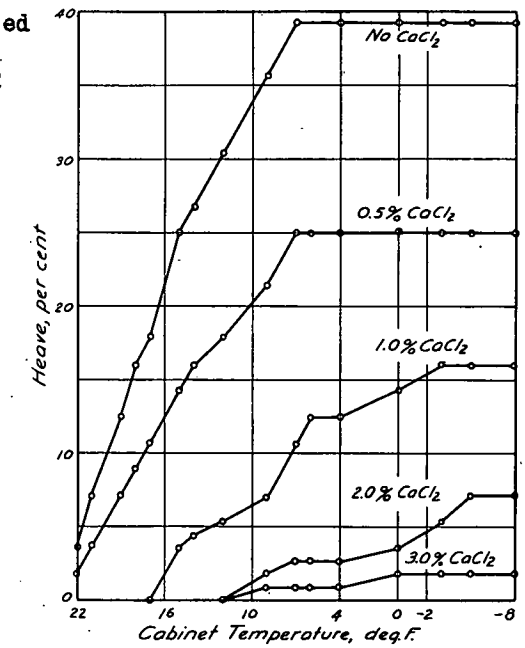


Figure 98. Effect of Calcium Chloride on Frost Heave (After Slate)

Sodium Chloride - Bouyoucos /1913-1 showed that solutions of common salt admixed with soil reduced the freezing point and increased the percentage of water which failed to freeze at a certain temperature. This has been mentioned previously.

Beskow /1935-1 made freezing tests on a silt soil to which he admixed concentrations of salt solutions ranging from $\frac{1}{60}$ normal to $\frac{1}{10}$ normal. He found that sodium chloride had a complicated effect: "In weak concentrations the salt solution increases frost heave over ordinary distilled water...at a certain concentration a maximum is reached; and then for increasing concentrations the rate of heaving decreases." Beskow

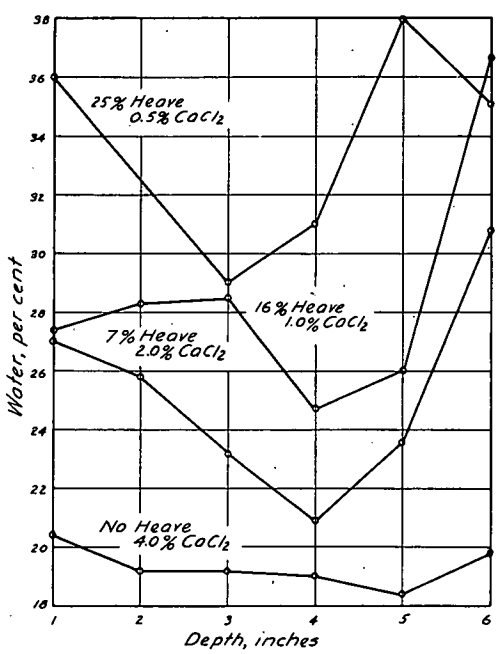


Figure 99. Shift of Water in Soil Due to Frost Heave (After Slate)

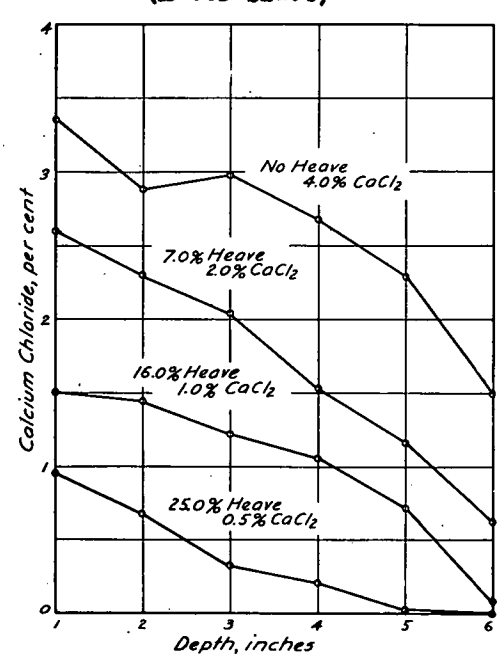


Figure 100. Migration of Calcium Chloride as Affected by Frost Heave (After Slate)

noted that these observations agree with Mattson's /1932-5, /1933-7 work on the influence of electrolytes on films measured by the variation in volume of fine colloids. In explanation, Beskow /1935-1, as quoted from Osterberg's translation /1947-12, stated, "For certain colloids there is a maximum thickness of the adsorption films for a certain concentration of a hydrating salt...with NaCl the particles adsorb the sodium ions which then form part of the film, swelling them up increasing the force of adsorption when the ion concentration of the pore water is very small. (This effect is interpreted by Mattson as an osmotic phenomenon, a sort of swelling of the films due to the difference in ion concentration between the films and the pore water). However, a further increase in the total concentration causes an increase in the ion concentration of the pore water and therefore a decrease of the difference in concentration, and thus the osmotic pressure difference between the films and pore water is diminished, which results in a decrease in the size of the adsorption films." The results of Beskow's tests using salt are shown in Figure 101.

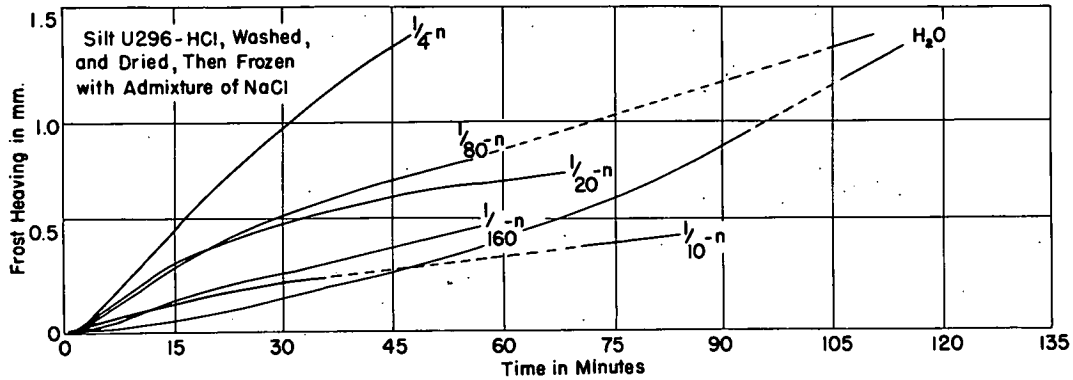


Figure 101. Diagram showing frost heaving rates when the soil specimen is frozen with solutions of different concentrations. This is the same soil as is indicated in Figure 103 but prepared with HCl and thereafter carefully washed and dried at + 105 deg. C., frozen with diluted water (H_2O) and with NaCl solution, the concentration of which is given in normality, I-n NaCl = 58.46 gr per 1,000 cm^3, corresponding to about 6 percent.

As seen from this diagram, the frost-heaving rate is increased by a very diluted salt-solution, and, concentration being gradually increased, reaches a maximum, from which it begins to fall.

The bend of the individual curves may be referred to alterations of ion-concentrations in the course of the freezing process. (After Beskow /1935-1)

Winn /1938-9 and Winn and Rutledge /1940-7 ran parallel tests with sodium and calcium chlorides in their studies of chloride salts. The following conclusions are drawn from their tests:

(1) That sodium chloride was more effective in reducing frost action when used with well-graded soil mixtures /15 than when used with a natural silty clay /15.

(2) That NaCl (natural silty clay /15 plus 2, 3, and 6 percent salt and graded-soil mixtures /15 plus 1, 2, 3, and 4 percent salt) provided good resistance to frost action primarily because of the freezing-point lowering effected by them.

(3) Silty clay /15 plus 3, 4, and 6 percent salt and graded-soil mixtures plus 0.5, 1, 2, 4, and 6 percent salt showed no frost damage irrespective of initial moisture content.

/15 Data on silty clay and graded-soil mixtures are given previously under Calcium Chloride.

Rowat /1939-3 described the use of salt in the prevention of frost heave on railroad tracks. He cites that soil temperatures at 15- to 24-in. depths seldom go below 15 F., although the air temperatures may fall to -50 or -60 F., and that temperatures at 6 to 7 foot depths are near the freezing point. Because a 5 percent salt solution will not freeze above 15 F. it could be expected to reduce the amount of heaving.

He stated that during 1936, 1937, 1938, and 1940, locations on Canadian Railways subject to "recurrent and persistent frost heave" were treated with applications of salt.

Rowat describes four methods by which the salt has been applied.

1. Spread on ballast. Work out of sight with hose and shovels.

2. Spread on ballast. Build 9-in. dike at bottom of ballast section to prevent washing away of salt.

3. Remove ballast and place salt at bottom level of ties. For small applications replace ballast immediately. For large application leave open and refill at later date.

4. Dig 6-in. holes 18-30 in. deep at 2 ft. intervals center to center. Place 3 in. of salt in bottom of hole and refill with ballast.

Apply salt in late summer or early fall. Provide thinner salt applications on entering and leaving heaved areas. Apply salt to the hump where maximum heave occurs.

Rowat states the salt applications have reduced the amount of "shimming" required subsequent to treatment "in approximately 90 percent of the locations, varying from 25 percent reduction to complete elimination, the latter in 28 percent of the treatments."

Concerning application of salt to highway subbases, Rowat states, "This procedure has been followed...in Nova Scotia...for the past 2 years, where 20-40 tons per mile have been mixed into the subbases of newly constructed highways...The results, we understand, have been entirely satisfactory."

Migration of Soluble Salts - Some of Slate's /1942-1 results pertaining to migration have been mentioned under "Calcium Chloride" and "Sodium Chloride". Slesser /1943-9 extended the work begun by Slate, first surveying the literature, second, conducting field and laboratory investigations of migration and third, conducting laboratory freezing tests. His field studies pertaining to migration under surfaced roads (Purdue University Test Road No. 2) is of interest here.

Test Road No. 2 was built in the summer of 1938. It contained two chemical sections each 50 ft. long, 20 ft. wide and 6 in. deep. They contained (1) 88.5 percent pit-run gravel + 11.5 percent soil + 0.5 percent (based on full depth) sodium chloride (section 21) and (2) The same combination of gravel and soil + 0.5 percent calcium chloride (based on top 2 in.)(Section 22). In section 21 the salt was mixed full (6 in.) depth while in section 22 one half the calcium chloride was mixed with the top 2 in. and one half applied to the surface. The test road was given a bituminous surface in August and September 1939 and was resampled in March 1940.

Slesser in reporting the results states that "the migratory characteristics of both calcium chloride and sodium chloride...were definitely influenced by the presence of an impervious pavement. (See Figure 102). Whereas the surface sample showed the highest chemical content in the absence of the pavement it showed the lowest chemical content in the presence of the bituminous pavement." Thus a surface definitely accelerates downward vertical movement of both chlorides from coarse textured bases into underlying fine grained soils.

Slesser also studied lateral migration. He concluded that "lateral migration of sodium chloride is greatly reduced by an impervious pavement; Calcium chloride shows little lateral movement either with or without a pavement."

The Corps of Engineers /1949-23 conducted tests using calcium chloride and sodium chloride to prevent frost action in soils. Although the calcium chloride did prevent frost action in nearly every instance, the report stated that "the use of salts for frost action preventatives affords only temporary protection due to their tendency to migrate or leach out of the soil; therefore, they are regarded as unsatisfactory for use under pavements where permanent protection is essential."

Sulfuric Acid - Beskow /1935-1 found from laboratory freezing tests that the effect of sulfuric acid (H_2SO_4) is noticeable for small concentrations but soon reaches a constant rate of heaving regardless of the increase in concentration of the solution. This is indicated in the test results shown in Figure 103. The soil used in Figure 103 is a silt. No data are given concerning its characteristics. However, generally similar freezing results were obtained on another silt having 36 percent fine sand, 50 percent silt and 14 percent clay size. The coarser the soil the quicker the constant rate is reached with increasing concentration.

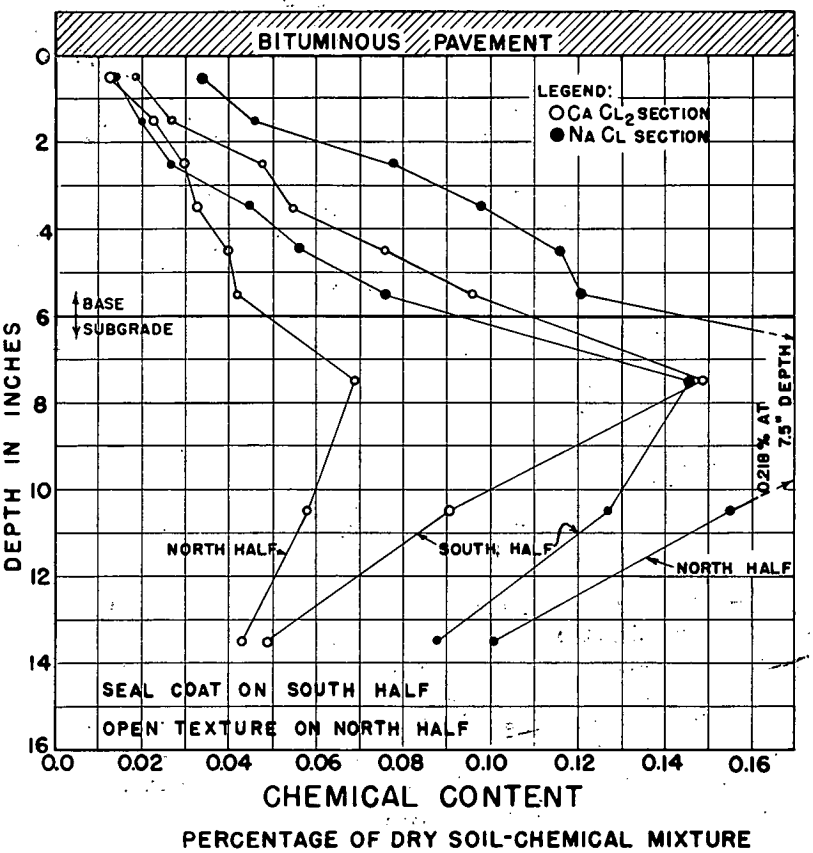


Figure 102. Variation of chemical content with depth: second sampling of Test Road No. 2. (After Slesser)

Sulfite Liquor - Beskow also experimented with sulfite liquor to reduce heaving. He moistened the sulfite leach to a syrupy consistency with 40 percent water. His results are shown in Figure 104.

Resinous Materials - Clare /1948-22 added Vinsol resin in amounts up to 1 percent to pulverized and recompacted chalk from England. Specimens were prepared in the standard compaction mold, and then subjected to an air temperature of -10 deg. C. (14 F.) while the lower surfaces were immersed in water at slightly above freezing temperature. Untreated specimens contained numerous ice lenses causing marked heaving while specimens treated with 1 percent Vinsol remained unchanged.

Beskow /1947-23 used solutions of alkaline resin, adding quantities ranging up to 0.5 percent of the dry weight of the soil. He stated "Such treatments would reduce frost-heave rate and water suction to a trifle, but only if the soil was dried out after treatment. If once dried out, the soil sample, even after mixing with water to full capillary saturation, and in suction connection with free water, will show but a small part of the frost heave rate of raw soil". (See Figure 105).

The Corps of Engineers /1949-23 experimented with Darex AEA in laboratory tests to prevent frost action. It was concluded as being unsatisfactory as a preventative of frost action in the soil with which it was used.

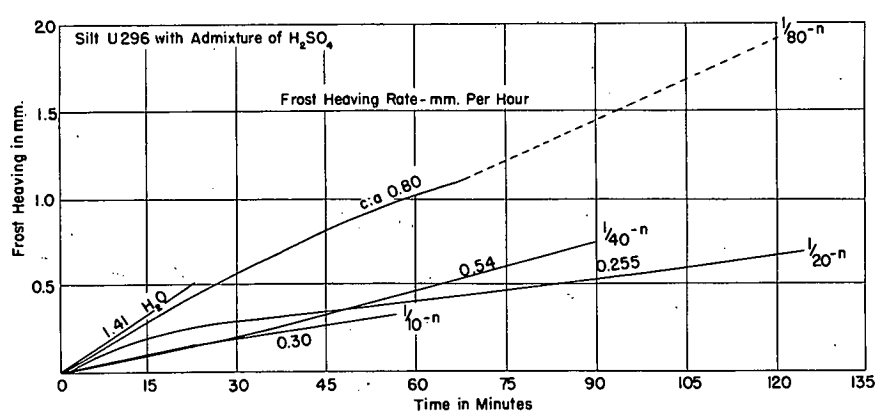


Figure 103. Diagram showing different frost-heaving rates when the soil-specimen is frozen with solutions of different concentrations. Silt U 296, unprepared, frozen with pure water (curve marked H_2O) and diluted with H_2SO_4, the concentration of which is measured in normality (e.g. 1/10-n). As I-n H_2SO_4 is 49.04 gr pro 1 000 cm^3, or about 5 percent, 1/10-n is only ½ percent. As shown by the diagram the effect of also very diluted sulfur-acid is a marked suppression of the frost-heaving rate, which for this soil is especially great. (After Beskow /1935-1)

Sodium Silicate - Taber /1930-9 summarized his findings on the affect of sodium silicate as follows: "Sodium silicate mixed into clay lowers the freezing point, increases the percentage of water that does not freeze, and reduces the permeability. In practical use it would be slowly leached out of soils that are alkaline. In acid soils sodium silicate decomposes with the precipitation of insoluble gelatinous silica which on drying would act as a cement and help to bind particles together".

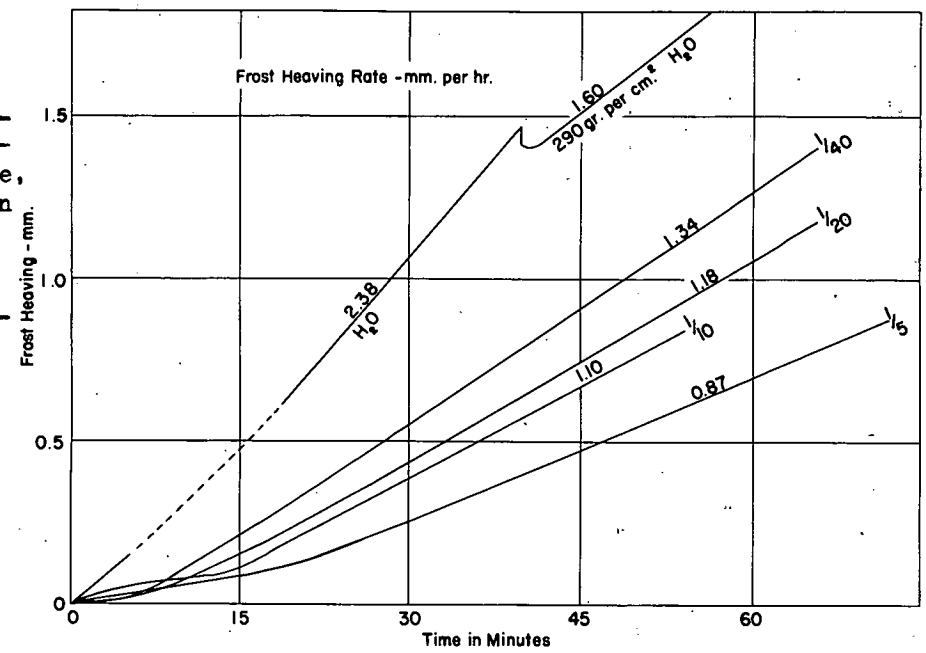


Figure 104. Frost-heaving curves of clayey loam H 28, with pure water and with sulphite-leach, i.e. a byproduct of the sulphite-process of paper-pulp manufacture. Concentration in this diagram not in normality, but in part of evaporated viscous leach (standard trade-article), with a concentration of 60 percent dry substance. E.g. 1/20 thus represents a concentration of 3.0 percent dry substance, 1/5 a concentration of 12 percent. (After Beskow /1935-1)

Liquid Binders

Road Oils - Minnesota /1945-8 in its investigation of the effect of frost action in producing "high joints" in cement-concrete pavements conducted an experiment to determine the effect of an admixture of No. 2 road oil plus 25 percent kerosene to the subgrade of an existing pavement. The mixture was forced under the pavement in the amount of one

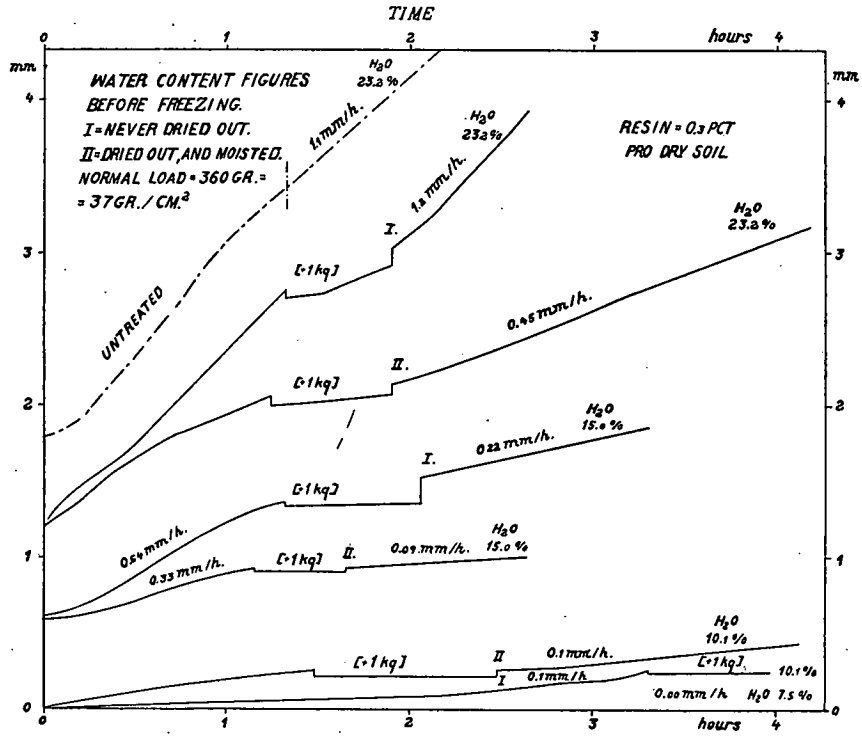


Figure 105. Effect of Resin Preparation on Frost Heave Speed in Coarse Silt - Silt columns 70 mm., placed on sand base, water filled, and in connection with free water level, 10 cm. below bottom of silt column. Each pair of curves represents the same water content before freezing started, in each pair No. II was dried out after treatment and then moistened, No. I was not dried out. Treatment with alkaline resin solution (water content = 23.2 percent (full saturation), resin = 0.3 percent, all percentages per dry soil weight. (After Beskow /1947-23)

gallon per sq. yd. through holes drilled in the pavement. Elevations observed in December showed greater uplift on treated than on untreated sections. The report states that apparently the oil "increased the water holding capacity of the soil, thereby increasing the uplift." Maximum penetration of oil into the subgrade was about 2 ft. Additional data on bituminous materials is given later.

The Corps of Engineers /1949-23 experimented, in laboratory tests, with Bunker "C" oil, tar (RT-2) and with calcium chloride in combination with Bunker "C" oil and with the tar.

It was held from the results that the water-proofing property of bituminous materials and the percentage of soil finer than 0.02 mm. by weight are two factors from which a quantitative method of design for bituminous admixtures might be developed. That possibility was based on the premise that if water proofing is regarded as being effected when the voids are filled with bituminous material so water cannot enter, there the void ratio of the soil gives a measure of water proofing. The void ratio here is defined as the ratio of volume of voids to the total volume of solids (including the admixture). Then, for any soil there could be a critical void ratio with admixture at which frost action would be prevented. Tests with Bunker "C" oil and RT-2 defined that critical void ratio. The void ratio (with admixture) at which no frost action occurs has been plotted against the percentage by weight of particles finer than 0.02 mm. as is shown in Figure 106.

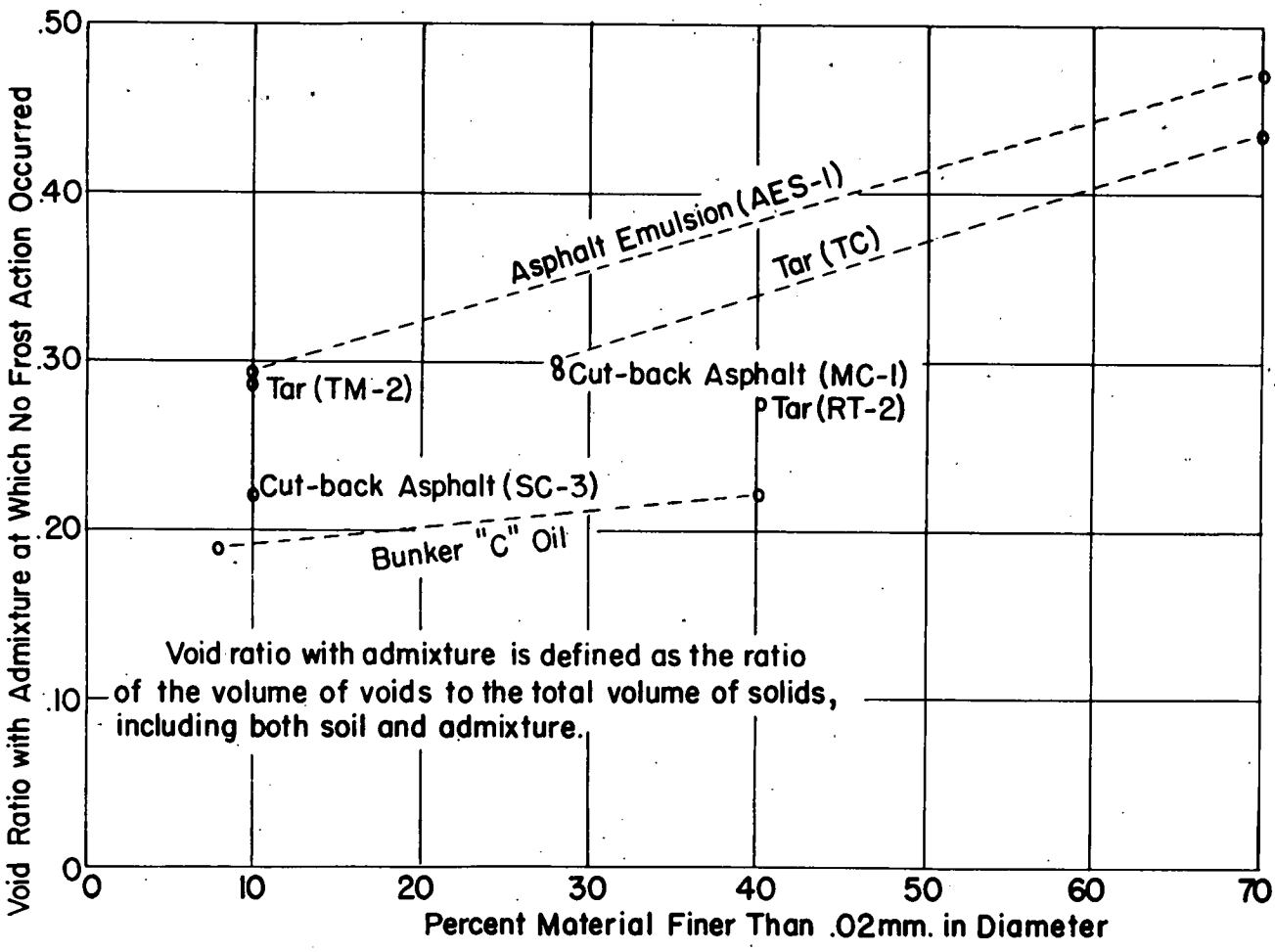


Figure 106

The results indicate that the asphalt emulsion AES-1 gives best results followed by tar, asphalt cutbacks, and Bunker "C" oil. It was stated that additional tests using a wider variety of oils were necessary before conclusions could be reached.

Tar - The Minnesota studies /1945-8 also included an experiment with tar. Prior to laying a new pavement (July 1932) a clay till (A-6 soil group) was treated with 1 gallon per sq. yd. of TC-2 road tar. The tar penetrated to depths of 1½ to 2 in. The subgrade was well compacted prior to paving. The report concludes, "It was found that ice lenses formed in the soil penetrated by the tar as well as in the untreated portion and that an impervious membrane was not formed". Added data on tar are given later.

Influence of Cementing Materials

Portland Cement - The War Department's Report /1949-2 on Frost Investigation 1944-1945 showed that no ice lenses and no frost crystals formed in soil-cement bases on two projects, one at Houlton, Maine, the other at Fargo, North Dakota. Added data on cement admixtures are given later.

Comparison of Effectiveness of Various Admixtures - Winn and Rutledge /1940-7 (also Winn /1938-9) conducted an extensive series of laboratory freezing tests to determine the effects of various stabilizing admixtures on the frost-heave characteristics of two Indiana clays and of mixtures of sand and clay, and gravel and clay, each with and without admixes. The admixes used were calcium oxide (CaO) (quick lime), sodium chloride, calcium chloride, Portland cement, vinsol resin, and the following bituminous materials: Cut-back asphalts, MC-1, RC-3, and SC-3; tars TC and TM-2; asphaltic emulsion AES-1; and also a "bituminous stabilizer."

The tests were made under controlled conditions of molding, curing, and freezing, which makes the results of value from a comparative standpoint. Hence, the report /1940-7 is reviewed under this heading, rather than the separate results being presented under each of the appropriate subject headings which have preceded. Therefore, the report /1940-7 will be reviewed under this heading instead of under each of the appropriate subject classifications.

The soils used, which were essentially silty and clayey, were sampled from two test roads. The grain-size distribution curves for the two soils, a sand, and a gravel are shown in Figure 107. Grain size curves for typical graded-soil mixtures used in the freezing studies are shown in Figure 108.

All soils and soil mixtures were air dried and pulverized as far as practicable without crushing soil particles. All admixtures were based on dry weight of soil. Portland cement, vinsol and quicklime were dry mixed with the soil before adding water. Bituminous materials were added after water had been completely mixed with the soil. Chloride salts were dissolved in water and added. All specimens (except those used in study of effect of density) were compacted in four equal layers by 14 blows of a 5½-lb. hammer (2in. diam. face) dropping 12 in. in a mold 3 in. in diam. and 7 in. high (1/30 cu. ft.).

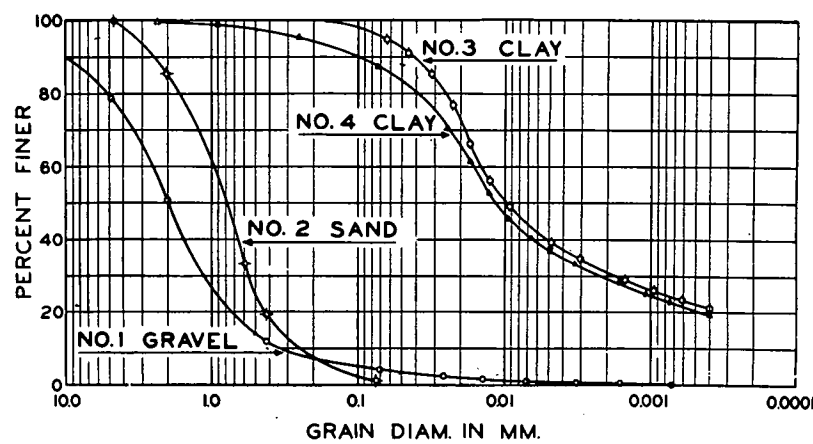


Figure 107. Grain-Diameter Distribution for Individual Soil Types. (After Winn and Rutledge)

All soil-cement mixtures were cured in a moist room for 10 days or more. Bituminous mixtures were air cured in laboratory at 70 F. for 10 days; vinsol specimens were cured by air drying. Since different methods of curing gave different moisture contents the specimens were divided into two groups; one group was air dried to a low moisture content and the other group consisting of specimens that did not require curing, which were tested as molded at the optimum moisture content, and those that required curing, which were resaturated to near the optimum water content. Freezing took place in a special cabinet where specimens were placed on porous discs and insulation placed around the specimen so freezing was from the top downward.

In stating the results the authors believe that "percentage of heave data from individual tests should not be used as criteria for rigid comparisons of the frost action resistance of natural soils, treated soils, or stabilized soils but may be used as a basis for general classification of the materials into heaving and non-heaving groups."

The reviewer quotes from the Winn-Rutledge report /1940-7 in summarizing the conclusions of their findings:

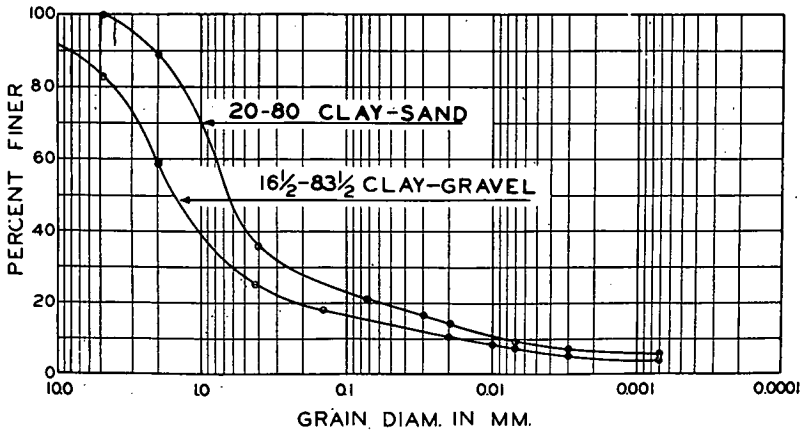


Figure 108. Grain-Diameter Distribution for Typical Graded Soil Mixtures (After Winn and Rutledge)

"All the admixtures tested are much more effective in reducing frost action when used with well-graded soil mixtures than when used with natural natural sandy clay.

"Calcium oxide (2, 6, 10 percent in sandy clay; 4 percent in graded soil mixture) does not increase the mixture's resistance to frost action or moisture-content fluctuation sufficiently to warrent its use for these purposes.

"Sodium chloride (natural clay plus 1, 2, 3, 6 percent; graded soil mixture plus ½, 2, 1, 3 and 4 percent) and calcium chloride (natural sandy clay plus 2, 4 percent; graded soil mixture plus ½, 1, 2, 3, 4 percent) provide good resistance to frost action primarily because of the lowering effect of the admixture on the freezing point. The data indicate that as long as the soil retains the chemical in its full concentration, 2 percent or less chemical prevents freezing at -10 to -15 F. and thereby prevents frost damage.

"The resistance to frost action of a soil-cement mixture (natural clay plus 4, 6, 7, 10, 12 percent; graded-soil mixture plus 4, 6, 7, 10 percent) is inversely proportional to the degree of saturation of the mixture at the beginning of the freezing period.

"In general, the resistance to frost action to bituminous mixtures is inversely proportional to the degree of satruation at the beginning of the freezing period.

"Portland cement, tar, cutback asphalt, road soil, emulsified asphalt, and vinsol add stability to a sandy clay by inhibiting capillary motion of the water to various degrees, the amount being closely related to the percentage of admixture and moisture content of the mixture at the time it is exposed to the water.

"Vinsol is effective as a waterproofing agent and frost action preventive when the moisture content of the clay-vinsol mixture is between 4 and 10 percent.

"On the basis of the data presented in this paper, the following group classifications can be made:

"Group No. 1, damaged by frost action at all percentages of initial moisture content. Sandy clay (natural); graded mixtures of clay plus gravel and clay plus sand; clay plus 2, 6, 10 percent CaO; graded soil mixture plus 4, 6 percent CaO; clay plus 1 percent NaCl; clay plus 1 percent $CaCl_2$; graded soil mixture plus ½ percent $CaCl_2$; sandy clay plus 4 percent portland cement; clay plus 2, 4, 6 percent TC; sandy clay plus 2, 4, percent AES-1; sandy clay plus 2, 4 percent MC-1.

"Group No. 2, damaged only when initial moisture content was approximately 100 percent saturation. Clay plus 6, 7, 10, 12 percent portland cement; clay plus 4, 6, 8, percent TM-2; clay plus 4, 6, 8, percent AES-1; clay plus 4 percent Sc-3; graded soil mixture plus 2, 4, 6 percent RC-3; clay and graded soil mixtures plus 1, 2, 3, 5 percent vinsol (also when moisture content is below 4 percent).

"Group No. 3, no frost damage at all degrees of initial moisture content. Clay plus 3, 4, 6 percent NaCl; graded soil mixture plus ½, 1, 2, 4, 6 percent NaCl; clay plus 3, 4, 6 percent $CaCl_2$; graded soil mixture plus 1, 2, 4 percent $CaCl_2$; graded soil mixture plus 4, 6, 8 percent portland cement; graded soil mixture plus 4, 6 percent TM-2; graded soil mixture plus 4, 6 percent AES-1; graded soil mixture plus 4.4 percent Bitumuls Stabilizer; clay plus 6, 8 percent SC-3; graded soil mixture plus 2, 4, 6 percent SC-3."

Slesser /1943-9 supplemented the work of Winn and Rutledge by conducting additional laboratory freezing tests on the effectiveness of calcium and sodium chloride on frost heave in a LaPorte (Indiana) silt (80 percent silt sizes 0.05 - 0.005 mm; P.I. = 3.3 optimum moisture content 21 percent; AASHO max. density =102 pcf) and also on a mixture of 80 percent concrete sand and 20 percent sandy clay. His results are shown in Figure 109.

It may be seen from Figure 109 that as little as 2 percent of either sodium or calcium chloride will reduce heaving when subjected to temperatures as low as -15 F. No heaving occurs when the chemical content is approximately 5 percent. Slesser found the critical chemical content for the sand-clay (80-20 mixture of concrete sand and sandy clay) mixture at which no heave occurs is about 2 percent for both chlorides.

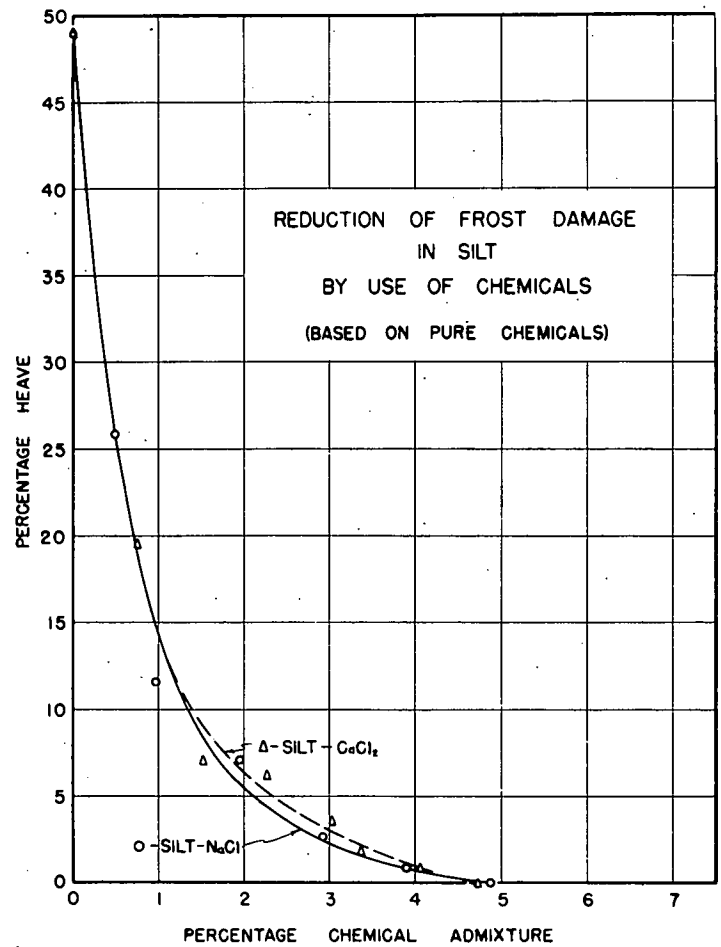


Figure 109. Reduction of frost damage in silt by use of calcium chloride and sodium chloride (based on pure chemicals). (After Slesser)

The Corps of Engineers /1949-23 studied the results obtained by others with admixtures of calcium oxide, water repellents (Stabinol, 321, and ferrous sulfate plus 321), salts (sodium and calcium chlorides), portland cement, vinsol resin, and bituminous materials (asphalt emulsions, asphalt cut-backs, tars and oil). That study indicated that "only two groups of admixtures, the salts and the bituminous materials were effective in preventing frost action in the soils tested."

Electrochemical Stabilization

Solntzev and Sorkov /1941-11 attempted to stabilize soils in the field by electrical means to prevent heaving. The location was one where previous heaving had reached a magnitude of 37 cm (14.5 in.). They added calcium chloride to one section and sodium chloride to a second section. No materials were added to the third section. The conclusions were that a diminished swelling occurred in grounds stabilized in the presence of calcium chloride and particularly sodium chloride. The effect of electrical stabilization alone was not noted.

Patents on Use of Admixtures

Sourwine /1939-12 was issued a patent (No. 2162185) June 13, 1939 which provides for means of preventing the occurrence of ground freezing. The patent covers the use of liquids other than water and carrying in suspension less than 1.5 percent of solid matter, having a freezing point lower than water and possessing the physical capacity to form a thin capillary film for the purpose of incorporating with soil to insure against ground freezing and heaving. The claimant specifically exempts bituminous or asphaltic oils or emulsion, any form of "sulfide wastes," or any admixture of calcium chloride or sodium chloride as "not suitable for use as any part of this invention." The patent states that "several alcohols, either undiluted or in solution or in chemical combination with other liquids, are suitable for use." The patent also covers the use of potassium hydroxide, kerosenes, ammonia compounds, or other compounds and solutions of nitrogen. The method of preparing the foundation is also specified.

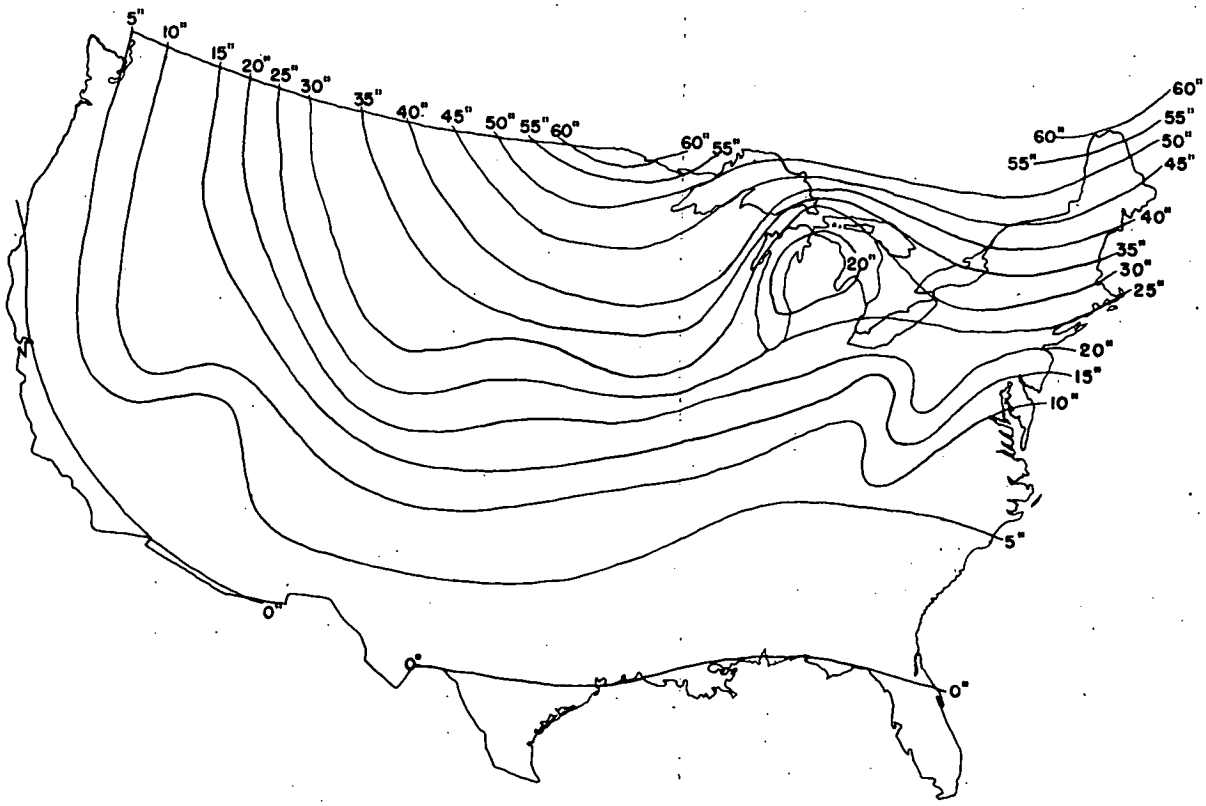


Figure 110
Average Annual Frost Penetration in Inches. (Based upon State Averages) (After U. S. Weather Bureau)

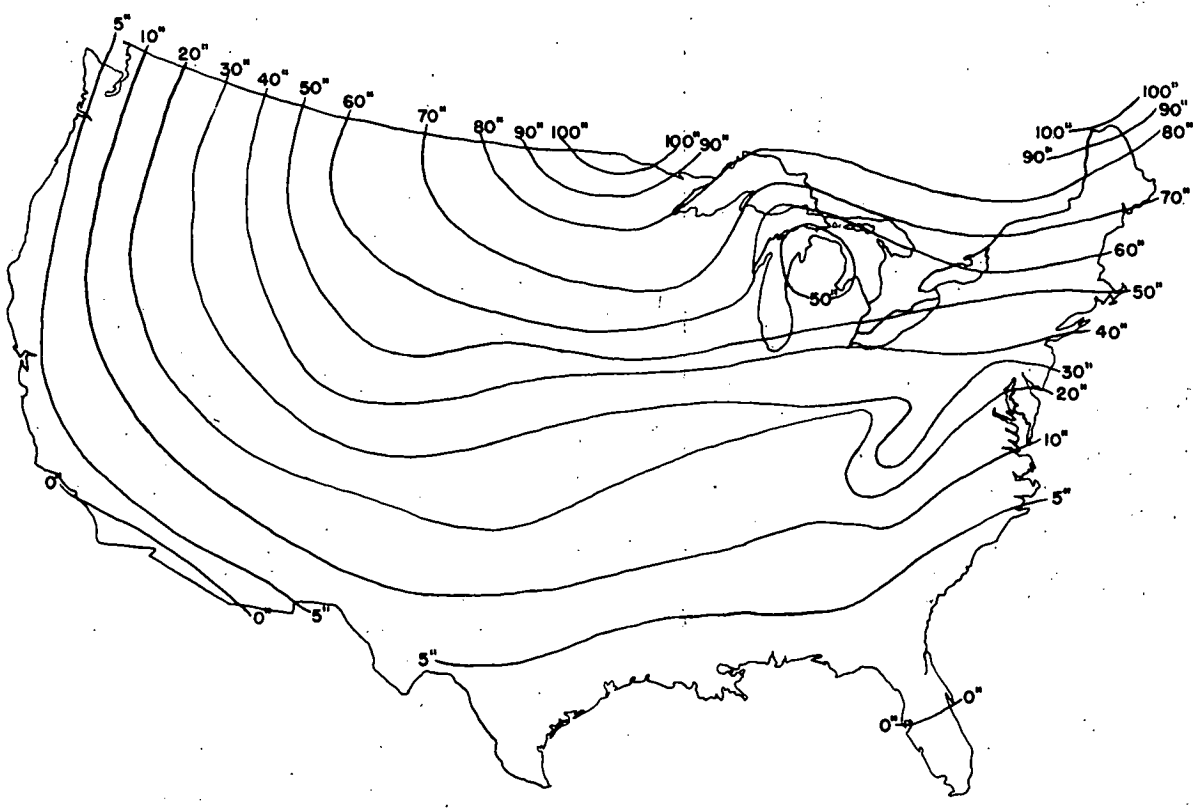
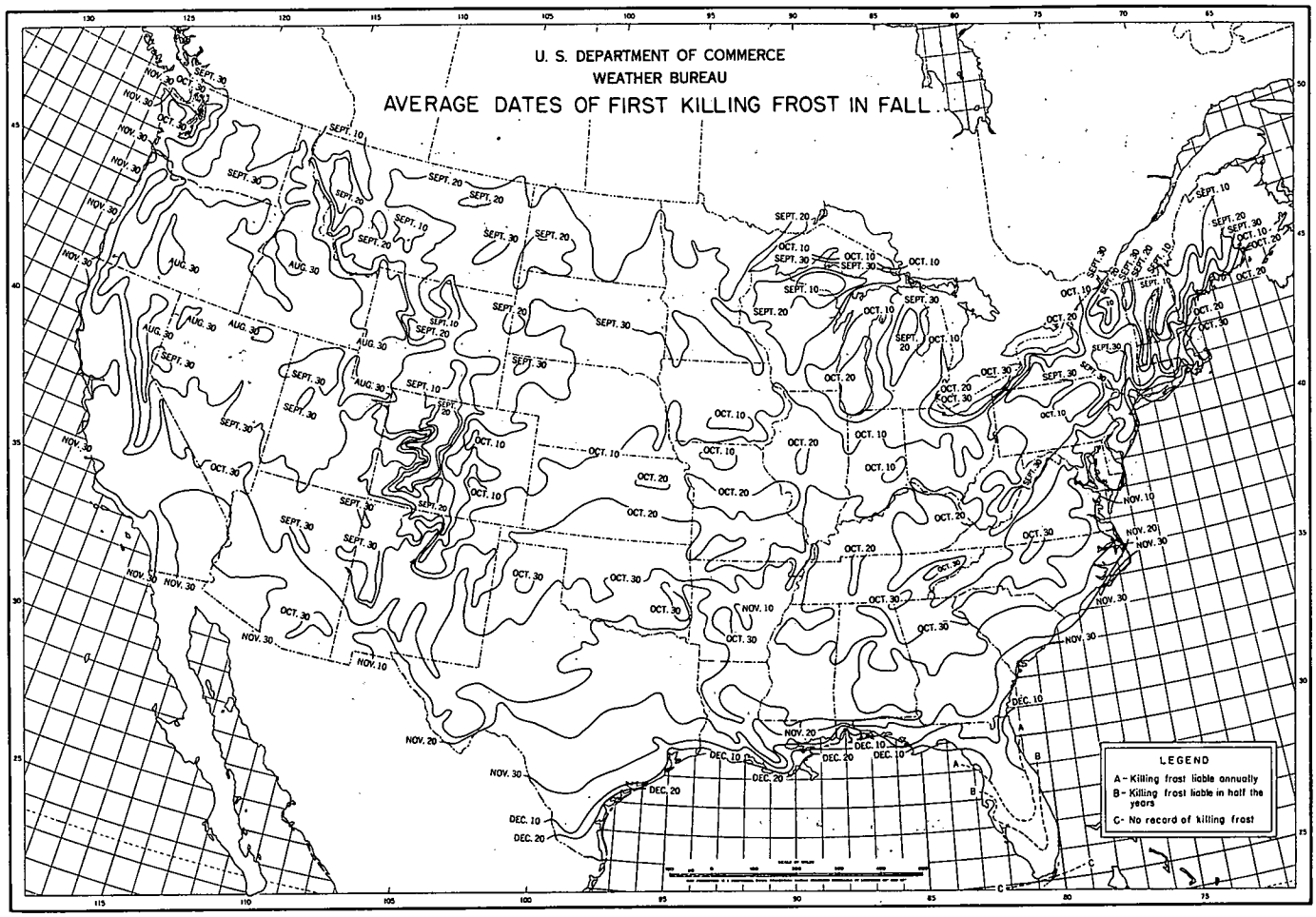


Figure 111
Extreme Frost Penetration in Inches. (Based upon State Averages) (After U. S. Weather Bureau)


U. S. DEPARTMENT OF COMMERCE
WEATHER BUREAU
AVERAGE DATES OF FIRST KILLING FROST IN FALL
LEGEND
A - Killing frost liable annually
B - Killing frost liable in half the years
C - No record of killing frost


U. S. DEPARTMENT OF COMMERCE
WEATHER BUREAU
AVERAGE DATES OF LAST KILLING FROST IN SPRING

PENETRATION OF FROST

Average Annual and Maximum Depth of Frost

Data on depth of soil freezing are useful in arriving at adequate designs for roadways, minor structures, and underground conduits. Part of the available data are from the U. S. Weather Bureau and part from municipalities, the latter largely from water department records.

The U. S. Weather Bureau has prepared maps of the United States showing isolines of average annual frost penetration (Fig. 110) and extreme frost penetration (Fig. 111). The reviewer has been unable to determine the date of publication of the maps shown in Figures 110 and 111 or their relative reliability in terms of period of observations on which they are based and whether based on depth of frost under snow-covered pavements, cleared pavements, or turf cover. The U. S. Weather Bureau has published the following additional maps which may be of some value in the study of climate as related to ground freezing:

1. Average dates of first killing frost in fall.

2. Average dates of last killing frost in spring.

3. Average length of frost free period (days).

Heating and Ventilating Magazine /1938-8 presented a map (Figure 112) showing the maximum depth of frost penetration in the United States. The map is plotted for several-hundred points for which data are available through the U. S. Weather Bureau. The magazine stated that "the map is believed to be reasonably accurate, excepting in the Rocky Mountain regions where few climatic maps are highly accurate." Table 37, also from Heating and Ventilating Magazine, gives specific data on maximum frost penetration in 100 cities, and Table 38 gives approximate average depth of frost penetration in 45 states and the District of Columbia. The article does not state whether measurements represent penetration under paved or grassed areas.

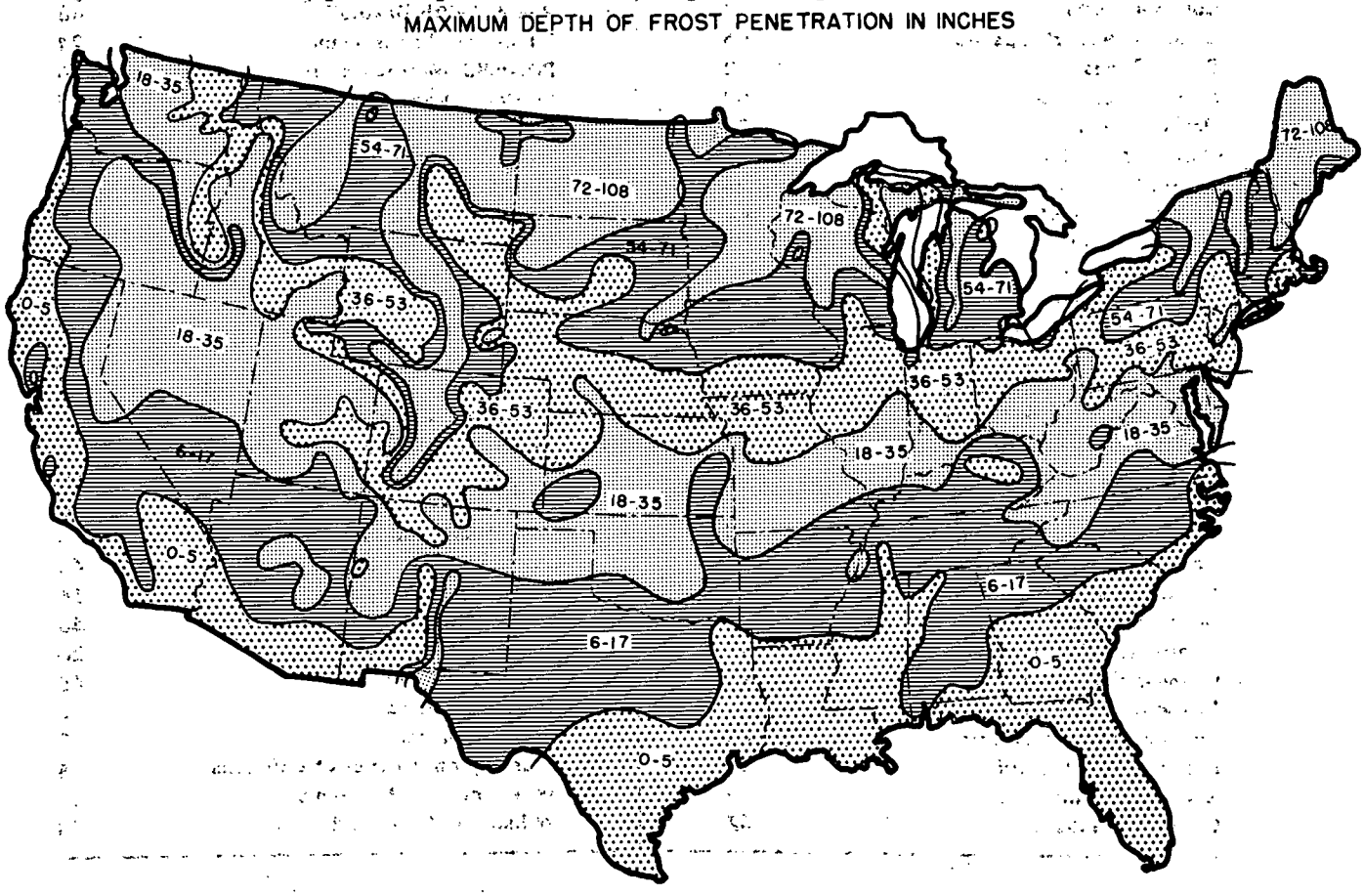


Figure 112
Maximum Depth of Frost Penetration in Inches
(Courtesy Heating and Ventilating Magazine)

TABLE 37

MAXIMUM FROST PENETRATION (INCHES) IN GROUND IN 100 CITIES

City	Depth, Inches	City	Depth, Inches
Albany, New York	48	Lincoln, Nebraska	36
Albuquerque, New Mexico	4	Los Angeles, California	2
Atlanta, Georgia	6	Louisville, Kentucky	5
Baltimore, Maryland	20	Marquette, Michigan	24
Binghamton, New York	72	Memphis, Tennessee	20
Birmingham, Alabama	12	Milwaukee, Wisconsin	60
Bismarck, North Dakota	84	Minneapolis, Minnesota	108
Boise, Idaho	42	Missoula, Montana	84
Boone, Iowa	60	Mobile, Alabama	6
Boston, Massachusetts	48	Monroe, Louisiana	8
Buffalo, New York	48	New Orleans, Louisiana	2
Burlington, Vermont	66	Newport News, Virginia	4
Butte, Montana	84	New York, New York	42
Cedar Rapids, Iowa	60	Ogdensburg, New York	72
Charleston, South Carolina	2	Ogden, Utah	48
Charlotte, North Carolina	8	Oklahoma City, Oklahoma	22
Chattanooga, Tennessee	15	Paducah, Kentucky	11
Cheyenne, Wyoming	60	Pensacola, Florida	3
Chicago, Illinois	48	Peoria, Illinois	48
Cleveland, Ohio	60	Philadelphia, Pennsylvania	40
Columbia, South Carolina	4	Phillipsburg, New Jersey	32
Columbus, Ohio	30	Pierre, South Dakota	48
Concord, New Hampshire	66	Pittsburgh, Pennsylvania	24
Dallas, Texas	12	Pittsfield, Massachusetts	96
Denver, Colorado	60	Portland, Maine	72
Dayton, Ohio	36	Portland, Oregon	6
Detroit, Michigan	48	Prescott, Arizona	24
Dover, Delaware	18	Providence, Rhode Island	60
Duluth, Minnesota	96	Pueblo, Colorado	36
El Paso, Texas	18	Richmond, Virginia	24
Eugene, Oregon	8	Rochester, New York	48
Flagstaff, Arizona	20	Sacramento, California	2
Fort Wayne, Indiana	40	Salina, Kansas	24
Frederick, Maryland	24	Salt Lake City, Utah	34
Grand Canyon, Arizona	6	San Antonio, Texas	5
Grand Forks, North Dakota	84	San Diego, California	2
Grand Island, Nebraska	36	Savannah, Georgia	2
Grand Rapids, Michigan	58	Scranton, Pennsylvania	60
Green Bay, Wisconsin	60	Seattle, Washington	26
Greensboro, North Carolina	10	Shreveport, Louisiana	4
Harrisburg, Pennsylvania	48	South Bend, Indiana	48
Hartford, Connecticut	66	Spokane, Washington	66
Houston, Texas	3	Springfield, Illinois	48
Huntington, West Virginia	30	Springfield, Missouri	20
Indianapolis, Indiana	48	Syracuse, New York	48
Jacksonville, Florida	2	Tulsa, Oklahoma	16
Johnstown, Pennsylvania	48	Urbana, Illinois	33
Kansas City, Missouri	25	Washington, District of Columbia	18
Knoxville, Tennessee	9	Wheeling, West Virginia	36
Lafayette, Indiana	47	Williamsport, Pennsylvania	48

TABLE 38

Average Frost Penetration, Inches (Approximate)

ALABAMA	
Birmingham	3
Mobile	1
Montgomery	2
ARIZONA	
Flagstaff	13
Globe	2
Phoenix	0
ARKANSAS	
Fort Smith	12
Little Rock	4
CALIFORNIA	
Eureka	3
Red Bluff	2
Sacramento	2
San Diego	1
San Francisco	1
San Jose	1
Stockton	2
COLORADO	
Boulder	20
Denver	24
Durango	18
Fort Collins	36
Grand Junction	18
Greeley	24
Leadville	66
CONNECTICUT	
Bridgeport	21
Hartford	30
New Haven	23
DELAWARE	
Dover	8
Wilmington	11
DISTRICT OF COLUMBIA	
Washington	10
GEORGIA	
Athens	3
Atlanta	3
Columbus	2
Macon	2
Rome	5
Savannah, less than	1
IDAHO	
Boise	15
Lewiston	8
Pocatello	30
Twin Falls	12
ILLINOIS	
Aurora	33
Bloomington	25
Cairo	6
Chicago	35
Decatur	20
Peoria	26
Quincy	22
Rock Island	32
Springfield	20
Urbana	22
INDIANA	
Elkhart	29
Evansville	7
Indianapolis	20
Lafayette	22
Marion	20
Michigan City	30
Richmond	15
South Bend	30
Terre Haute	20
Vincennes	10
IOWA	
Cedar Rapids	35
Clinton	33
Council Bluffs	30
Davenport	32
Des Moines	32
Dubuque	26
Fort Dodge	33
Sioux City	35
KANSAS	
Atchison	20
Emporia	12
Hutchinson	13
Leavenworth	16
Salina	17
Topeka	14
Wichita	11
KENTUCKY	
Bowling Green	5
Frankfort	10
Louisville	10
Owensboro	6
Paducah	5
LOUISIANA	
Baton Rouge, less than	1
New Orleans, less than	1
Shreveport	2
MAINE	
Eastport	36
Lewiston	54
Portland	48
MARYLAND	
Annapolis	10
Baltimore	12
Frederick	16
Hagerstown	18
MASSACHUSETTS	
Boston	36
MICHIGAN	
Alpena	24
Battle Creek	42
Detroit	36
Grand Rapids	36
Ironwood	36
Lansing	25
Marquette	9
Saginaw	42
Sault Ste. Marie	10
MINNESOTA	
Duluth	41
International Falls	66
Mankato	39
Minneapolis	66
Moorhead	51
Winona	41
MISSISSIPPI	
Corinth	3
Natchez	1
Vicksburg	2
MISSOURI	
Hannibal	20
Kansas City	15
St. Joseph	20
St. Louis	13
Springfield	8
MONTANA	
Anaconda	60
Butte	60
Great Falls	36
Helena	66
Kalispell	6
NEBRASKA	
Hastings	30
Lincoln	27
North Platte	22
Omaha	30
Sidney	33
NEVADA	
Carson	13
Elko	10
Reno	17
Tonopah	27
NEW JERSEY	
Atlantic City	6
Newark	17
Trenton	12
NEW MEXICO	
Albuquerque	6
Roswell	3
Santa Fe	27
Silver City	4
NEW YORK	
Kingston	32
Newburgh	22
New York	17
Poughkeepsie	25
NORTH CAROLINA	
Asheville	10
Charlotte	5
Raleigh	4
NORTH DAKOTA	
Bismarck	60
Dickinson	48
Fargo	54
Jamestown	54
Minot	48
OHIO	
Akron	15
Cincinnati	9
Cleveland	22
Columbus	10
Dayton	10
Toledo	23
OKLAHOMA	
Guthrie	8
McAlester	6
Oklahoma City	7
OREGON	
Ashland	1
Ontario	12
Pendleton	7
Portland	2
PENNSYLVANIA	
Easton	15
Erie	30
Harrisburg	19
Johnstown	15
Philadelphia	11
Pittsburgh	15
Reading	14
RHODE ISLAND	
Newport	26
Providence	26
SOUTH CAROLINA	
Charleston, less than	1
Columbia	2
Greenville	5
SOUTH DAKOTA	
Aberdeen	30
Pierre	37
Sioux Falls	40
TENNESSEE	
Chattanooga	6
Knoxville	8
Memphis	3
Nashville	5
TEXAS	
Dallas	3
El Paso	2
Fort Worth	4
Houston, less than	1
San Antonio	1
UTAH	
Logan	24
Nephi	16
Ogden	12
Salt Lake City	18
VIRGINIA	
Lynchburg	5
Norfolk	3
Richmond	4
Roanoke	10
WASHINGTON	
Astoria	1
Seattle	5
Spokane	24
Tacoma	4
Walla Walla	7
WEST VIRGINIA	
Bluefield	15
Charleston	10
Huntington	8
Parkersburg	9
Wheeling	12
WISCONSIN	
Green Bay	24
Madison	42
Milwaukee	42
Racine	40
Superior	48
WYOMING	
Cheyenne	24
Sheridan	26
Yellowstone	48

Garneau /1939-10 lists the following maximum depths of frost penetration for Canadian cities. Data are for city streets with snow removed.

Location City	Depth Ft.
Calgary, Alta.	12
Edmonton, Alta.	6½
Moose Jaw, Sask.	9
Regina, Sask.	7
Winnipeg, Man.	7
Windsor, Ont.	7
Ottowa, Ont.	6
Quebec, Ont.	4 - 6

The Corps of Engineers Manual /1946-5 presents a map on which isolines of "freezing index" are conducted to show the aereal distribution of ground freezing. That map is shown later under "Correlation of Frost Depth and Temperature Data."

Factors Which Influence Depth of Soil Freezing

Several factors influence soil temperatures and the depth of soil freezing. Some of the factors influence ground freezing by affecting air temperatures, sunshine, wind, or precipitation. Other factors include those which determine the degree in which the soil is exposed to the elements of climate, e.g., the ground cover, and the response of the soil to exposure due to the thermal properties of the soil. These factors may, for convenience, be placed into two broad categories: position and condition.

The positional factors which effect soil temperature through their influence in climate or its effects are: (1) Latitude, (2) Altitude (of importance in mountainous terrain but of no significance in prairie states), (3) Proximity to bodies of water (Great Lakes and Oceans have a marked effect in modifying temperatures), (4) Terrain (some protection is afforded by hilly country), and (5) Slope of ground (south slopes often freeze later do not freeze as deep and are first to thaw).

The above factors bear a direct relation to the climate and its impact on ground temperatures. (The term climate as here used, includes the combined effects of temperature, sunshine, wind and precipitation). In addition there are factors of condition which also influence soil freezing. They are:

1. Extent and type of ground cover (this may include brush, grasses, snow, type and thickness of pavement, etc.;
2. Soil moisture content. (an important factor due to high specific heat of water);
3. Soil density;
4. Soil color;
5. Soluble salts in soil moisture;
6. Thermal properties (specific heat, thermal conductivity) of soil mass.

Bouyoucos /1916-3 classified the factors which influence soil temperature as (1) those which are intrinsic and (2) those which are external. The intrinsic, i.e., the internal, are those contained by the soil and include the specific heat, heat conductivity, radiation, water content, density, evaporation of water, concentration of the soil solution, topographic position, condition of the surface, etc. The external factors comprise the meteorological elements and include air temperature, sunshine, wind, barometric pressure, precipitation, etc.

Climate

Temperature in Air and Soil

Effect of earth's radiation on soil temperatures - Forbes (Transactions of the Royal Society of Edinburgh, 1846, 16, p. 189) and Kelvin (Transaction of the Royal Society of Edinburgh, 1862, 23, p. 157) showed an average increase of temperature with depth (in England) of about 1 C. for 110 ft. The final conclusion of the British Association Committee was that an average of "41.4 gramme-degrees of heat escape annually through a sq. cm. of a horizontal section of the earth's substance." This was believed to have negligible effect on soil temperatures at depths subjected to freezing.

Temperatures in soil may change daily, periodically during a season, seasonally, and annually. The daily changes may be non-uniform or they may be cyclic in nature where the temperature variations take distinct wave patterns. The daily change in soil temperature is usually limited to small depths. Callendar /1895-1 and Callendar and McCleod /1896-1 reported some of the early measurements of soil temperatures made in North America. They used electrical resistance thermometers for their measurements at Montreal, and extended their observations throughout the different seasons. Figure 113 indicates the distinct cyclic nature of soil temperatures at a depth of 4 in. during April 17 to April 22; the much dampened wave for the 10 in. depth, and the absence of daily cyclic variations at the 20 and 40 in. depths.

A large proportion of the many measurements of soil temperatures referred to in the literature have been made by those studying the effect of soil temperature on the growth of plants. The results of Keen and Russell /1921-4 are of interest because they illustrate the time lag in summer and the marked absence of a cyclic change in soil temperature even at a 6-in. depth at certain periods in the winter when the soil is only slightly above freezing temperature. They also showed that the temperature fluctuation in the summer below the ground line is a "reduced image" of that at the surface and that at the 6-in. depth there is a time lag of several hours between the beginning of change in temperature at the surface and its appearance at a given depth. The daily temperature fluctuations in winter were small in comparison.

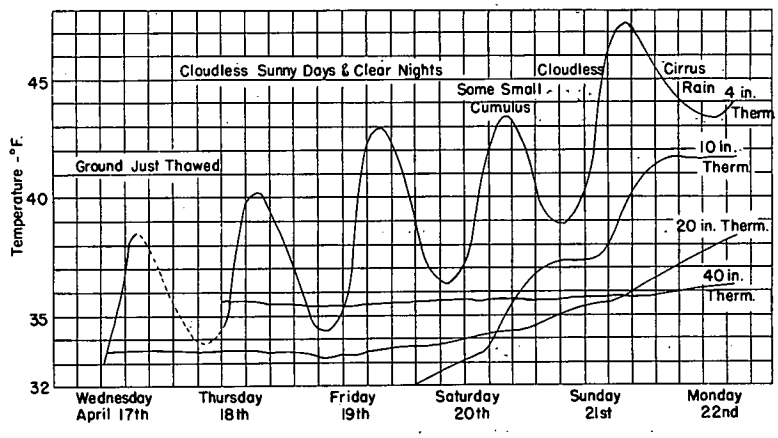


Figure 113. Diurnal Temperature Variations (After Callendar and McCleod)

Kimball, Ruhnke, and Glover /1934-1, West /1932-1, Smith /1932-2, Mattimore and Rahn /1935-2, Baver /1940-2, and many other investigators have presented data on daily changes in ground temperatures.

Seasonal temperature movements - Data on seasonal temperature movements, like those for daily movements are available in many articles in the published literature. Callendar /1895-1 measured seasonal changes to depths of 100 in. Bouyoucos /1913-1, /1916-3, Harrington /1928-3, Kimball, Ruhnke and Glover /1934-1, Thomson /1934-5, Mattimore and Rahn /1935-2, Winn and Rutledge /1940-7, Belcher /1940-14, Hieronymous /1944-9, Swanberg /1945-7, and the Corps of Engineers /1947-2 are among the many who presented data on seasonal changes in temperatures in soils for various depths below the ground surface. Some of the work done by these authors is of interest to those who study soil freezing. Thomson /1934-5 made measurements at depths of 4, 10, 20, 40, and 66 in. and 9 and 15 ft. below ground surface at Winnepeg, Canada. Figure 114 shows the average values for a 3-year period. Of interest is the annual "overturn" in soil temperatures and the relative change in temperature for the various depths. Swanberg's data /1945-7 covers the cold weather season for a period of 5 years and gives soil temperatures at depths to 60 in. under a concrete pavement in Minnesota. Swanberg's data which are shown graphically in Figure 115, permit the determination of the temperature gradient at various seasons of the year.

The investigations by the Corps of Engineers /1947-2 included measurements of temperature at seven airfields under rigid and flexible pavements and turf cover. Figure 116 shows typical

subsurface temperature data for the two types of surfacing and for the turf cover for depths to 6 ft. at Dow Field, Bangor, Maine for the period from January to April 1945. The graphs also indicate the depth of frost at different times during the 4-month period.

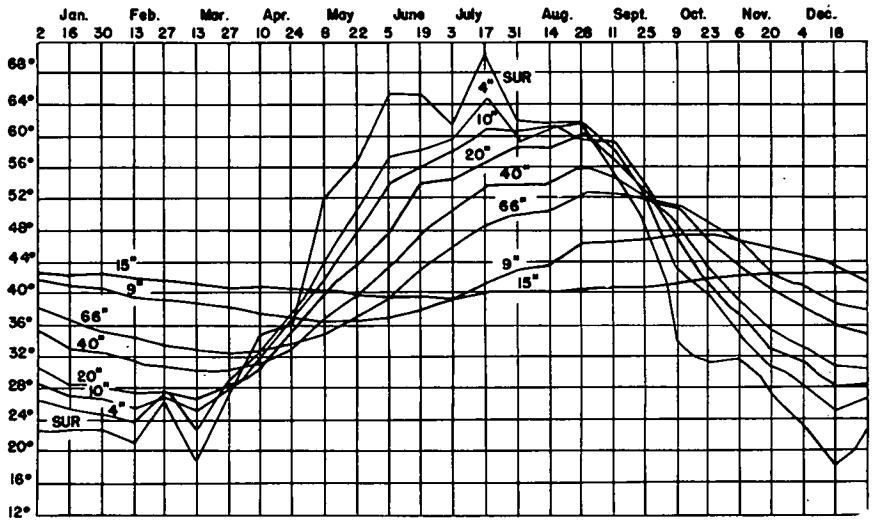


Figure 114. Soil temperature variations between its surface and 15 feet; 3 year average. (After Thomson /1934-5)

Maximum depth of annual temperature changes - Several investigators have attempted to obtain data which aid in defining the maximum depth of seasonal and annual temperature changes in the earth. Thomson's data /1934-5 showed an average annual change of about 3 F. at a depth of 15 ft. at Winnipeg. Heironymous /1949-9 observed air and earth temperatures over a 7-yr. period for the purpose of obtaining data to facilitate computation of underground-electric-cable loading limits. The study was made in a grassed plot in Kansas City in loessial soils of the Marshall series. (60 percent silt, 27 percent clay). His temperature observations extended to a depth of 12 ft. He found little change in cyclic levels at the 12-ft. depth. His method of determining the sink point, i.e., the value of depth at which there is no apparent response to the variation in surface air temperature, is indicated in Figure 117. The value obtained for Kansas City by his method is about 25 ft.

Temperature at the frost line - Early in this review consideration was given to the freezing point in soils determined by laboratory testing. It was found that fine-grain soils generally had lower freezing points than did coarse-grain soils and that a saturated clay solidifies at a higher temperature than does a similar clay having a moisture content well below saturation. Beskow /1935-1 found in nature a supercooling effect, the temperatures at which freezing begins being slightly below freezing. He presented data shown in Table 39 showing temperatures at which freezing occurs.

TABLE 39

Type of Soil	Frost Line Temperature Limits deg. C.	Average deg. C.
Medium Sand (6% moisture)	0.015-0.02	---
Silty Moraine, Stockholm	0.02—0.08	0.05
Silt Norrbotten	0.15—0.2	---
Lean Clay, Vasternorrland	0.4—1.0	

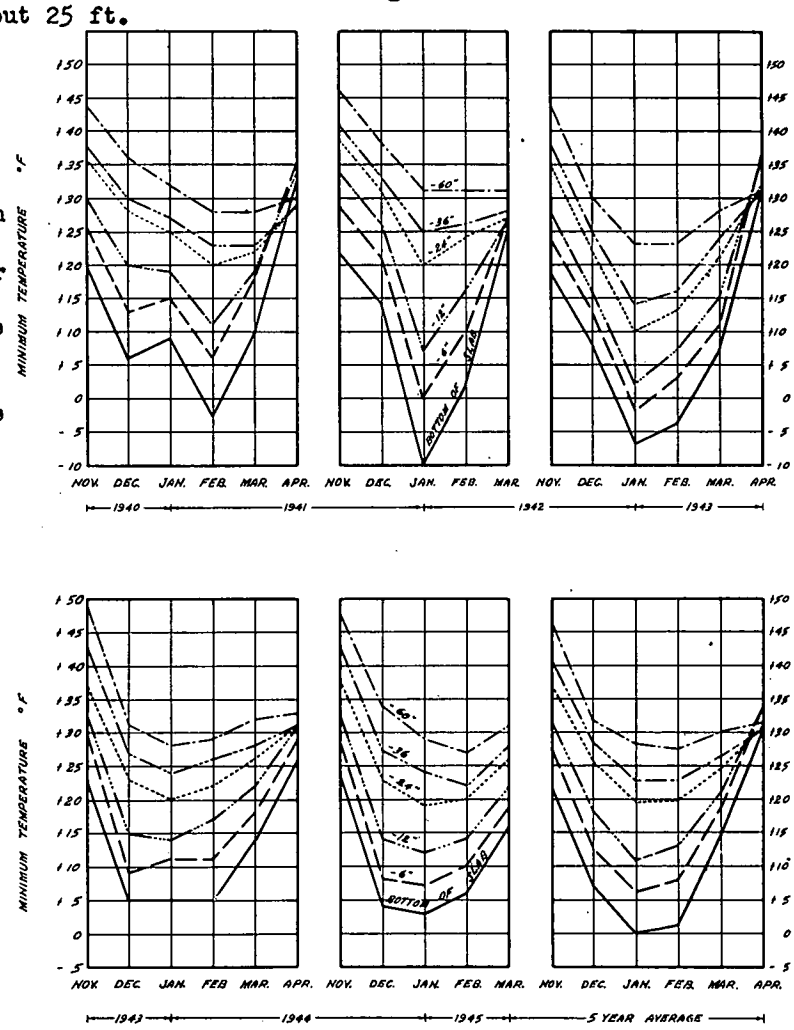


Figure 115. Minimum Temperatures at Various Points in Subgrade Under 7-in. Concrete Slab in Laboratory Driveway. (After Swanberg /1945-7)

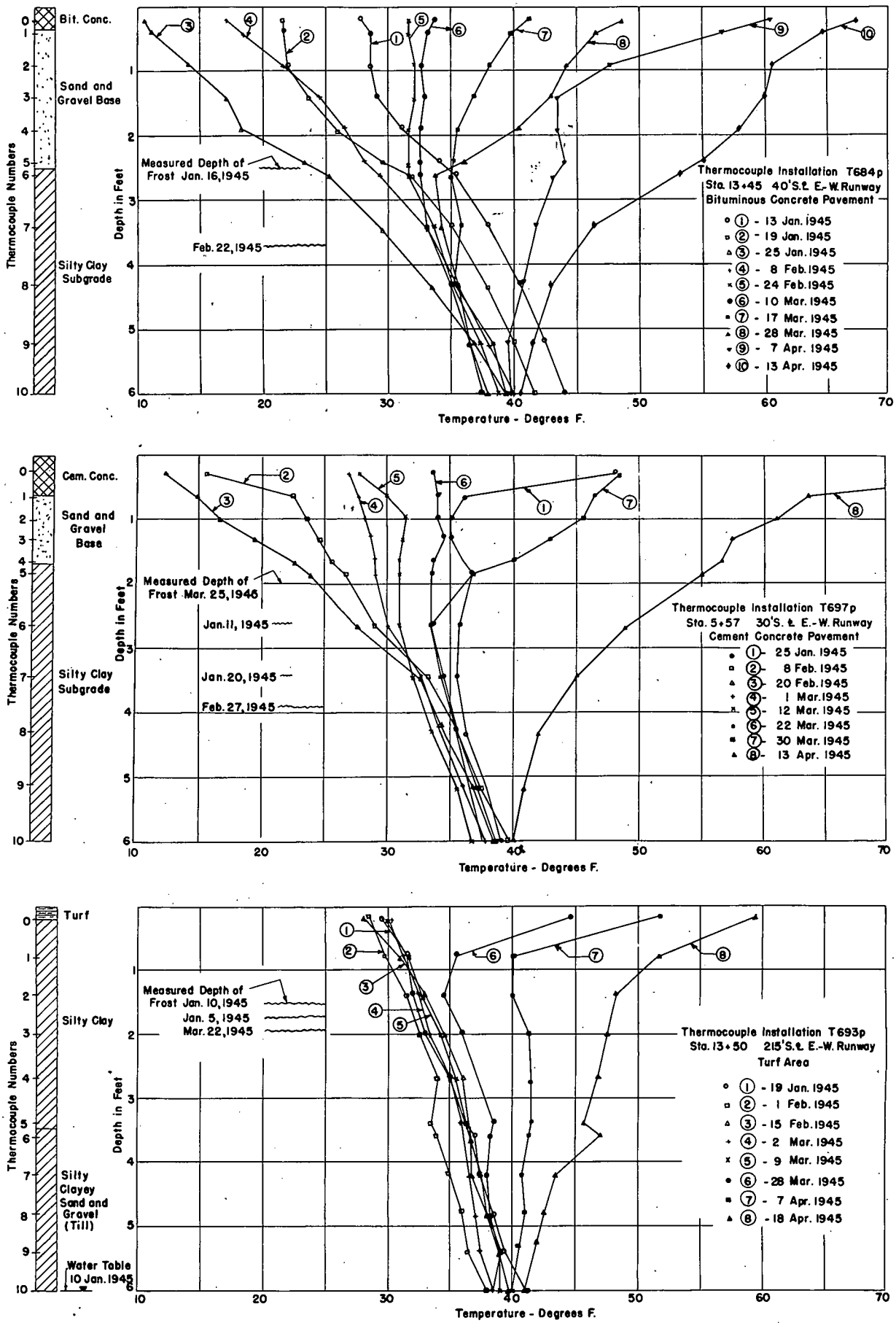


Figure 116
Typical Subsurface Temperature Data
(After Corps of Engineers)

When thawing takes place, however, the action is different, especially if ice lenses are present in the soil. Then, since the ice melts at 32 F., the receding frost line (from below) is the 32 F. line.

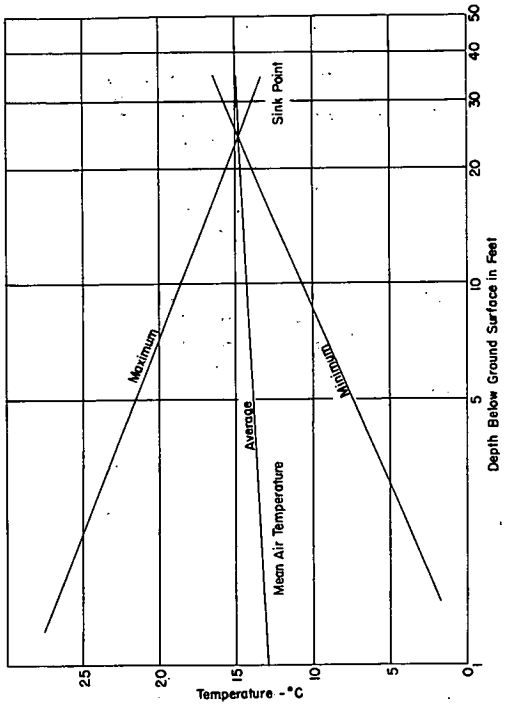


Figure 117
Relation Between Air and Ground Temperatures. (After Heironymous)

Beskow /1935-1 attempted to measure the temperature at the frost line by exact means. He obtained a cylinder of lean clay, installed thermoelements, insulated the sides so it would freeze from the top down, and allowed it to freeze outdoors. He found the frost line was slightly below the -1 C. line (about 30.2 F.). Mackintosh /1936-9 made tests at Harvard on a cylinder of clay and found the 0 C. line about 2½ cm (about 1 in.) below the limit of frost line. The temperature at the frost line was about -0.75 C. (30.65 F.)

Data from field studies are apparently not sufficiently accurate to use as a basis for determining the temperature at the freezing line. A review of the Corps of Engineers field studies /1947-2 on airfields shows a reasonably close agreement between depth of penetration of frost and the 32 F. line. However, where the frost and the 32 F. lines did not coincide, the depth of frost was greater than the depth of the 32 F. line in more instances than less. Only at the Fargo and Pierre airfields was the 32 F. line materially below the depth of frost. Where the frost line was found to be below the 32 F. line the soils were usually lighter in texture than those found at Fargo and Pierre.

The 1945-1947 studies of the Corps of Engineers /1949-23 made comparisons of temperatures observed by means of thermocouples with those obtained by means of thermometers. A study of typical readings at three airfields showed that the thermocouples generally gave higher temperature readings than the thermometers at comparative depths. Differences between the two methods generally ranged from 1 to 3 deg. although some readings differed by 6 deg. The higher readings by the thermocouples could well account for the discrepancy between depths of the 32 F. line and the lower limit of frozen soil. In later studies /1951-32 the Corps found that the soil moisture in a number of soils tested, freezes at temperatures ranging from 29.1 F. to 32 F. the lower values occurring in the silty and clayey soils. During laboratory studies it was found that the 32 F. temperature, after progressing into the sample for a depth of 2 to 4 in., suddenly receded, usually 1 to 2 in. before proceeding downward again. It was believed that "this phenomenon is due to the release of latent heat of fusion when the soil moisture begins to freeze at the top of the sample as a result of some triggering action, after having become subcooled to below the normal freezing point and heaving commenced only after the temperature recession had occurred."

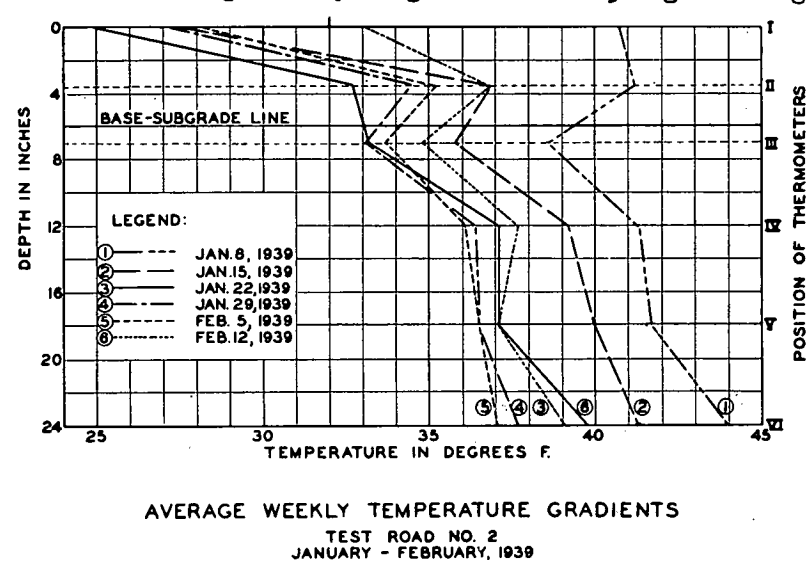


Figure 118. Average Weekly Temperature Gradients (After Winn and Rutledge /1940-7)

Temperature gradients in freezing soil - The magnitude and the rate of change in the temperature gradient are largest at the surface and decrease with depth. In freezing soil, however, if the soil is well below the surface, the gradient often becomes quite small, especially in clay soils. Beskow /1935-1 concluded that "at the beginning of freezing the gradient at the frost line is about 0.1 C. per cm (0.46 F. per in.) but decreases during the winter rather rapidly,

and at the turn of the year is about 0.05 C. per cm. (0.23 F. per in.) and at the end of winter is about 0.02 - 0.03 C. per cm. (0.09 - 0.14 F. per in.). Winn and Rutledge /1940-7 observed temperatures of the air and at depths of 3½, 7 (bottom of stabilized base), 12, 18 and 24 in. below the surface of a test road in Indiana. Freezing temperatures reached a depth of about 3½ in. during the two months of observations (Jan., Feb. 1939). However the temperature gradients reached a maximum of 3 F. per in. near the surfaces and up to about 3/4 F. per in. in the subgrade. The average weekly temperature gradients which they obtained are shown graphically in Figure 118.

Swanberg's studies /1945-7 of conditions in Minnesota showed temperature gradients up to 1 F. per in. and greater immediately under the slab, yet the gradient was only 0.2 F. per in. at depths of 36 to 60 in. Swanberg computed the maximum temperature gradients shown in Table 40.

TABLE 40

SUBGRADE TEMPERATURE GRADIENTS

Air Temperature -10 F. to -5 F. Bottom of Slab
Temperature -5 F. to +10 F. (After Swanberg /1945-7)

Depth Below Slab	Temperature Gradient Deg. F. per inch			
	Dec.	Jan.	Feb.	Mar.
in.				
0 to 6	1.17	1.17	1.27	1.67
6 to 12	1.17	1.11	.67	.75
12 to 24	.65	.95	.62	.25
24 to 36	.29	.36	.15	.08
36 to 60	.20	.27	.15	.10

Cycles of freezing and thawing of subgrade soils - There is general agreement among writers that several cycles of thawing and refreezing result in greater accumulation of water, greater heave, and greater reduction in load carrying capacity than result from a single cycle. Therefore it is of interest to know something of the number of cycles of freezing and thawing which take place in soils under pavements. Data on cyclic freezing and thawing of subgrades obtained on the Lawrence experimental concrete pavement in Kansas were reported by Arndt /1943-11. He gave the results of temperature observations in air, in top and bottom of a 9-7-9-concrete pavement, and at a depth of 6 in. in the subgrade for the period January 1936 to December 1940. Arndt stated that "one freezing and thawing cycle...is defined as thawing from 31 F. to 33 F." His results for the 5 year period are given in Table 41. It may be seen that during 2 of the 5 years the subgrade did not go through a single cycle of freezing and thawing, yet during one year (1937), the subgrade went through 8 cycles of freezing and thawing.

Swanberg's tests /1945-7 on a 7-in. concrete pavement in Minnesota also covered a 5-year period. His temperature observations were made to depths of 60 in. The results showing number of cycles of freezing and thawing in air, at the top and bottom of the slab and at depths of 6, 12, 24, 36 and 60 in. below the slab are shown in Table 42. It may be seen that at a depth of 6 in. below bottom of slab there occurred an average of five freezing and thawing cycles per yr. for the 5-year period, and that two cycles of freezing and thawing occurred at the 12-in. depth.

Influence of Duration and Intensity of Cold - The preceding review of seasonal air and soil temperature has shown that the depth to which freezing temperatures penetrate the ground is dependent on not only the intensity of cold but also the duration of the cold period. It has been shown that the degree of damage resulting from frost action is usually associated with a fairly long cold period, provided that the moisture contained in the soil or otherwise available for freezing is adequate. Thus the relationship between duration and intensity of cold and the depth of freezing is one worthy of study by engineers working in frost areas. The literature reviewed shows that the interest in that relationship is high in several branches of science. Of the work done in sciences other than highway and airport engineering, perhaps the work of waterworks engineers is of most interest. In all sciences, however, the work of developing the relationship is comparatively recent. It is worthy of note that some of the early observations of the intensity-duration relationships were by highway engineers. Illinois /1924-3, found that

the approximate depth of frost penetration is related to the average temperature below freezing and its duration being markedly less in Central Illinois than for Northern Illinois. Sourwine /1930-5 presented a theoretical study of the relationship in which he gave consideration to "the intensity, duration, and frequency of low temperature occurrence over a period of years" in an effort to develop a method by which climatological records can be used as a guide in predicting on a relative basis the probable highway damage.

TABLE 41

FREEZING AND THAWING CYCLES

(31 deg. F. to 33 deg. F.) (After Arndt 1943-11)

Year (Jan. to Dec.)	Air	Pavement		Sub-grade	Remarks
		Top	Bot-tom		
1936	90	39	25	2	First 30 days of year coldest on record. Summer second hottest on record. Maximum 112 deg. Lowest 10 deg. below zero.
1937	80	76	60	8	Average temperature below normal. 4 months above normal. No extreme temperatures. Lowest zero. Maximum 103 degrees.
1938	77	58	42	0	All but 2 months above normal. Maximum 106 degrees. Minimum zero.
1939	62	47	34	0	Average temperature above normal. Maximum 106 degrees. Minimum 4 degrees below zero.
1940	58	54	37	1	Mean temperature 2 degrees below normal. Maximum 102 degrees. Minimum 13 degrees below zero.

TABLE 42

NUMBER OF FREEZING AND THAWING CYCLES

Investigation No. 132
(After Swanberg 1945-7)

Location of Point	November						December						January						February						March						April						Total					
	1940	1941	1942	1943	1944	Average	1940	1941	1942	1943	1944	Average	1941	1942	1943	1944	1945	Average	1941	1942	1943	1944	1945	Average	1941	1942	1943	1944	1945	Average	1941	1942	1943	1944	1945	Average	1941	1942	1943	1944	1945	Average
—Top of slab	6	12	16	7	0	8.2	6	10	0	11	8	7.0	2	12	0	13	2	5.8	4	8	7	11	5	7.0	23	29	10	12	6	16.0	2	3	4	7	1	3.4	43	74	37	61	22	47.4
—Between slab and subgrade	3	6	7	5	0	4.2	0	5	0	11	6	4.4	0	4	0	6	0	2.0	0	1	6	7	4	3.6	11	16	7	7	3	8.8	0	0	0	5	0	1.0	14	32	20	41	13	24.0
—6" below bottom of slab	1	1	0	3	0	1.0	0	1	0	4	5	2.0	0	0	0	3	0	0.6	0	0	2	0	0	0.4	1	1	1	0	1	0.8	0	0	0	1	0	0.2	2	3	3	11	6	5.0
—12"	0	0	0	0	0	0.0	0	1	0	2	4	1.4	0	0	0	0	0	0.0	0	0	0	0	0	0.0	1	0	1	0	1	0.6	0	0	0	1	0	0.2	1	1	1	3	5	2.2
—24"	0	0	0	0	0	0.0	1	1	0	0	0	0.4	0	0	0	0	0	0.0	0	0	0	0	0	0.0	0	1	1	0	1	0.6	1	0	1	0	0	0.4	2	2	1	1	1	1.4
—36"	0	0	0	0	0	0.0	1	0	0	0	1	0.2	0	0	0	0	0	0.0	0	0	0	0	0	0.0	0	1	0	0	2	0.6	1	0	1	1	0	0.6	2	1	1	1	3	1.6
—60"	0	0	0	0	0	0.0	0	0	0	0	0	0.0	0	1	0	1	1	0.6	0	0	0	1	1	0.4	0	0	0	0	0	0.0	1	0	1	0	0	0.4	1	1	1	2	2	1.4
—Air temperature	6	15	15	12	6	10.8	9	7	1	13	8	7.6	4	17	2	12	4	7.8	3	5	7	10	9	6.8	26	26	9	10	7	15.6	2	8	7	8	2	5.4	50	78	42	65	36	54.2

Critical air temperatures for ground freezing - Sourwine /1930-5 reviewed the results obtained by Bouyoucos /1920-1 and Petit (whose results are discussed by Bouyoucos) and stated that the period of supercooling required to produce freezing in soils ranged from 160 min. for sand to about 190 min. for "average" clay. From climatological data he found that absolute-minimum air temperature seldom endured longer than 3½ hrs. (210 min.). Applying this and the results of Bouyoucos /1920-1 and Petit he arrived at a critical initial air temperature of 23.0 to 23.5 F. for sand and 21.4 to 22.4 F. for clay and adopted 23 F. as the critical air temperature

to produce freezing and a depth of 3 in. as the critical depth. He then makes an arbitrary assumption of an allowable frequency of the critical air temperature of 5 percent — which means that "for any locality all cold periods sufficient to cause freezing of average surface soil, ground freezing below 3-in. depth may occur 1 time in 20." A frequency of more than 5 percent, he designates as "objectionable frequency." He then determines the "critical value of the lowest monthly average of daily minimum temperatures which corresponds with an average of 5 percent frequency of occurrence of highway ground freezing to a depth greater than 3 in." Again applying the results of Bouyoucos /1920-1 he arrives at an "absolute-minimum soil temperature of approximately 26.4 F. coincident with inception of ground freezing."

Again referring to the work of Bouyoucos /1916-3, Sourwine /1930-5 finds that the average minimum air temperature which is equivalent to a soil temperature of 26.4 F. at a 3-in. depth is 16 F. That is, "16 F. is the actual average equivalent minimum air temperature at which ground freezing begins at the 3-in. depth." with no allowable frequency permitted at the 3-in. depth.

He analyzed the air-temperature data from 15 stations well distributed across the United States for all periods where the temperature fell below 23 F. for a duration of 8 hrs. or more and for a temperature drop greater than 3 F. The records were summarized and curves plotted showing the relationship between the minimum temperature and frequency of cold periods as indicated in Figure 119. Thus in 5 percent of the cold periods the temperature in Amarillo reached a low of 9 F. Sourwine then tabulated these data together with absolute-minimum temperatures as indicated in Table 43.

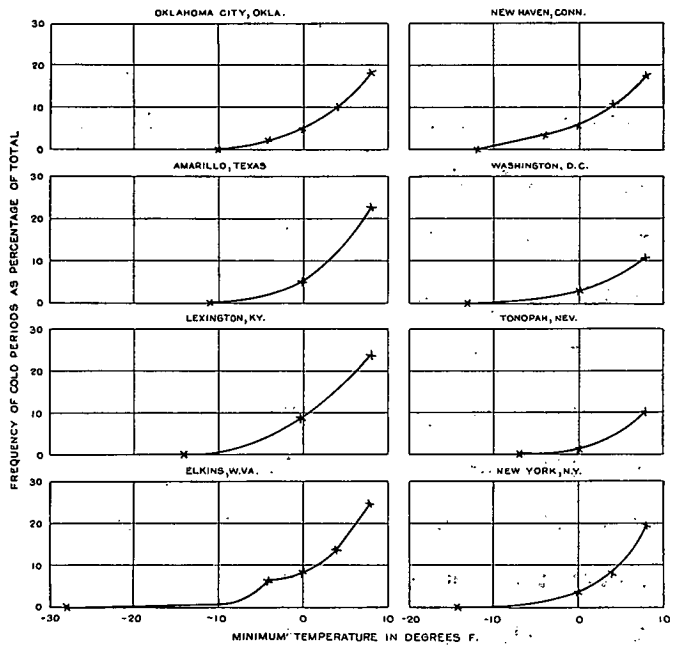


Figure 119. Minimum Temperature-Frequency Curves for Selected Stations Showing Relation Between Minimum Temperature Occurring with a Frequency of 5 Percent, and Absolute Minimum Reported. (After Sourwine /1930-5.)

From Table 43 Sourwine found an average difference of about 13 F. between the minimum temperature with a frequency of 5 percent and the absolute-minimum temperature. By subtracting 13 from 16 (the temperature at which ground freezing begins at 3-in. depth) he found a value of 3 F. as the critical absolute-minimum air temperature coincident with 5 percent frequency of ground freezing at a 3-inch depth. Then going back to the records of stations, charts similar to Figure 120 were prepared showing the relation between the monthly average of daily minimum temperatures and absolute-minimum temperature for each station. From the line drawn (Figure 120) there was obtained for each station the monthly average of daily minimum temperature corresponding to an absolute-minimum temperature of 3 F. For example, for Amarillo this was 24.4 F. which corresponds with the average. Due to variations in the data thus obtained, Sourwine made a correction factor of -1.4 F. making the value 23 F., which represents the lowest monthly average of daily minimum air temperature, as a critical design value for highway ground freezing.

Sourwine /1930-5 then made isothermal maps for each state and combined them to arrive at Figure 121. According to Sourwine the 23 F. isothermal line in Figure 121 "marks a general critical-temperature line below which should lie areas relatively safe and above which should lie areas relatively dangerous from highway ground freezing." Sourwine considered the 23 F. line "a general line only...and subject to modification or approval for any local area...covering local conditions of soil, drainage, surface cover and exposure".

Duration of cold - Sourwine /1930-5 stated, "It is well known that several days duration of low temperature, even with a rather moderate minimum temperature, may produce effects equally damaging and possibly even more damaging than those produced by a minimum temperature several degrees colder but having only a short duration". He expressed cold duration in degree-hours and stated that the approximate value of degree-hours for any cold period is:

$$\text{Degree-hours} = \tfrac{1}{2}\,DT$$

TABLE 43

Minimum Temperature of 5 Percent Frequency Occurrence
Vs. Absolute Minimum Temperature

Station	Period of Study	Minimum Temperature with frequency of 5 per cent (1)	Absolute Minimum Temperature (2)	Difference (1) - (2)
		Deg. F.	Deg. F.	
New Haven, Conn.	11 yrs. 1911-1921	-0.5	-12.0	-11.5
New York, N.Y.	do	+1.0	-14.0	-15.0
Philadelphia, Pa.	do	+4.0	- 4.0	- 8.0
Washington, D. C.	15 yrs. 1907-1921	+5.0	-13.0	-18.0
Elkins, W. Va.	11 yrs. 1911-1921	-5.0	-28.0	-23.0
Lynchburg, Va.	do	+4.0	- 7.0	-11.0
Lexington, Ky.	15 yrs. 1907-1921	-2.0	-14.0	-12.0
Nashville, Tenn.	11 yrs. 1911-1921	+1.3	-10.0	-11.3
Evansville, Ind.	do	-1.8	-16.0	-14.2
Cairo, Illinois	do	-2.0	-15.0	-13.0
St. Louis, Mo.	do	-3.5	-16.0	-12.5
Bentonville, Ark.	do	-2.5	-20.0	-17.5
Oklahoma City, Okla.	do	-0.5	-10.0	- 9.5
Amarillo, Texas	15 yrs. 1907-1921	+1.0	-11.0	-12.0
Tonopah, Nev.	14 yrs. 1908-1921	+5.0	- 7.0	-12.0
	Average			-13.37

where D is duration in hours below the critical initial air temperature (23 F.) and T is the degrees Fahrenheit of absolute minimum air temperature below 23 F. He studied data from 35 stations lying in the general proximity of the 23 F. line (Fig. 120). He then summarized the data according to frequency of occurrence, both relative to duration of cold period and to "minimum temperature of cold period." He plotted for each station a duration-frequency curve and a minimum temperature-frequency curve. From these two curves was derived a third curve, the degree-hour-frequency curv Examples of the above relationships are shown in Figure 122.

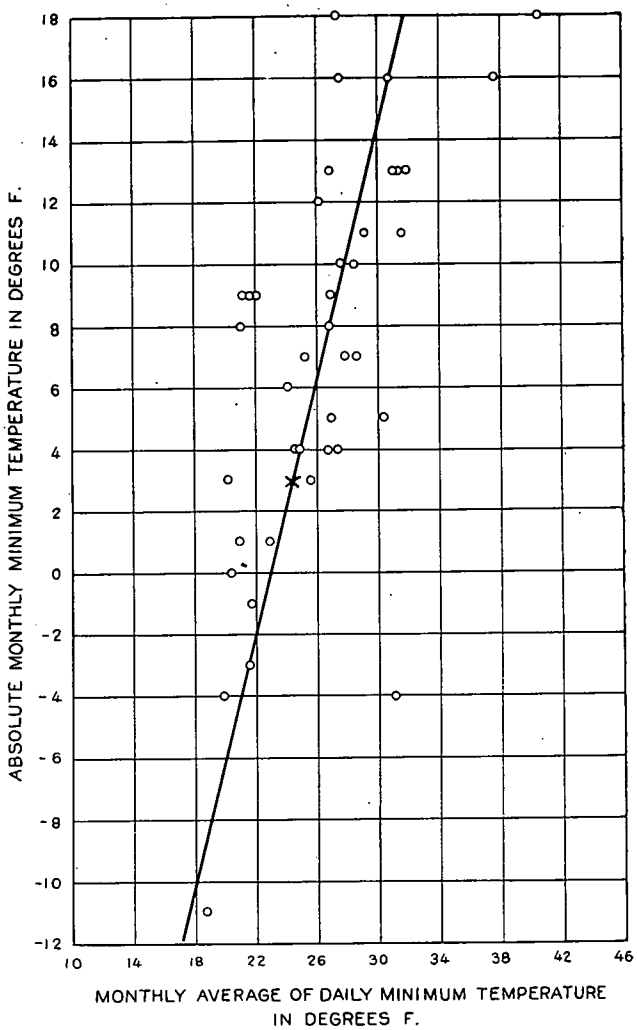


Figure 120. Relation Between Monthly Average of Daily Minimum Temperature and Absolute Minimum Temperature at Amarillo, Texas. (After Sourwine)

As a measure of cold quantity, Sourwine selected as an index "the degree-hours frequency curve. These were plotted on a map and lines of equal "degree-hour index" for 5 percent frequency drawn as shown in Figure 123.

Sourwine finally selected a degree-hour index of 900 as representing "the danger line for highway ground freezing, based on the combined study of low temperature intensity, cold period duration, and cold period frequency." He then prepared a "highway ground-freezing index map" of the United States as shown in Figure 124.

Casagrande /1931-14 cast concrete slabs on a New Hampshire silt soil and recorded the elevation of water table, height of heave, and average daily air temperature. He presented one of the first correlations between cumulative temperatures, frost heaving, and depth of penetration of frost. He used the average daily temperature (less 1 F.) and plotted cumulative degree-days of temperature against time. In plotting the cumulative temperatures he considered temperatures below 32 F. as positive and those above 32 F. as negative. Casagrande's Duration and Intensity vs. Time Chart is shown earlier under "Factors Influencing Magnitude, Rate and Nature of Frost Action and Reduction in Load Carrying Capacity".

Mabee /1937-5 correlated duration and intensity of cold with depth of frost penetration and also with the number of cases of frozen-water services in Indianapolis, Indiana. The results of some of his studies are shown in Figure 125. In the figure Mabee shows the relationship between cumulative degree-days below 32 F. and depth of frost for a normal winter and for the winter of 1935-1936, which he cited as the coldest winter in Indianapolis in 65 years. The results show a reasonably good correlation between the duration and intensity of cold and depth of frost. For the winter of 1935-36, they also show a relation to the number of frozen services reported each day during the prolonged severe cold spell. The frost depths were measured from excavations. It was not noted whether the excavations were under paved or grassed areas or whether a snow cover existed.

The duration and intensity of cold also bears a relation to the magnitude of frost heave if other conditions are equal. Gardner and Wright /1939-5 observed air temperatures and heaving in their studies of heaving on a section of road north of Dover. New Hampshire. The road was paved with a 9 in.-6 in.-9 in. X 18 ft. concrete slab laid in a gravel base 4 to 8 in. thickness. Figure 126, which was drawn from data given by Gardner and Wright, illustrates the relationship between accumulated temperatures and frost heave for two soils, one a clay and the other a sandy loam. Skelton /1940-12 also reported data on this same project.

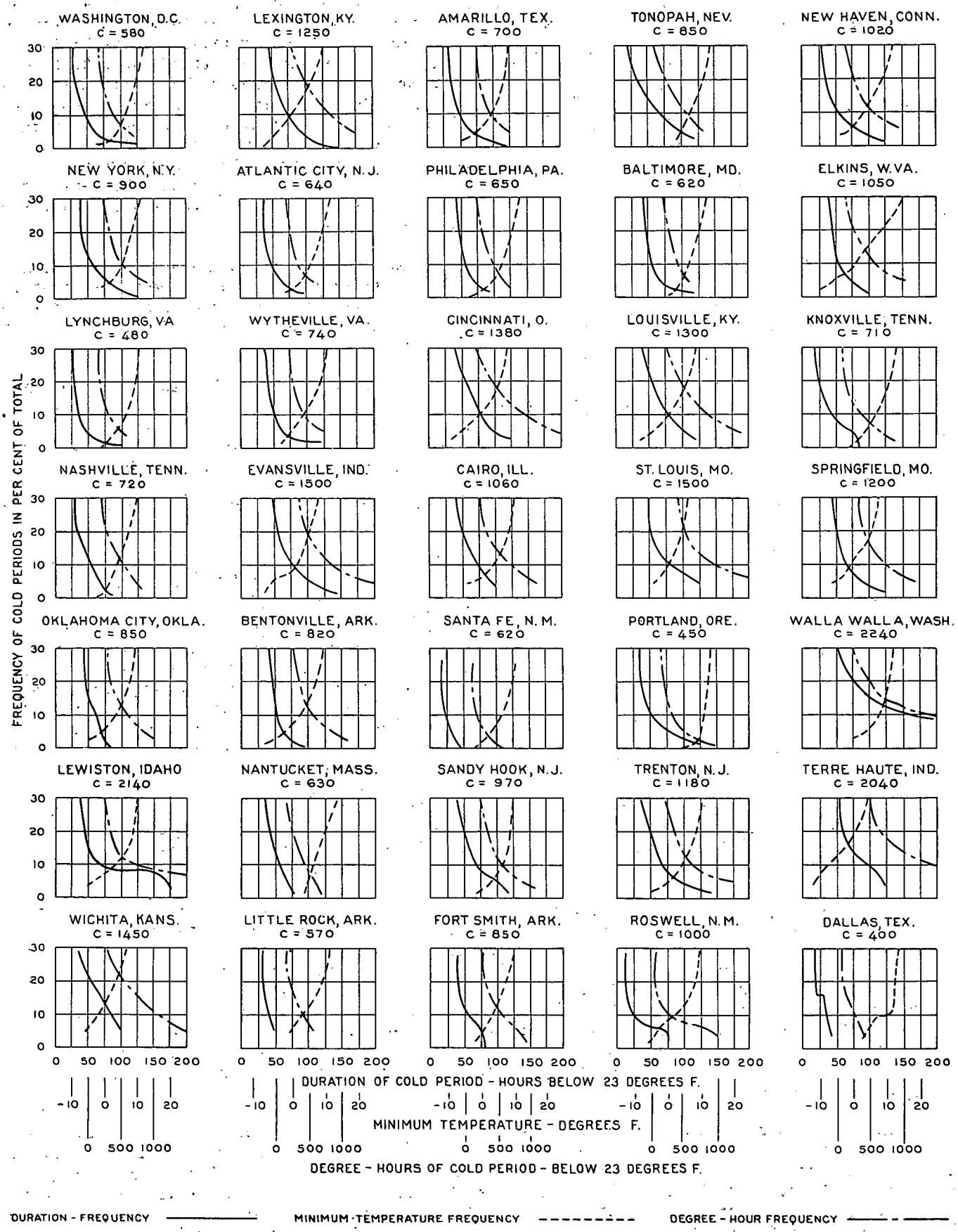


Figure 122. Duration-Frequency Curves, Minimum Temperature-Frequency Curves, and Degree-Hour-Frequency Curves for 35 Selected Weather Bureau Stations. (After Sourwine /1930-5)

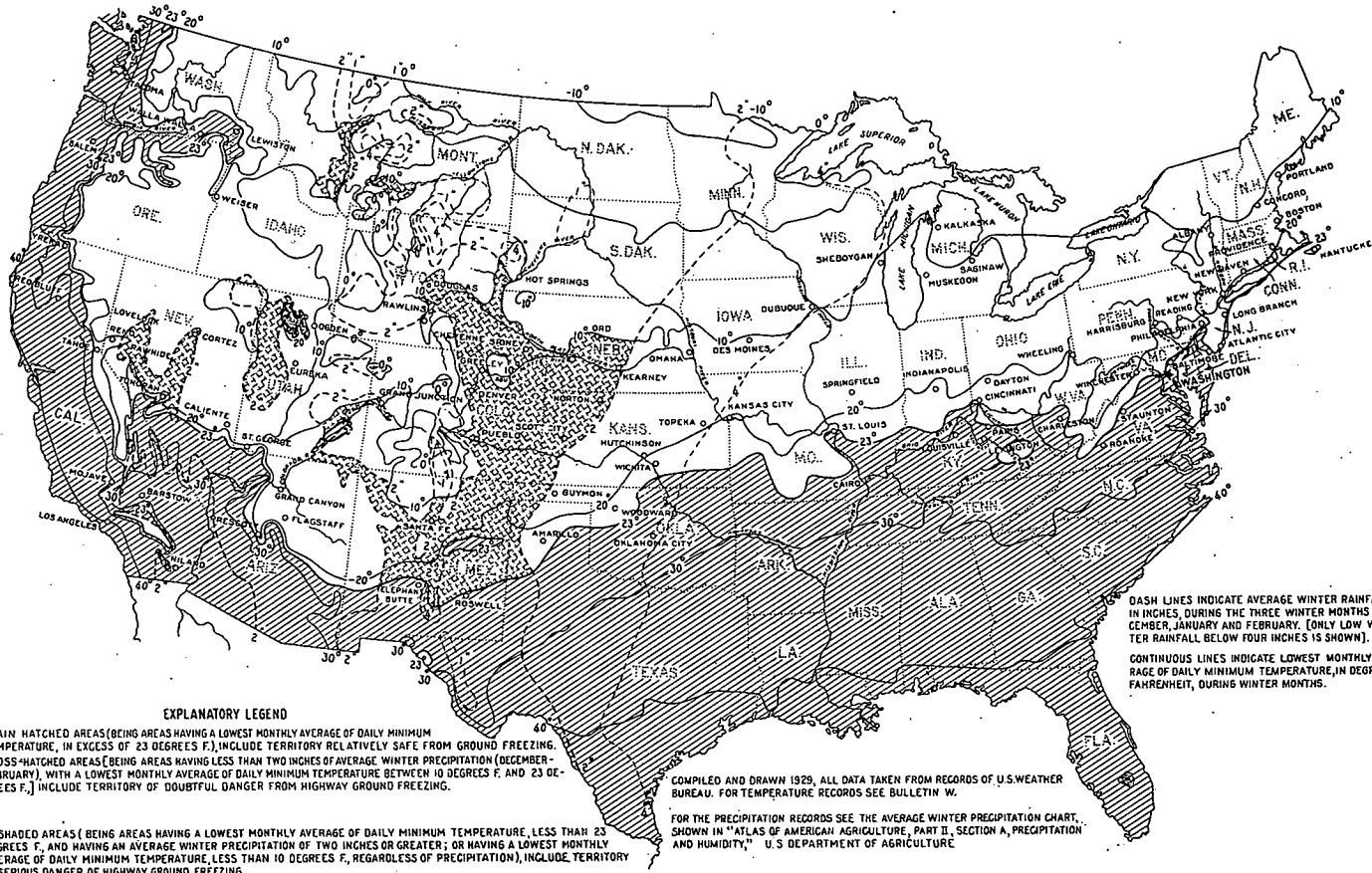


Figure 121. Isothermal Guide Map for Use in Determining Areas Subject to Ground Freezing. Compiled From U. S. Weather Bureau Records. (After Sourwine)

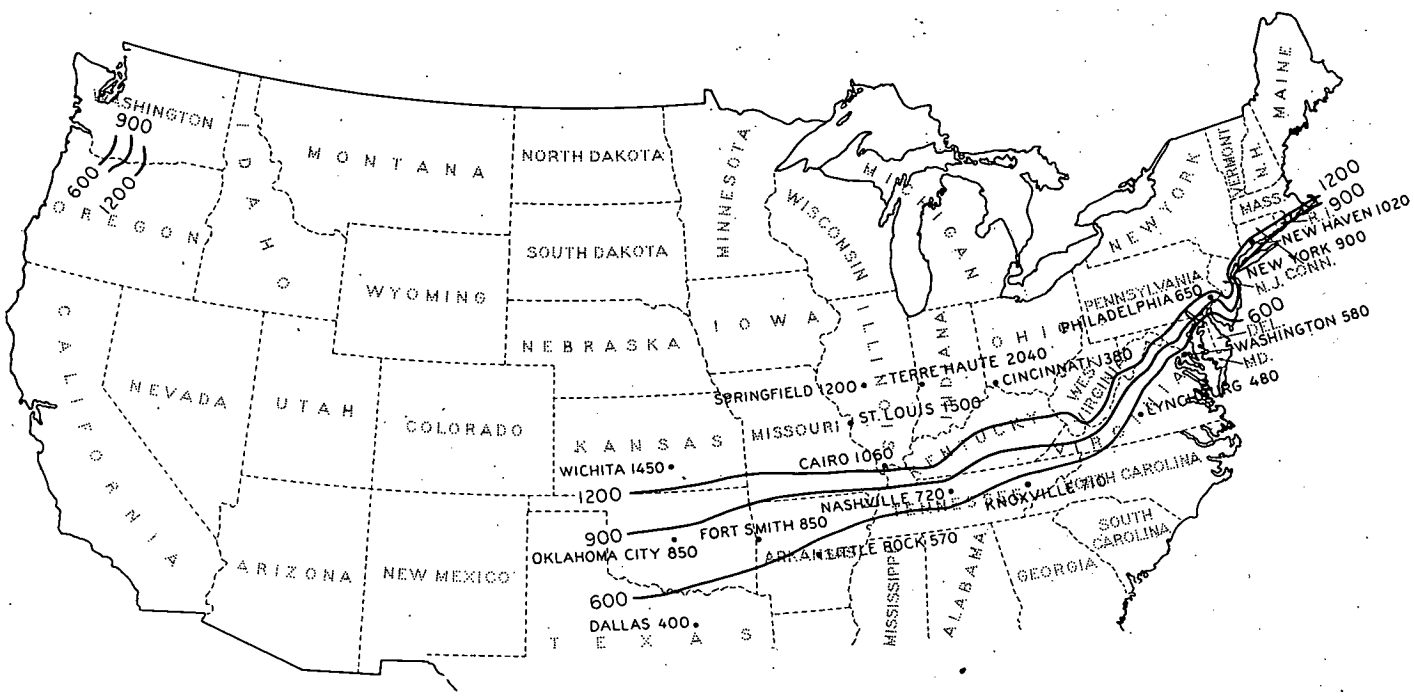


Figure 123. Isothermo-Chronal Map of the United States Showing Lines of Equal Degree-Hours of Cold Period Occurrence (Below 23 Deg. F.) (After Sourwine /1930-5)

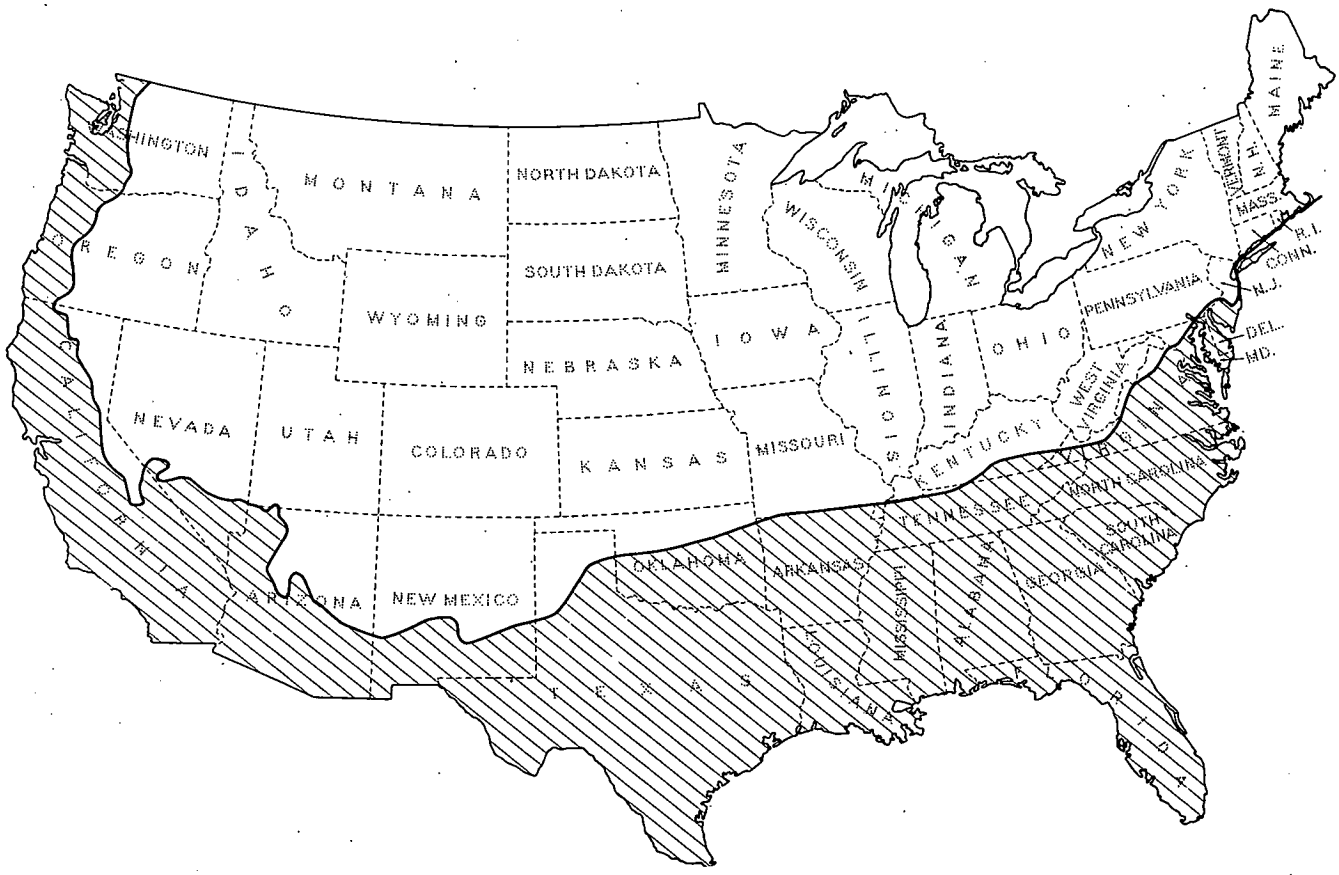


Figure 124. Map Showing Critical Index Line for Highway Ground Freezing, Based on Most Adverse Existing Conditions. (After Sourwine /1930-5)

Croney /1951-36 reported that the experience in the last 20 years indicates that in Great Britain the air temperature must remain at or about the freezing point for some 40 consecutive days before roads suffer serious damage. Temperature records taken over the last 100 years show that on this basis serious damage to roads (as presently constructed) would have occurred during 9 winters, of which five occurred in the 22 years 1874-1895 and three in the period 1929-1947. During those prolonged periods freezing occurred to depths of approximately 18 in. Where the thickness of construction exceeded 18 in., damage was almost negligible.

Influence of Sunshine on Subgrade Soil Temperature - Smirnoff /1923-4, in his studies of the relation between soil temperatures and electric-cable rating, charted the solar radiation and the accompanying soil temperatures under asphalt surfaced pavement in Washington, D. C. However, he showed no comparative data for other types of pavement. His observations were made to a depth of 18 in. below the pavement surface. He stated that the warmest soil temperature exists under black asphalt, which besides absorbing the sun's heat, prevents moisture from penetrating into the soil.

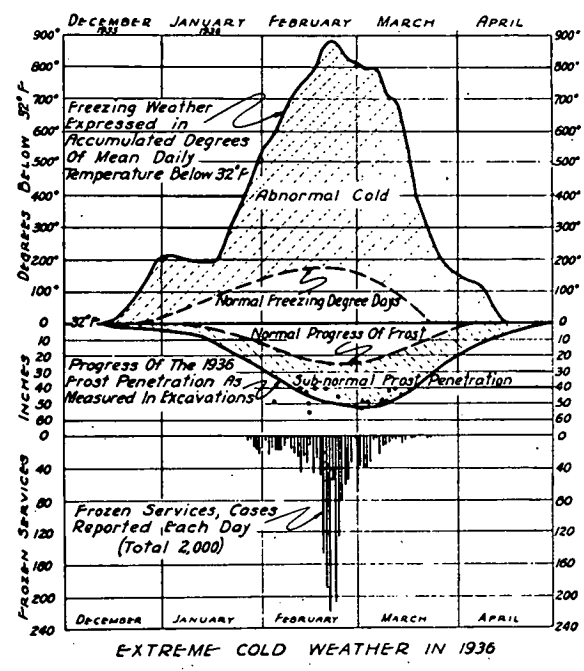


Figure 125. Frost Penetration and Frozen Services, Indianapolis, Indiana (After Mabee /1937-5)

Highland /1926-1, in studies of soil temperatures associated with frozen water pipes at Davenport, Iowa, reported that frost penetration on the south side of an east-west street was greater than on the north side. He gave no data on the relative difference in sunshine available on the two sides.

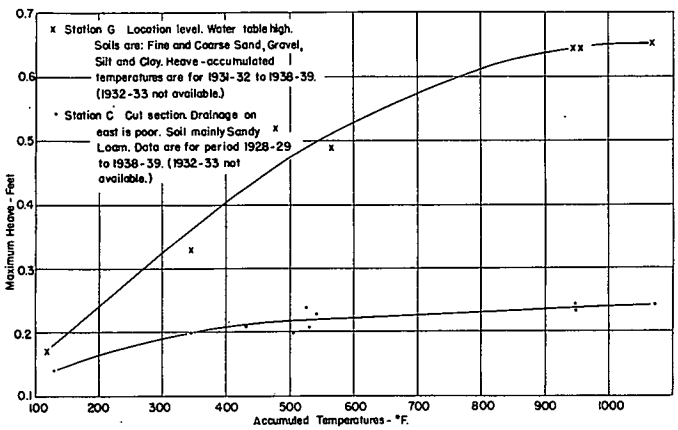


Figure 126
Relation Between Frost Heave and Accumulated Temperature. (After Gardner and Wright)

Influence of Precipitation and Melting Snow on Soil Temperature /16 - Generally speaking, precipitation causes an increase in the soil moisture and, because the rain is often cooler, has a cooling effect on the soil. Increasing the soil moisture increases the conductivity of the soil. However, since water has a specific heat almost four times that of dry soil, increasing the moisture increases the capacity of the soil to absorb heat and thus retard the rate of temperature change. Actually, the greatest rate of movement of freezing temperature is between the two extremes of dry and complete saturation, where the effect of moisture on conductivity is greater than that of the increased specific heat and the increased heat which must be removed in freezing the soil water (approx. 80 cal. per gram).

Some disagreement exists regarding the influence of rainfall and melting snow on soil temperature. Callendar /1895-1 regarded rainfall and percolation as one of the greatest of the factors influencing soil temperature. He observed, in Montreal, that a heavy rainfall on November 3 caused a rapid decrease in temperature in sandy soil. The drop was checked on November 10 by a 4-in. snowfall. A severe frost on November 19 froze the soil to a depth of 4 in. The fall of temperature near the surface was accelerated on November 23 by a rainfall percolating through partly frozen soil. The most remarkable sudden fall of temperature began on December 12 and was caused by a rainfall, which melted about 3 in. of snow and percolated through the partly frozen soil. That type of fall in temperature is more rapid than that due to thermal diffusion.

Rainfall was found to raise the value of thermal diffusivity considerably. On the basis of a 3-year record, Callendar and McCleod /1896-1 found that the diffusivity of the soil during Febraury, when the ground was so frozen that there was practically no percolation, had a value of about 1/3 the annual average. It was concluded that the February value represented the diffusivity due to pure thermal conduction and that more than half the average value is therefore due to the effect of percolation.

Bouyoucos /1913-1, whose views differ from Callendar's, suggested that the importance of rainfall had been over-estimated. He has stated that although rain is commonly considered a warming agent, his records reveal that spring rain lowers the soil temperature, not only by eliminating sunshine but by subsequent evaporation of the rain.

Franklin /1920-2 considered rain a great equalizer of temperature between the surface soil and that at depth, due to percolation. In sand, due to rapid percolation of rain, the subsurface temperature will change very rapidly during a rainfall and afterwards return to normal. These changes take place with decreasing rapidity in loam and in clay.

Keen and Russell /1921-4 felt that rainfall not only cooled the soil in summer due to lower temperature but also prevented warming of the soil due to associated cloudiness. It was their belief that rain reduced the maximum summer temperature but tended to raise the minimum autumn temperature. The effect on the mean temperature was therefore somewhat less than might have been expected.

/16 Taken in part from a review of the literature on Soil Temperatures by Carl B. Crawford /1951-14.

Smith /1929-1 found rainfall to have marked effects on soil temperature. When rainfall was above normal, soil temperatures showed distinct variations. Keen /1931-1 pointed out that rain is usually at a lower temperature than the soil and therefore has a cooling effect. Rain may percolate through the soil, distributing the temperature more evenly. The water movement will, of course, change the diffusivity as well. Atkinson and Bay /1940-3 observed rainfall to have a hastening effect on the time of frost thaw in spring. It was noted that the rainfall was usually accompanied by higher air temperatures.

Although opinions of its importance differ, it appears that rainfall has a definite modifying effect on soil temperatures, promoting both cooling and warming depending on the season and the soil condition. Some observers have felt that a rainfall map could be closely correlated with a frost depth map. Permeable soils, of course, are affected to a greater extent than impremeable ones, due to freer percolation. Percolation seems to have the effect of equalizing the temperature of the soil at various depths. The increase in moisture content, resulting from rainfall, naturally raises the value of thermal diffusivity considerably.

To a certain extent the effect of rainfall can be controlled. Modern highways prevent the percolation of rain, and drainage therefore determines the extent to which rainfall will affect the soil moisture conditions. Further studies may show that control of rainwater is an important determinant of soil temperatures.

Prediction of Depth of Frost Penetration

Several investigators saw possibilities in the use of the relationship between the duration and intensity of cold and the depth of frost penetration in delineating areas subject to frost and predicting depth of frost penetration. Some developed the relationship from experimental data and set up empirical relationships, while others sought to develop theoretical relationships as a basis for calculating depth of freezing from known or assumed data.

Calculation of Theoretical Methods - Berggren /1943-1 developed theoretical relationships, in an idealized system, for computing the depth of frost penetration. His computations take into account density, moisture content, latent heat of fusion, specific heat, and the thermal properties of the soil in frozen and unfrozen state. The Corps of Engineers /1947-2 expanded Berggren's solution in graphical form, reproduced here in Figure 127. The Corps also presented an example for computing the depth of frost penetration. In presenting the Berggren analysis the Corps of Engineers stated:

> "It is realized that the prediction of frost penetration depends on the further study of thermal properties of soils in the frozen and unfrozen states and that present theories for analyzing frost penetration are complicated by the changing water content while the soil freezes. However, it may be shown that the depth of frost penetration varies as the square root of the thermal conductivity of the frozen soil and the square root of reciprocal of the total heat required to freeze the soil."

The recent work of Kersten /1949-13 on determining the thermal properties of soils should make the method more reliable in predicting depths of freezing.

Shannon /1945-1 refers to Berggren's rigorous solution of the problem of computing depth of frost penetration and suggests that Berggren's equation may be written as follows:

$$X = 2 B \sqrt{ta}$$

in which X = depth of frost penetration
t = time that the air temperature T is less than the freezing point of the soil
a = thermal diffusivity = $\frac{\text{thermal conductivity}}{\text{heat capacity}}$ and,
B is a function of (1) the thermal conductivity and heat capacity, (2) temperature conditions, of air and soil; and (3) latent heat of fusion of water in the soil. He did not show examples giving values for the various components of the equation.

Different writers presented some differences in opinion on the relative accuracy and usefulness of computed values for depth of soil freezing. Keil /1938-3 concluded from his studies

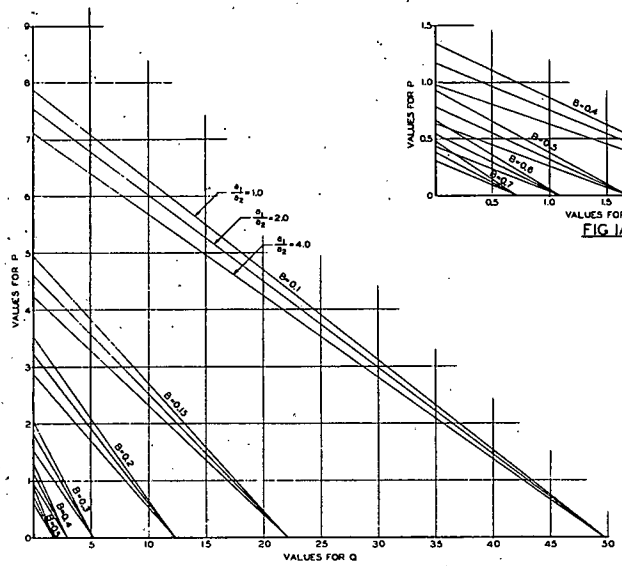


FIG. 1

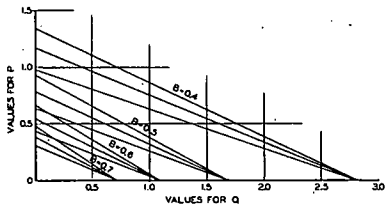


FIG. 1A

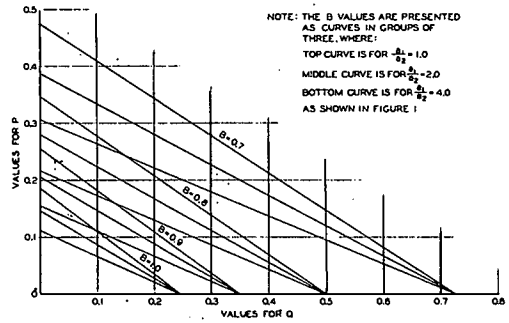


FIG. 1B

NOTE: THE B VALUES ARE PRESENTED AS CURVES IN GROUPS OF THREE, WHERE:

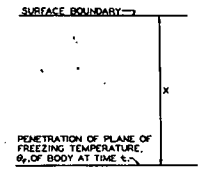
TOP CURVE IS FOR $\frac{\theta_1}{\theta_2}=1.0$

MIDDLE CURVE IS FOR $\frac{\theta_1}{\theta_2}=2.0$

BOTTOM CURVE IS FOR $\frac{\theta_1}{\theta_2}=4.0$

AS SHOWN IN FIGURE 1

B FOR VARIOUS VALUES OF Q, P AND $\frac{\theta_1}{\theta_2}$



TEMPERATURE AT SURFACE BOUNDARY EQUALS θ_0 = 32°F UNTIL TIME t_0 WHEN SURFACE TEMPERATURE θ_s < 32°F SUDDENLY APPLIED

ISOTROPIC, HOMOGENEOUS BODY OF INFINITE EXTENT WITH SURFACE BOUNDARY PLANE AT TEMPERATURE θ_0 AT TIME t_0 AND OF FOLLOWING PROPERTIES:

γ = UNIT DRY WEIGHT, LBS/CU.FT.
w = WATER CONTENT PERCENT DRY WEIGHT
C_1 = VOLUMETRIC HEAT CAPACITY (FROZEN) = $\gamma(c+\frac{0.5w}{100})$ = BTU/(CU.FT)(DEG.F.)
C_2 = VOLUMETRIC HEAT CAPACITY (NON-FROZEN) = $\gamma(c+\frac{w}{100})$ = BTU/(CU.FT.)(DEG.F.)
WHERE c IS THE SPECIFIC HEAT OF DRY SOIL (ASSUMED TO BE 0.2 BTU/(LB)(DEG.F.))
k_1 = THERMAL CONDUCTIVITY (FROZEN) BTU/(FT)(HR)(DEG.F.)
k_2 = THERMAL CONDUCTIVITY (NON-FROZEN) BTU/(FT.)(HR)(DEG.F.)
L = LATENT HEAT OF FUSION BTU/CU.FT
a = DIFFUSIVITY FT^2/HR WHERE:

$$a_1=\frac{k_1}{C_1} \quad \text{AND} \quad a_2=\frac{k_2}{C_2}$$

IT IS ASSUMED THAT THE WATER CONTENT OF THE BODY AT EVERY POINT IS CONSTANT DURING THE TEMPERATURE CHANGES.

FIG. 2-CONDITIONS

$X = 2B\sqrt{a_1 t}$ WHERE:
X = PENETRATION IN FEET OF PLANE OF EQUAL TEMPERATURE BELOW SURFACE BOUNDARY
t = THE TIME IN HOURS THAT TEMPERATURE θ_s HAS BEEN APPLIED TO THE SURFACE.
B = A CONSTANT FOR A PARTICULAR SET OF CONDITIONS AND IS DEFINED BY THE EQUATION.

$$\frac{e^{-B^2}}{G(B)} - \left(\sqrt{\frac{k_2 C_2}{k_1 C_1}}\left(\frac{\theta_0-\theta_f}{\theta_f-\theta_s}\right)\right)\frac{e^{-B^2\frac{a_1}{a_2}}}{1+G\left(B\sqrt{\frac{a_1}{a_2}}\right)} - \left(\frac{L}{C_1(\theta_f-\theta_s)}\right)B\sqrt{\pi}$$

IN WHICH:
e = BASE OF NATURAL LOGARITHMS.
G = PROBABILITY INTEGRAL, KNOWN AS GAUSS "ERROR-FUNCTION"

$$\sqrt{\frac{k_2 C_2}{k_1 C_1}}\left(\frac{\theta_0-\theta_f}{\theta_f-\theta_s}\right)=P$$

AND

$$\frac{L}{C_1(\theta_f-\theta_s)}=Q$$

VALUES FOR B FOR VARIOUS VALUES OF P AND Q HAVE BEEN COMPUTED AND ARE PLOTTED IN FIGURES 1, 1A, AND 1B

FIGURES 3, 4 AND 4A GIVE VALUES FOR THE LATENT HEAT OF FUSION AND THE VOLUMETRIC HEAT CAPACITY OF SOILS FOR VARIOUS UNIT DRY WEIGHTS AND WATER CONTENTS.

FIG. 2A- EQUATION

EQUATION FOR PENETRATION "X" OF PLANE OF EQUAL TEMPERATURE

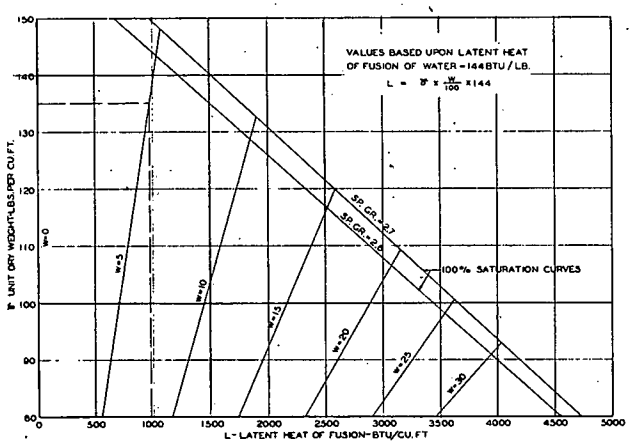


FIG. 3

LATENT HEAT OF FUSION VS UNIT DRY WEIGHT

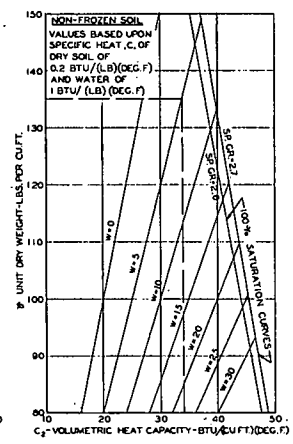


FIG. 4

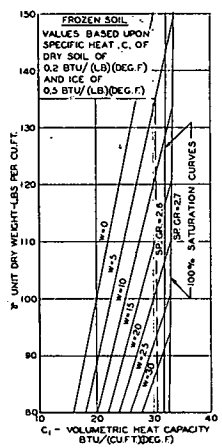


FIG. 4A

HEAT CAPACITY VS UNIT DRY WEIGHT

IT IS ASSUMED THAT THE SOIL UNDER CONSIDERATION IS ISOTROPIC AND HOMOGENEOUS AND HAS THE FOLLOWING PROPERTIES AT THE CONDITIONS NOTED:

γ = 135 LBS/CU.FT.
w = 5 PERCENT
θ_0 = 40 DEGREES F.
θ_f = 32 DEGREES F.
k_1 = 2.3 BTU/(FT)(HR)(DEG.F.)
k_2 = 1.3 BTU/(FT.)(HR)(DEG.F.)

FROM FIGURES 4 AND 4A VALUES ARE OBTAINED FOR:
C_1 = 30.5 BTU/(CU.FT) (DEG.F.) AND
C_2 = 34 BTU/(CU.FT.)(DEG.F.)

THUS: $\frac{a_1}{a_2}=\frac{k_1 C_2}{k_2 C_1}=\frac{(2.3)(34)}{(1.3)(30.5)}=1.97$

L IS OBTAINED FROM FIGURE 3: L = 970 BTU/CU.FT.

IT IS THEN FURTHER ASSUMED THAT THE SURFACE TEMPERATURE θ_0 IS SUDDENLY REDUCED TO θ_s, 19 DEGREES FAHRENHEIT, AND MAINTAINED FOR 100 DAYS. THUS:

θ_s = 19 DEGREES F.
t = 100 DAYS × 24 HRS/DAY = 2400 HRS

$$Q=\frac{L}{C_1(\theta_f-\theta_s)}=\frac{970}{(30.5)(32-19)}=2.44$$

$$P=\sqrt{\frac{k_2 C_2}{k_1 C_1}}\left(\frac{\theta_0-\theta_f}{\theta_f-\theta_s}\right)=\sqrt{\frac{(1.3)(34)}{(2.3)(30.5)}}\left(\frac{40-32}{32-19}\right)$$

$$=\sqrt{0.63}\left(\frac{8}{13}\right)=0.488$$

WITH P = 0.488 AND Q = 2.44 B IS OBTAINED BY INTERPOLATION IN FIGURE 1 USING THE MIDDLE CURVE OF THE GROUPS OF THREE SINCE $\frac{\theta_1}{\theta_2}$ EQUALS APPROXIMATELY 2.0.

B = 0.37

FROM EQUATION: $a_1=\frac{k_1}{C_1}$, $a_1=\frac{2.3}{30.5}=0.0754$

SUBSTITUTING IN EQUATION:

$X = 2B\sqrt{a_1 t}$
$X = 2(0.37)\sqrt{(0.0754)(2400)} = (0.74)(13.46)$
$X = 9.98$ FT.

EXAMPLE FOR USE OF EQUATION

* REFERENCE:
BERGGREN, W.P. "PREDICTION OF TEMPERATURE DISTRIBUTION IN FROZEN SOILS" TRANSACTIONS AMERICAN GEOPHYSICAL UNION, 1943.

Figure 127
Prediction of Frost Penetration. (After Corps of Engineers)

of frost in German highways that "with a given soil it is impossible to arrive in advance at a mathematical determination of the expected damage from frost which will enable suitable preventive measures to be taken, especially as each successive frost alters the conditions on which such a calculation would be based." Beskow /1947-12 held the opinion that frost depth depends not on cold quantity alone and that exact mathematical determination of frost depth is not possible. However, he saw the practical usefulness of computed values as means for comparing different materials whose properties are known. He believed Stefan's formula furnished the simplest method of computing frost depth as a function of time. That formula is based on the growing of an ice layer on a surface of still water, assuming a constant freezing temperature. The formula follows:

$$S = \sqrt{\frac{2\ t\ C_1\ T}{d\ q}}$$

Where:

- S = thickness of ice after time T
- t = air temperature in deg. C.
- C_1 = coefficient of thermal conductivity of ice
- d = density
- q = specific heat of ice

The formula is recognized as an approximation, but it can be adapted for the freezing rate of wet soil by using appropriate constants. It is necessary to take into account the sum of the latent heat of fusion and heat capacity of both soil solids and soil moisture, which Beskow denotes "frost storing capacity." That is,

$$Q = 80\ F + \frac{t}{2}\ (0.45\ f + 0.55\ s)\ \text{cal./cm}^3$$

Where:

- t = surface temperature
- f = water content per unit volume
- s = mineral matter per unit volume
- 80 cal./gram = latent heat of fusion of ice
- 0.45 cal./cm^3 = volumetric heat capacity of ice
- 0.55 cal./cm^3 = volumetric heat capacity of soil solids.

He suggests that for most cases (mineral soils, except fat clays) the following is a good approximation.

$$Q = 80\ f + 0.4\ \frac{t}{2}\ \text{cal./cm}^3$$

Thus the formula becomes

$$S = \sqrt{\frac{2\ t\ C_1\ T}{Q}}$$

Where Q is the "frost-storing capacity" given above.

The above formulas neglect heat-exchange conditions between surface and air, radiation into space, heat conduction from below, the effect of snow covers, and the fact that not all water in fine grained soils freeze. Of this group of factors, influencing depth of freezing, Beskow considered that the effects of the first two are rather small and can be neglected. He did find that heat conduction from below and the effect of snow cover are important. He accounted for heat conduction from below by substituting for the temperature t, a quantity $t - h_f G$ where h_f = depth of frozen layer, G = temperature gradient below the frozen layer (theoretically G should be multiplied by $\frac{C_u}{C_f}$ where C_u = coefficient of thermal conductivity for unfrozen and C_f for frozen soil but the correction is so small it may be neglected considering other approximations.) Beskow then suggests that for practical purposes, frost depth is not necessarily proportional to the frost quantity tT, because "moderately cold temperatures over a long time period do not cause the same frost depth as a severe cold spell over half the time, even though the products tT are the same." From this he concludes that "any exact mathematical treatment is not possible."

Beskow /1947-12 also reported that in Norway use is made of calculations of frost depth for design purposes by determining the relative "freezing resistance" of different layers in terms of the "cold quantity" of each. For the top layer, this freezing resistance FR is given from the formula for frost depth:

$$FR_1 = tT = \frac{Q_1 h_1^2}{2 C_1} \text{ deg.-hr. C.}$$

for the next layer

$$FR_2 = \frac{Q_1 h_2^2}{2 C_2} + Q_2 h_2 \frac{h_1}{C_1}$$

Where FR = freezing resistance of the layer
h = thickness of the layer
Q = frost storing capacity
C = coefficient of thermal conductivity

For any deeper layer the formula would become:

$$FR_n = \frac{Q_n h_n^2}{2 C_n} + Q_n h_n \sum \frac{h_o}{C_o}$$

Where $\sum \frac{h_o}{C_o}$ in the sum of $\frac{h}{C}$ for all overlying layers.

The Corps of Engineers /1949-23 continued its work on the analysis of frost penetration by mathematical methods. That work is so extensive that it cannot be reviewed here in a manner which will be of value to technologists who wish to make a detailed study of that phase of the frost problem. For those who wish to study the work in detail the reviewer wishes to call attention to Appendix A of the report /1949-23.

Thermal conductivity tests were made for the Corps of Engineers by Kersten indicated that values of thermal conductivity for cohesionless materials, as found in base courses, range from 1.0 to 1.8 BTU per ft. per hr. per deg. F. That range does not include organic, volcanic, or cohesive soils, which may differ in thermal properties. A value of 1.3 was used in computations of the depth of frost penetration.

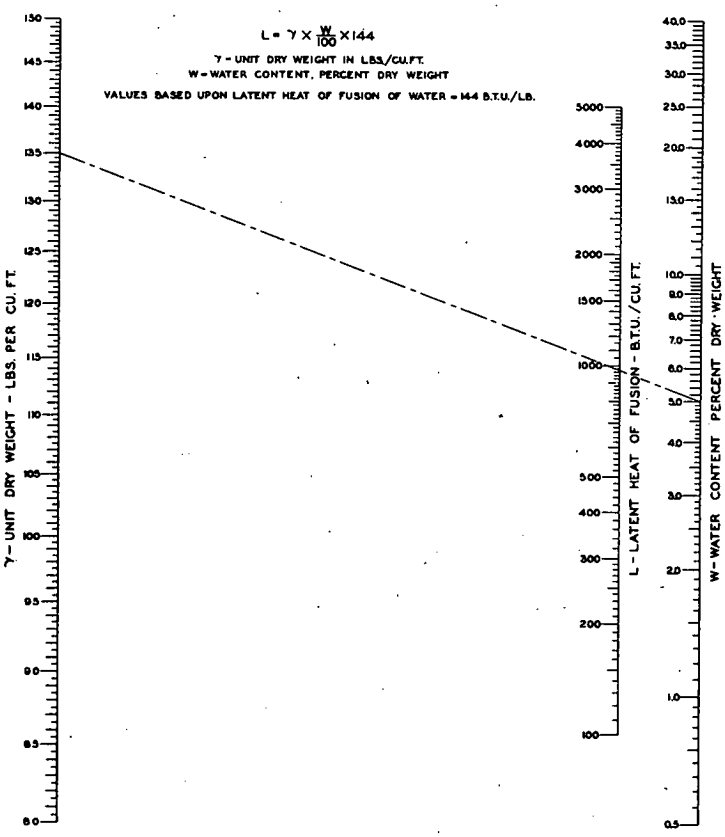


Figure 128
Latent Heat Determination
(After Corps of Engineers)

The latent heat of fusion of soil moisture is proportional to the percentage of soil water which freezes. For practical purposes, all soil water in clean, cohesionless soils of the gravel and sand textures will freeze at approximately 32 F. Figure 128 shows the relationship between density in p.c.f. and latent heat of fusion in BTU per cu. ft. for various moisture content bound on the assumption that all water freezes. Where several soil layers occur, each having different moisture contents, the average latent heat may be obtained from the equation:

$$L = \frac{L_1 d_1 + L_2 d_2 + L_3 d_3 + \ldots + L_n d_n}{d_1 + d_2 + d_3 + \ldots + d_n}$$

Where:

L is average latent heat of soil moisture in BTU per cubic foot.

L_1, L_2, L_3, are latent heats of soil moisture in BTU per cubic foot in layers 1, 2, 3, etc.

d_1, d_2, d_3, are the thicknesses of layers 1, 2, 3, in feet etc.
(note that $d_1 + d_2 + d_3 + \ldots + d_n$ equals the depth of freezing.)

An average value of 0.2 BTU/(lb.)(deg. F.) for specific heat of soil, and values of 1.0 and 0.5 for water and ice have been used in the calculations for predicting depth of frost penetration.

The relationships between density and volumetric heat in BTU/(cu. ft.)/(deg. F.) for various moisture contents (frozen and unfrozen) are given in Figure 129. The volumetric heat of saturated nonfrozen soil ranges from 40 to 45 BTU/(cu. ft.)(deg. F.) within reasonable limits of unit dry weight; and for frozen soil is approximately 32 BTU/(cu. ft.)(deg. F.). The following equation is used to determine an average value of volumetric heat for several layers having different densities and moisture contents.

$$C = \frac{C_1 d_1 + C_2 d_2 + C_3 d_3 + \ldots + C_n d_n}{d_1 + d_2 + d_3 + \ldots + d_n}$$

Where: C is the average volumetric heat in BTU/(cu. ft.)(deg. F.).

C_1, C_2, C_3, etc. are volumetric heats in frozen or unfrozen states for layers 1, 2, 3, in BTU/(cu. ft.)(deg. F.).

d_1, d_2, d_3 etc. are thickness of layer 1, 2, 3, in feet, etc.

Four equations (83, 93, 154, and 158) are given below. These were used to compute depth of frost penetration for comparison with observed depths of penetration. Comparison of the relative merits of each forumla is given below:

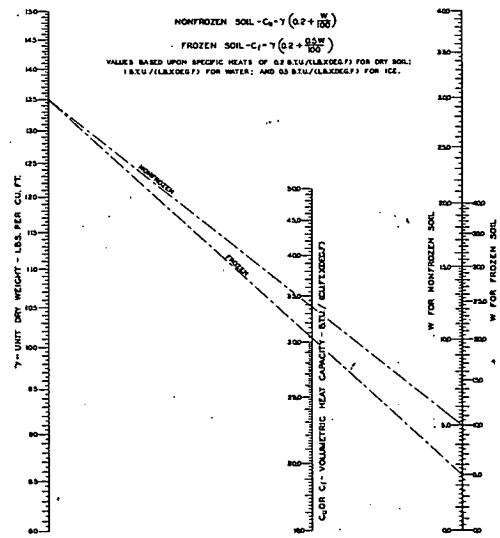


Figure 129
Volumetric Heat Capacity Determination
(After Corps of Engineers)

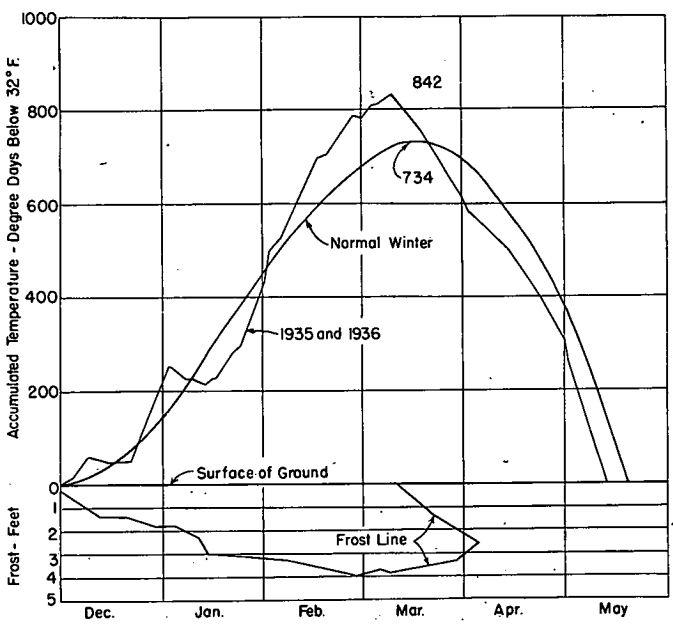


Figure 130. Relation Between Accumulated Temperature Deficiency and Frost Penetration. Winter of 1935 and 1936, Portland, Maine. (After Fuller 1940-9)

Equation	Ratios of Predicted to Observed Depths Avg.	Max.	Min.	Observations Within 6 In. Percent
83	1.60	3.39	1.09	6
93	1.32	2.69	.83	29
154	.94	1.89	.58	42
154 + pave.	1.10	2.31	.67	54
158 (Turf)	1.95	5.67	.92	50

EQUATIONS:

83 $$x = \sqrt{\frac{48kF}{L}}$$

93 $$x = \sqrt{\frac{48kF}{L + C\left(V_0 - 32 + \frac{F}{2t}\right)}}$$

154 $$x = \sqrt{\frac{24kF}{L + C\left(V_0 - 32 + \frac{F}{2t}\right)}}$$

158 $$x = -\frac{d}{2} + \sqrt{\left(\frac{d}{2}\right)^2 + \frac{24kF}{L + C\left(V_0 - 32 + \frac{F}{2t}\right)}}$$

x = DEPTH OF FROST PENETRATION IN FEET.
k = THERMAL CONDUCTIVITY IN B.T.U./(FT.)(°F)(HR.)
F = FREEZING INDEX IN DEGREE-DAYS
L = AVERAGE LATENT HEAT IN B.T.U./FT.³
C = AVERAGE VOLUMETRIC HEAT IN B.T.U./(FT.³)(°F)
V_0 = MEAN ANNUAL AIR TEMPERATURE IN °F
t = DURATION OF FREEZING INDEX IN DAYS
d = THICKNESS OF INSULATION LAYER IN FEET

AN AVERAGE VALUE FOR THERMAL CONDUCTIVITY (k) = 1.5 B.T.U./(FT.)(°F)(HR.), IS USED THROUGHOUT THESE CALCULATIONS.

VALUE FOR "d" USED IN EQUATION 158 IS THICKNESS OF TOPSOIL IN FEET.

Equations 83 and 93 gave values which were consistently too high. The results from equation 154 bracketed observed values. Equation 158 for areas of turf cover bracket observed depths but the dispersion of values is great.

Several other investigators have developed theoretical methods for computing depth of freezing for known or assumed conditions. The work of Ruckli /1943-2 has been translated and is available to those who wish to go deeper into the mathematical approach to determining frost depth.

Correlation of Frost Depth with Temperature Data - The work of Sourwine /1930-5 and Casagrande /1931-14 developed the concept of relating duration and intensity of freezing temperatures with depth of soil freezing. Since these early studies, more effort has been given to developing the relationship by expressing cold in terms of cumulative degree-days below freezing. Later investigators have named that relationship the freezing index.

The work of Gardner and Wright /1939-5 has been mentioned. In addition to the work of highway engineers, waterworks engineers have contributed to data on freezing index. Examples of their work are those of Mabee /1937-5 (cited previously) and Fuller /1940-9. Fuller subtracted the mean temperature of the day from 32 F. and added the deficiency for each day to that of the preceding day, obtaining a cumulative temperature-deficiency curve for a winter as shown in Figure 130. From long-time records he prepared the relationship between depth of frost penetration and accumulated deficiency in temperature. (Figure 131).

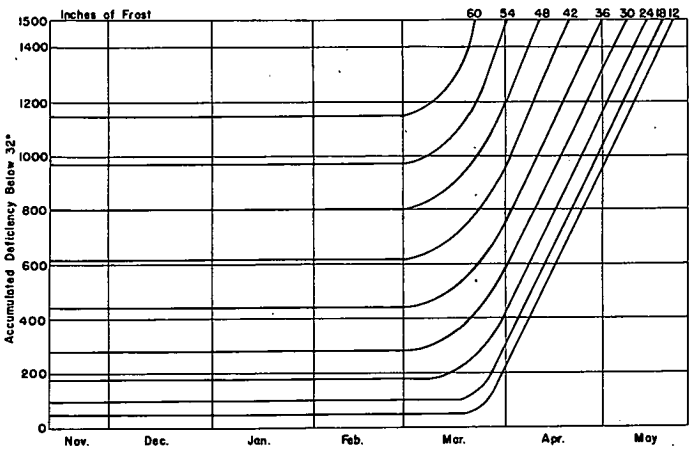


Figure 131. Temperature Accumulated Deficiency Curves and Frost Penetration (After Fuller 1940-9).

Belcher /1940-14 concluded from his studies of temperature-duration intensity vs. depth-of-freezing data obtained from the Manitoba Agricultural College and the Canadian Meterological Service, that "the depth of penetration of a given temperature is not a pure function of the algebraic summation of temperatures above and below 32 F." He presented data to show that at depths of 40 in., or less, the number of degree-days below freezing required to produce a temperature of 32 F. varies considerably. He added that "a study of the rate of change of temperature and the temperature-penetration curve indicates that there may be some correlation in that respect with more rapid accumulation of temperatures below freezing, producing a greater depth of penetration per time-temperature unit. The average number of degree-days per in. of penetration required at depths of 10, 20, 40 and 65 in. is, in this case, 48. Undoubtedly variations in moisture and cover conditions from year to year account for the extreme variation in the amount of cold required to lower the soil temperature at a given depth. Therefore, it would seem unwise to base important prediction upon air temperature alone."

Lang /1941-7 and Swanberg /1945-7 made observations of air temperatures and depth of freezing temperatures under a 16½-ft. x 17-ft. x 7-in. concrete slab in Minneapolis. The slab was placed on a loamy fine-sand subgrade, had good surface drainage, and was kept free of snow and ice. The penetration of the frost into the subgrade is indicated in Figure 132 for each of 5 years during the period 1940-1945 and for the average 5-year period. Above the graph of penetration are given cumulative temperature-deficiency data calculated by summing up the number of degrees below 32 F. for each day beginning November 1.

Figure 132 shows a relationship between depth-of-frost penetration and freezing index. However, since readings were taken only to a depth of 5 ft. and frost penetrated in excess of 5 ft., it is not possible to determine a close relationship. It may be noted, however, that the cumulative degree-days at the time the frost reaches a depth of 5 ft. differs markedly from year to year.

Shannon /1945-1 constructed his temperature-deficiency curve by beginning with the first fall freezing temperatures. Differences (between mean daily temperature and 32 F.) for days having mean temperatures above 32 F. he assigned negative values. He termed the difference between the largest negative value and the largest positive value the freezing index. He indicated his belief that it was possible to correlate, with reasonable accuracy, the depth of frost penetration and the freezing index. He suggested further that an approximate value for freezing index may be computed from the monthly average temperatures using the formula:

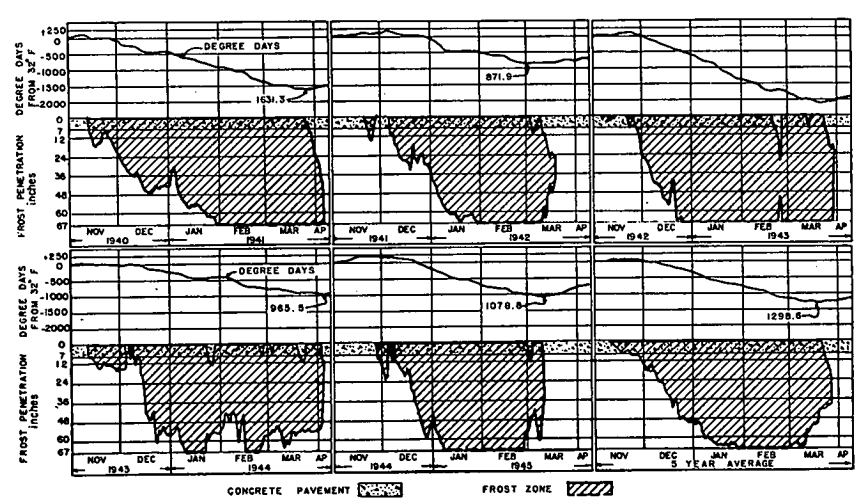
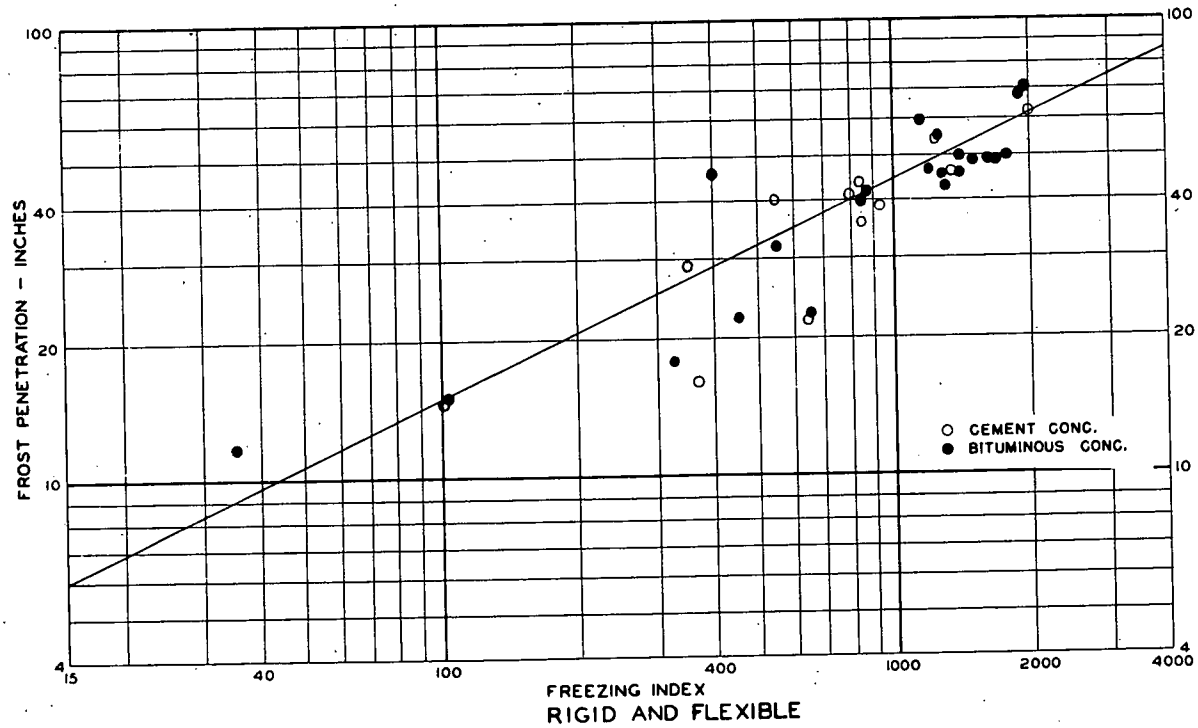


Figure 132. Frost Penetration as Related to Degree Days. (After Swanberg 1945-7)

$$\text{Freezing Index} = (32\,y - X)\,30.2$$

in which y equals the number of months during which the average temperature is less than 32 F., X is the sum of the average temperatures for these months, and 30.2 is the average number of days for the months of December, January, February, and March. The value computed using this formula will always be less than the true index but will approach it with increasing magnitude of the freezing index. This approximate method should not be used at locations where the mean value for the freezing index is less than about 300.

The Corps of Engineers' /1947-2 extensive studies of subgrade freezing on airfields showed a reasonable correlation between freezing index and depth of freezing. They plotted all observations of frost penetration beneath paved areas versus freezing index. The trend of the observations is a straight line when plotted on a logarithmic scale as shown in Figure 133. Figure 133 shows data for both rigid and flexible pavements. The same relationship is presented for design purposes in the Corps of Engineers Manual /1946-5. The Corps states, "This curve may be used to predict the depth of frost penetration beneath rigid and flexible pavements which are maintained snow free and have bases constructed of non-insulating materials such as sand, gravel, or crushed rock." The Corps of Engineers also prepared maps showing isolines of mean annual air temperature and duration of normal freezing index as shown in Figure 134.



Notes:

Freezing index obtained from degree-day diagram on date frost penetration was measured.

For this study the freezing index is not necessarily the maximum value of negative and positive values on the degree-day diagram.

Straight line equals the design curve showing combined thickness of pavement and base required to prevent freezing of subgrade recommended in revisions to engineering manual.

Combined thickness of pavement and base is indicated by numbers adjacent to plotted values.

Figure 133. Relation between Frost Penetration and Freezing Index for Rigid and Flexible Type Pavements (After Corps of Engineers 1947-2)

Sources of Climatological Data

Some well-known sources of climatic data prepared in summary form are:

1. Precipitation and Humidity - Kincer, J. B., Atlas of American Agriculture, Part 2, Climate, Section A, Precipitation and Humidity, Advance sheets, No. 5, March 15, 1922.

2. Temperature, Sunshine and Wind - Kincer, J. B., Atlas of American Agriculture, Part 2, Climate, Section B, Temperature, Sunshine and Wind, Advance sheets, No. 7, November 1928.

3. U. S. Weather Bureau - Climatic Summary of the United States. Bulletin W, Sections 1 to 105, 1930.

4. U. S. Weather Bureau - Monthly weather summaries by states.

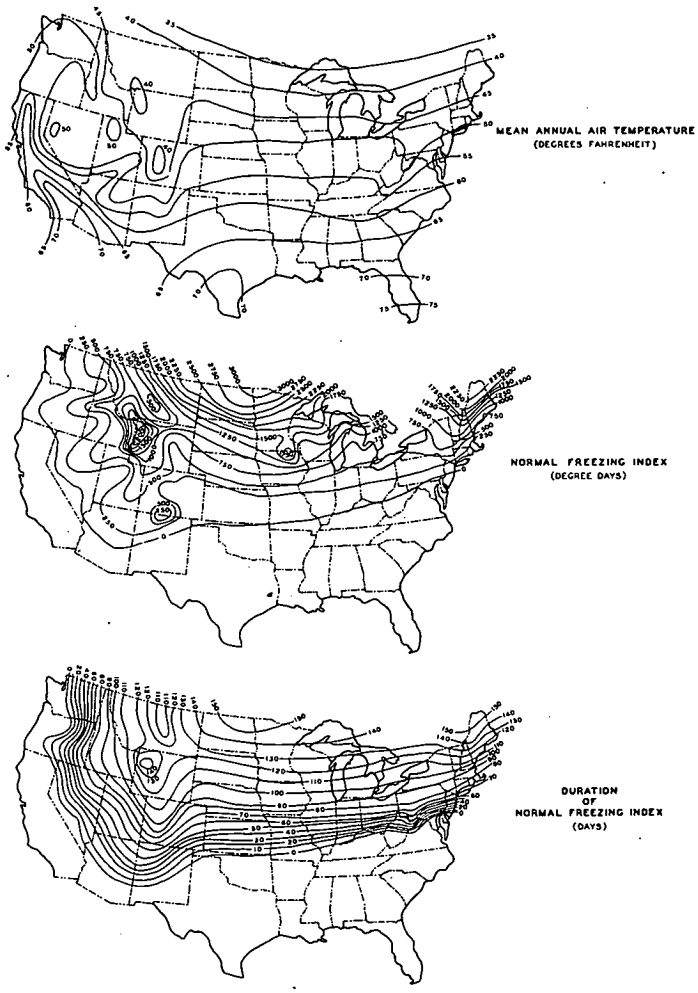


Figure 134. Isograms for Prediction of Frost Penetration (After Corps of Engineers 1947-2)

Item 1 is made up largely of charted maps of the U. S. giving data on precipitation and humidity on basis of monthly seasonal and annual values. Records for the uniform 20 year period 1895 to 1914 form the basis for the principal charts and diagrams.

Item 2 presents charts and charted maps of temperature, sunshine, and wind on the basis of monthly, seasonal, and annual values. Records cover a 20 year period 1895-1914.

Item 3 contains complete climatic data from the establishment of the various stations up to 1930 inclusive. The sections refer to sections in states, e.g., South Dakota is divided into two sections, the eastern half (east of the 100 meridan) being section 37, the western half, section 36. This bulletin is available at local offices of the U. S. Weather Bureau.

Item 4 gives the more recent climatic data from the monthly summaries by states. It also is available at local offices of the Weather Bureau.

5. Eno, F. H. - The Influence of Climate on Building, Maintenance and Use of Roads in the United States. Proceedings, Highway Research Board, Vol. 9, pp. 211-223, 1929.

Item 5 shows copies of typical maps presented in items 1 and 2.

6. The U. S. Department of Agriculture, Yearbook of Agriculture, Climate and Man, /1941-3 contains the following information on climate:

Kincer, J. B. Climate and Weather Data for the U. S. General Background on Collection of Climate and Weather Data in the U. S. and Territories. Forty-six maps showing distribution of temperatures, precipitation, snowfall, snow cover, humidity, sunshine, frosts, etc., pp. 701-748.

Climates of the States. Tabulated data and seven maps for each state showing variation in climate within each state. pp. 749-1228.

Reed, W. W. The Climates of the World. pp. 665-684, mean and extreme temperatures and monthly and yearly precipitation for 387 stations throughout the world. Discussion of principal features of climates of Eastern and Western Hemispheres. Three maps.

Influence of Location, Exposure and Topographic Position

Latitude - The influence of latitude on climate and the resulting soil temperatures and depth of soil freezing is generally known. Some maps indicate depth of frost penetration as related to latitude.

Altitude - Average temperatures generally decrease with increase in altitude. According to Keen /1931-1 the rate of decrease is one degree C. per 550 ft. increase in elevation. Below 3000 ft. elevation the decrease is slightly greater in summer than in winter. Some authors indicate that local conditions alter the general rule, e.g., Moore /1910-1 explains that cold air has a tendency to settle down the slopes to bottom lands.

Distribution of Land and Water - Masses of water give more equable air temperatures due to the high specific heat of water plus the increased water vapor in the air in the vicinity of large lakes and seas. The modifying effect of water may be seen readily by observing the modified temperatures in the vicinity of the Great Lakes.

Effect of Slope of Ground on Soil Temperatures - Keen /1931-1 explained that radiation per unit area is directly proportional to the cosine of the angle between a line perpendicular to the surface and the direction of radiation. The larger the angle the less the radiation. He shows the following temperatures on 1:12 north and south slopes.

Depth in.	North Slope deg. C.	South Slope deg. C.
1½	4.0	12.4
3	3.2	8.6

Shreve, F. "Influence of Slope Exposure on Soil Temperature," Carnegie Trust Year Book (Washington) 1924, 23, pp. 140-141, reported temperatures on south slope (alluvial clay, slope 30 deg. at Desert Laboratory, Tucson, Arizona) to be 5 to 7 C. warmer than north slopes. This agrees closely with Keen's observations. Atkinson and Bay /1940-3 indicated there may be greater depth of frost penetration on north exposures, but their studies at the LaCrosse Station showed the difference to be insignificant.

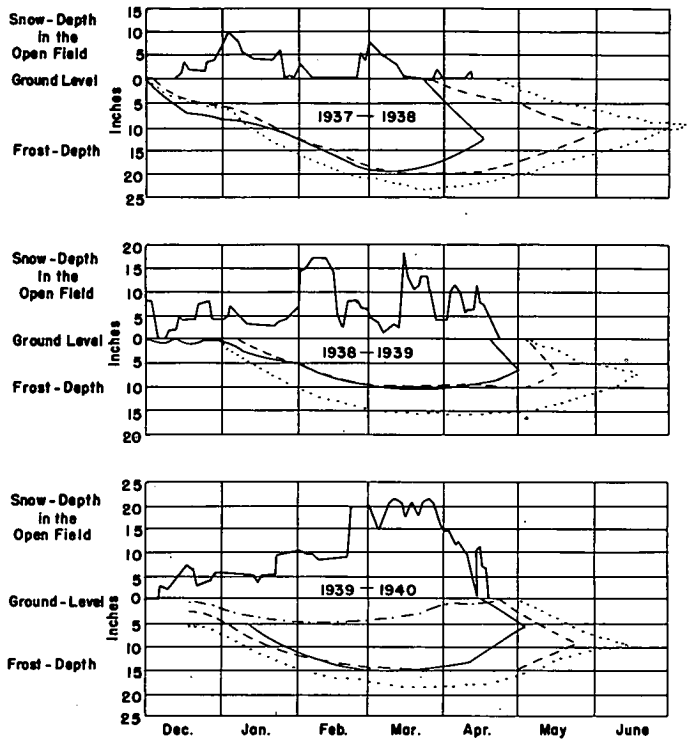
Influence of Soil Cover on Soil Temperature and Depth of Freezing

Effect of Soil Mulch (Depth of Cultivation) - West /1932-1 (Australia) found that cultivation to a depth of 10 cm. (about 4 in.) to form a mulch influenced soil temperatures both above and below the depth of cultivation. The uncultivated soil warmed more quickly than did the cultivated soil. In autumn the uncultivated soil cooled more rapidly. During winter there was a tendency for the temperature in the cultivated soils to fall below those which were not cultivated. The differences in daily temperatures at a depth of 15 cm. ranged from a maximum of about 7 C. to a minimum of about 1 C. between the two conditions, being greatest in the afternoon when temperatures were highest.

Effect of Vegetative Cover - Baver /1940-2 cites the work of Woolney in Germany who found a considerable difference in soil temperature at a depth of 10 cm. under grass cover compared to bare earth, the temperature of the bare soil being up to 8 C. higher in midafternoon. Keen /1931-1 brought out that during winter vegetative cover reduced the rate of heat loss. Atkinson and Bay /1940-3 studied the effects of vegetative cover on the depth of frost penetration. During 1939-1940 when the snow depths were practically the same for an ungrazed woodlot and an open bluegrass-pasture area, they found "the frost depth in the open pasture area was approximately twice as deep as in the protected woodlot area." The woodlot contained 1½ in. of forest litter. Frost depths in plowed areas averaged 25 in. or about five times that in the protected woodlot. Between the two extremes of plowed and protected woodlot areas are grazed woodlots, open pastures, hay, grain, and corn stubbles depending on the density and height of vegetation. The results are shown in Figure 135.

Belotelkin /1941-4 observed frost penetration (during period 1937-40) in a spruce flat, a spruce swamp, in hardwood forest, and in an open field at the Gale River experimental forest in Northern New Hampshire. He found that (1) forest cover delays ground freezing prior to snowfall; (2) frost penetration was least in the hardwood plot and greatest in the spruce swamp; and, (3) thawing was completed earliest in the hardwood plot.

It may be that the soil type and condition influenced frost penetration, hence it is worthy of note that he found that frost penetrates more deeply and stays longer in soils with good drainage and that fine-textured soils resemble poorly-drained soils and coarse-textured soils resemble well-drained soils in their influence on soil freezing.



TEMPERATURE						
Month	1937-38		1938-39		1939-40	
	Mean	Av. Min.	Mean	Av. Min.	Mean	Av. Min.
November	32	25	33	23	24	16
December	18	9	21	13	20	11
January	15	4	14	4	10	2
February	20	13	20	8	14	5
March	27	17	22	10	22	13
April	42	31	33	24	34	26

LEGEND
Open Field
Spruce Flat
Spruce Swamp
Hardwoods

Figure 135. Depth of ground-frost in different cover-types (Snow-cover and temperature as recorded at the weather station in the open field). (After Atkinson and Bay 1940-3)

licate plots with 6, 12, and 24 in. of snow and kept two plots free of snow. Snow densities (expressed as inches of water per inch of snow) were 0.256, 0.215, and 0.247 in. for the 6, 12, and 24 in. covers. The authors observed:

"Measurements during the first 10 days indicate the presence of frost under the three different snow-depths and the bare plots. The frost disappeared from the 12-in. and 24-in. plots, but continued to deepen on the six-inch and bare plots. There were a few instances in later measurements indicating the presence of frost under the 12-in. snow cover. The snow settled to some extent on the plots, and warm weather during the first week of March caused some thawing of the snow. It was necessary to add more snow to build the plots up to the original levels. The bare plots were swept clean of snow as

Beskow's /1947-12 experience in northern Sweden showed that the effect of vegetation is generally to lower the temperature. That is true for moss and turf, whose effect is large, because during summer they are dry and poor conductors while in winter they are wet and good conductors. Beskow found that on mossy ground, frost could often be found at shallow depths.

The investigations of the Corps of Engineers /1947-2 showed clearly the marked difference in depth of freezing and in temperature distribution at various periods during the winter in turfed areas and paved areas. That has been illustrated earlier under "Seasonal Temperature Movements." At Bangor, Maine, the maximum depth of freezing temperature was about 2 ft. under turf while it was 3.5 to 4 ft. under cement concrete and bituminous concrete respectively. Temperature gradients were generally greater under paved areas during freezing and also during thawing periods. More recent investigations /1949-23 added to the data but did not alter, materially, the findings.

Effect of Snow Cover - The literature presents many reports of investigators who have recorded observations or provided data on the effect of snow cover as an insulator in reducing the depth of penetration of frost. Callendar's /1895-1 temperature measurements showed the remarkable effect of snow in reducing depth of freezing and in producing uniform temperatures in the soil, although air temperatures fluctuated widely. Thompson /1934-5 and Mail /1936-4 made similar observations of the effect of snow. Atkinson /1940-3 made controlled field tests to observe the effect of snow cover on depth of freezing. They covered a plowed field (having a north exposure) with straw before freezing began. When sufficient snow had fallen, they covered dup-

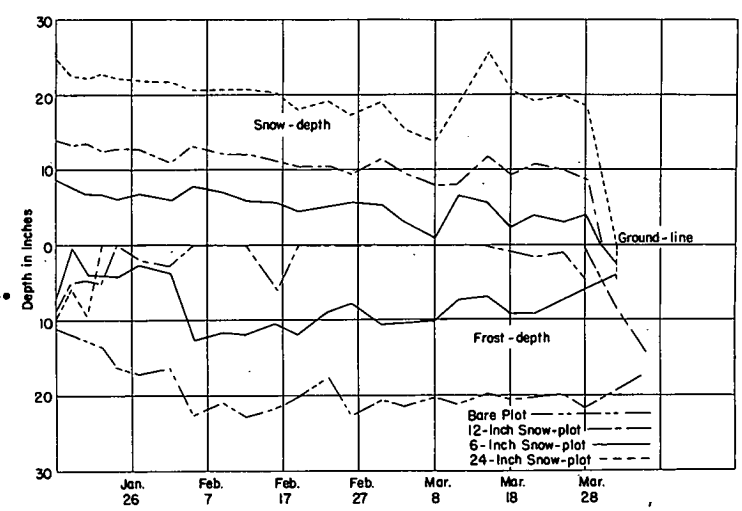


Figure 136. Depth of Frost Penetration Under Different Thicknesses of Snow Cover. (After Atkinson and Bay)

soon as possible after each snowfall. That thawing may take place at the lower frost-line, as well as at the surface, was quite definitely indicated on the 6-in. and bare plots, beginning March 25 and April 1, respectively.

"The two bare plots used in the snow depth experiment quite consistently maintained the same frost-depth and so one of them was covered with a 12-in. depth of snow on February 26 and frost-measurements were continued. This was done to determine whether a 12-in. depth of snow would have sufficient insulating value so that the frost could be drawn from the soil by warmth of the earth. Within a few days the frost in the newly covered plot softened considerably and less difficulty was encountered in driving the soil-tube into the frozen soil than on the bare plot where the frost was extremely hard. Figure 136 shows what took place under the bare and newly covered plots. Along with the softening of the frost there was a gradual rise toward the surface of the lower frost-line on the snow-covered area while that on the bare plot maintained very nearly the same frost-depth until the first of April when it began to rise.

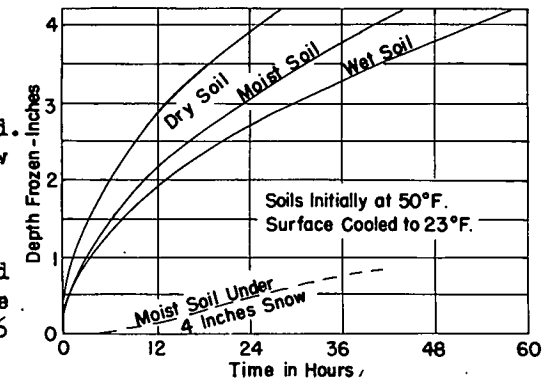


Figure 137. Depth Frozen (After Berggren)

"Snow, when of sufficient depth, will protect the soil against frost if the snow covers the area before frost has a chance to penetrate the soil. It has been found that if frost does penetrate the soil and snowfall of sufficient depth follows, frost may be drawn from the soil."

Geslin /1942-6 found that 6¼-in. of snow was necessary to completely prevent the freezing of the underlying soil in France in January 1941.

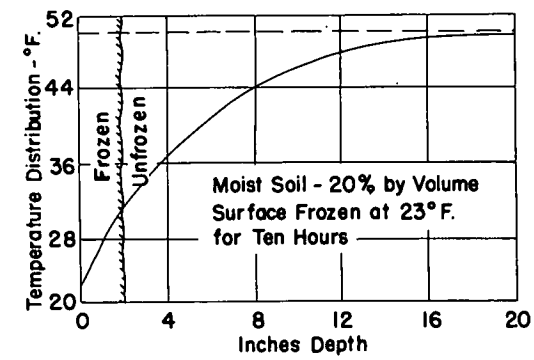


Figure 138. Temperature Distribution. (After Berggren)

Berggren's /1943-1 theoretical studies on frost depths included calculation of the effect of snow cover on rate of freezing of soils. He calculated the depth frozen after different periods of time for dry, moist, and wet soil. He estimated the effect of a 4-in. cover of fresh snow by "computing the time required, under steady conditions, for the snow to conduct enough heat to lower its surface-temperature to 23 F., establishing a linear gradient to 32 F. at the ground level plus removing the latent heat from given depths of soil. Since the method neglects the heat removed from the unfrozen soil, the actual depth frozen under a snow or vegetal cover at any given time is certainly lower (possibly by 40 percent) than the estimated depth." The rates of freezing for the various conditions given above are shown in Figure 137. The temperature distribution in the "moist" soil after 10 hours when frozen 2-in. deep is shown in Figure 138.

Beskow /1947-12 held that the effect of a layer of snow on the surface could be most easily evaluated by considering it in terms of an equivalent (in terms of its heat-transmitting properties) thickness of frozen soil. The rate of heat movement is proportional to the temperature gradient in the frozen layer. After equilibrium has been reached as heat passes through a snow layer and a frozen soil layer, the temperature gradients are inversely proportional to their thermal conductivities. Thus, in calculating the effect of snow cover, Beskow simply added to the depth of frozen soil the thickness of snow cover multiplied by the ratio of the conductivity of the frozen soil to that of snow (conductivity of snow is 1/10 to 1/7 of that of soil). This exaggerates the effect of snow but the error is not great for wet soils. The manner in which Beskow /1947-12 computes the effect of snow is illustrated in Figure 139.

The Corps of Engineers investigation /1947-2 showed a marked difference in penetration of frost in turfed areas with snow cover compared to frost penetration under paved areas. The results are summarized in Table 44.

Although the data in Table 44 do not separate the effect of the turf and the snow, it is believed they indicate generally, the effect of snow cover on depth of freezing.

TABLE 44

Comparison of Freezing Depth in Snow Cover Turf and Paved Surfaces
(After War Dept. 1947-2)

Location of Turf Test Areas	Average Snow Cover During Winter in Turfed Areas (ft.)	Average Total Frost Penetration in Feet		
		Turf	Pavement	
			Bit.	P.C.C.
Dow Field	1.8	2.0	4.7	4.5
Presque Isle	2.5	3.0(a)	5.9	5.3
Watertown	0.75	3.5(a)	4.1	3.4
Pierre	0.75	0.5(a)	2.1(b)	3.5

(a) From subsurface temperature readings at 32 F.
(b) Frost penetration February 3, 1945.

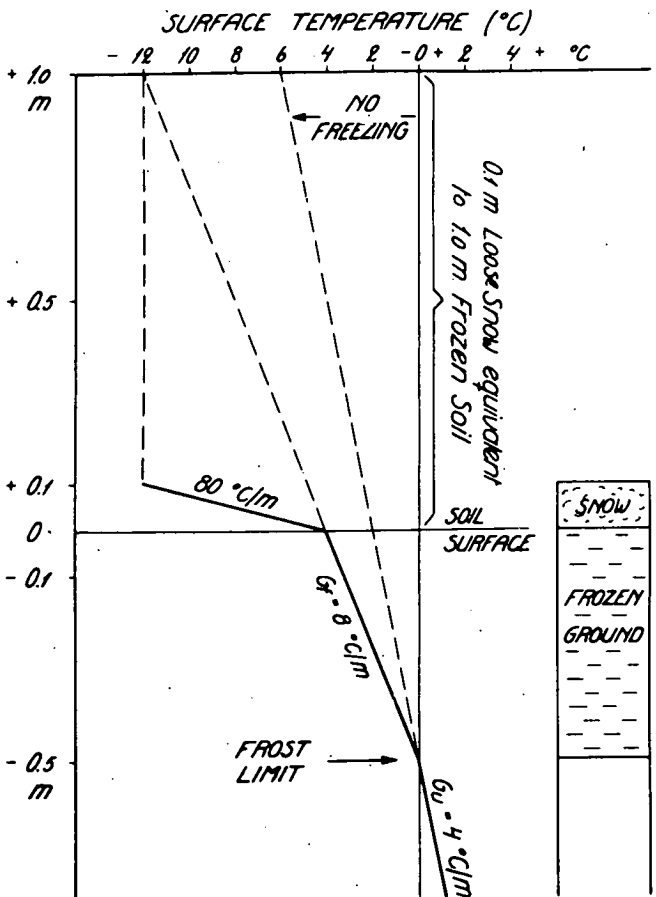


Figure 139. Effect of snow. Heat conductivity of frozen soil assumed - 10 times that of snow ($\lambda_f / \lambda_s = 10$); thus a 0.1 m layer of snow corresponds to 1.0 m layer of soil. Shown also is the surface temperature required to maintain freezing (in the given example - 6 C; in case of no snow it would have been - 2 C). (After Beskow)

The importance of the density of snow cover in influencing its insulating properties was shown by Crawford /1951-14. It was illustrated by observations of Bouyoucos on one cold day when the minimum temperatures at 3-in. depth were 7.5 F. under bare soil, 15.6 F. under compact snow cover, and 32.3 F. under uncompacted snow and a layer of vegetation.

Beskow /1935-1 presented data obtained by H. Abels showing the monthly variation of the average density of snow cover. In addition, he published a curve of the variation of the heat conductivity of snow as a function of its density. This is a graphical summary of data from Landolt-Bornstein, and especially Jansson (1901). Table 45 illustrates the insulating effect of snow cover during a normal winter. Both the thermal conductivity and the density of snow increase rapidly in early spring.

During the early part and middle of winter snow conductivity is about one-tenth that of soil. Franklin /1920-2 estimated the average conductivity of snow to be about one-fifteenth that of soil.

The protective effect of snow in reducing frost penetration and soil temperature fluctuations has been realized for many years. Unfortunately, as previously pointed out, it is unreliable protection and may have to be removed. Nevertheless, in some engineering work the insulating effect of snow cover should be considered for the sake of economy. It is clear that the density of snow will have a considerable effect on its protective qualities. Even so, 1 foot of snow cover will normally provide as much protection against frost

penetration as several feet of soil. Snow cover on roads is, of course, much more dense than the above figures would indicate.

TABLE 45

Variation of the Average Density and Thermal Conductivity of Snow Cover
(After Crawford 1951-14)

Month	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.
Average Density of Snow Cover	.139	.182	.193	.189	.233	.279
Thermal Conductivity(a)	.00029	.00033	.00037	.00035	.00045	.00057

(a)Cal. per cm. per sec. per deg. C.

Influence of Soil Characteristics and Soil State on Depth of Freezing

The thermal properties of soil, that is, heat conductivity and heat capacity, determine the relative depth to which freezing occurs. Those properties are determined by the nature of the soil and the state of moisture content and density in which it exists. The influence of thermal properties on frost action has been mentioned earlier in this review under "Thermal Properties of Soils and Pavements" and is also mentioned later under "The Flow of Heat in Soils and Its Relation to Frost Action." The paragraphs which follow immediately concern cases where investigators have found that soil composition and soil state influence depth of freezing. Soil color, although it has an influence on soil temperature, is not a factor in freezing of subgrades and is not covered in this review.

Soil Composition and Texture - Moore /1910-1 found differences from 4 to 13 F. in air temperatures over bogs compared to other land (having different soil composition). He attributed the difference to higher specific heat and poorer conductivity of bogs. In 1916 Bouyoucos /1916-3 reported in detail soil temperature data taken since his first report /1913-1. The reviewer prepared, from Bouyoucos /1916-3, graphs showing relative temperatures at various depths in gravel, sand, and clay, as well as average daily differences between maximum and minimum temperature. Bouyoucos found that the difference in temperatures for similar depths for various types of soil is quite small. It may be seen from Figure 140 that the differences occurred during summer when soil temperatures normally show the greatest response to fluctuations in air temperature.

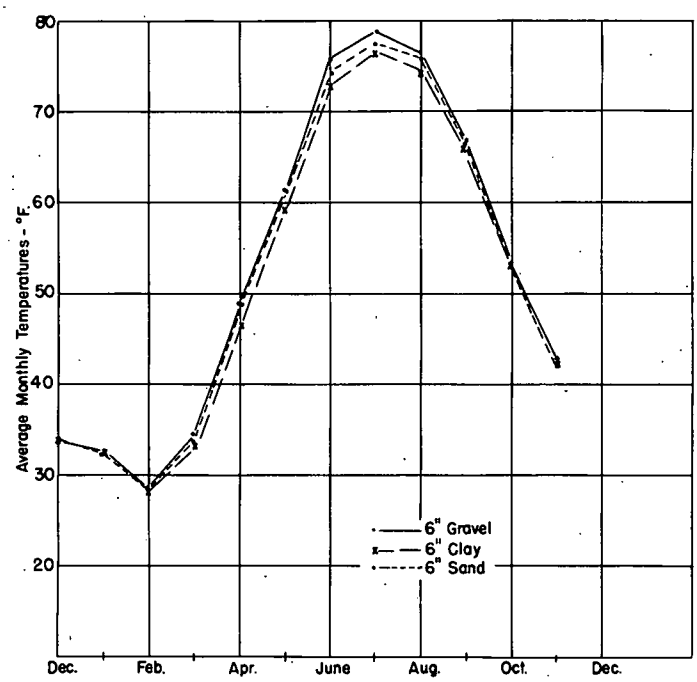


Figure 140. Average Monthly Temperatures of Sand, Gravel, and Clay at 6-in. Depths. (After Bouyoucos)

Highland /1926-1, in reporting soil temperatures from his studies of frozen water pipes, gave the following general relationships between freezing and texture of soils:

1. Moose Jaw, Sask., found the greatest depth of frost occurred in wet clays, the penetration being 50 percent less in sandy soils.

2. Leadville, Colo., found a smaller depth of penetration in clay soils.

3. Syracuse, N. Y., found that clay soils do not freeze as soon as sand and gravel.

4. Calgary, Alta., found that frost penetrated in 1916 to a depth of 12 ft. in gravel and 8 ft. in loam.

5. Glens Falls, N. Y., found that no freeze-up troubles were encountered in rock trenches, the trouble being limited to areas where clay or heavy loam were encountered.

Smith and Byers /1938-6 found that organic matter and texture are important in regulating thermal conductivity of soil. They state that "a sandy texture indicates a large ability to transfer heat, while a high clay content suggests less ability to transfer heat. High organic matter indicates least ability to transfer heat". Belcher /1940-14 points out that fine-grained soils having a high void ratio normally resist freezing to a greater extent than coarse soils containing less water. He cites records of frost penetration in 29 Indiana cities (Table 46) and shows that of 16 cities reporting observations in both types of soil 10 cities reported deeper penetration of frost in sandy soil, 4 reported the same in sand and clay, and 2 reported deeper penetration in clay soil.

TABLE 46

Frost Penetration During Winter of 1935-36*

Location (29 Indiana Cities)	Maximum Frost Sandy Soil	Penetration in inches Clay Soil
Bedford	—	36
Bloomington	—	42
Columbus	—	48
Crawfordsville	—	48
Danville	40	40
Elkhart	60	—
Evansville	36	36
Fort Wayne	54	48
Galveston	—	36
Gary	54	60
Goshen	54	60
Indianapolis	50	48
Jasonville	—	30
Kendallville	40	40
La Porte	—	54
Ligonier	60	54
Marion	40	30
Michigan City	63	—
Newburgh	30	30
Noblesville	48	—
Osgood	37	28
Richmond	—	45
Rochester	52	50
Rockport	32	30
Scottsburg	—	24
Terre Haute	60	—
Union City	54	48
Valparaiso	54	48
Washington	30	22

* Reported at 29th Annual Meeting of Indiana Section of American Water Works Association at Purdue University, April 7, 8, and 9, 1936.

Schaible /1941-1 found that coarse soils freeze more rapidly and more deeply than do fine soils. In contrast, Belotelkin /1941-4 found that fine textured soils freeze deeper and remain frozen longer than do coarse textured soils.

Soil Moisture Content - According to Crawford /1951-14, the studies by Bouyoucos /1913-1 and /1916-3 included experiments on soils having different amounts of organic matter. Organic matter in a soil alters its color and its water-holding properties. Bouyoucos planned his studies to determine the extent to which those characteristics would oppose each other in influencing soil temperature, because with increase in organic content the soil color darkens and the water-holding capacity increases. Tests were run on sand with various amounts of organic matter and on peat. The variation of percentage moisture content with seasons and the striking effect of organic content on moisture content is shown in Table 47.

TABLE 47

Moisture Content of Natural Sand Soil with Different Amounts of Organic Matter (%)
(After Crawford 1951-14)
5-in. depth

	Organic Content					
	1.81%	2.01%	3.32%	5.47%	6.95%	Peat
April 3	16.96	12.95	21.80	26.90	32.53	256.5
July 27	2.08	3.69	6.78	12.83	17.42	236.4
Nov. 4	2.46	5.85	8.63	14.46	21.8	247.8

Four years of observations showed that soils either white in color and with low moisture content or black with high moisture content had a lower average temperature during the spring and summer than soils with these properties in medium proportions. From these observations it would appear that both surface color and moisture content have considerable influence on soil temperature. From the point of view of color, peat should absorb the greatest amount of heat and natural soil the least. On the other hand, the natural soil has a low specific heat and should therefore change temperature more readily. The 3.3 percent soil (Table 47) became warmest in summer and had the greatest yearly amplitude. This is probably caused by the most favorable balance between surface color and specific heat due to moisture content.

Bouyoucos /1913-1 gives the specific-heat properties of soils shown in Table 48. It is seen that peat has the lowest specific heat by volume and yet the highest specific heat by weight. It is the specific heat by volume that is important when considering soil temperature variation with depth.

TABLE 48

Soil	Specific Gravity	Specific Heat	
		Equal Weight	Equal Volume
Sand	2.664	.1929	.5093
Gravel	2.707	.2045	.5535
Clay	2.762	.2059	.5686
Peat	1.755	.2525	.4397

Moisture content has great effect on the specific heat of soil in place, since water has a specific heat approximately five times as great as dry soil. As shown by Table 49, dry peat will heat or cool about twice as readily as sand or gravel with equal application of heat. In its natural, wet condition it will heat or cool only about one-third as readily. Hence, it is seen that moisture content will over-shadow in importance density and specific heat of dry soil.

TABLE 49

Effect of Moisture on Soil Temperature
(After Bouyoucos)

Soil	Weight of a cu. ft. (lb.)	Percent Moisture	Specific heat by equal weight	Rise of Temp. of dry soils by 100 heat Units	Rise of Temp. of moist soils by 100 heat Units
Sand	102.7	16.96	.1915	.011170 C.	.005876 C.
Gravel	109.2	10.45	.2045	.009854	.006520
Clay	76.35	29.16	.2059	.013990	.005790
Loam	72.93	40.7	.2154	.014010	.004848
Peat	36.76	256.5	.2525	.023740	.002127

Bouyoucos deduced that although sand and gravel have a higher specific heat by volume than peat and thus will heat or cool more slowly in the dry condition, when field moisture content is considered the sand and gravel will cool or heat three times as rapidly as peat (Table 49). For this reason sand and gravel would be expected to warm up more rapidly in the spring and cool more rapidly in the fall. This is true in the spring because the air temperature has a daily upward trend and sand and gravel warm up early, but the peat, having the greatest heat capacity, warms up slowly and finally reaches the temperature of the sand and gravel. In the fall, the trend of air temperature is downward on the average, but the fluctuations are great. On certain days the temperature falls very low and the sand and gravel cool most. Next day the temperature may rise considerably and the sand and gravel will heat most. These alternate cold and warm days tend to keep the sand and gravel as warm as the other soils. If, however, there is a sudden continual drop in air temperature the sand and gravel would cool faster than the other soils, viz., clay, loam, and peat.

Keen /1931-1 points out that dry soil has a low conductivity due to poor contacts between grains, and hence temperature falls off rapidly with depth. Moisture improves the grain-to-grain contact and the conductivity increases, but specific heat also increases so the actual rise in temperature is small. Moreover, the vaporization of water will slow warming, since the latent heat of water is about 500 calories per gram. The latent heat may even cool the soil. The greatest temperature rise from a given application of heat occurs between the extremes of wet and dry soil where the conductivity increases more rapidly than the specific heat.

Smith /1932-2 attributes variation in temperature lag at depth from year to year to variation in moisture content. Beloteklin /1941-4 observed that frost penetrates deeper and remains longer in poorly-drained soils than in well-drained soils. Fine-textured soils resemble poorly drained soils in their influence on soil freezing; coarse-textured soils resemble well-drained soils.

Belcher /1940-14 suggests that fine-grained soils normally resist freezing to a greater extent than coarse soils having less water. Belcher classified factors influencing soil temperatures into two groups, those of position and those of condition. He held that moisture content is the most important of the "conditional" factors due to its high specific heat. He wrote:

"Dry soil itself furnishes a better conductor for heat than water, yet water is a better thermal conductor than air. In a dry state the soil particles transmit heat or cold through the thermal contact between soil grains, the air in the voids acting as an insulator. Although water is a poor conductor, when it partially displaces air in the voids it improves the conductivity of that phase of the soil mass and the thermal contact between grains, and therefore, the soil is in an optimum condition for conducting heat. Increasing the moisture content further creates a volume of water that is in excess of the amount required to obtain maximum thermal conductivity. This excess of water then acts as a modifier by requiring more heat to raise the temperature, or a greater amount of cold to lower the temperature of the soil."

Smith /1942-8 agrees that moisture content is an important variable, far more important than variation in mechanical composition or particle arrangement. He observed that although the influences of moisture content can be sorted out and dealt with in order, they are so interdependent that under field conditions the total effect is complex. Leggett and Peckover /1949-28 suggested the mechanism of water-vapor movement in soil is possibly a main determinant of soil temperature variation.

From the standpoint of thermal conductivity it would appear that depth of frost in a dry soil might be less than if the same soil were saturated. This is not the case, for the depth of freezing is approximately proportional to the square root of the thermal conductivity and inversely proportional to the square root of the latent heat of water in the soil and the total volumetric heat of the soil. A change in water content from 5 to 10 percent will increase the sum of the volumetric heat and latent heat by about 100 percent, while the same change in water content will increase the thermal conductivity only about 20 percent. Thus the greatest depth of freezing will be at the lowest water content and decrease with increase in water content.

It has been shown previously that capillary moisture in soils flows from regions of warm to regions of cold. Thus, the existence of temperature gradients influences soil moisture content. Also, it has been shown that water flows from regions of low to regions of higher capillary potential. Thus, the existence of large moisture gradients may influence the movement of soil temperatures.

Soil Density - The influence of soil density on depth of frost penetration has been reviewed under the item "Influence of Physical Characteristics and Properties of Soils on Frost Action - Effect of Soil State - Soil Density".

Influence of Type of Pavement and Base on Depth of Freezing

Highland /1926-1, in his studies of soil temperature with relation to frozen water services, found that frost reached greatest depth under roadways and least under grass. That was borne out by the comparative studies of the Corps of Engineers /1947-2 and has been mentioned previously. The Corps of Engineers studies showed that for practical purposes the depth of freezing is about the same for bituminous concrete and for cement concrete. That is based on the premise that the underlying materials used in bases and subbase and the soils have similar thermal properties. The War Department's report holds that the thermal conductivity of slag and cinders is about one-half that of other materials (sand, sand-gravel, or crushed rock). Slag or cinders should reduce penetration under pavements, so they approach turf more closely in resisting depth of freezing.

Measurement of Depth of Freezing

Depth of freezing has, in most instances, been determined from excavations, or from soil temperature data, assuming that freezing occurred at the 32 F. level. Two other methods for determining depth of freezing have come out in recent writings. Cailleux and Thellier /1948-3 used porcelain tubes (80 cm. long by 30 mm. in diameter with 2-mm. walls) closed at the lower end and fixed vertically in the soil with the top covered and barely above the ground surface. They placed thin, graduated strips of plastic or wood in the tubes so the zero marks coincided with the ground surface. On one side of the strips they fitted a series of open-ended capillary glass tubes (20-mm. long by 4-mm. in diameter) containing water. The water in the horizontal tubes freezes without breaking the tubes. The limit of freezing is then found by quickly removing the strip and taking a reading on the scale. It is claimed that data so obtained are reliable.

Colman /1946-9 developed a resistance-type soil-moisture meter (described in the U. S. Forest Service Manual of Instructions for the use of the Fiberglas Soil Moisture Instrument /1948-9) which he feels has great promise in determining depth of frost penetration. He states /1946-9: "Electrical soil moisture instruments can be used to detect freezing and melting because of the very great difference in resistivity between the solid and liquid phases of water." He had had no experience with the meter for that purpose but felt it had promise.

DESIGN METHODS FOR PREVENTING DETRIMENTAL FROST ACTION

Paved Surfaces - Flexible and Rigid Type

The subject of design is an important part of this review for it covers that part of the literature which has practical value towards the solution of problems involving soil freezing, heaving, and reduction in load carrying capacity of frost susceptible soils. The subject of frost susceptibility has been reviewed directly and in some detail earlier under the subject "Factors Influencing Magnitude, Rate and Nature of Frost Action and Reduction in Load Carrying Capacity," and in a less direct manner under "Penetration of Frost." Those data on frost susceptibility presented in the literature specifically as criteria for design purposes are restated in part here for purpose of convenience. The reader should bear in mind that criteria for non-frost susceptibility as they pertain to intense differential heaving need not lie identical to criteria pertinent to reduction in load carrying capacity during the thawing period. Accordingly the reader is asked to distinguish between adequacy of designs based on need.

The reviewer has attempted to classify design requirements according to the purpose for which the design was intended. The writings are not always clear in stating whether the designs presented are for general use on certain types of soils or whether they pertain to special moisture and drainage conditions associated with those types of soils which cause severe differential heaving. The literature is not always clear on whether the design is intended to alleviate conditions of heaving from the standpoint of heaving alone or from the standpoint of reduction in load-carrying capacity, or both. The reader may draw his own conclusions as to the correctness of the grouping and not be swayed by the manner in which data are presented, for that grouping has been done merely to present the overall picture with some degree of orderliness. The review of older literature brings out designs no longer adequate or improved upon in recent years. Therefore, the reader is asked to consider the time at which each design was current, so he may better appreciate the progress of design and the manner in which it has kept pace with increase in knowledge of frost phenomena.

Design to Prevent Damaging Detrimental Heaving - The nature of any design used will depend on (1) the degree of the condition, (2) the aerial extent of the condition, and (3) the relative cost of correcting the condition by various methods. Perhaps the most important single item which influences the economics of design to prevent damaging heaving is the availability of suitable natural or processed non-frost susceptible materials at reasonable cost. The literature recognizes the economics of designs, yet gives few data on costs.

Prevention of damaging differential heaving may in some instances be accomplished by change in road location. However, in most instances the solution is not that simple. It may involve change in grade elevation, subsurface drainage, excavation, and replacement (sometimes accompanied by drainage) or it may involve the use of insulation, or other design methods.

Change in Grade Elevation - Lang /1935-3 shows a condition (see Figure 141) where Minnesota topsoils are of a relatively stable nature and not susceptible to severe heaves and boils. Figure 141 shows that heaves and boils will occur if the grade line is placed in the position indicated by the broken line, while no severe differential heaving need be anticipated if the grade line is raised to the position indicated by the upper (solid line) alternate grade line. Casagrande /1938-5 suggests that where it is possible to "obviate the danger of damage due to frost by raising the road bed, this should be allowed for in preparing the plans." Stokstad /1938-10 stated that in Michigan the elimination of bad drainage conditions is done by proper elevation of the grade line wherever possible, but occasionally perched water tables makes drainage necessary. Goeltz /1948-15, after reviewing the seriousness of spring breakup conditions in St. Clair Co., Michigan, holds that it will be necessary to dig new ditches, build up the grade to at least 1½ to 2 ft. above the surrounding country, and to place a granular fill 12 to 18 in. thick so the "wearing surface would be at least 5 ft. above ditch location." Livingston /1951-34 brought out that, in high mountain valleys in Colorado where the ground-water table is near the ground surface, design grade lines are held 4 to 5 ft. above side-drainage ditches or the natural ground line to prevent intensive frost damage.

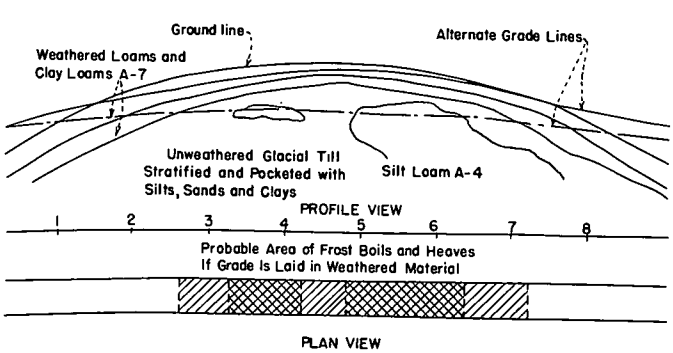


Figure 141. Grade-line adjusted where soil survey preceding grading reveals inferior subsoils. (After Lang /1935-3)

Subsurface Drainage - Wilson /1918-3 found the worst heaving in Duluth occurred on subbases generally of the type recommended by Harrison /1918-2. As a substitute, Wilson suggested adequate provision for draining the subgrade by the use of edge tile drains suggested by the design in Figure 142. Wilson believed the drains should be deep enough to lower the elevation of the zone of saturation below the subgrade. Burton /1931-5 made the following comments regrading the effectiveness of drainage in preventing heaving:

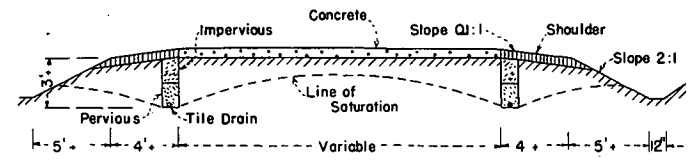


Figure 142. Pavement Section to Eliminate Frost Action. (After Wilson)

"It has been a matter of opinion that frost heaving could not be corrected by tile drainage because the phenomenon is purely capillary by nature, and to a certain extent this is true. Many serious heaves, however, have occurred in Michigan in fine to fairly coarse sands to which water is fed under hydrostatic head by springs or in which water is held up in the frost area by an impervious layer of soil below. Such heaves can be prevented by a proper and intelligent design of tile drainage. On the other hand, heaves which occur in silts and in some clays are the result of a capillary phenomenon and cannot be corrected by tile drainage."

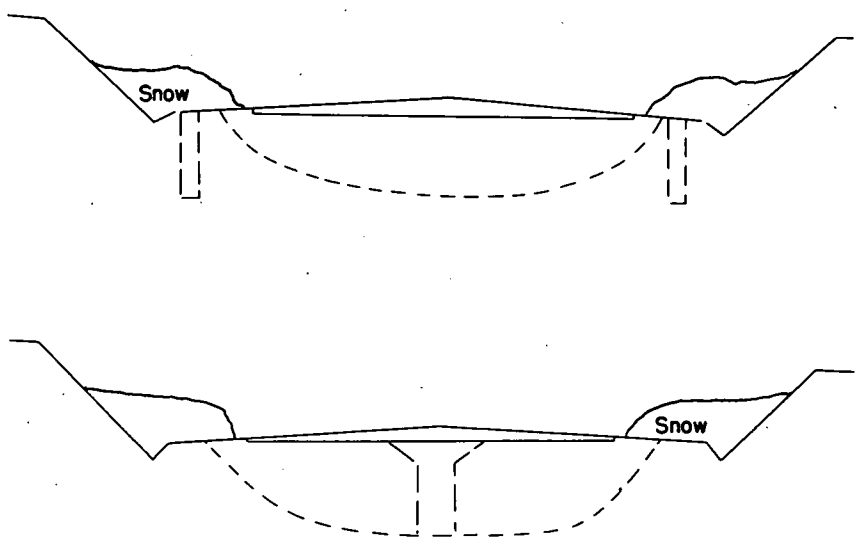


Figure 143. Center Trench Installation

An American Road Builders Association committee on subgrades and pavement base /1930-16 illustrated the relative effectiveness of side and center trench subsurface drainage. (Figure 143). The drawing shows sections of side and center trench types or drains. The dashed line indicates approximate depth of frost penetration. As the thaw begins in the center and moves downward, it can be seen that no water can reach the side drains until thawing is complete. The center trench is said to keep the roadway dry prior to freezing and in a condition to receive thaw water as rapidly as it forms. The committee recommended this type of drainage in "a capillary frost-heaving soil."

A Michigan soil exhibit illustrated three commonly found field conditions associated with heaving and gave proper and improper methods for correcting the conditions by subsurface drainage. The field conditions and suggested corrective measures are indicated in Figure 144.

Aaron's /1934-3 report of the cooperative studies of the Bureau of Public Roads and the State highway departments of Minnesota, Wisconsin, and Michigan illustrated a number of designs including subsurface drainage which had been used up to 1934. They included (1) drainage, (2) excavation and replacement, and (3) a combination of 1 and 2. The subgrade designs used are shown in Figure 145. The effectiveness of the various preventive measures is summarized in Table 50. The table shows that drainage of the types used was of little benefit in preventing heaving. It should be stated, however, that in most cases the drains were placed at arbitrary depths of 3 to 6 ft. "without regard to the type of soil or the arrangement of the soil layers. Type M, when installed in water bearing sandy soils in such a manner as to conform to the soil profile, served to intercept and carry away enough of the water to prevent detrimental heave. This same type, when used in stratified silts, fine sands, and clays was of no value whatever." Failure to obtain the desired stability is definitely attributable in many instances to the attempt to apply arbitrary standards of spacing and depth in placing the drains without regard to prevailing conditions".

The problem of drainage in bedrock was discussed by Paradis /1934-4. He suggested, "Probably the most practical method of drainage in those cases is to crack the rock by blasting below the frost level in the ditches."

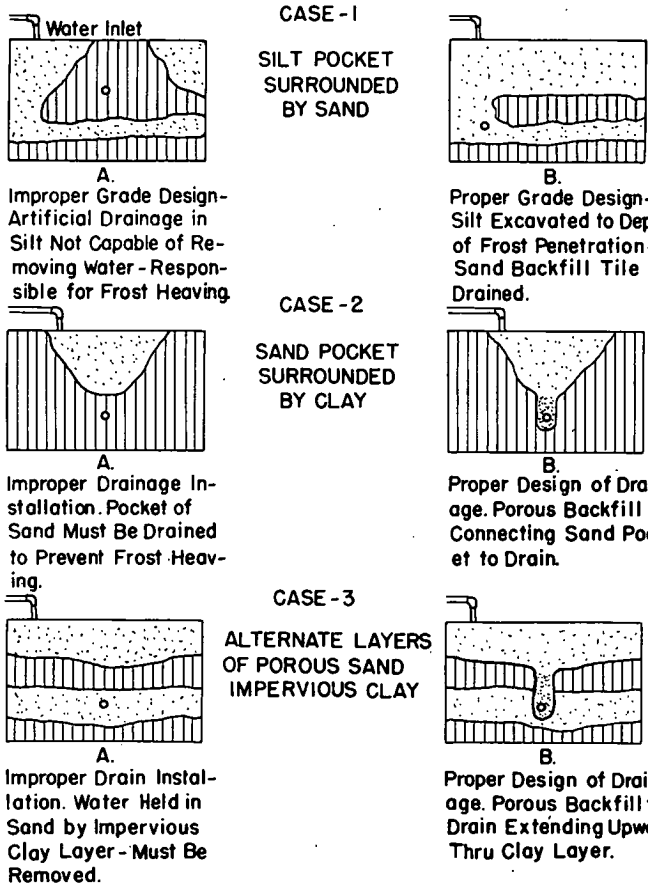


Figure 144. Typical Field Conditions and Grade Design Measures for Elimination of Frost Heaving

Beskow /1935-1 and Osterberg /1940-5 found that the relationship between the capillarity of the soil and the depth to groundwater is significant in drainage of soils to prevent heaving. (This item has been discussed in part previously). Beskow /1935-1 found that the rate of flow was inversely proportional to the distance to free water and varied greatly for different grain sizes. If the depth to free groundwater is greater than the capillarity of the soil, there can be no flow to the freezing layers and no heaving will occur (other than any minor heaving which might occur due to contained water). For coarse soils of low capillarity, heaving can be prevented by lowering the groundwater below the limit of capillary rise. Even for finer soils Beskow held that the groundwater need be lowered only a few feet to decrease the heave materially. His earlier work showed that the depth for drainage to be effective in preventing frost heave in roads is about 1.8 meters (6 ft.) under the road surface. This means when the subgrade begins to freeze the water is about 5 ft. and the load is about 50 grams per sq. cm. (0.7 psi.). He points out, however, that only in special cases does such drainage make the soil completely non-frost-heaving. Both his earlier and later works, /1938-11 and /1947-12, point out that for preventing frost boils, the effective depth of drainage is at least 1.7 meters for silts (5.6 ft.) and 1.5 meters (5 ft.) for clay. He found that roads on side slopes react favorably to drainage and often can be drained with one deep drain while on flat ground two deep drains as indicated in Figure 146 may be necessary.

TABLE 50

Results of Frost Heave Preventive Measures

Type of treatment [1]	Type of surfacing	Soil profile [2]	Results	Remarks
A	Gravel	F, D	Heaved on sides during winter; surface softens during thaw. Carries moderate amount of traffic satisfactorily.	County roads; center trench 3 by 3 feet.
	Bituminous surface treatment	I, G	Heaved excessively on sides; no serious break-up during thaw.	Center trench 3 by 3 feet.
	Concrete	H	Heaved sufficiently to cause considerable cracking	Center trench 6 by 2 feet.
B	Gravel	B	Heaved a small amount on sides; no break-up	Center trench 8 by 3.5 feet.
C	do	J, L	Heaved on sides; slight softening and rutting during thaw. Carries traffic satisfactorily.	Center trench 3 feet deep, 6 feet at top, 4 feet at bottom.
	Concrete	L	Heaved on sides removing crown; considerable cracking	Do.
D	Gravel	D	No heaving observed; smooth and firm; no break-up	Trench 24 by 1 foot.
E	Bituminous surface treatment	D, I, J	Heaved on sides in soil profiles D and J; surface broken but no rutting. Heaved badly in soil profile I; no rutting.	Treatment is 12 feet wide on soil profiles D and J and 26 feet wide on soil profile I, V-type trench.
F	Concrete	A to O, inclusive	Heave negligible, very small amount of cracking	Slightly wider than pavement.
G	do	L, M	No detrimental heaving; very small amount of cracking	Do.
H	do	D, M	Heave negligible; very small amount of cracking	Do.
I	do	B, D; L, M	Heave negligible; small amount of cracking	From ditch to ditch.
J	Gravel	D	Uneven and rough; small amount of rutting; carries traffic satisfactorily.	Do.
	Concrete	B	Slight amount of heaving; fairly uniform; small amount of cracking.	Do.
K	Bituminous surface treatment	N	Heaved on sides of trenches leaving a narrow depression along the center line and over cross trenches; reduced surface break-up.	Rough riding and dangerous at times.
L	Gravel	D	Reduced break-up in most cases so that traffic could move across, although difficult; some sections impassable.	Very unsatisfactory in this soil condition.
	Concrete	D	Considerable cracking typical of frost heave	
M	Gravel	I	Impassable during thaw	This method apparently of no value in this soil.
	Concrete	C, E	Heave negligible; small amount of cracking	Depth of drains and location of cross drains varied to conform with soil profile.
N	Gravel	D	Soft and rutted in spots during thaw; carries traffic with difficulty.	
O	do	D	Impassable during thaw	No benefit derived from this method.
P	Bituminous surface treatment	D	Heaved breaking surfacing; carried traffic without breaking through.	
	Concrete	K	Heaved and cracked considerably	One section heaved to such an extent that the pavement had to be replaced.

1 Refers to designs in Figure 145 2 Refers to soil profiles in Figure 93

Stokstad /1938-10 found that where perched water tables were encountered in cut sections within 5 ft. of the road surface the common practice in Michigan was to place intercepting tile-edge drains 5 ft. deep and 2 to 4 ft. from the edge of the road metal on the side from which the water was coming. He found it important to place at least a 12-in. outfall at the outlet to avoid excessive maintenance. Roadside ditches are inadequate for drainage except when 6 ft. or more deep.

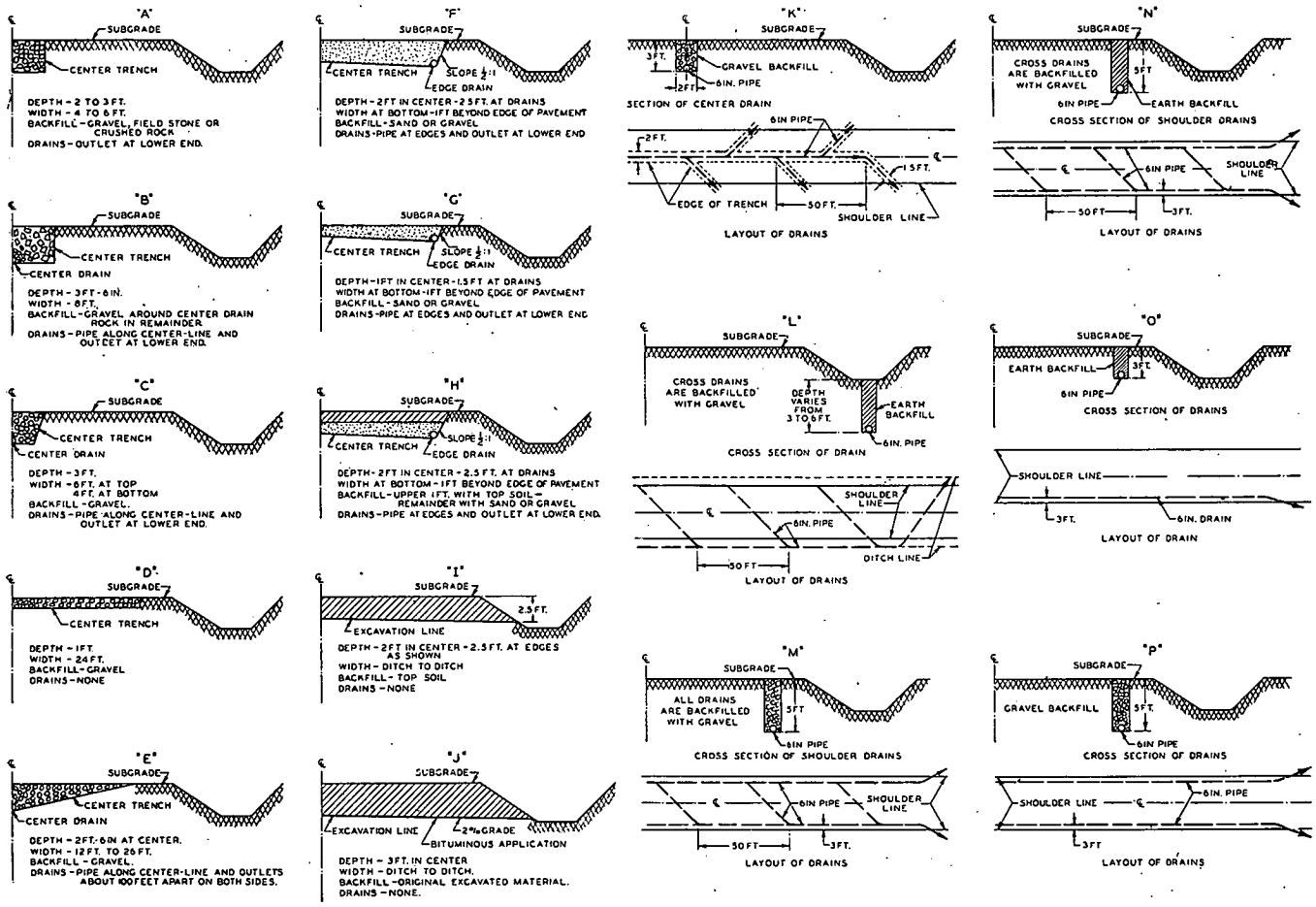


Figure 145. Subgrade Designs Used to Prevent Frost Heave
(After Bureau of Public Roads)

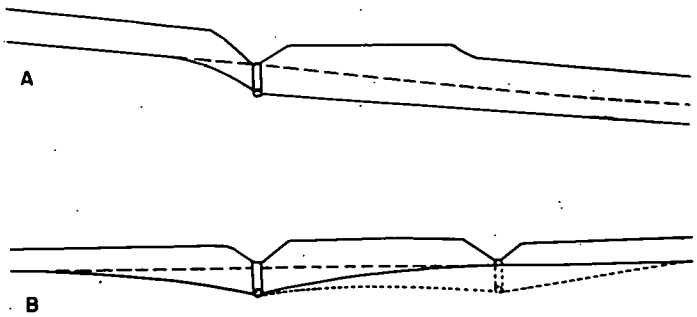


Figure 146. Effect of deep drainage on ground water level. Broken line: ground water level before drainage. A: Side slope. Only one drain, along the upper road side. B: No side slope. Drain causes a depression in the ground water surface, the narrower the depression, the denser the soil. Thus as a rule two drains are necessary. (After Beskow)

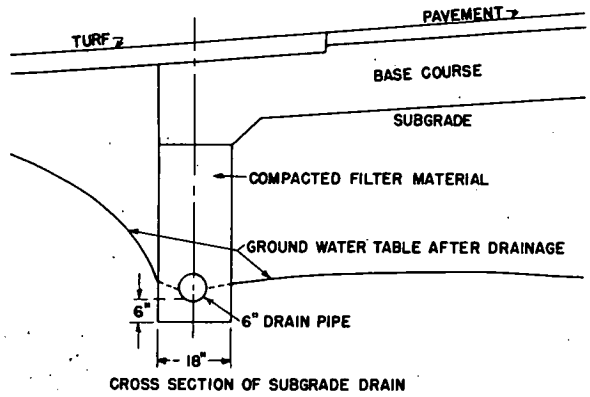


Figure 148
Typical Subgrade Drainage Details
(After Corps of Engineers)

L. Casagrande /1938-2 observed two experimental road sections in Germany to determine value of subsurface drains. Borings showed that on an alluvial soil the drains were effective only to a distance of 20 in. on either side of the drains and heaving was noticeable on drained as well as undrained sections. Later, /1938-5, he added, "Lowering the groundwater by providing ditches or seepages is possible only with relatively permeable soils. The provision of seepage ditches and underground drainage is successful on slopes with water conducting courses located adjacent to the surface."

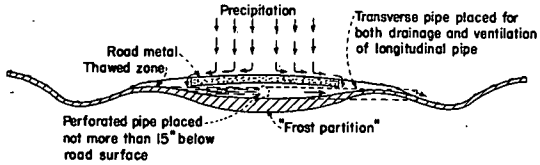


Figure 147
Installation of perforated pipe to relieve upper thawed zone of excessive free water. (After Williams)

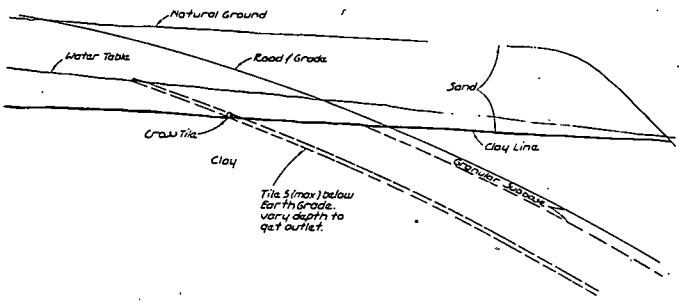


Figure 149
Drainage of Wet Sand over Clay - Berrien Soils
(After Stokstad)

Schmid /1942-4 found that deep drainage extending below the frost penetration will not check frost heaving but will control the thickness of ice segregated. Williams /1945-5, after presenting his theory on water being trapped in a trough under the pavement above the "frost partition", suggests that to be effective the drain must be placed where it will carry off the excess water after it accumulates. He suggests that the flow line should be not lower than the depth of the thawed subgrade above the frost partition, i.e., not more than 15 in. below the road surface. (Fig. 147).

The Michigan Field Manual of Soils Engineering /1946-16 provides soil engineering data and recommendations for each of the agronomic soil series in Michigan. Included in the data are estimated lineal feet (per 1000 ft. of cut below natural ground elevation) of frost heave excavation and open joint sewer pipe to take care of drainage where needed.

The Corps of Engineers Manual /1946-19 describes two types of subsurface drainage: Subgrade drainage and intercepting drainage.

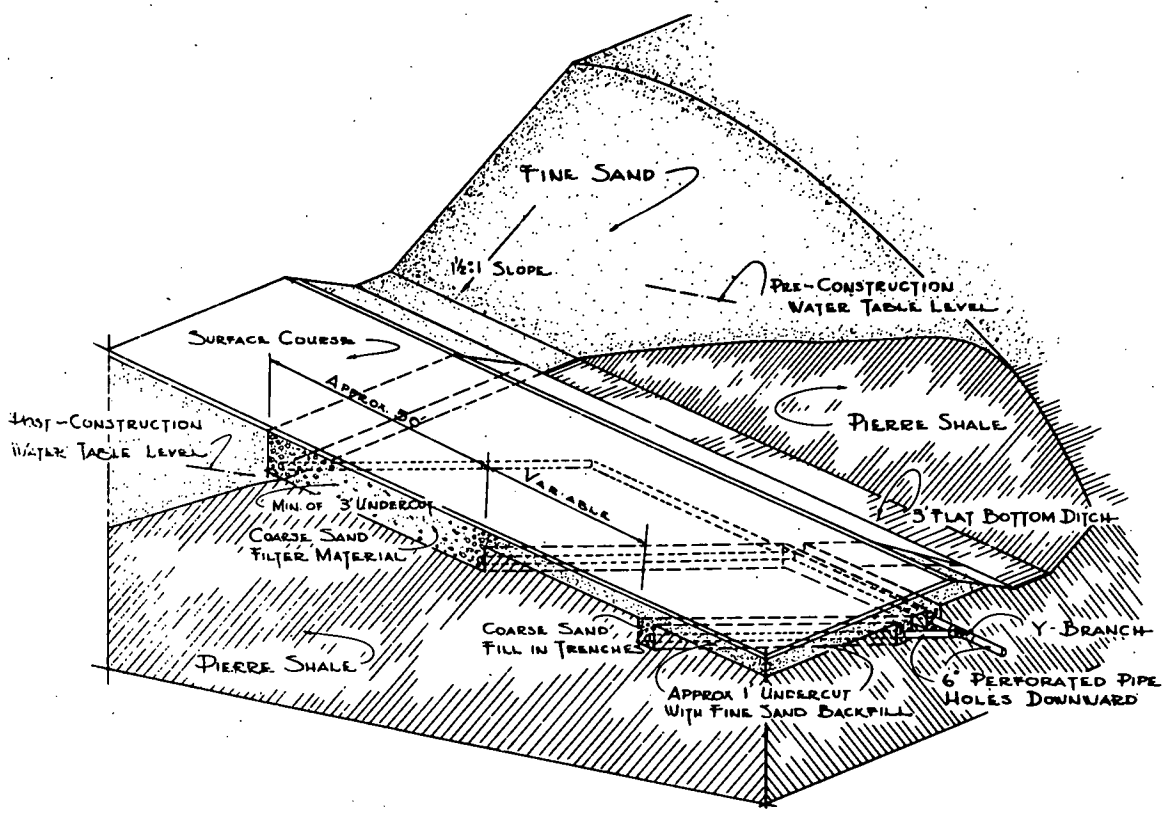


Figure 150. Subsurface Drainage Installation. Pictorial View of section taken at centerline showing left half of roadway. (After Lund)

Subgrade Drainage - is required at locations where seasonal fluctuations of groundwater may be expected to rise in the subgrade beneath a paved area to less than one foot below the bottom of the base course. It is provided primarily to drain subsurface water in areas of high groundwater. The drains may consist of open ditches or of subsurface drain pipes. The type, location, depth, and spacing of drains depend upon the soil characteristics and depth to groundwater. A cross section of a subgrade drain is shown in Figure 148.

Stokstad /1951-41 states that where adequate height of grade above the water table cannot be obtained, subsurface drainage (although a poor substitute for a high grade) is necessary. Figure 149 illustrates the use of subsurface drains to solve a typical drainage problem presented by a sand-over-clay type of profile.

Stokstad stresses the difficulty of maintaining a free outlet for drains and illustrates how tile-drain outlets without proper outfall soon become blocked.

Lund /1951-42 described three conditions in Nebraska where heaves occur in areas where a pervious material is underlain by a less pervious material. Those areas are described under "Effect of Natural Formations - The Soil Profile." Subsurface drains can be used effectively in those areas provided: (1) the water bearing stratum is drainable, and (2) the topography provides sufficient slope. He mentions that "in practice, subdrains are now being installed only in upland or slope situations where free water is observed in the bottom of soil survey borings." An example of a design of such an installation is shown in Figure 150. It may be seen that the design includes an extra thickness of permeable granular base.

Otis /1951-33 reports that 2-ft. gravel bases are used in ledge-rock excavations. Gravel base courses contain a maximum of 5 percent passing the No. 200 sieve to permit drainage. Underdrains are installed in ledge areas if there is indication that water will enter the roadway.

Keene /1951-43 reports that on new construction in Connecticut an underdrain (occasionally two) is placed in wet cuts to drain the subbase, intercept sidehill seepage, lower the water table, or do all three. The underdrain is located under the gutter or the middle of the shoulder. The depth of the underdrain depends on frost penetration, type of soil, and thickness of pavement and subbase; usually it is 5 ft. below top of shoulder in earth cuts. If the subbase extends to full depth of frost, the underdrain can be shallower, serving only to drain the subbase, but this will increase the cost of construction materially. In rock cuts the underdrain is 4 ft. below top of shoulder, serving only to drain the subbase. Six-in. pipe is used--usually perforated, asphalt-coated metal pipe--except that larger pipe is used if it is to carry surface water also. Perforations are usually on the top side of pipe, but if the water table is about level they are on the bottom side; the recently-approved design with perforations below middle of pipe is now being used. Backfill is ½-in. stone, except that where the soil is fine-grained, with no coarse fraction, washed concrete sand is used. The choice is usually determined by the "piping ratio" of backfill to soil.

Intercepting Drainage - is required where seeping water in a pervious stratum will raise the ground water locally to a depth of less than 1-ft. below the bottom of the base course. This flowing water may occur in pervious stratified soil layers, in exposed rock cuts, etc. The object is to intercept flow and collect water before it reaches the paved area. The type and depth of drain depend upon the soil and ground water conditions and may be open ditches or subsurface drain pipe. Sketches of typical installations are shown in Figure 151.

Riis /1948-25 held that drains are useful only in preventing capillary absorption from a high ground water level and up into the subgrade immediately beneath the paving. He believed that while drains only seldom counteract frost heave, "it is possible that by drawing off the excess water formed by the melting of the ice lenses, they may reduce thaw damage." He suggested that drains should never be placed under the riding surface as they create nonuniform drainage and nonuniform heaving.

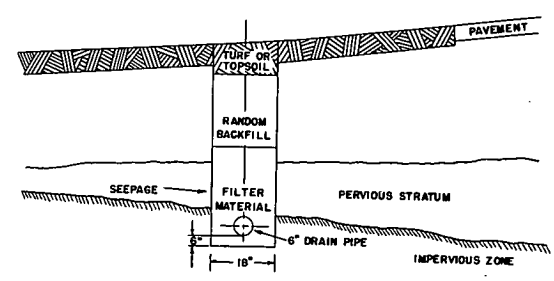


Figure 151
Typical Installation of Intercepting Drains (After Corps of Engineers)

Joint drains under Portland cement concrete pavements - Minnesota reported to the Highway Research Board Committee on Warping of Concrete Pavements /1945-8 data on the effectiveness of tile drains placed under joints in concrete pavements. A test section was built on a highway east of Brandon on a clay till subgrade similar to the soil on an adjoining project, on which heaving at joints during winter had occurred. Seepage drains were constructed at several transverse joint locations in an effort to intercept water entering through leaky joints and carry it to side ditches in order to prevent an increase in soil moisture content of the subgrade adjacent to joints. After fine grading was completed trenches were dug 1-ft. wide and 2-ft. deep at the center line, 2½-ft. deep at pavement edges and sloped down to the bottom of the side ditches, which were 3½-ft. deep. Coarse aggregate backfill was used.

Permanent "frost proof" bench marks were installed. The studies included periodic observations of elevations and subgrade moisture content.

The conclusions reached from the study were:

(1) "Seepage drains as constructed in this case were not successful in keeping moisture out of the subgrade near the joints.

(2) "Warping was produced as a result of high moisture condition in the subgrade soil near the joints combined with subsequent frost action.

(3) "Indications are that warping may be produced by a combination of frost heaving at joints and slight shrinkage of soils at mid-portions of slab."

Of interest in the experiment was the subsequent introduction of water (through holes drilled in the pavement) to mid-portions of the slab and the uplift of that portion, materially lessening the warping effect.

Backfill for subsurface drains - It would be of interest to review all available writings on backfill for drains and also to summarize data on backfill material for drains from specifications of state highway departments in the frost area. Space does not permit that to be done here. Therefore only one fairly recent publication is reviewed here. The Corps of Engineers Manual /1946-19 suggests the following theoretical design of filter material for backfilling subsurface drains:

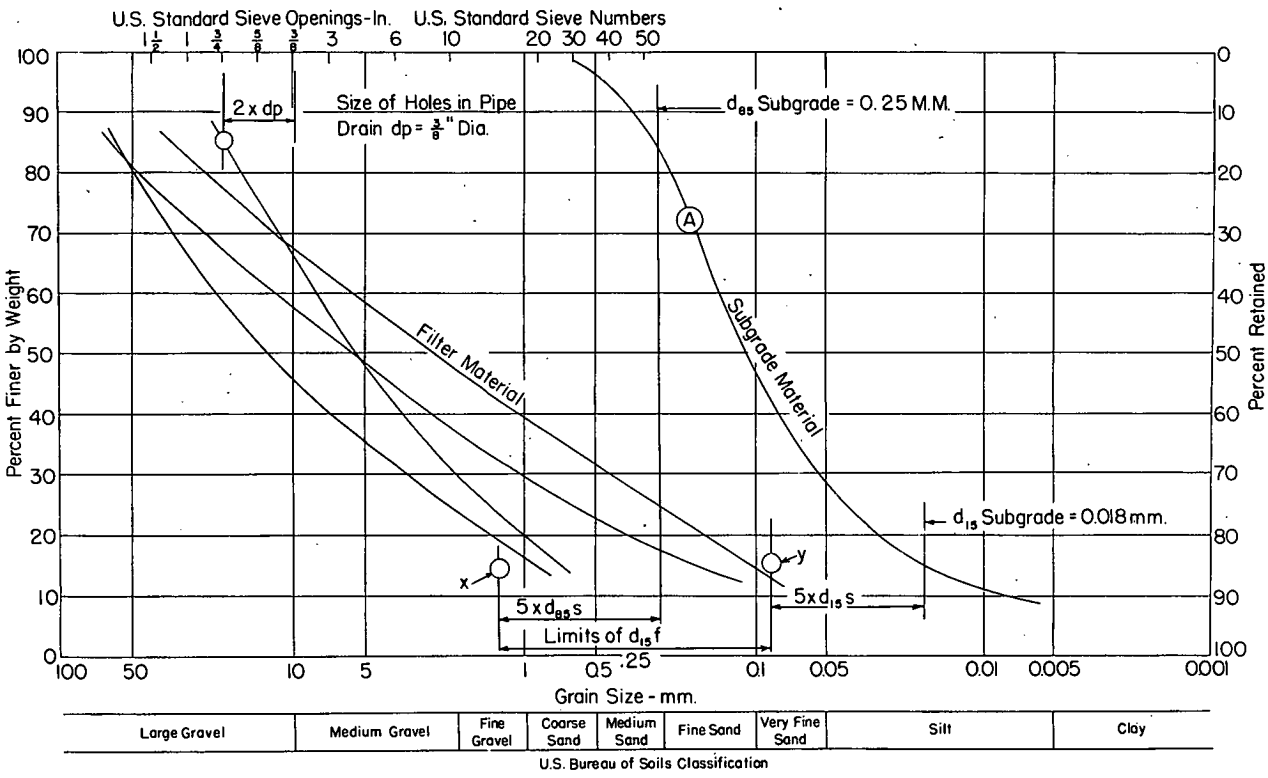


Figure 152
Design Example for Filter Materials
(After Corps of Engineers)

(1) To prevent clogging the pipe with small particles entering the openings, the following ratio of grain size to opening is suggested:

$$\frac{\text{85 percent size of filter material}}{\text{Size of opening in pipe}} \geqq 2$$

(2) To prevent movement of particles in the protected soil the following ratio of size of filter material to size of protected soil is suggested:

$$\frac{\text{15 percent size of filter material}}{\text{85 percent size of protected soil}} \leqq 5$$

"When the protected soil is plastic and without sand or silt partings, the 15 percent size of the filter material need not be less than one mm."

(3) To permit free water to reach the pipe, the filter material needs to be several times more pervious than the protected soil. The following ratio of size of filter material to size of protected soil is suggested:

$$\frac{\text{15 percent size of filter material}}{\text{15 percent size of protected soil}} \geqq 5$$

Ordinarily for most fine, sandy, and silty soils a concrete sand meets the design requirements. If the concrete sand contains a sufficient amount of fine gravel sizes (for pipes with small openings) it will also prevent infiltration into the pipe. An example of design meeting the above criteria is shown in Figure 152.

Excavation and Replacement - Excavation of wet frost-susceptible soils and replacement with non-frost-susceptible granular materials is often the simplest, surest and most economical method of preventing the occurrence of detrimental differential heaving. The literature mentions many instances where the method has been used effectively and economically. The writings reviewed here do not necessarily refer to this method as a design item. In fact most of the references reviewed discuss the subject in broad, general terms, showing typical profiles where it has been used, depth and width of excavation, etc. There is some difficulty in distinguishing from the literature whether the work described is excavation and replacement for a specific location or whether it is more appropriately an item of base-course construction. Nevertheless, an effort has been made to distinguish between the two and to review here the work of excavation and replacement as a design item.

Burton /1931-4 stated that Michigan has corrected serious heaves "simply by removing the undesirable textures and backfilling with a sandy loam obtained from the cut in question, above the level of the heave material." He illustrated typical profiles on which the offending soils were excavated. Three alternate designs for excavation and backfilling, two of which are combined with subsurface drainage, are shown in Figure 153. Figure 153 also gives data on the relative costs of the three types at that time (1931). Burton cautioned that the thickness of the ballasting required depends upon the conditions involved.

Aaron's /1934-3 report of the cooperative studies between Michigan, Minnesota, and Wisconsin and the Bureau of Public Roads showed typical sections of seven subgrade designs involving excavation and replacement. Those have been shown previously under "Subsurface Drainage". Referring to the cooperative studies, Aaron stated that "Except under the extreme climatic conditions of the northern parts of Michigan and Wisconsin, heaving is negligible in treatments where the excavation and backfill is not less than 2 ft. Noticeable but uniform heave has been measured in treatments 1-ft. deep." He cautioned that such treatments should not end abruptly but should be tapered out and that 50-ft. tapers had been the most satisfactory. Lang /1935-3 illustrated a condition in which the grade line passed through inferior topsoils and how those topsoils were excavated and backfilled with selected materials. Morton /1938-10 reported the following measures were being taken in New Hampshire to prevent excessive heaving:

1. Construction of gravel bases 4 ft. deep through silt or clay deposit or the use of a gravel base course 18 in. deep through these areas where uniform frost heave is to be permitted.

2. The construction of gravel bases 12 to 24 in. deep through graded soils.

3. The construction of 6- to 12-in. gravel bases through cohesionless soils.

4. Fill sections less than 4 ft. above the original ground are treated as cut sections. On fill sections over 4 ft., base courses of gravel 6 to 12 in. deep are constructed.

Stokstad /1938-10 brought out the value of mixing non-uniform soils to obtain uniform heaving. He stated that simple excavation was the most common method of treating frost heaving materials in Michigan. He stated the 1938 practice of "backfilling the excavation with material similar to the adjacent soils and then placing a 12-in. sand subbase over the entire area of clayey soils." That is done to a width 4 ft. greater than the proposed road metal and to depths of $2\frac{1}{2}$ to 4 ft., the 4-ft. depth being used only in very bad areas in northern Michigan. A later report /1951-41 again stresses the manner in which mixing of soils has been used to prevent detrimental heaving.

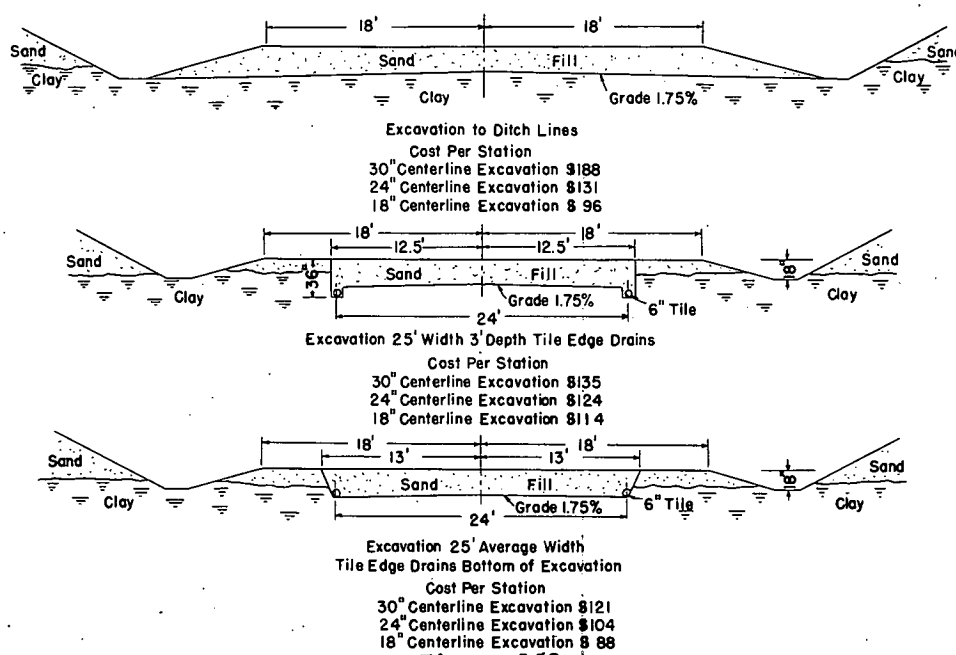


Figure 153
Alternate Methods of Subgrade Design
(After Burton)

L. Casagrande /1938-10 stated that in Germany the most common treatment was the use of pure sand, gravel, or cinders under the pavement. Depth of 15 to 20 in. was usually sufficient. When thinner layers are used he suggested they be protected with a filter layer to prevent penetration of fine materials. Skelton /1940-12 concluded from his studies in New Hampshire that the granular replacement should extend to the full depth of the frost zone for deep deposits of frost dangerous soil, and that a sand bedding course will alleviate most of the mixing of the base course and the underlying finer grained soil. Hansen /1943-12 stated that it was the established practice to excavate to depths of from 2 to 5 ft. in Minnesota and to backfill with granular material. He gave the following specifications for sand-gravel fill.

Percent Passing	Sieve
100	3 in.
over 80	2 in.
0-60	No. 40
0-20	No. 270

Shannon /1944-1 quotes from the Corps of Engineers Manual as follows: "If the subgrade consists of both frost heaving and non-frost heaving soils, highway experience has indicated that all the frost heaving soil should be removed to the full depth of average frost penetration. The area should be backfilled with compacted select material to provide uniformity." A later Corps of Engineers Manual, /1946-5, held that "conditions conducive to irregular heaving occur at locations where subgrades vary from clean sands to silty soils with ground water close to the surface" and suggested that "where conditions are conducive to irregular heaving, freezing of the subgrade should be prevented; this is especially true for soils of the ML and SF groups, /1945-10, for which experience indicates that excessive differential heaving results if full thickness is not employed."

The Michigan Field Manual of Soils Engineering /1946-16 provides basic information on amount of excavation of deposits of highly capillary soils and fine sands. The amounts designated are based on many years' experience and indicate the lineal feet of frost heave excavation for each 1000 ft. of cut, below natural ground elevation, through any particular soil series. The Manual also provides for additional excavation for those soils which require excavation in the transition from cut to fill.

McLeod /1947-22 in his report on airport evaluation in Canada states that "the Department of Transport...endeavors during construction to remove pockets of silt or fine sand occurring in the subgrade, where they are likely to develop frost heaving or frost boils."

Stokstad's most recent report, /1951-41, describes current requirements concerning excavation of materials capable of destructive heaving of pavements. The bottom width of the excavation is 4 ft. wider than the pavement. The depth of excavation ranges from $2\frac{1}{2}$ to 3 ft. in the grey brown Podzolic soils to 3 to 4 ft. in the Podzol soils. These depths indicate average frost penetration. Damage resulting from frost penetration below the depth of excavation has been found to be negligible. Figure 154 shows the Michigan standard section for frost heave excavation. Frost heave excavations are backfilled with soil materials similar to the material surrounding the frost heave pocket. Rock cuts are undercut 1-ft. from ditch bottom to ditch bottom, crowned so as to drain to the sides, and then backfilled with granular material.

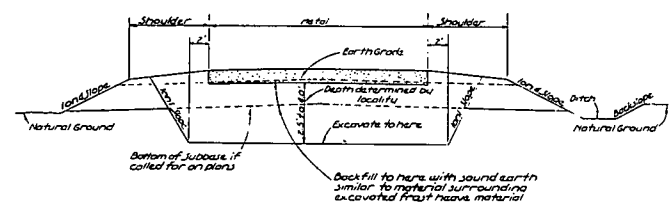


Figure 154
Section Showing Frost Heave Excavation
(After Stokstad)

Fuller /1951-44 recommends that when a cut is made through an impervious, moisture-retaining soil it should be under cut to a depth of at least 40 to 60 percent of the depth of normal frost penetration and backfilled with a layer of properly drained medium sand. Excavation in rock cuts is seldom uniform and may form a rough, non-uniform surface which holds water and is a source of heaving. Fuller describes the use of a fine-grain bituminous mix for backfilling rock excavation areas. The backfill is brought up in well-compacted lifts to form a smooth surface which will drain. He then suggests placing a minimum thickness of 12 in. of run-of-bank gravel between the pavement and the backfill. Fuller also reported that heaving of boulders occurred under pavements in New York State. In the area affected, the subgrade is undercut to a depth of 4 ft. below grade to remove boulders, then backfilled and recompacted.

Norwegian and Swedish engineers (Beskow /1947-12) have long practiced excavation and backfill to prevent excessive heave. The backfill may consist of sand or sandy gravel to depths of 24 in. or more, or may include materials having insulating value. The use of insulating layers is reviewed later. Skaven-Haug /1951-45 reported that uneven swelling of ground due to frost is one of the main problems of the Norwegian railroads, which employ three corrective methods, drainage, lifting of rails (shimming), and soil replacement. Inasmuch as the soils are replaced with materials having an insulating value, these methods are described later.

Lund /1951-42 reports that Nebraska removes the depth of offending subgrade material and replaces it with soil of better quality, or base course material. This method is used in those cases where it is determined that a relatively thin stratum of unsatisfactory soil occurs imme-

diately below the base course and that its removal would solve the problem. The most frequently encountered situations of this type are found on the older roads in the sand-hill area, in cases where the upper 6-in. or so of the original subgrade had been constructed of clay or binder soil for the purpose of providing temporary wearing surfaces. To repair such failures, the clay layer is removed and wasted and fine sand is backfilled in this space and the bituminous pavement replaced.

The method of undercutting and wasting poor subgrade material is also employed in certain instances where deep strata of poor subgrade material cause failures for short distances. Cretaceous shales of the Pierre, Niobrara, Carlyle, Granerous, and Dakota formations are sometimes encountered in excavations of the eastern half of the state. Usually Pleistocene deposits overlie those of Cretaceous age.

If these projects were graded today, the shales would be undercut and replaced with granular materials of the younger formations. But this was not done on the older projects, and as a result, the maintenance forces sometimes find it necessary to perform this operation to avoid being confronted with repeated failures in the same locations after each spring thaw.

Design to Prevent Damage Due to Reduction in Load Carrying Capacity - It has been found that where freezing of wet, fine-grain soil occurs, there is usually associated with frost melting a reduction in the load-carrying capacity of the soils. It would be unwise to attribute all spring-season reduction in load-carrying capacity, even in frost areas, to freezing. Other factors of drainage, precipitation, and conditions of the surface exert their influences on the subgrade moisture content, yet they also aid in setting the stage so detrimental frost action can occur if climatic conditions become favorable. In any instance, subgrade-moisture content has a habit of reaching its yearly maximum during the cold season and carries that high moisture content into the period of thawing.

The designer who designs a road for year-round use needs to know what range in load-carrying capacity he might expect in different soils and base materials in different seasons of a given year, or during a period of several years, and how that range is influenced by frost action. There are several aids to designing for a minimum reduction in bearing capacity during the frost-melting period. Adequate drainage, while usually considered a means of preventing excessive heave, may also prevent excessive reduction in bearing capacity. The most universally-used method is the construction of base courses of material least susceptible to detrimental frost action of adequate thickness and proper cross-section. Drainage of bases may also be beneficial. Capillary cut-off courses may aid in holding down the water content during freezing. Insulation courses may lessen the depth of penetration of frost, and admixtures may be used to prevent freezing.

Granular Bases and Subbases - If a granular base or subbase is used to lessen reduction in bearing capacity it must be non-frost susceptible. Much has already been said in preceding paragraphs to show the relationship between frost action and the various soil characteristics and soil properties as a means of distinguishing frost-susceptible from non-frost-susceptible materials. That will not be reviewed here. Only those limits stated directly in design criteria and those inferred from design data are given here. They include data on characteristics of soils (including granular materials) and on climate (depth of freezing or intensity and duration of freezing temperatures).

Texture of bases and subbases - The Michigan Field Manual of Soils Engineering /1946-19 states that granular subbases have two principal functions: (1) to prevent concentration of water under the travelled roadway, and (2) to distribute wheel loads. It states that "the best material is coarse sand or sand-gravel mixture having less than 7 percent passing the 200 mesh sieve".

The Civil Aeronautics Administration Standard Specifications /1948-42 shows the following textural grading and plasticity requirements for bases and subbases:

Bases

Type of Aggregate	Percent Passing No. 40 Sieve	Percent Passing No. 200 Sieve	L.L. /b	P.I. /b
Aggregate	10-30	5-15 /a	30 max.	6 max.
Crushed Aggr.	10-25	3-10 /b	25 max.	6 max.
Caliche	15-35	0-15	35 max.	10 max.
Shell	-----	0-15	25 max.	8 max.
Sand Clay, Fine Aggregate Type	35-70	8-25 /a	25 max.	4 max.

/a The amount of the fraction passing the No. 200 sieve shall not exceed one-half of the fraction passing the No. 40 sieve.

/b Values based on testing in accordance with A.A.S.H.O. Methods T-89, T-90, and T-91

Subbases

Type A Subbase (10-in. or greater frost penetration)

Sieve designation	Percent passing by weight
3 inch	100
No. 40	Not more than 70

The material passing the No. 10 mesh sieve shall meet the following requirements:

No. 10	100
No. 40	25-70
No. 200	0-15

Liquid limit 25 maximum, plasticity index 6 maximum. If more than 45 percent of the entire sample is retained on the No. 10 mesh sieve, the amount of material passing the No. 200 mesh sieve may be increased to 25 percent if the liquid limit and the plasticity index are not increased.

Type B Subbase (less than 10-inch frost penetration)

Sieve designation	Percent passing by weight
3 inch	100
No. 40	Not more than 70

The material passing the No. 10 mesh sieve shall meet the following requirements:

No. 10	100
No. 40	25-70
No. 200	0-35

Liquid limit 35 maximum, plasticity index 9 maximum. If more than 45 percent of the entire sample is retained on the No. 10 mesh sieve, the amount of material passing the No. 200 mesh sieve may be increased to 45 percent if liquid limit and plasticity index are not increased.

The Corps of Engineers Manual /1951-51 holds that the potential intensity of ice segregation in a soil is dependent in a large degree on its void sizes and expresses frost susceptibility in terms of grain size as follows:

Inorganic soils containing three percent or more of grains finer than 0.02 mm. in diameter by weight are generally frost-susceptible. Although uniform sandy soils may have as high as 10 percent of grains finer than 0.02 mm. by weight without being frost-susceptible, their tendency to occur interbedded with other soils makes it generally impractical to consider them separately.

The Manual classifies frost susceptible soils into four groups, in the order of increasing susceptibility. (The textural descriptive names are defined in accordance with the Airfield Classification /1945-10.)

Frost-susceptible soils have been classified into the following four groups, listed in order of increasing susceptibility. The soils in Group F4 are of especially high frost susceptibility. Soil names are as defined in the Corps of Engineers Uniform Soil Classification.

Group	Description
F1	Gravelly soils containing between 3 and 20 percent finer than 0.02 mm. by weight.
F2	Sands containing between 3 and 15 percent finer than 0.02 mm. by weight.
F3	(a) Gravelly soils containing more than 20 percent finer than 0.02 mm. by weight and sands, except fine silty sands, containing more than 15 percent finer than 0.02 mm. by weight. (b) Clays with plasticity indexes of more than 12, except varved clays.
F4	(a) All silts including sandy silts. (b) Fine silty sands containing more than 15 percent finer than 0.02 mm. by weight. (c) Lean clays with plasticity indexes of less than 12. (d) Varved clays.

Varved clays consist of alternating layers of inorganic silts and clays and in some instances fine sand. The thickness of the layers rarely exceeds one-half-inch, but occasionally very much thicker varves are encountered. The constituents of varved clays were transported into fresh-water glacial lakes by melt waters at the close of the ice age. They are likely to combine the undesirable properties of both silts and soft clays. Varved clays are likely to soften much more readily than homogeneous clays with equal average water contents.

Inorganic soils containing less than 3 percent of grains finer than 0.02 mm. in diameter by weight are generally non-frost-susceptible.

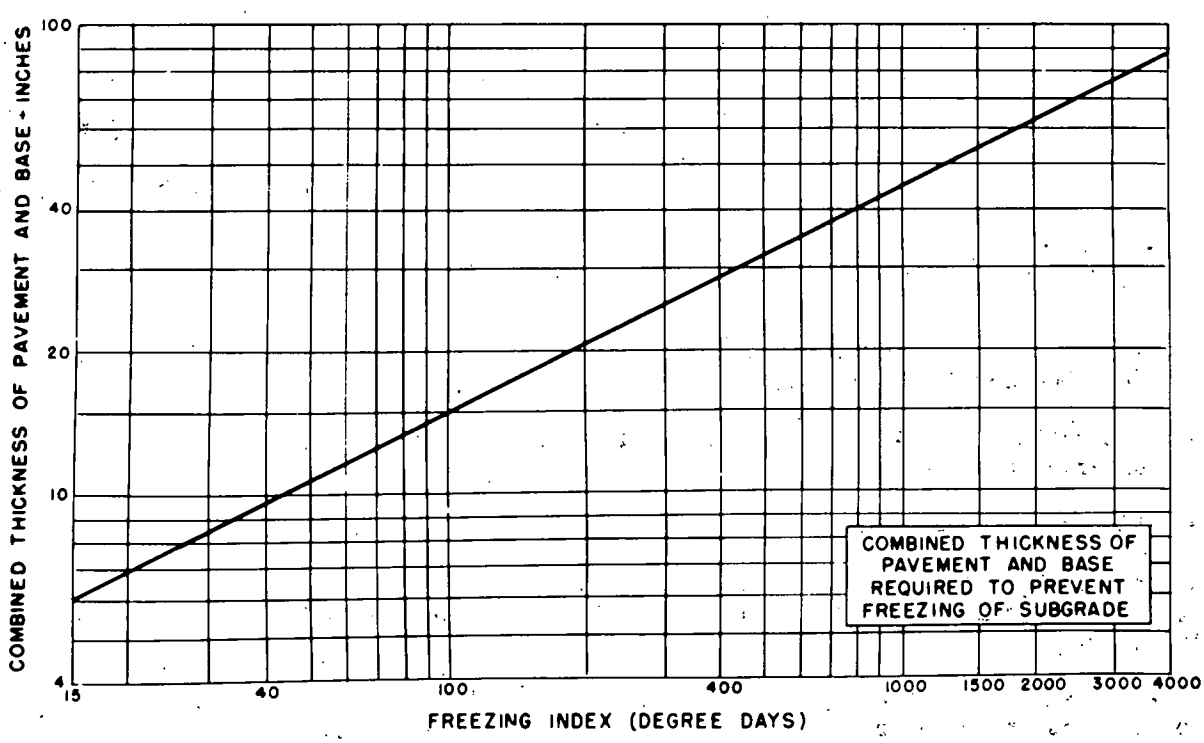


Figure 155
Combined thickness of pavement and base required to prevent freezing of subgrade. (After Corps of Engineers)

The Corps of Engineers Manual requires that all base-course materials lying within the depth of frost penetration shall be non-frost-susceptible. Where the combined thickness of pavement and base over a frost-susceptible subgrade is less than the depth of frost penetration determined from Figure 155 the following additional design requirements shall apply:

a. For both flexible and rigid pavements, the bottom 4 in. of base shall consist of any non-frost-susceptible gravel, sand, or crushed stone and shall be designed as a filter between the subgrade soil and overlying base-course material. The gradation of this filter material shall be determined in accordance with criteria presented in paragraph 2-11 of Chapter 2, Part XIII, of the Engineering Manual, with the added overriding limitation that the filter material shall in no case have more than 3 percent by weight finer than 0.02 mm.

b. For rigid pavements, the 85 percent size of filter or regular base-course material placed directly beneath the pavement shall be equal to or greater than ½ in. in diameter.

The purpose of the latter two requirements is to prevent mixing of the frost-susceptible subgrade with the base during and immediately following the frost-melting period and to prevent loss of support by pumping soil through the joints of rigid pavements.

The Corps of Engineers Manual, The Michigan Manual, and the Civil Aeronautics Administration Manual and Specification are the only manuals for design which the reviewer has found which state definite values of grain size or grading of materials for bases or subbases for use where ground freezing occurs. The reviewer recognizes that consideration of frost action has been given in setting up the A.A.S.H.O. gradings (AASHO Designation M 56-42 materials for stabilized base course and M 75-42, crushed stone and crushed slag for base course) for base courses and also that consideration of frost action has been given in setting grading limits for base courses in most state highway specifications. However, the reviewer has found only few published data on textural gradings which make specific reference to frost action, and those are reviewed herein.

General information on design of base courses - Under this heading is presented general information and data on dimensions of bases used and experience with various designs. Formulated design methods used to determine design thicknesses are given in later paragraphs.

The literature contains many writings which include statements of a general nature on drainage, textural grading, thickness, width, and other characteristics of base courses. The reviewer makes no pretext of having made a complete search of that literature but believes the following brief reviews and quotations arranged chronologically will give the reader a fair cross-section of what the writings contain.

Harrison /1918-2 believed many of the designs used up to that time were faulty because they did not take into account the freezing of subgrades. He held that a design should include the following points:

1. Ditches should be deep enough to prevent any leakage from them into any part of the subgrade which thaws during the winter.

2. The base should permit drainage to the ditch of any water infiltrating through the shoulders or cracks in the pavement.

3. The combined thickness of base and surface should at least equal the total depth of any winter thaws.

4. All ditches should have sufficient slope to permit rapid surface drainage.

Wilson /1918-3, in discussing Harrison's writings, believed the solution lay in the use of drains (see Subsurface Drainage) placed sufficiently deep to lower the zone of saturation well below the subgrade.

An American Road Builders Association Committee on Subgrades and Pavement Bases /1930-16 cited the success which had been had in Rhode Island with the use of 6-to 24-in. depths of 1½ in. to 20-mesh pit-run sand-gravel as a "subbase." The "subbases" were built 20 feet wide for an 18-ft. road. They stated it seemed to exhibit the following benefits as regards frost action:

1. Eliminates capillary water movement for its depth and decreases the extent of heaving.

2. Eliminates strata, pockets, and laminated layers of unequal heaving material to the depth of the subbase and thus tends to equalize heaving, decreasing the detrimental action, and

3. Decreases the depth of frost penetration by an amount equal to its thickness.

The Committee cautioned that porous subbases, unless well drained, may act as reservoirs for water and be detrimental.

Aaron /1934-3 reported the results of cooperative studies by the Bureau of Public Roads and Michigan, Minnesota, and Wisconsin. Many of the designs used were considered to be replacement of excavated frost heave material. Therefore, his report has been reviewed under "Excavation and Replacement."

Paradis /1934-4 reported some experiences in Canada with sand bases. He advocated the use of sand "as it has a much lower thermal conductivity than any other road building material." He reports use of 6-to 12-in. depths, the most-used depth being 8-in. The bases are built either in 22-ft. trench sections (for 20-ft. road) or full width (32 ft. for 20-ft. pavement). The results have been satisfactory. Some of the earliest "sand cushions" of 6-in. depth were used in Quebec in 1918 and 1922. Seven additional projects were built in 1932-34 with thicknesses ranging up to 12 in. Where the sand cushion was used the roads were reported in good condition.

Keil /1938-3 reported the use of coarse sandy gravel or fresh granite quarry waste ($\frac{1}{2}$ in. to $2\frac{1}{2}$ in.) in depths of 6, 12, 18 and 24 in. and to a width of 30 ft. (road surface width 24-ft., 6 in.) in Germany. Pipe drains were provided throughout under the outer margins and transverse channels 4 in. wide and 2 in. deep were cut in the soil at the bottom of the excavations for the base courses. The surfacing consisted of 8 in. of concrete with 33-, 43-, and 56-ft. slabs. The average subgrade soil had 11 percent colloids; 42 percent clay (0.002 to 0.02 mm.); fine silt (0.02 - 0.05) 37 percent; coarse silt (0.05 - 0.5) 5.5 percent and the remainder sand. It showed a capillary rise of 26 ft. and had a maximum and minimum permeability of 3.35×10^{-4} and 2.35×10^{-7} respectively. Maximum and minimum P.I. 28.8 and 7.6 and a moisture content ranging from 12.3 to 29 percent. As a result he reached the following conclusions regarding use of "protective" courses (base courses) from the study of the experimental road: "The main line of defense against frost is a protective course of gravel or ballast below the surfacing. The required minimum thickness of a protective course on cohesive soil, assuming a frost of two months' duration and a frost penetration depth of 32 in., is 18 in". He did not believe that thickening the slab will overcome deficiences in the subsoil.

Morton reported /1938-10 using thick granular bases to prevent frost damage. Data on their use are given under "Excavation and Replacement." Stokstad /1938-10 reported using 12-in. sand subbase over the entire subgrade area of clayey soils in Michigan. The Michigan Soils Manual /1946-16 states that the thickness of the base varies with the type of subgrade material and the traffic, but "in no case should the granular base be less than 12 in. thick." More recently /1951-41 he reemphasizes that the most effective means for controlling spring breakup is an adequate granular subbase. Most subbases in Michigan are built 12 to 18 in. thick except an 8-in. depth is sometimes used where advantage can be taken of an existing gravel road. The material used for this purpose is the best sand or gravel most readily available.

L. Casagrande /1938-10 reported the use of a 15-to 20-in. sand layer in Germany in 1938. A little later /1940-8, after his study of the effect of the severe winter of 1939-1940 on German roads (frost penetration to depths over 40 in.), he concluded that, if for reasons of economy complete protection against a very hard winter is not generally possible, an insulation course of sand, gravel, or slag should be provided to a minimum thickness of 24 to 32 in. for motor roads and 16 to 24 in. for provincial roads. Tremper /1938-10 reported the use of blankets from 12 to 24 in. deep in eastern Washington, although frost penetrates considerably deeper and frost boils are of common occurrence on "unballasted" roads. He reported using a layer of sand or rock screenings on the subgrade to prevent infiltration into the overlying ballast. Hansen /1943-12 stated that "for preventing frost boils where differential heaving does not occur, excavation and backfill of granular material to depths of from 1 to 2 ft. has usually been satisfactory. Shannon /1944-1 cited Chapter 20 of the Corps of Engineers Manual then in use, which required that base courses in areas subject to frost action "extend to a depth of at least 50 percent of the average penetration in order to provide suitable reinforcement to take care of reduction in bearing." The base course thickness so determined will govern only when it exceeds the thickness governed by other requirements.

Minnesota /1945-8 constructed a number of test sections and experimental projects using different types and thicknesses of base courses (up to 2 ft.) under Portland cement concrete pavements to prevent the occurrence of high joints due to differential heaving resulting from greater soil moisture contents under leaky joints and cracks. The effectiveness of base courses ranging in thickness from 3 to 9 in. is indicated in Table 51. It was concluded that granular base courses 9 or more inches thick proved successful in Minnesota in preventing pavement distortions attributable to frost action.

TABLE 51

Type of base	Expansion Joints				Contraction Joints				Length of section (ft.)
	No. of Joints	Per cent warped			No. of Joints	Per cent warped			
		½ in. to ¾ in.	¼ in. to ½ in.	0 in. to ¼ in.		½ in. to ¾ in.	¼ in. to ½ in.	0 in. to ¼ in.	
3-in. sand over 3-in. stab. sand	66	0	0	100	129	0	2.33	97.67	6,151
6-in. sand base	56	0	1.8	98.2	111	0	1.8	98.2	5,221
9-in. sand base	58	0	0	100	112	0	0.9	99.1	5,280
3-in. stabilized sand base	47	0	0	100	95	0	1.1	98.9	4,469

In more recent writings Beskow /1947-12 stressed that the main cause of road damage is the softening of soil from melting ice lenses, causing a reduction in bearing capacity of the soil: Scandinavian design measures are aimed principally at guarding against that softening and use sand and sand-gravel for that purpose. Experience has shown that the minimum depth from road surface to bottom of sand-fill for Swedish highway traffic (2½-ton wheel load for flexible type pavements) is 60 cm. (23.6 in.) for very frost sensitive soils (silts and light silty clays) 50 cm. (19.7 in.) for moderately sensitive soils (clay and moraine soils) (Note: those thicknesses correspond approximately to Corps of Engineers values for 2.5 ton wheel load for C.B.R. values of 1.0 and 1.5 respectively).

Beskow held that the sand layer cuts the capillary connection between the overlaying soil and the subgrade, preventing capillary suction to the frost line. He stressed that the sand layer must not become water saturated but must be above the free-water table. Two examples of how granular materials are used are given in Figure 156. In alternative 2 (Figure 156) the sand may be a comparatively thin layer, the minimum thickness permitted being 12 cm. (4.7 in.). The refill of very frost sensitive soil (silt) should not exceed 20 cm. (7.9 in.). That method presumes good vertical drainage and that the distance to ground water exceeds the capillarity of the sand by at least 10 cm. (3.9 in.). He suggested that for ordinary, thick sand subbases even a fine or "dirty" sand can be used if it is non-frost-heaving. However the thin sand cut-off layer must be clean, coarse sand.

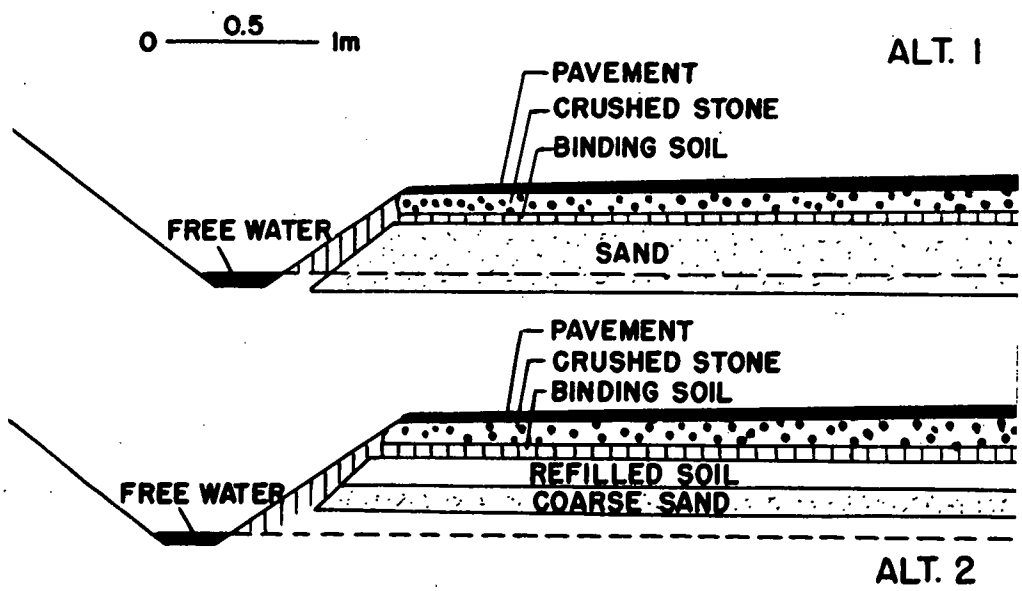


Figure 156
Use of sand layer to prevent frost damage. (After Beskow)
Alternate 1: Thick sand fill, acting as a non-frost sensitive backfill and subbase.

Alternative 2: Thin insulating bottom layer of rather coarse sand, acting as an isolation layer, breaking capillary connection with overlying refilled frost sensitive soil material.

Godskesen /1948-23 brought out that the total frost ranges from 6500 deg.-hr. to 12,400 deg.-hr. in Denmark. That is equivalent to 487.5 to 930 Farenheit-deg.-days. Frost penetrates from 1 to 1.85 meters (3.28 to 6.1 ft.). He reported that roads having a 37 cm. (14.6 in.) cover have suffered only minor damage. Others with 40-45 cm. (15.7 - 17.7 in.) cover have suffered "considerable damage." Roads having covers up to 50-55 cm. (19.7 - 21.6 in.) were rarely affected while those having 60 cm. or more (23.6 in.) cover were safe against frost damage. A more recent report /1950-3 showed that although frost may be expected in some cases to penetrate to a depth of 6 ft., damage is not expected to result, even in frost sensitive soils, throughout this full distance. Depths below which damage is unlikely to occur are: in unsurfaced roads, 40 in; in railway tracks, 20 to 24 in; in concrete surfaced roads, 20 in; and in roads with other types of surfacing, 24 in. Riis /1948-25 reported Denmark is building subbases 60 cm. thick in cuts (23.6 in.) and 45 cm. on fills (17.7 in.)

Bleck /1949-11 in his report of extensive observations on the performance of "ballast" (free-draining granular bases and subbases) in Wisconsin found that ballast courses ranging from 9 to 15 in. thick "are now considered for general use for both rigid and flexible type pavements in most cases where the use of ballast courses are indicated by the soil-moisture-freezing and subsequent reactions. In those areas where extreme differential frost heave conditions are manifested these can be increased to be in the range of from 12 to 15 or 18 in." He states further that in Wisconsin "the practice now is to stabilize the upper surfaces of these ballast courses with several inches of crushed and graded gravel or crushed stone, the thickness ranging from 3 to 5 in., depending upon the traffic characteristics of the highway."

Stokstad /1951-41 believes the use of granular bases to be the most effective means for controlling spring breakup. He stated that most subbases in Michigan are built 12 to 18 in. thick, except that an 8 in. depth is sometimes used where advantage can be taken of an existing gravel road. The material used is the best sand or gravel readily obtainable. Figure 157 illustrates the type of section commonly employed.

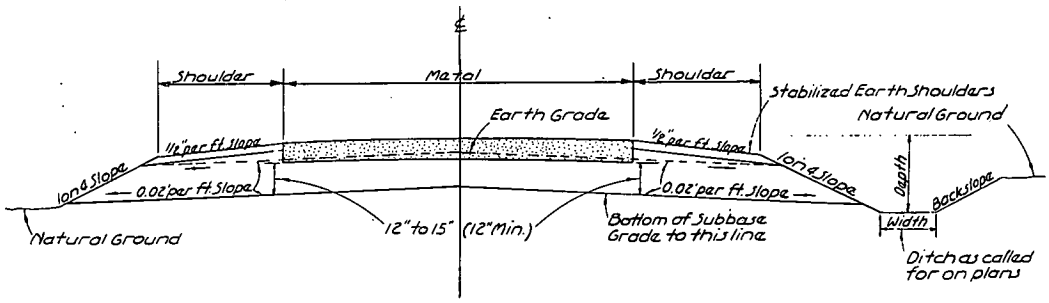


Figure 157
Section Showing Granular Sub-base
(After Stokstad)

Lund /1951-42 reports that in Nebraska base course mixtures are "designed with a view towards providing sufficient stability during the spring thawing period." In this connection, it should be pointed out that the aggregates available in Nebraska for use in base-course construction are almost wholly the rounded water-worm sands and gravels which were rolled from the Rocky Mountains at the bottoms of Tertiary and Pleistocene channels. Some coarse sands found in the Nebraskan and Kansan Till sheets are slightly more angular in shape. All of these materials are relatively fine in texture, having maximum fineness moduli of about 5. With such aggregates particular attention must be given to gradation. The trend through the years in design has been towards less binder, and the concensus now is that these base courses should be laid with the minimum clay content necessary for stability during construction. The following table indicates the typical test characteristics of granular mixtures considered to be stable during and after the thawing period, and at the same time having sufficient cohesion so that surface course mixing and laydown operations can proceed on the compacted base course:

TABLE 52

TYPICAL BASE MIXTURES

PI	Percent Sand	Percent Silt	Percent Clay	Percent Passing No. 4	No. 10	No. 20	No. 40	No. 100	No. 200
2	94	4	2	75	50	35	25	12	9

Four different admixtures have been used in granular base courses in Nebraska in attempting to provide sufficient load-carrying capacity through the thawing period. These are asphaltic oils, tar, Portland cement, and calcium chloride. Sufficient success has been had with the first of these that its use has become commonplace. It should be pointed out that more minus 200 material (15-20 percent) is added to these base course mixtures.

Granular base courses for concrete pavements are constructed in Nebraska in all cases where the percentage of minus 200 material in the subgrade soil is greater than 35 or the plasticity index is higher than 6.

Keene /1951-43 states that the object of designs and treatments in Connecticut is not to eliminate frost action but to reduce it so maintenance will be small and load restrictions will not be necessary.

On new construction the primary requirement is an adequate subbase, as an additional amount cannot be added after the pavement is placed. The subbase is invariably clean, bank-run gravel; crushed stone with a sand filler is also included in the specifications but is never used because the gravel is abundant and probably will continue to be for a considerable time. The gravel specification is as follows:

Sieve size	5 in.	1/4 in.	No. 40	No. 100
Percent Passing	100	30 to 65	5 to 30	0 to 10

The portion passing the No. 100 sieve shall not have sufficient plasticity to permit performing the plastic limit test, A.A.S.H.O. Number T-90.

Frost action in such gravels appears to be negligible. Little difficulty has been experienced in obtaining gravel to meet this severe specification, except in a narrow area of about 200 sq. mi. around Hartford where the red gravels (of Triassic origin) have considerable clay. Specifications for subbase in use before 1942 were identical with A.A.S.H.O. Specification M-147-49 for base course; some of the red gravels passed that specification and were used at that time and subsequently have given some trouble in slight heaving, break-up, pumping at joints, etc., but probably the chief reason for changing to the more severe specification is that the better gravels are abundant.

Gravel-base course and gravel-surface (wearing) course, each 4 in. thick and together constituting the pavement, have the same specification as for gravel subbase, except that 100 percent shall pass the 3½ in. sieve.

The lack of silt or clay binder in these pavement courses may be questioned by highway engineers. Such binder appears unnecessary in Connecticut highways as the gravel has excellent gradation and thus can be densely compacted, to about 130 or 135 lb. per cu. ft. and also the surface is bound by oil, tar, or asphalt.

On new construction of primary roads, such as parkway, federal aid, and trunk line, the combined thickness of pavement (surface and base) plus subbase in earth cuts is from 20 to 32 in. which is 2/3 or 3/4 of frost penetration; the combined thickness is 32 in. in rock cuts and about 14 in. in earth fills. The last may be reduced to 8 in. if cuts exceed fills. On secondary roads these thicknesses are reduced slightly. On tertiary roads (town aid), these thicknesses of pavement plus subbase total 8 to 20 in. depending on conditions. In low fills over wet ground, subbase requirements are the same as for earth cuts. The reason for the latter is that the water table is as close to the roadway surface (say 5 ft.) at the low wet fill as it is in a wet cut with

under-drainage. Therefore, unless such a fill is composed of clean material throughout its height, conditions for frost action are about the same in both cases and subbase requirements are the same.

Depth of subbase also is influenced by soil types, the greater depths given above being used in silts and often in clays and the lesser depths in tills. In clean sand and gravels, subbase is reduced to 6 in. or zero.

Subbase is carried out to the gutter on each side in cuts and to full width of shoulders in fills. Also the thickness in cuts is carried without diminution into fills for a minimum distance of 50 ft. before tapering down to the normal thickness in the fills. In the past this was not done, and bad heaves resulted in the topsoil at the grade points.

Pavement Thickness Design Methods - Several State highway departments /17 and Federal departments have developed criteria and methods evaluating the influence of frost for the design of pavement thickness. The methods are reviewed separately according to type of pavement.

Flexible type pavements

Civil Aeronautics Administration Method /1948-13

The Civil Aeronautics Administration Manual on Airport Paving /1948-13 states that it is possible to "draw up an engineering classification of soils related to their field behavior" and presents a classification which forms a part of the C.A.A. method of determining thickness of pavement and base. The method is based on the assumed use of bases and subbases meeting appropriate specifications /1948-43 for the type of base and subbase material used. The specifications are reviewed under "Textural Grading of Base Courses." The method is also based on certain construction requirements (compaction, lift, thickness, etc.) not reviewed herein but nevertheless which should be given consideration if a comparison more detailed than space permits here is to be made with other methods. The thickness design is also based on certain conditions of drainage and frost discussed later.

The soil-classification system is based on only three tests: mechanical analysis, liquid limit, and plastic limit (for P.I.). The limits of each of the three test values defining the various soil groups are given in Table 53. Figure 158 aids in classification of fine grained soils.

Only the portion of the sample passing the No. 10 sieve is considered in classifying a soil. However, provision is made for upgrading a soil from 1 to 2 classes when the "total sample retained on the No. 10 sieve exceeds 45 percent for soils of the E-1 to E-5 groups and 55 percent for the others, provided the coarse fraction consists of reasonably sound material," and is well graded down to the No. 10 sieve.

Inasmuch as soil texture is an indication of the permeability of a soil, a separate textural classification, similar to that of the U. S. Department of Agriculture, is used to determine the need for and location of underdrains. The methods used or the limits which govern drainability of soils are not given.

Table 53 serves to classify the soil under conditions of good drainage or poor drainage for no frost or severe frost. The appropriate references to the different conditions as they affect design thickness of rigid type pavements are given under the subject "Rigid Type Pavements."

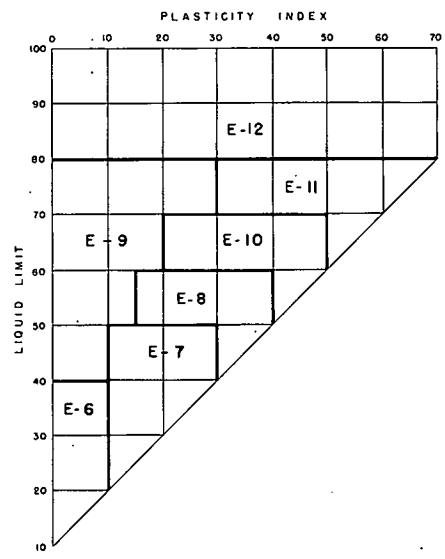


Figure 158. Classification Chart for Fine Grained Soils
(After Civil Aeronautics Administration)

/17 Only those whose methods have been published are reviewed here.

"To distinguish between good and poor drainage requires a knowledge of the topography of the site, the properties and arrangement of the different layers of the soil profile, and the elevation of the water table. Poor drainage is defined as a condition where the subgrade may be rendered unstable owing to (1) inadequate internal drainage caused by the character of the soil profile, (2) capillary rise from a high water table, (3) topographic features, such as a flat terrain at elevations only slightly above sea level, or (4) any other cause that may result in instability or produce saturation of the subgrade.

"Good drainage may be defined as a condition where (1) the internal drainage characteristics are such that there will be no accumulations of water that would develop spongy areas in the subgrade, (2) the water table is at such an elevation that the soil will not become water logged either by percolation from above or capillarity from below, and (3) the topography is such that surface water will be removed rapidly. These conditions can be met by any of the soil groups with the exception of E-13.

"Good drainage cannot be distinguished from poor drainage on the basis of test results alone. This criterion has been used in the past and has led to the erroneous conception that a good drainage condition cannot exist when the soil consists of clays and silty clays of the E-7 to E-12 groups. There is no logical reason for such an assumption, although it is recognized that soils of these groups are expansive and subject to detrimental volume change with variations in moisture content. The degree to which these detrimental properties may develop will depend on the drainage conditions as indicated by a study of the topography and the entire soil profile.... It is important to remember that while the soil group may be determined by means of the laboratory test results, the drainage conditions must be established by the surface and subsurface characteristics of the site.

"With respect to frost action, a 'severe frost' condition exists if the depth of frost penetration for the particular site is greater than the anticipated thickness of surfacing, base and subbase as determined for 'no frost' and the drainage condition as defined above. Otherwise the condition of 'no frost' prevails.

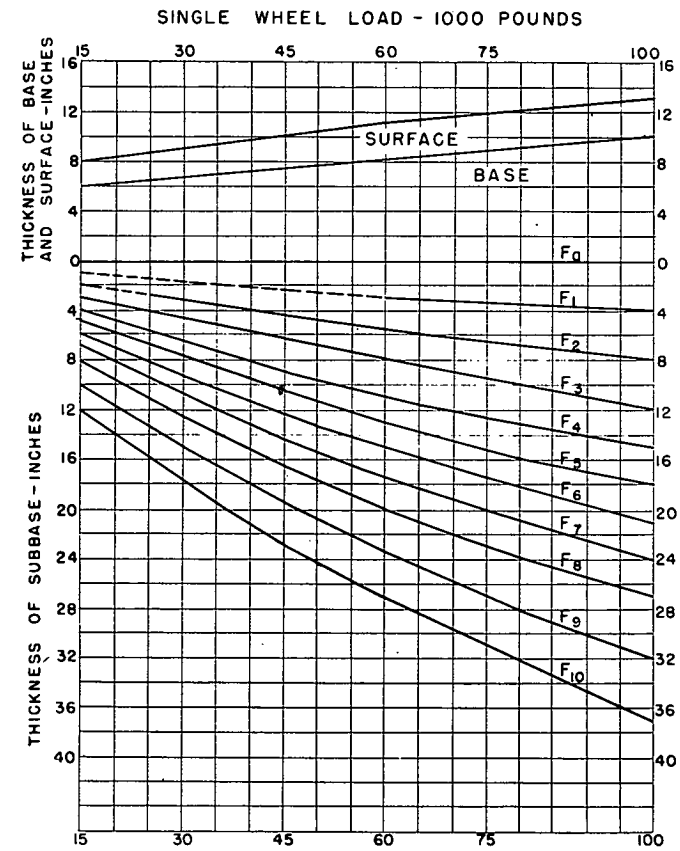


Figure 159. Design Chart for Flexible Type Pavement with Non-Bituminous Base for Airfield Runways. (After Civil Aeronautics Administration)

"Some explanation may be necessary relative to a combination such as 'good drainage' and 'severe frost'. It is obvious that a good drainage condition may exist in a location where the average depth of frost penetration exceeds the required thickness of surface, base and subbase for a 'no frost' condition. In this case, the subgrade class would be determined on the basis of 'severe frost'. The importance of 'severe frost' in a location having good drainage has been questioned. From the standpoint of detrimental frost heave, it may not be important but it is well known that even under the best drainage conditions, the penetration of frost below the non-heaving subbase material, together with alternate freezing and thawing, produces a softening of the subgrade soil immediately under the subbase. The only remedy for this situation is additional subbase thickness to overcome the loss in stability."

The Civil Aeronautics Administration method of design /1948-13 provides flexible pavement thickness designs for single wheel loads from 15,000 to 100,000 lbs. Design curves are given for (1) non-bituminous base, runways; (2) non-bituminous base, taxiways, aprons and runway ends; (3) bituminous base, runways; (4) bituminous base, taxiways, aprons, and runway ends; (5) bituminous coated aggregate base, runways; (6) bituminous coated aggregate base, taxiways, aprons and runway ends.

TABLE 53

CLASSIFICATION OF SOILS FOR AIRPORT CONSTRUCTION

Soil Group	MECHANICAL ANALYSIS				Liquid Limit	Plasticity Index	SUBGRADE CLASS			
		Material Finer Than No. 10 Sieve					Good Drainage		Poor Drainage	
	Retained on No. 10 Sieve/a	Coarse Sand Pass No. 10 Ret. No. 60	Fine Sand Pass No. 60 Ret. No. 270	Combined Silt & Clay Pass No. 270			No Frost	Severe Frost	No Frost	Severe Frost
	Percent	Percent	Percent	Percent						
E-1	0-45	40+	60-	15-	25-	6-	Fa R1a	Fa R1a	Fa R1a	Fa R1a
E-2	0-45	15+	85-	25-	25-	6-	Fa R1a	Fa R1a	F1 R1a	F2 R1a
E-3	0-45	--	--	25-	25-	6-	F1 R1a	F1 R1a	F2 R1a	F2 R1a
E-4	0-45	--	--	35-	35-	10-	F1 R1a	F1 R1a	F2 R1a	F3 R2a
E-5	0-45	--	--	45-	40-	15-	F1 R1a	F2 R1b	F3 R1b	F4 R2b
E-6	0-55	--	--	45+	40-	10-	F2 R1a	F3 R2b	F4 R2b	F6 R2b
E-7	0-55	--	--	45+	50-	10-30	F3 R1b	F4 R2b	F6 R2b	F7 R2c
E-8	0-55	--	--	45+	60-	15-40	F4 R1b	F6 R2c	F7 R2c	F8 R2d
E-9	0-55	--	--	45+	40+	30-	F5 R2b	F7 R2c	F7 R2c	F9 R2d
E-10	0-55	--	--	45+	70-	20-50	F5 R2b	F7 R2c	F8 R2c	F9 R2d
E-11	0-55	--	--	45+	80-	30+	F6 R2c	F8 R2d	F9 R2d	F10 R2e
E-12	0-55	--	--	45+	80+	--	F8 R2d	F9 R2e	F10 R2e	F10 R2e
E-13		Muck and Peat - Field Examination					Not Suitable for Subgrade			

/a Classification is based on sieve analysis of the portion of the sample passing the No. 10 sieve. When a sample contains material coarser than the No. 10 sieve in amounts equal to or greater than the maximum limit shown in the table, a raise in classification may be allowed provided the coarse material is reasonably sound and fairly well graded.

Figure 159 illustrates the design curves for (1) non-bituminous base-runways. In Figure 159 the Fa line represents the subgrade class which requires no base. Referring to Table 53 it will be seen that the Fa subgrade class applies to all conditions of drainage and frost when the soils fall in the E-1 group, and only under good drainage conditions for soils of the E-2 group. When the subgrades fall in Classes F1 to F10, subbases of varying thickness are required.

A soil of a particular group may fall in several subgrade classes, depending on drainage and frost conditions. For example, soils of the E-5 group may be classed as F1 subgrade for good drainage and no frost, F2 for good drainage and severe frost, F3, for poor drainage and no frost, and F4 for poor drainage and severe frost. An F4 subgrade, on the other hand, is the result of one of the following combinations:

Soil groups	Drainage	Frost
E-5	Poor	Severe
E-6	Poor	No
E-7	Good	Severe
E-8	Good	No

The method of developing the design curves is entirely empirical and is based on service records of pavements on different types of soils under variable drainage and climatic conditions.

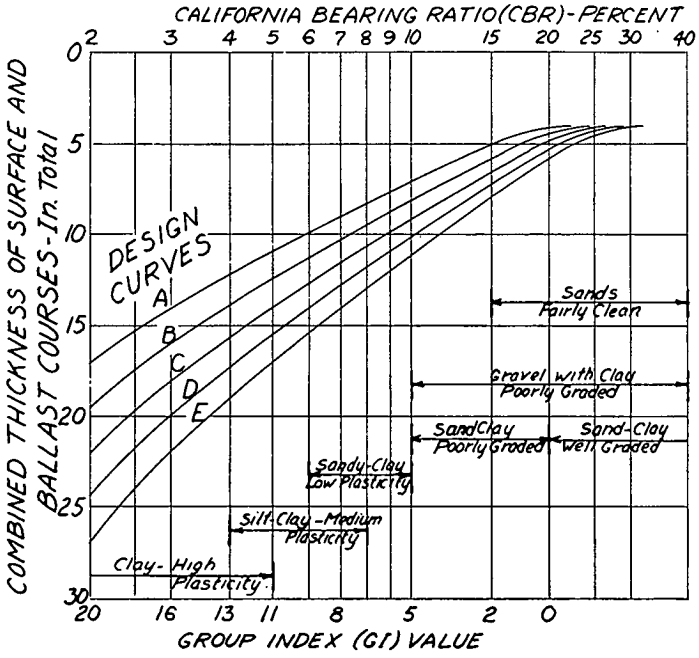


Figure 160. Colorado State Highway Department Design Chart for Thickness of Surface and Ballast Courses - Note: This is a dual purpose chart to indicate required thicknesses using either CBR or GI values. The coincidence of the values on the chart does not mean that they are equal. When design is based on CBR, the GI values should be ignored and vice versa.

ITEM	LIMITS	VALUE
ANNUAL PRECIPITATION	5 TO 10 INCHES	0
	10 TO 15 INCHES	1
	15 TO 20 INCHES	3
	20 TO 25 INCHES - OR LIGHT IRRIGATION	6
	25 TO 50 INCHES - OR HEAVY IRRIGATION	10
WATER TABLE	NONE EVIDENT	0
	10 TO 6 FEET BELOW GRADE	1
	6 TO 4 FEET BELOW GRADE	3
	4 TO 2 FEET BELOW GRADE	5
FROST ACTION (INDUCING HEAVE)	NONE	0
	LIGHT	1
	MEDIUM	3
	HEAVY	8
EXISTING CONDITIONS	EXCELLENT	0
	FAIR	2
	ADVERSE	6
TRAFFIC (DESIGN REPETITIONS TRAFFIC IN ONE DIRECTION - IN EQUIVALENT 5000 POUND WHEEL LOADS)	0.0 TO 1.0 MILLION	0
	1.0 TO 2.0 MILLION	2
	2.0 TO 3.0 MILLION	4
	3.0 TO 5.0 MILLION	6
	5.0 TO 7.0 MILLION	9
	7.0 TO 9.0 MILLION	12
	9.0 TO 11.0 MILLION	15
	11.0 TO 13.0 MILLION	18
	13.0 TO 15.0 MILLION	21
	15.0 PLUS MILLION	24

MINIMUM CENTER SECTION OF SELECTED MATERIAL SURFACING PLUS BASE COURSE SURFACING PLUS TOP COURSE SURFACING WITHIN TRAFFIC GROUP

	F.I. IN.	F. IN.	S. IN.
	3	3	3
	3½	3	3
	4	3½	3
	6	4	3
	9	4	3
	12	4½	3½
	15	4½	3½
	18	5	4
	21	5	4½
	24	5½	4½

TRAFFIC STUDY (SEE ATTACHED REPORT FOR TRAFFIC LOAD COMPUTATION)

ROAD U.S. 87 SHERIDAN - BUFFALO
SECTION BANNER - SOUTH
LOADOMETER STATION (1) AVERAGE OF THE 10 STATIONS FOR 1946.
AVERAGE DAILY TRAFFIC (1946) 528
AVERAGE DAILY COMMERCIAL TRAFFIC (1948) 143
ESTIMATED AVERAGE DAILY COMMERCIAL TRAFFIC (1966) (2) 286 } AVERAGE 215

WHEEL LOAD GROUPS (IN POUNDS)

1. 4500 - 5500 (3) 13.17% x 215 x 365 x 20 x 1 = 206,703
2. 5500 - 6500 15.30% x 215 x 365 x 20 x 2 = 480,267
3. 6500 - 7500 11.76% x 215 x 365 x 20 x 4 = 738,293
4. 7500 - 8500 14.11% x 215 x 365 x 20 x 8 = 1,771,652
5. 8500 - 9500 6.21% x 215 x 365 x 20 x 16 = 1,599,455
6. 9500 & OVER 5.84% x 215 x 365 x 20 x 32 = 2,933,082

TOTAL ESTIMATED 5000 LB. WHEEL LOADS IN 20 YEARS 7,689,452
DESIGN REPETITIONS - TRAFFIC IN ONE DIRECTION 3,844,726 } EQUIVALENT 5000 POUND WHEEL LOADS

(1) ACTUAL REPORT, ON INDIVIDUAL PROJECTS, MAY BE USED WHEN AVAILABLE.
(2) 100% INCREASE IN 20 YEARS (VARIABLE, DEPENDING ON THE ROAD).
(3) FACTORS FOR CONVERTING TO WHEEL LOADS (FROM LOADOMETER STATION)

IF	4	5	6	7	8	9	12	15
USE DESIGN CURVE								
TOTAL VALUE IS WITHIN LIMITS OF	0-2	3-6	7-11	12-17	18-24	25-32	33-41	42-53

EXAMPLE 1

SECTION THRU A HEAVILY IRRIGATED FLAT; WATER TABLE 3 FEET BELOW GRADE; HEAVY FROST ACTION EVIDENT; EXISTING CONDITIONS ADVERSE DUE TO DRAINAGE PLUS OTHER CONDITIONS; TRAFFIC, AS COMPUTED IN THE TRAFFIC STUDY ABOVE, 3.8 MILLION; TOTAL VALUE 35 DESIGNATES USE OF THE DESIGN CURVE 12.

EXAMPLE 2

SECTION THRU HIGH BENCH; ANNUAL PRECIPITATION LESS THAN 10 INCHES; NO WATER TABLE EVIDENT; NO FROST APPARENT; EXISTING CONDITIONS TO BE CONSIDERED EXCELLENT DUE TO DRAINAGE PLUS OTHER CONDITIONS; TRAFFIC 3.8 MILLION; TOTAL VALUE 6 DESIGNATES USE OF DESIGN CURVE 5.

ASSUMING SOILS WITH A C.B.R. LESS THAN 3%, SEE FIGURE 2 FOR CONTINUATION.

DIFFERENT DESIGN CURVES ARE USED, THROUGHOUT A PROJECT, AS CHANGING CONDITIONS SHOW TOTAL VALUES INDICATING THE USE OF A DIFFERENT DESIGN CURVE IN ANY SECTION.

Figure 161
Wyoming Highway Department
Supplemental Design Data

To illustrate the use of the design curves, assume a single wheel load of 45,000 pounds, an E-6 soil, good drainage and severe frost. The corresponding subgrade class for these conditions (from Table 53) is F3. For a non-bituminous base the pavement requirements are:

Runways

Bituminous surface	2½ in.
Base	8
Subbase	6
Total	16½ in.

Following are brief summaries of published literature on state highway department design methods which make specific reference to frost action or its effects. Although the design methods were published at earlier dates, the data given herein on the several methods were taken from the most recent publication (Highway Research Board, Current Road Problems /1949-9, No. 8-R, November 1949) to insure that this review contained any recent revisions.

Colorado /1949-9

Colorado recognizes, in addition to soil bearing values, three additional variables which may affect the thickness of flexible pavements:

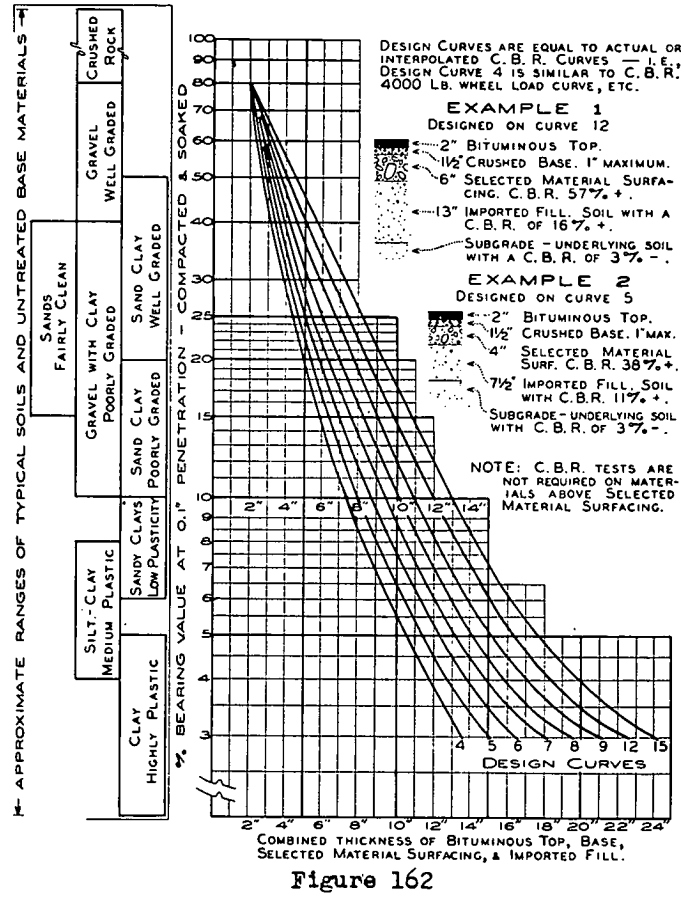


Figure 162
Wyoming Highway Department Design Data for Embankment, Selected Material, Surfacing, Subbase, Base, and Bituminous Top

1. Frost Conditions. Values are assigned on the basis of depth of frost penetration and the frost susceptibility of the soil as determined from its grain size distribution.

2. Moisture conditions. Values are assigned on the assumed possibility of relative saturation whether from surface or subsurface water.

3. Traffic Conditions. See reference 1949-9 for method of assigning values.

TABLE 54

THICKNESS OF SURFACING AND BALLAST COURSES

STA. _____ TO STA. _____ DATE _____

NOTE: Use check marks to indicate proper condition

	Assigned value	Used in design
Frost Conditions:		
Penetration of 0 to 12 in. and low frost potential	1	.....
Penetration of 0 to 12 in. and high frost potential	3	.....
Penetration of 13 to 24 in. and low frost potential	2	.....
Penetration of 13 to 24 in. and high frost potential	5	.....
Penetration of 25 to 36 in. and low frost potential	4	.....
Penetration of 25 to 36 in. and high frost potential	7	.....
Penetration of over 36 in. and low frost potential	6	.....
Penetration of over 36 in. and high frost potential	10	.....
Moisture Conditions:		
Arid or high table land not subject to standing water	2	.....
Ground subject to occasional standing water during storms	4	.....
Ground subject to saturation only during periods when frost is not present	7	.....
Ground subject to saturation during periods when frost is present	10	.....
Traffic Conditions:		
Traffic of 0 to 50 vehicles per day	0	.....
Traffic of 51 to 100 vehicles per day	1	.....
Traffic of 101 to 200 vehicles per day	2	.....
Traffic of 201 to 300 vehicles per day	3	.....
Traffic of 301 to 400 vehicles per day	4	.....
Traffic of 401 to 700 vehicles per day	5	.....
Traffic of 701 to 1000 vehicles per day	6	.....
Traffic of 1001 to 1500 vehicles per day	8	.....
Traffic of over 1500 vehicles per day	10	.....

Total Assigned Value:

Sum of Assigned Values	Design Curve to be Used
From 0 to 8	Use Curve A
From 9 to 13	Use Curve B
From 14 to 18	Use Curve C
From 19 to 24	Use Curve D
25 and Over	Use Curve E

Laboratory Information:

CBR Value _____ GI Value _____

COMBINED THICKNESS OF BALLAST AND SURFACING _____ .....IN.

THICKNESS OF SURFACING USED _____ .....IN.

REQUIRED THICKNESS OF BALLAST _____ .....IN.

Prepared by _____ Checked by _____

Table 54 indicates how frost conditions are evaluated and weighted for use in arriving at the total thickness of pavement and base. Figure 160 shows the design curves on which weighted values are applied to arrive at the pavement thickness.

Michigan /1949-9

The Michigan Manual of Soil Engineering /1946-16 catalogs the various design factors of location of grade line with respect to depth to natural ground surface, normal depth to ground water, drainage requirements, an estimate of the quantity of frost heave excavation, suitability of the soil, and minor factors of importance in evaluating the adequacy of the supporting foundation. The method /1949-9 states that "soil materials considered subject to detrimental frost action under final environmental conditions will be removed and replaced with suitable material. Finally, where conditions are not adaptable to subgrade improvement consistent with the requirements of adequate subgrade support, a sand sub-base of some thickness may be recommended depending on characteristics of the peculiar type." The Michigan method presents no charts, graphs or formulas for determining pavement thickness.

North Carolina /1949-9

The North Carolina method is based on a formula relating pavement thickness to tire-contact pressure, diameter of equivalent circular tire-contact area and the bearing value of the subgrade soil. The method states that "factors such as climatic conditions, drainage, frost action, and the physical characteristics of the soil itself are taken into consideration before selecting the final design bearing value." It does not state how those factors are evaluated or how the values are applied.

Wyoming /1949-9

The Wyoming method is based on California Bearing Ratio (CBR) tests and the use of eight different curves relating wheel load and CBR. The selection of the proper curve is indicated by a total emperical value obtained from evaluating job conditions. The conditions considered are annual precipitation, water table, frost action, traffic, existing conditions of surface drainage, subdrainage, and snow conditions which may influence required thickness.

Frost action is evaluated in terms of the amount of heaving known to occur in an area and the "apparent detrimental effect of such heaving in respect to subgrade support. Frost action is light if only slight heaving is observed, medium heaving approaches 2 in. with some evidence of weakening of subgrade support, and heavy if heaving is in excess of 2 in. with noticeable weakening of subgrade support."

An example of how data are assigned for different conditions of frost action and drainage is shown in Figure 161 and 162. The quality of the base course is indicated by the CBR value except that it should be borne in mind that a thin leveling course of high-quality base material is used in the top 1½ in. of base.

Wisconsin /1949-20

The Wisconsin Design Standards /1949-20 states: "Ballast courses are conceived to be permeable lift or intermediary courses of sand, sand-gravel, stone debris, and similar materials obtained from roadway and drainage excavation, when available, or from outside sources. These courses shall be freely draining, extending across the full width of the roadway from ditch to ditch, or if this is not feasible, they shall be drained through the installation of conduits, such as perforated pipe or tile placed in a slight depression in the earth grade immediately under the ballast course. French drains or stone drains shall not be used.... Base courses composed of graded materials, ranging from coarse through the intermediate sizes to and including the fines, constructed of gravel or crushed stone, are essentially load-carrying and load distributing structures." The Manual provides for the use of a layer of materials with finer void spaces over the subgrade soils to prevent their intrusion into the base.

TABLE 55

Thickness of Pavement Surfaces
(Wisconsin)

Current Annual 24 hour average traffic volumes	Flexible Pavement: Type /18		Flexible Pavement: Thickness (exclusive of base course)	Rigid Pavement: Thickness
Under 100	L		1 or less	
100-500	L or I		2	
500-1000	L or I		2½	7
1000-2000	I or H		3	8
2000-3000	H		3	9
over 3000	Initial Min. H	Ultimate H	3	9

Wisconsin Design Standards /1949-20 also list the thicknesses of ballast required for flexible and rigid surface types for six different groupings of average 24-hr. traffic volumes as indicated in Table 56. The remaining part of the design standards which determine the thickness of ballast and/or base course used is set up into five design classes depending on the nature of the soil encountered. The five design classes are usually used under the following conditions for the flexible and rigid types:

Class 1 Design. Indicated where native soils are of a character equivalent to the materials used for sand or sand-gravel lift or ballast courses, provided detrimental stratification is absent and permanent water table is deep. No base courses are used under rigid pavements when soils are adequate.

Class 2 Design. Indicated for till of sandy and/or gravelly nature containing only small quantities of fines and "of a nature so as not to become sufficiently plastic to result in deformation under load when wet," also indicated for sandy loam outwash material and for sand if toward fine side of gradation, and for residual sands of finer textures if silt is not dominant. Class 2 design may also prevail for rigid pavements when the grade line would be within or near silty overburden. Under those conditions Class 3 design would be used for flexible pavements.

Class 3 Design. Indicated for till when the grade line lies within the drift material and not in the overlying mantle of windborne material; in outwash materials consisting largely of silty material; in residual or loessial materials for rigid pavements except when the grade line would lie in silty soils or near underlying clays conducive to saturation of soils in grade line, in which case Class 4 design would be used for rigid pavements.

Class 4 Design. Is employed when the grade line lies in a mantle of windborne material (over till) or within till if there is evidence of differential frost heave in the area; in outwash materials consisting largely of silty material; and, in residual or loessial soils with high porosities and low bearing capacity.

Class 5 Design. Applies in areas where the grade line lies in silty or silty clay overburden where overburden over impervious bed rock or waterbearing formations is thin, or for flexible pavements where poor internal drainage is evidenced by mottling of the soil; also in glacial

/18 L - low type, includes surface treatments, road mix mats
I - intermediate type, includes road blade or travel plant mix - hot central plant mix using single aggregate or 2 bin saturation
H - high type - hot central plant mix, usually asphaltic concrete - 3 bin separation.

TABLE 56

ROADWAY STRUCTURAL DESIGN - THICKNESS OF BASE COURSES IN INCHES

DESIGN HOUR CLASSIFICATION (DHC)		25	100	200	400	800	1200	DHC-DUAL
ANNUAL 24 HOUR AVERAGE TRAFFIC VOLUMES		Less than 100	100-400	400-800	800-1600	1600-3000	3000-5000	Over 5000
DESIGN HOUR TRAFFIC VOLUMES		Less than 25	25-100	100-200	200-400	400-800	800-1200	Over 1200
Roadway Design Class and Thickness of Ballast or Lift (Subbase) Courses.								
Class 1 - Ballast Required	None							
Base Course for Flexible Pavements		5	6	8	10			
Base Course for Rigid Pavements					4	4	4	4
Class 2 - Ballast Required	None							
Base Course for Flexible Pavements		6	8	10	12			
Base Course for Rigid Pavements					4	4	6	6
Class 3 - Ballast Required	9 in.							
Base Course for Flexible Pavements		5	6	8	10			
Base Course for Rigid Pavements					4	4	4	4
Class 4 - Ballast Required	12 in.							
Base Course for Flexible Pavements		5	6	8	10			
Base Course for Rigid Pavements					4	4	4	4
Class 5 - Ballast Required	15 in.							
Base Course for Flexible Pavements		5	6	8	10			
Base Course for Rigid Pavements					4	4	4	4

NOTES: Under certain conditions under Class 1 - the base course for Rigid Pavements need not be used - See Text.
When the surface course is to consist of bituminous surface treatment, the thickness of the base course shall be increased 2 inches.
When the material of the ballast course is inherently quite unstable, the thickness of the base course shall be increased by 25 percent.

The above Design Classes are essentially for Rural Pavements. For urban design contemplating, full width construction with curb and gutter, a design classification one step lower than would be indicated for Equivalent Rural Exposures may be used.

outwash and lacustrine deposits, if there is evidence of severe differential frost heave; in residual or loessial soils of high porosity and low bearing capacity class 5 may be used for classes 3 and for rigid pavements; and in alluvial and colluvial soils in lower portions of terraces and flood plains.

Corps of Engineers /1951-51

The Corps of Engineers Manual /1951-51 classifies soils according to frost susceptibility, requires all bases to be of non-frost-susceptible soils, and provides for the use of a filter course between subgrade and base. The Corps requirements for those items are given under the heading "Texture of Bases and Subbases." The manual provides two acceptable methods for designing pavements over frost-susceptible subgrades. One is to prevent freezing of the subgrade and thereby prevent pavement heave and subgrade weakening. The other method is to allow freezing of the subgrade and to design on basis of anticipated reduced strength of the subgrade during the frost-melting period.

a. Preventing freezing of subgrade. The combined thickness of rigid or flexible pavement and non-frost-susceptible base should not be less than the depth of frost penetration as determined from Figure 155, (using the normal freezing index for the particular location) in the following cases:

(1) Over Group F4 subgrade soils.

(2) Over other frost susceptible subgrade soils when frequent and/or abrupt changes in soil and ground-water conditions occur which would result in detrimental nonuniform heave.

(Exception to the above rule is made for flexible pavements for areas of lesser importance where some non-uniform heave may not be detrimental, or where non-uniform soil may be removed to ameliorate conditions. In those conditions frost may be permitted to enter the subgrade and design is based on reduction in subgrade strength).

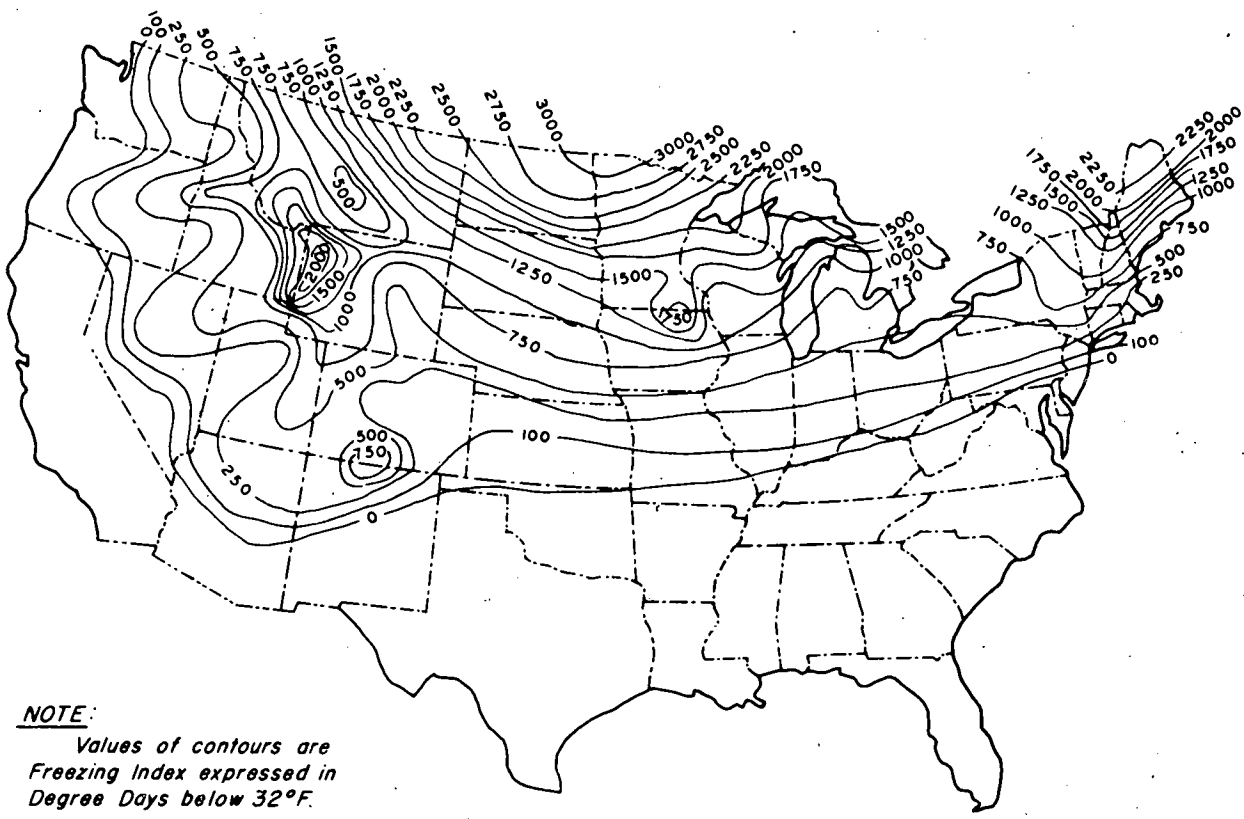


Figure 163. Freezing Index Data Influencing Frost Action
(After Corps of Engineers)

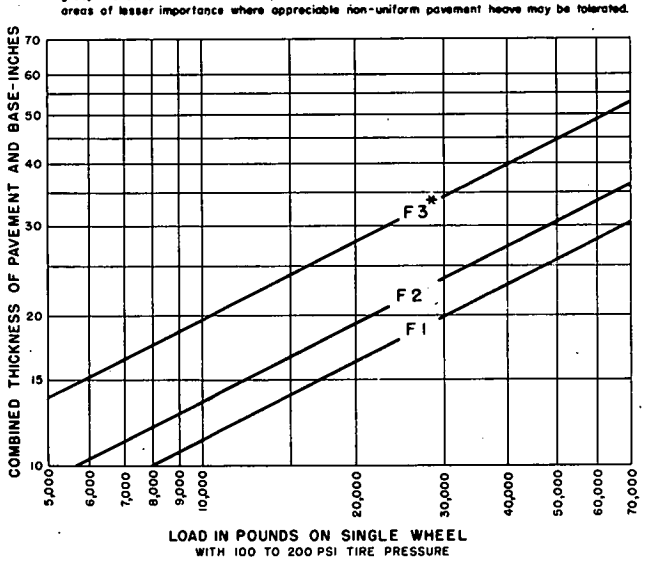
Where the normal freezing index in Figure 163 is less than 100, the freezing index should be computed for the coldest year of record for the past 15 yrs., and design should be based upon this value or 100, whichever is smaller. In mountainous areas, the normal freezing index should be computed for the particular location. In southern areas below the zero contour line on Figure 163, where detrimental frost occurs infrequently, the upper 4 in. of base course, if required, should be composed of non-frost-susceptible material.

b. Reduction in subgrade strength. Design may also be based on the reduction in strength of the subgrade due to frost action, thereby frequently permitting less depth of pavement and base than that required to prevent freezing of the subgrade. This method may be used for both flexible and rigid pavements on subgrade soils of Groups F1, F2 and F3 when subgrade conditions are sufficiently uniform to assure that objectionable differential heaving will not occur or where subgrade variations are correctible to this condition as previously noted. The method may also be used where appreciable non-uniform heave can be tolerated in pavements of lesser importance, used for slow-speed traffic, but in such cases it is limited to flexible-type pavements.

Flexible pavements - When freezing is permitted in the subgrade the curves in Figure 164 and 165 (Note: the Engineering Manual also provides curves relating thickness and wheel load for the ranges of load of 30,000 - 120,000 (dual wheels) and 100,000 - 200,000 lb. twin tandem assembly) should be used to determine the combined thickness of flexible pavement and non-frost-susceptible base required for various aircraft wheel loads and wheel assemblies and Figure 165 should be used for highways. These curves reflect the reduction in strength of soil during the frost-melting period. The reduction in strength of subgrades is believed to be greater in cuts than in fills. If field data and experience definitely indicate that the reduction of strength

GROUP	DESCRIPTION
F1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 20 PER CENT FINER THAN 0.02 mm. BY WEIGHT.
F2	SANDS CONTAINING BETWEEN 3 AND 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT.
F3	(a) GRAVELLY SOILS CONTAINING MORE THAN 20 PER CENT FINER THAN 0.02 mm. BY WEIGHT AND SANDS, EXCEPT FINE SILTY SANDS, CONTAINING MORE THAN 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT. (b) CLAYS WITH PLASTICITY INDICES OF MORE THAN 12, EXCEPT VARVED CLAYS.
F4	(a) ALL SILTS INCLUDING SANDY SILTS. (b) FINE SILTY SANDS CONTAINING MORE THAN 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT. (c) LEAN CLAYS WITH PLASTICITY INDICES OF LESS THAN 12. (d) VARVED CLAYS.

* When frost is permitted to penetrate group F4 soils, use same design curve as for group F3 soils. Frost should be permitted to penetrate F4 soils only under flexible paved areas of lesser importance where appreciable non-uniform pavement heave may be tolerated.



THE THICKNESS WILL BE REDUCED 10 PER CENT FOR CENTRAL PORTION OF RUNWAYS (AREA BETWEEN 1000 FT. SECTION AT EACH END)

Figure 164. Flexible Pavement Design Curves for Taxiways, etc. for Frost Action in Subgrade Soil. (After Corps of Engineers)

GROUP	DESCRIPTION
F1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 20 PER CENT FINER THAN 0.02 mm. BY WEIGHT.
F2	SANDS CONTAINING BETWEEN 3 AND 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT.
F3	(a) GRAVELLY SOILS CONTAINING MORE THAN 20 PER CENT FINER THAN 0.02 mm. BY WEIGHT AND SANDS, EXCEPT FINE SILTY SANDS, CONTAINING MORE THAN 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT. (b) CLAYS WITH PLASTICITY INDICES OF MORE THAN 12, EXCEPT VARVED CLAYS.
F4	(a) ALL SILTS INCLUDING SANDY SILTS. (b) FINE SILTY SANDS CONTAINING MORE THAN 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT. (c) LEAN CLAYS WITH PLASTICITY INDICES OF LESS THAN 12. (d) VARVED CLAYS.

* When frost is permitted to penetrate group F4 soils, use same design curve as for group F3 soils. Frost should be permitted to penetrate F4 soils only under flexible paved areas of lesser importance where appreciable non-uniform pavement heave may be tolerated.

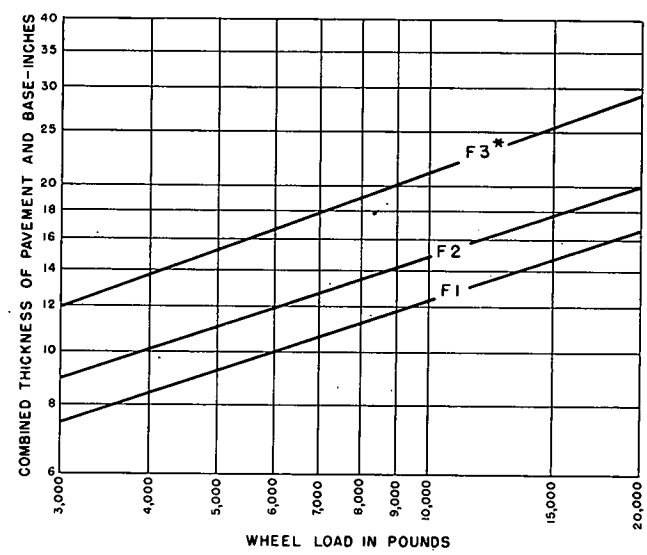


Figure 165. Flexible Pavement Design Curves for Highways, etc. for Frost Action in Subgrade Soil. (After Corps of Engineers)

in fill areas is less because of the height of fill and of greater depth to the water table below the fill, a reduction in combined thickness of base and pavement for the fill area may be permitted. In no case should the thickness of pavement and non-frost susceptible base be less than 9-in. where frost action is a factor.

c. Examples of flexible pavement design

Example 1. Design an airfield taxiway for flexible pavement to withstand a 40,000 lb. single-wheel load with 100 psi. tire pressure for the following conditions:

Normal freezing index - 400 deg.-days

Subgrade-uniform, lean clay, 40 percent by weight finer than 0.02 mm.

Highest ground water - 3 ft. below surface of subgrade

Subgrade CBR - 8 percent (normal period)

Base CBR - 80 percent

Preventing Freezing. From Figure 155 the combined thickness of pavement and base to prevent freezing of the subgrade is 29 in. This would prevent or reduce to a tolerable value pavement heaving and loss of subgrade strength.

Reduction in subgrade strength. The required total thickness of 39 in. according to Figure 164, is greater than the depth of frost penetration; therefore, this method of design is not applicable. Since the thickness of 29 in. required to prevent freezing of the subgrade is less than the value from Figure 164 and greater than the 22 in. required under non-frost conditions, 29 in. would be selected as the combined thickness of pavement and base.

Example 2. Design an airfield taxiway for flexible pavement to withstand a 40,000 lb. single-wheel load with 100 psi. tire pressure for the following conditions:

Normal freezing index - 2,000 deg.-days.

Subgrade - uniform, lean clay, 40 percent by weight of grains finer than 0.02 mm.

Highest ground water - 3 ft. below surface of subgrade

Subgrade CBR - 8 percent (normal period)

Base CBR - 80 percent

Preventing freezing of subgrade. From Figure 155 the combined thickness of pavement and base to prevent freezing of the subgrade is 62 in. This would prevent or reduce a tolerable value pavement heaving and loss of subgrade strength.

Reduction in subgrade strength. When allowing a reduction in strength of the subgrade due to frost action and allowing uniform heave of the pavement, a total thickness of 39 in. is required, according to Figure 164.

Rigid pavements

Corps of Engineers Method /1951-51

The general provisions of the Corps of Engineers Manual /1951-51 regarding frost susceptibility of soil, the design of filter courses, and determination of freezing index of the area given under Texture of Bases and Subbases and under Flexible Pavements also apply for rigid pavements. Where frost penetration is permitted into a frost-susceptible subgrade beneath a rigid pavement, a non-frost-susceptible base course equal in thickness to the concrete slab shall be used. However, the following conditions are specific exceptions to this requirement;

(a) Where soils of Groups F1, F2, and F3 occur under very uniform conditions in the subgrade and the freezing index is less than 500, the thickness of the non-frost-susceptible base may be reduced to 4 in., and it shall be designed as a filter as specified for filter courses under "Texture of Bases and Subbases."

(b) Where soils of Groups F1, F2, and F3 occur under very uniform conditions and the depth to the uppermost water table is greater than 10 ft., the thickness of the non-frost-susceptible base may be reduced to 4 in. and it shall be designed as a filter as specified for filter courses under "Texture of Bases and Subbases."

(c) Over Group F4 soils the combined thickness of pavement and base shall be equal to the full depth of frost penetration.

The thickness of concrete pavement shall be determined in accordance with Chapter 3, Part XII, for airfields and Chapter 1, Part X, for highways, using the subgrade modulus determined from Figure 166, which considers the reduced strength of the subgrade. If the tested k value is smaller than the subgrade modulus obtained from Figure 166, the test value should govern the design.

GROUP	DESCRIPTION
F1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 20 PER CENT FINER THAN 0.02 mm. BY WEIGHT.
F2	SANDS CONTAINING BETWEEN 3 AND 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT.
F3	(a) GRAVELLY SOILS CONTAINING MORE THAN 20 PER CENT FINER THAN 0.02 mm. BY WEIGHT AND SANDS, EXCEPT FINE SILTY SANDS, CONTAINING MORE THAN 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT. (b) CLAYS WITH PLASTICITY INDICES OF MORE THAN 12, EXCEPT VARVED CLAYS.
F4	(a) ALL SILTS INCLUDING SANDY SILTS. (b) FINE SILTY SANDS CONTAINING MORE THAN 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT. (c) LEAN CLAYS WITH PLASTICITY INDICES OF LESS THAN 12. (d) VARVED CLAYS.

Frost shall not be permitted to penetrate group F4 soils beneath rigid pavements.

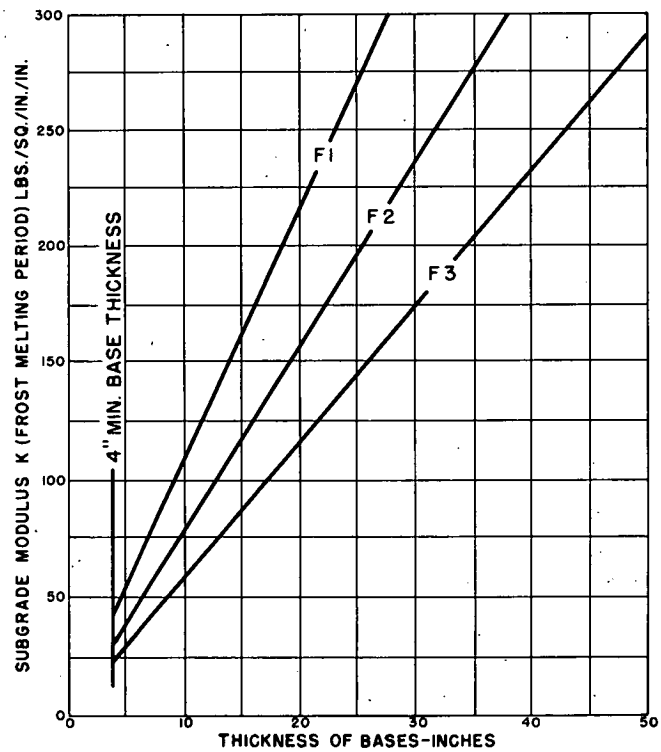


Figure 166. Rigid Pavement Subgrade Modulus Curves for Frost Action in Subgrade Soil. (After Corps of Engineers)

Examples of rigid pavement design

Example 1. Design an airfield taxiway for rigid pavement to withstand a 40,000 pound single-wheel load with 100 psi. tire pressure for the following conditions.

Normal freezing index - 400 deg.-days.

Subgrade - uniform - lean clay, 40 percent by weight of grains finer than 0.02 mm.

Highest ground water - 3 ft. below surface of subgrade

Subgrade CBR - 8 percent (normal period)

Base CBR - 80 percent

Subgrade modulus, k - 100 lbs. per sq. in. per in.

Concrete flexural strength - 650 lbs. per sq. in.

Preventing Freezing of Subgrade.

From Figure 155 the minimum thickness of pavement and base required to protect the subgrade from frost action is 29 in. The required slab thickness from Chapter 3 with no subgrade weakening is 11.4 in. Chapter 3 of the Engineering Manual specifies that when the thickness from the design curves indicates a fractional value greater than 1/4 in. the next full-inch thickness is used for construction. The adopted thickness should be 12 in. thereby resulting in a base course thickness of 17 in.

Reduction in Subgrade Strength. Since subgrade conditions are uniform and freezing index is less than 500, the exception in paragraph b above is applicable. A minimum base course of 4 in. is required as a filter to protect against loss of support by pumping. The subgrade modulus during the frost melting period is 25 lbs. per sq. in. as determined in Figure 166. The slab thickness required, in accordance with Chapter 3 of the Engineering Manual is 13 in.

Example 2. Design an airfield taxiway for rigid pavement to withstand a 40,000 lb. single-wheel load with 100 psi. tire pressure for the following conditions.

Normal freezing index - 2000 deg.-days

Subgrade - uniform - lean clay, 40 percent by weight of grains finer than 0.02 mm.

Highest ground water - 3 ft. below surface of subgrade

Subgrade CBR - 8 percent (normal period)

Base CBR - 80 percent

Subgrade Modulus, k - 100 lbs. per sq. in. per in.

Concrete flexural strength - 650 lb. per sq. in.

Preventing freezing of subgrade. From Figure 155 the minimum thickness of pavement and base required to protect subgrade from freezing is 62 in. The slab thickness according to the design curves in Chapter 3 of the Engineering Manual would be 11.4 in. A 12 in. slab thickness should be used in construction thereby resulting in a base course of 50 in.

Reduction in subgrade strength. If the combined thickness of pavement and base is to be less than the depth of frost penetration, the subgrade modulus as determined in Figure 166, assuming a 12-in. base thickness is 65 lb. per sq. in. per in. Using this value of subgrade modulus, the required slab thickness, from the design curves in Chapter 3, is 12 in. which requires a 12-in. base thickness in accordance with paragraph b under Rigid Pavements.

In the case of ground water table depth in excess of 10 ft. and all other conditions the same as given in Example 2, a 4 in. minimum thickness base is permitted in accordance with paragraph b under Rigid Pavements. The subgrade modulus in accordance with Figure 166 is 25 lb. per sq. in. per in. Using this value of subgrade modulus, the required slab thickness, from Chapter 3, of the Engineer Manual is 13 in.

Civil Aeronautics Administration Method /1948-13

The Civil Aeronautics Administration Method /1948-13 provides thickness designs for both the concrete pavement and the subbase for single wheel loads from 15,000 to 100,000 pounds for various classes of soils and soil conditions. The subgrade classes and the recommended design curves are indicated in Table 53 giving the C.A.A. soil classification. The curves showing the wheel load versus thickness relationships are given in Figure 167 for taxiways, aprons, and runway ends.

Line A represents the subgrade class which requires no subbase (E-1 to E-6 for good drainage, no frost; E-1 to E-5 for good drainage, severe frost; E-1 to E-4 poor drainage, no frost; and, E-1 to E-3 for poor drainage, severe frost).

An example of the use of the design curves in arriving at thickness design is given as follows:

"Assume a single-wheel load of 45,000 lb., and E-6 soil, good drainage and severe frost. The corresponding subgrade class for those conditions is R-2b which will require concrete pavement 11 in. thick on taxiways, aprons and runway ends. The subbase requirement is 8 in. Thicknesses are taken to the nearest inch."

Wisconsin Method /1949-20

Design standards for Wisconsin for rigid pavements are carried in the summary for Wisconsin /1949-20 under Flexible Pavements.

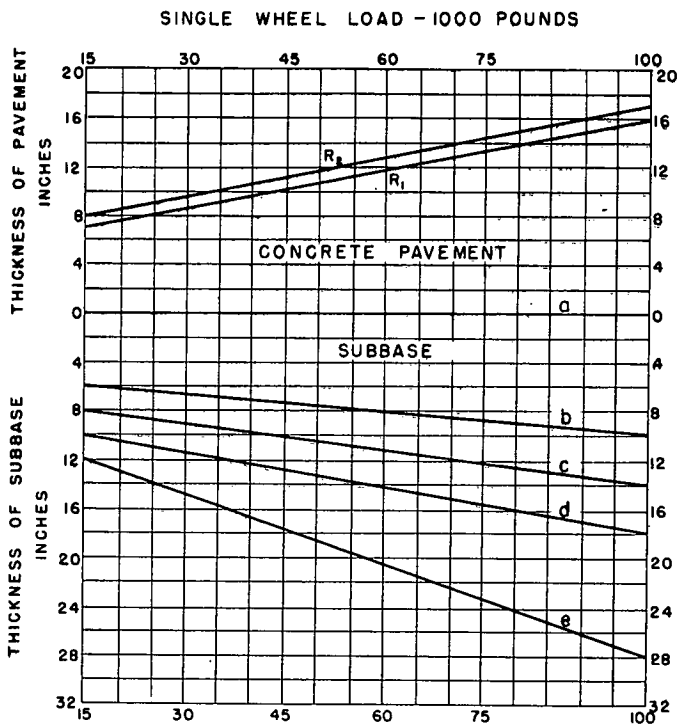


Figure 167. Design Chart for Rigid Type Pavements for Taxiways, Aprons and Runway Ends. (After Civil Aeronautics Administration)

Rigid pavement design details - Their influences on effects of frost action - Nearly all investigators concerned with design for frost action have limited their studies to the adequacy of thickness designs for the frost melting period. The literature includes few references which give information on design of pavement details in frost heave areas.

Keil /1937-12, /1938-3 and L. Casagrande /1940-8 after extensive studies in Germany stress the necessity of adequate dowelling of joints in rigid pavements when laid on frost-susceptible soils.

Minnesota's /1945-8 studies of high joints (warping) in rigid pavements showed that leaky joints resulted in higher subgrade moisture contents under joints and greater heaving at joints. It was found that slab length had an effect in that the uplift at joints on 20-and 30-ft. slab lengths was less than on 40-ft. slab lengths. The subgrade moisture distribution was more nearly uniform under the shorter slab lengths, and the differential uplift was smaller. The same studies also showed that where copper-seal expansion and contraction joints of the type used were properly installed "the special seals proved successful in preventing infiltration of surface water."

Drainage of bases - It has been brought out that saturated base courses, especially those constructed of poorly graded gravels, have a reduced strength to resist loads imposed by traffic as compared to drained base courses. Base-course saturation often occurs during the frost-melting period when the ice in the base and top portion of the subgrade is melted and escape downward is precluded by the frozen soil below. Prolonged rainy periods and ponding of surface run-off may further aid in saturating base courses.

Much talk has concerned the use of "free-draining" base courses as a means for preventing detrimental frost action but there has been little research. The only major field investigation which the reviewer came across in the literature is that of the Corps of Engineers /1946-10 which was initiated to establish criteria for the design of adequate base-course drainage for airfields. While primarily for airfield drainage the data obtained are equally as valuable for application to highways.

The following is an abstract type of review of the comprehensive report and four appendices to the report of the investigation by the Corps of Engineers. The study was divided into four parts, namely: Theoretical Studies; Viscous Fluid Model Tests; Full Scale Field Drainage Tests; and Field Investigations.

Theoretical Studies

The present theories of two-dimensional flow through soil were reviewed and applied to the base course drainage problem. The arbitrary design criterion that "a degree of drainage of 50 percent in the base course shall be obtained in not more than ten days" was established. The problem then became one of determining theoretically the relationships between time and percent

drainage for various combinations of material type, and the depth, width and slope of the base course.

The report reviews the present status of the theory, discusses various methods of analyzing drainage by use of different approximations, and gives the simplified method found by experiment to give results within the desired range of accuracy.

From the standpoint of practical application of the theoretical analysis of drainage of base courses it is in many cases sufficient to know the time required to reach 50 percent drainage.

The report gives two procedures for determining the time required to reach 50 percent drainage.

Procedure 1.

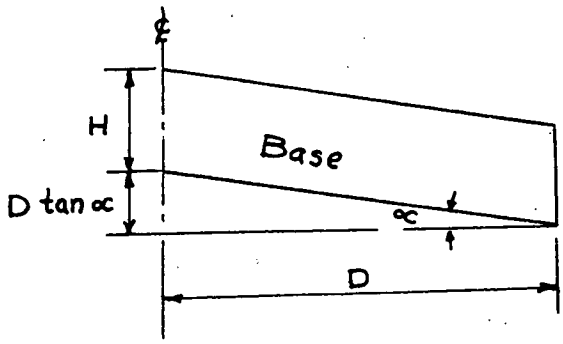


Figure 168

If k is coefficient of permeability
H is thickness of base
D is length of base being drained
α is angle of slope
S is slope factor

$$\frac{H}{D \tan \alpha}$$

t is time (t_{50} = time for 50% drainage/19)
T_{50} is time factor (Figure 169 shows the relation between time factor and slope factor.)
n_e is effective porosity = $\frac{\text{Volume of Voids drained}}{\text{Total Volume of soil}}$
c is parameter whose value depends upon $\frac{D}{H}$ and the slope. (Empirical relationships derived from facsimile model test data and given in Figure 170).

Then $t_{50} = \frac{c n_e T_{50} D^2}{2kH}$

An example showing the computation of the time required to obtain 50 percent drainage is shown in Figure 169.

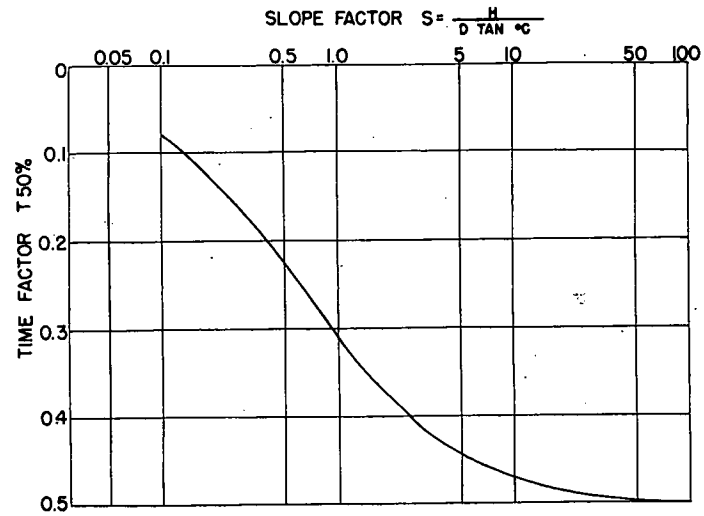


Figure 169

Procedure	Example
Compute $S = \frac{H}{D \tan \alpha}$ and from this chart read corresponding T_{50}	D = 75 ft. H = 2 ft. n_e = 0.3 k = 0.01 ft. per min. = 14.4 ft. per day Slope = 1.0% ($\tan \alpha$ = 0.01) Slope factor S = 2.67 T_{50} = 0.405
Compute time required for 50% drainage from $t_{50} = \frac{c n_e T_{50} D^2}{2 kH}$ where c is determined from Figure 170.	c = 1.87 t_{50} = 22.2 days

Figure 169. Relation Between Slope Factor and Time Factor for 50 Percent Drainage

/19 The degree of drainage is expressed here as a percent of the amount of water drained at a given time to the total amount of water which can be drained.

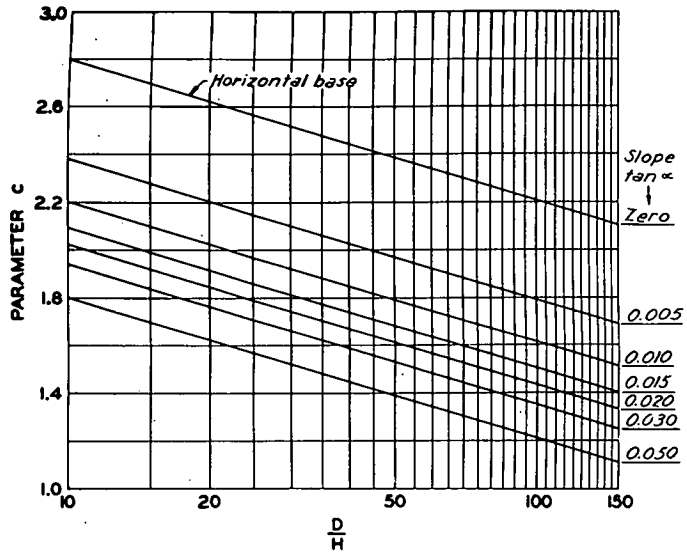


Figure 170. Empirical Relation Between Parameter c and Ratio D/H Derived from Viscous Model Tests

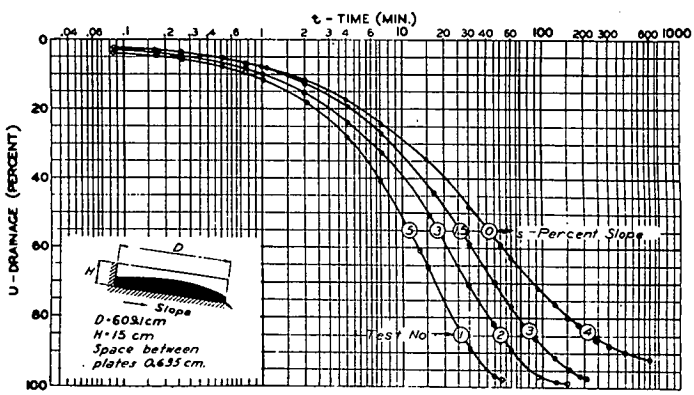


Figure 171. Percent Drainage 609 cm Model

Procedure 2.

For most practical purposes a further simplification is permissible by the use of the following equation:

$$t_{50} = \frac{n_e D^2}{2k(H + D \tan\alpha)}$$

By using the values given in the example on Figure 169. 50 percent drainage is obtained in 21.4 days compared to 22.2 days according to Procedure 1.

Viscous fluid model tests - The purpose of the model tests was to verify theoretical analysis of the drainage of base courses and particularly to determine constants used in those analyses.

The principal tests for the study of drainage was performed on facsimile models consisting essentially of two parallel glass plates separated by a uniform distance of ¼ in. and open to drain at one end. A temporary stop was placed in the open end and the space between the two plates was filled with glycerin to which had been admixed anhydrous sodium carbonate to increase viscosity and phenolpthalein to give the fluid a light red color. When the model was filled, it was suddenly inclined in the desired slope and the temporary stop removed, permitting the fluid to drain from the model.

Facsimile tests were performed on models with height to length ratio ranging from 1:1 to 1:40 and at slopes from 0 to 5 percent to study the effects of base dimensions and slope respectively. Measurements were made of the rate of discharge, the position of the surface of the draining fluid, and the temperature of the fluid, all versus time.

A series of tests were performed on two viscous fluid models to determine the coefficient of permeability which is a function of the viscosity of the fluid and plate spacing.

Figure 171 shows test data obtained from the 609 cm. model, (one of several sizes tested) used in the facsimile model studies.

Test No.	Slope s Percent	H/D	Test Temperature deg. C. Max.	Min.	Ave.	Glycerin Mix. No	Viscosity Sec. per 60 ml.	Coef. of Perm. k cm. per sec.
1	5	0.0246	21.7	21.1	21.4	6	192	7.74
2	3	0.0246	21.8	21.4	21.6	7	200	7.44
3	1.5	0.0246	20.6	20.3	20.4	8	217	6.85
4	0	0.0246	22.5	21.3	21.9	4	192	7.74

The results show remarkably close agreement between facsimile model test drainage curves and theoretical drainage curves of time vs. percent drainage as indicated in Figure 172.

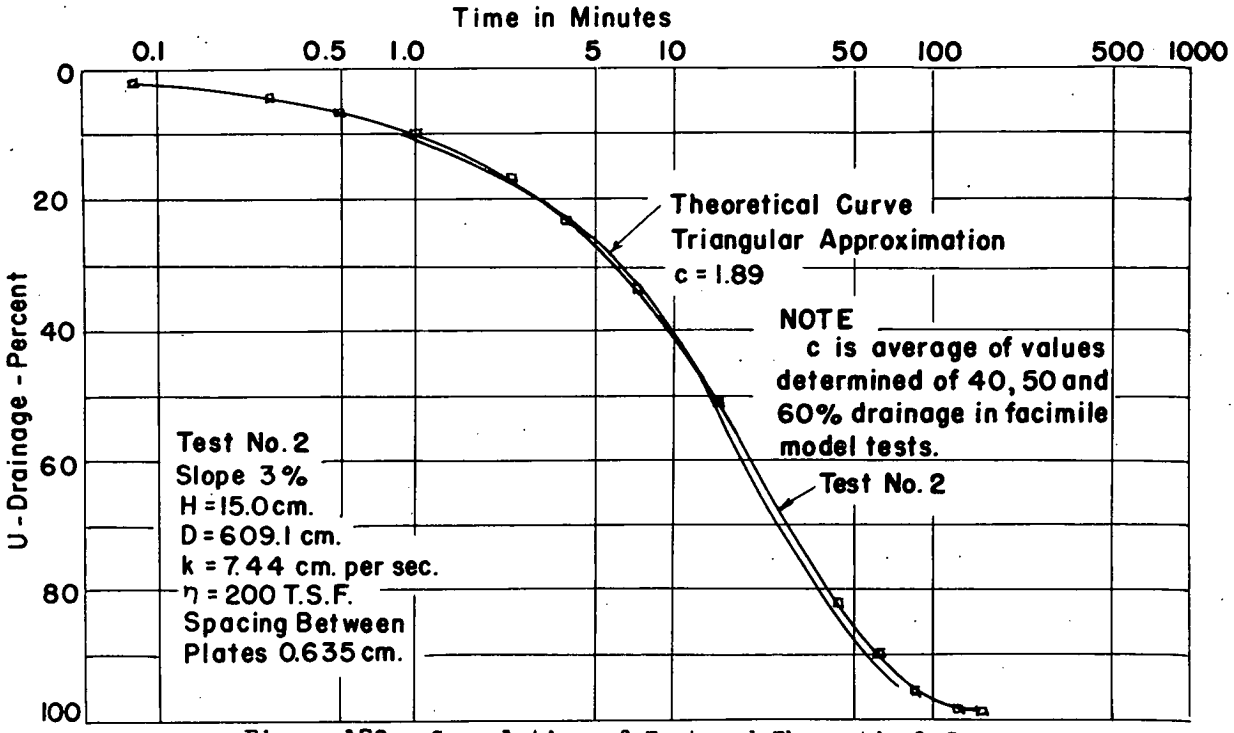


Figure 172. Correlation of Test and Theoretical Curves

Full Scale Field Drainage Tests - Base course drainage tests were performed on four full-scale test sections to verify the theoretical analysis. The test sections were constructed on an area adjacent to the Bedford Airfield, Bedford, Massachusetts. Each section was 10 by 75 ft. with a 1.5 percent slope in the direction of the long dimension. The sections were constructed using the following base course materials and design thicknesses:

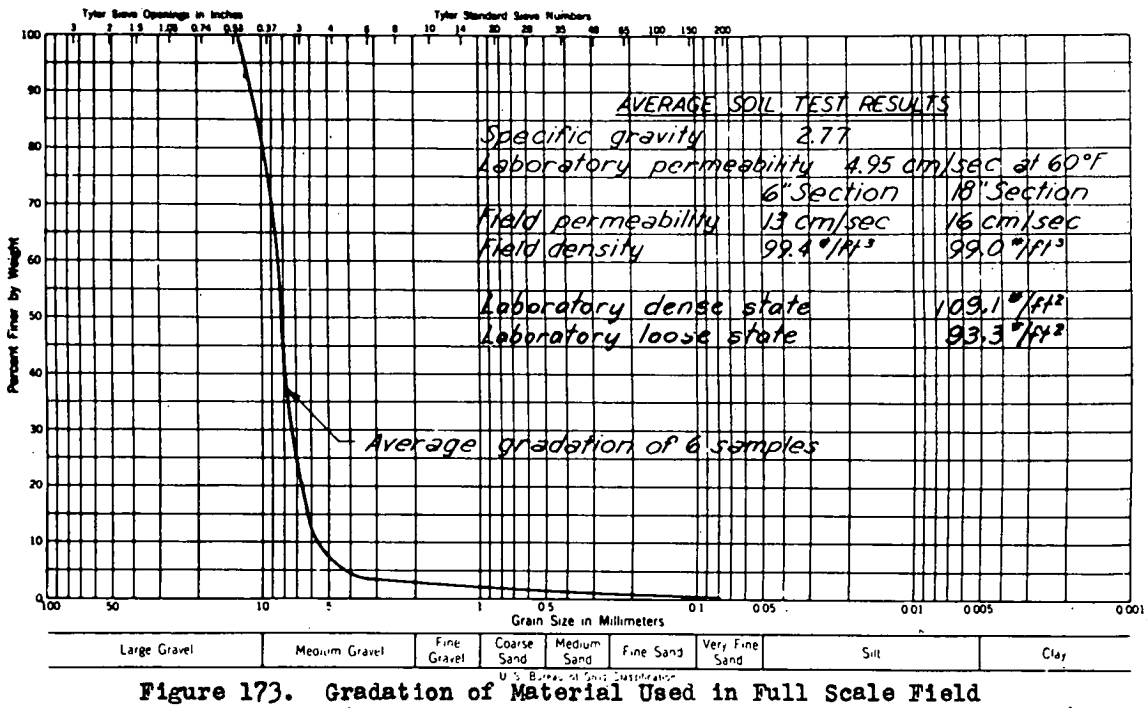


Figure 173. Gradation of Material Used in Full Scale Field Test Sections. 6 in. and 18 in. Depth of Base Section 1 and 4 Pea Gravel

Section No.	Thickness of Base in.	Base Materials
1	18	pea gravel
2	18	sand and gravel
3	6	sand and gravel
4	6	pea gravel

Grading curves of the pea gravel and the sand and gravel materials are shown in Figures 173 and 174.

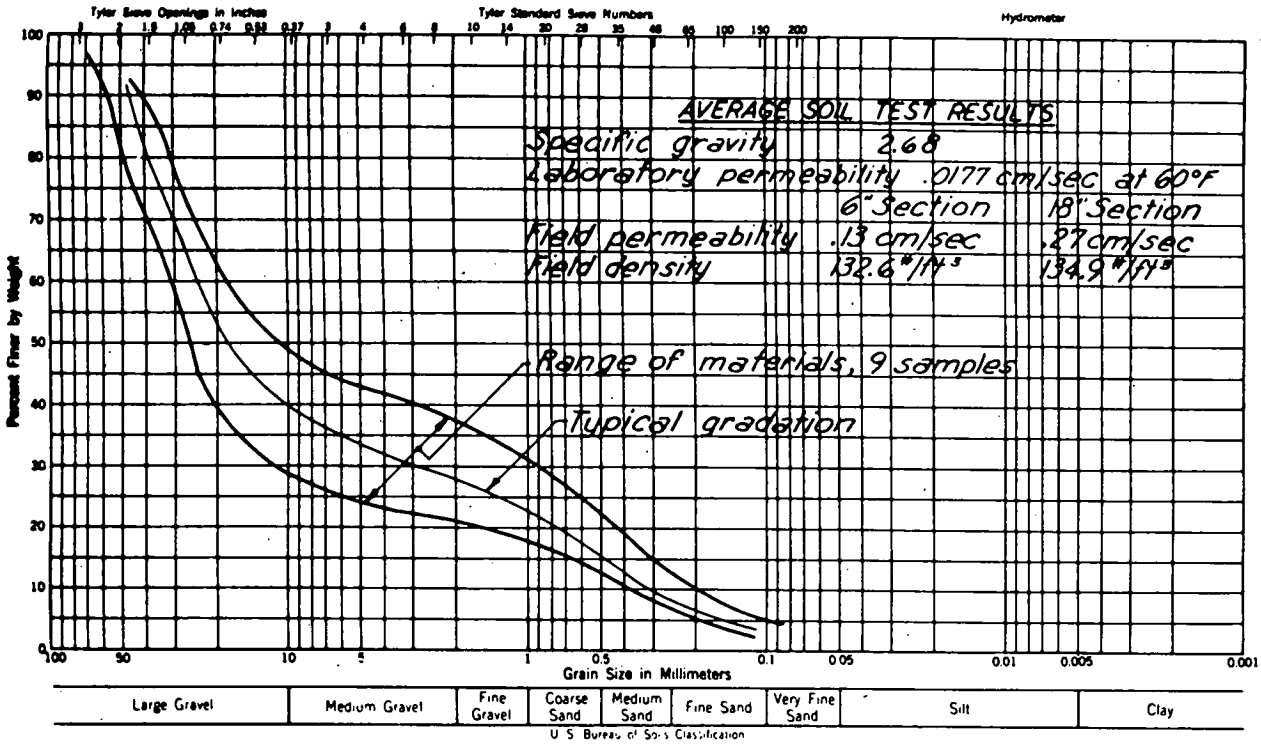


Figure 174. Gradation of Materials Used in Full Scale Field Test Sections. 6 in. and 18 in. Depth of Base Section, 2 and 3 Sand and Gravel

The base course in each section had impervious boundaries consisting of PBS (prefabricated bituminous surfacing) on the top, bottom, both sides, and at the upper end. A surcharge of sand and gravel was placed over the top layer of PBS to prevent uplift during saturation of the base courses. A drainage trench was constructed at the lower end of each section with an 8-in. corrugated metal drain pipe to collect and discharge the water from the base course. A 4-in. discharge pipe was connected to the middle of the drainage pipe and carried through the outer wall of the trench. Sixteen observation wells were installed in each section and two in each drain trench to determine the depth of the water in the base course and drain trench during the drainage test.

Two series of drainage tests were performed, the first in November and December 1946 and the second in May 1946.

In-place permeability tests and field density determinations were made. Laboratory tests were made to determine grain size distribution, specific gravity, coefficient of permeability, and effective porosity.

Comparison of theoretical and test section drainage results - The theoretical and actual drainage curves showed fair to close agreement. The results are summarized in Table 57.

Figure 175 shows the correlation between theoretical and actual drainage curves for Section 2. Table 58 of the comprehensive report shows the correlation of theoretical and measured rates of discharge of saturated base courses for all four test sections.

TABLE 57

Section No.	Thickness	Base Materials	Results
	in.		
1	18	Pea Gravel	Close agreement
2	18	Sand and Gravel	Check reasonably close for time of 50% drainage
3	6	Sand and Gravel	Vary appreciably except during first 20% of drainage
4	6	Pea Gravel	Close agreement except during first 40% drainage when actual drainage time was less than theoretical time

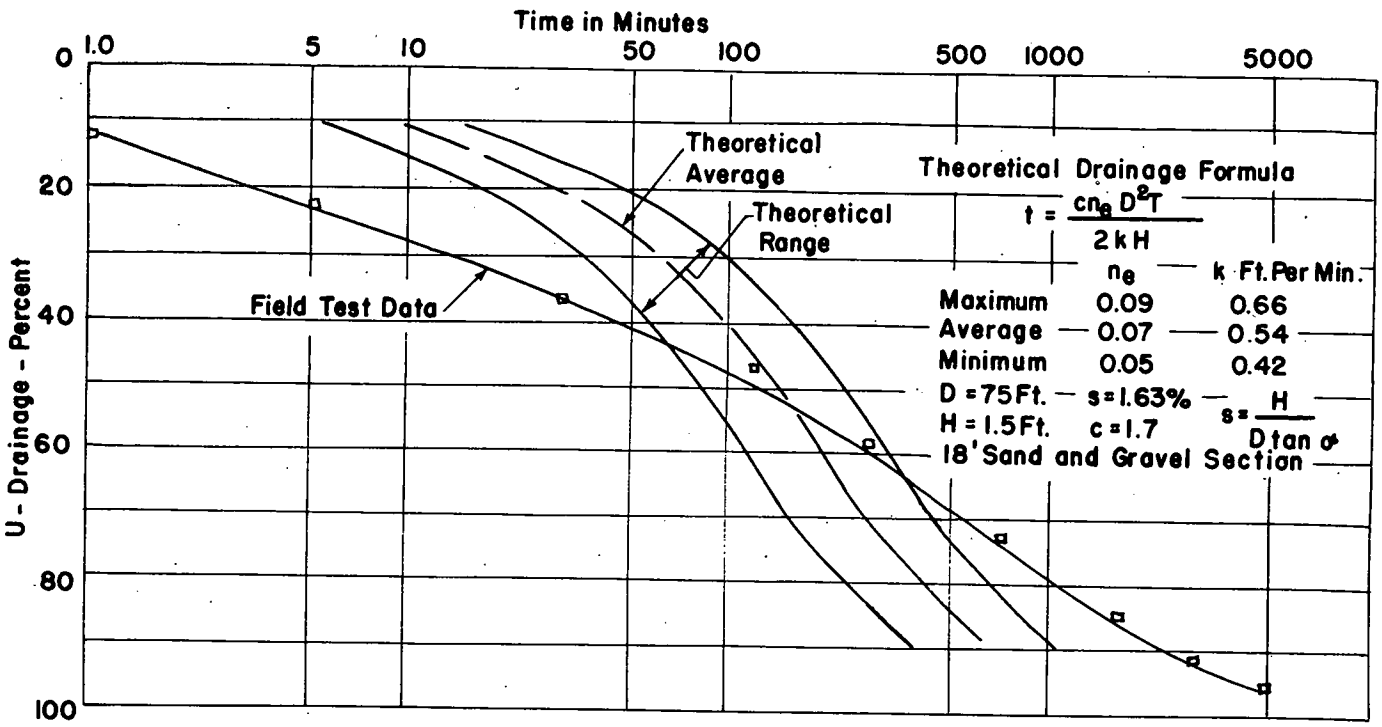


Figure 175. Drainage Correlation Between Bedford Test Section 2 and Theoretical Analysis

The reasons for variations between theoretical and actual test-section drainage curves are believed to be caused by a combination of factors. The theoretical percent drainage-time formula is predicated on the assumption that the portion of the base course above the theoretical draw down curve is at a degree of saturation consistent with the effective porosity of the base material and the material beneath the draw down curve is assumed to be saturated. The draw down curve which is defined as the top boundary of the saturated zone, therefore does not correspond to the piezometric surface as measured in the observation wells due to the existence of a zone saturated by capillary water above the piezometric surface.

Quantitative measurements of the height of the capillary zone were not made in the base materials used. It is estimated that in the sand and gravel the height would be three inches, the bottom two inches of which could be considered saturated. The pea gravel is estimated to have a capillary zone of 3/4 inch with the bottom ½ inch saturated.

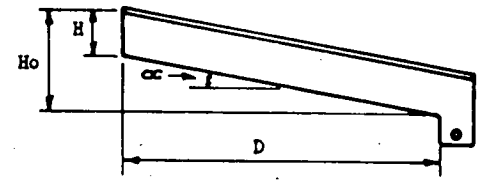
A variation in the coefficient of permeability throughout the depth of the section and (or) a difference between horizontal permeability would also cause a difference between the field test curves and the theoretical curves.

TABLE 58

CORRELATION OF THEORETICAL AND MEASURED RATES OF DISCHARGE OF SATURATED BASE COURSES

$$q = kH \frac{Ho}{D\ 60}$$

q = the peak discharge quantity in cfs per linial foot of drain



(a) Full Scale Field Test Sections, Bedford, Mass.

Material	D ft	H ft	Slope %	Ho ft	k from field tests (f/m)	(cfs) q Computed	(cfs) q Measured	Time After Start of Test (Minutes)	Date
PEA GRAVEL	75	1.5	1.46	2.59	34	0.30	0.22	3.5	27 Nov. 1945
PEA GRAVEL	75	0.5	1.40	1.55	26	0.044	0.034	2.5	28 Nov. 1945
SAND & GRAVEL	75	1.5	1.63	2.72	0.54	0.0049	0.0167	8.0	10 Dec. 1945
SAND & GRAVEL	75	0.5	1.50	1.62	0.26	0.0004	0.023	2.0	28 Nov. 1945

Field investigations - Two types of field investigations were made, namely survey investigations and comprehensive investigations. The former consisted of inspections of the fields to determine the conditions, operation, maintenance, and effectiveness of the drainage systems and were made on four airfields in Massachusetts and one in Connecticut. Comprehensive investigations included studying the operations of the subsurface drains, measuring the discharge of the drains at selected locations, gaging precipitation and observing ground water conditions by means

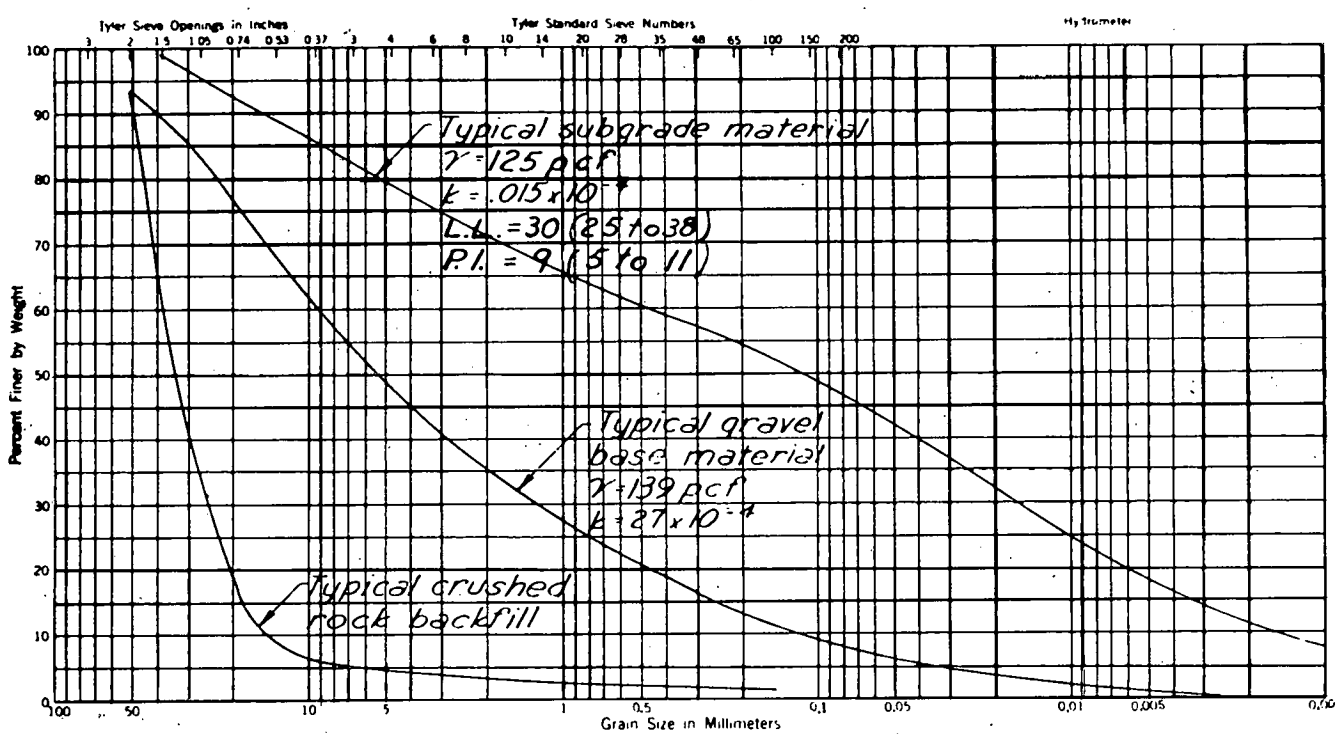


Figure 176. Presque Isle Airfield, Presque Isle, Maine - Typical Soil Characteristics

of observation wells, and observations on the operation of selected combination drains (Combination drains collect (1) surface water either by infiltration into a trench with pervious backfill exposed at ground surface or by catch basins or drop inlets and (2) subsurface water from the base course and subgrade by seepage into a trench with pervious backfill.)

Comprehensive investigations were made at seven sites. Of those, studies of subsurface drainage were made at five airfields located at Presque Isle, Maine; Bedford, Massachusetts; Bangor, Maine; Mt. Clemens, Michigan; and Madison, Wisconsin. Following are abstracts of the results obtained at those five fields:

Presque Isle, Maine. The pavement at Presque Isle consisted of 3 in. of bituminous concrete over a 2-in. dry bound macadam and a 30-in. sand and gravel (GW) base. The subgrade is a compact clayey silt sand and gravel (GF) till. The characteristics of the bases and soils are given in Figure 176.

Although discharge measurements were not practical, readings of water level in observation wells were made. The base course in the test area with subsurface drains was 56 percent saturated on April 11, 1946 and this water was for all practical purposes drained from the base on April 30, 1946. A correlation of theoretical and actual time for base course drainage is shown in Figure 177.

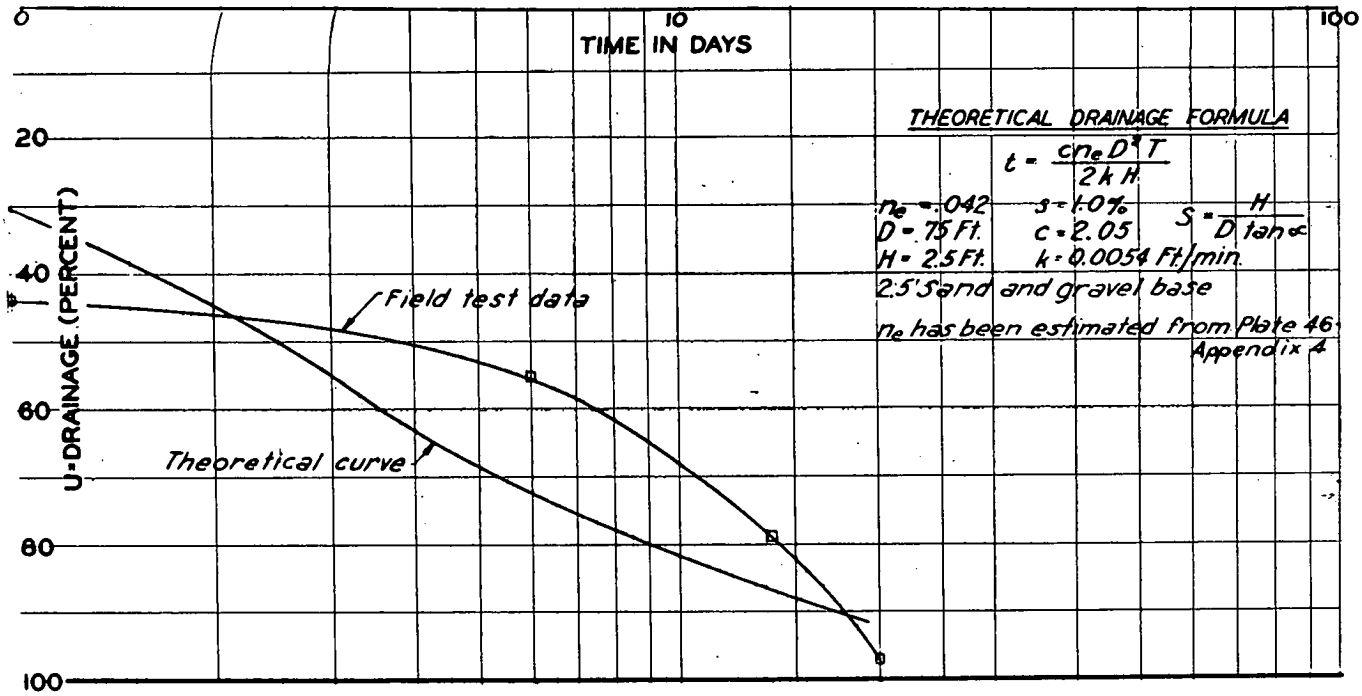


Figure 177. Correlation of Theoretical and Actual Time-Drainage, Presque Isle, AAF Test Area B

Bedford, Massachusetts. The pavement consisted of 3 in. of bituminous concrete over a bank run sand and gravel base course (GW) 6 to 9 in. thick over a medium sand (SW) which extends to a depth of 10 ft. It was indicated that although subsurface drains were installed, the water table is controlled by open ditches and the subsurface drains are not required.

Bangor, Maine. This field was selected for study because of the impervious clay subgrade and because extensive frost investigations were being conducted at this location. The 3½-in. bituminous concrete pavement is on a sand and gravel (GW) base from 3 to 4 ft. thick on a subgrade of silty clay (CL) with occasional silty, clayey sand and gravel (GC). The groundwater rose during the frost melting period suddenly a distance of 2 to 7 ft. to about 0.5 ft. above the bottom of the base. The groundwater to June 1st had not lowered appreciably.

Mt. Clemens, Michigan. The pavement consisted of 10 in. of non-reinforced concrete over 12 in. of bank run sand gravel (GW) base, on a subgrade of clayey, silty sand (ML) as indicated in Figures 178 and 179. Discharge studies and observation of water levels, by means of observation wells were made. The water level was high up in the base on March 15 but by March 18 all

observation wells indicated the water was well below the bottom of the base (Figure 180). Comparisons of computed and measured inflows from the base course and subgrade are shown in Table 59. They are in reasonably close agreement to those computed by the empirical formulae.

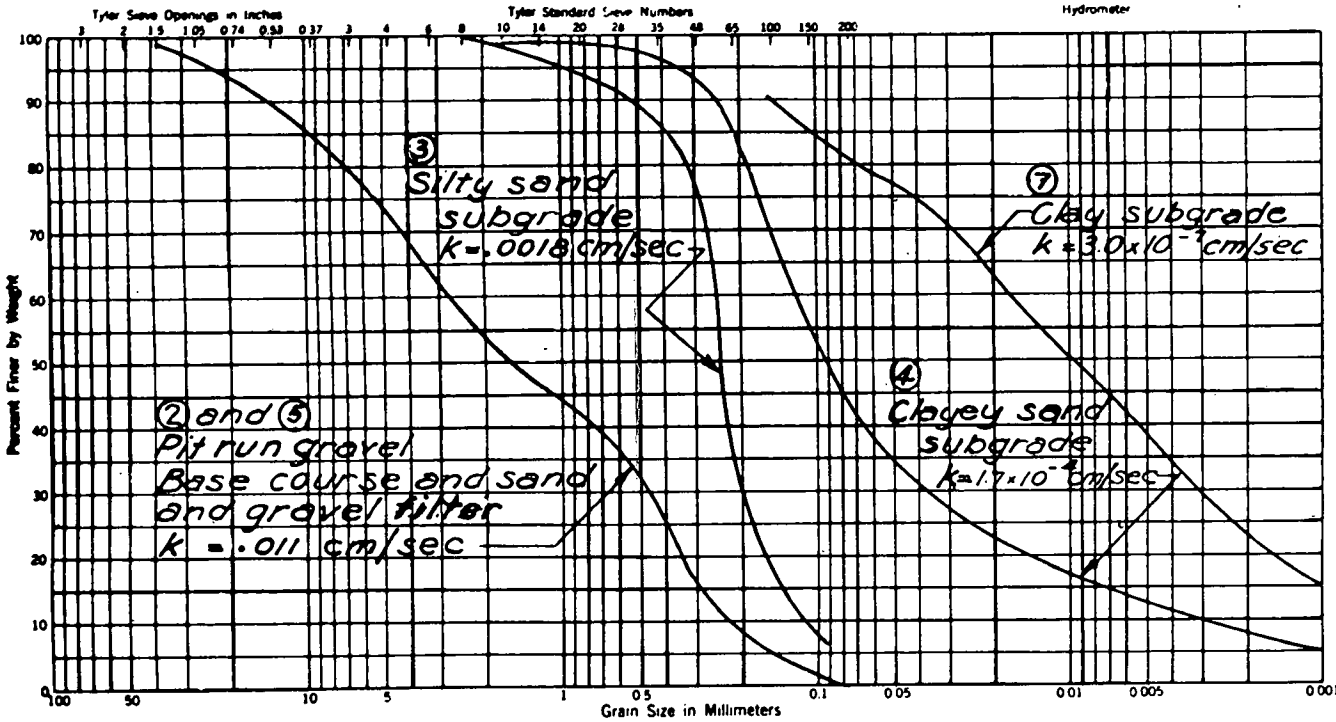


Figure 178. Mechanical Analysis of Soils, Selfridge Field, Michigan

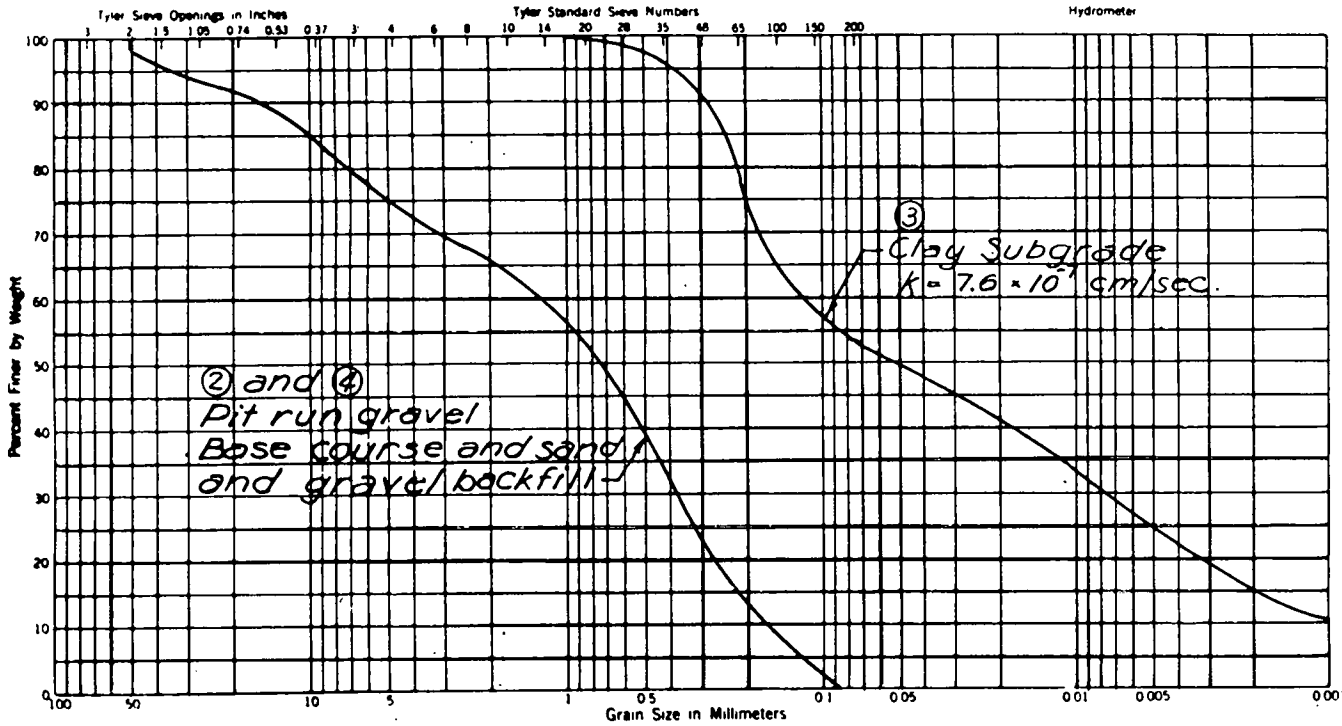


Figure 179. Mechanical Anslysis of Soils, Selfridge Field Michigan

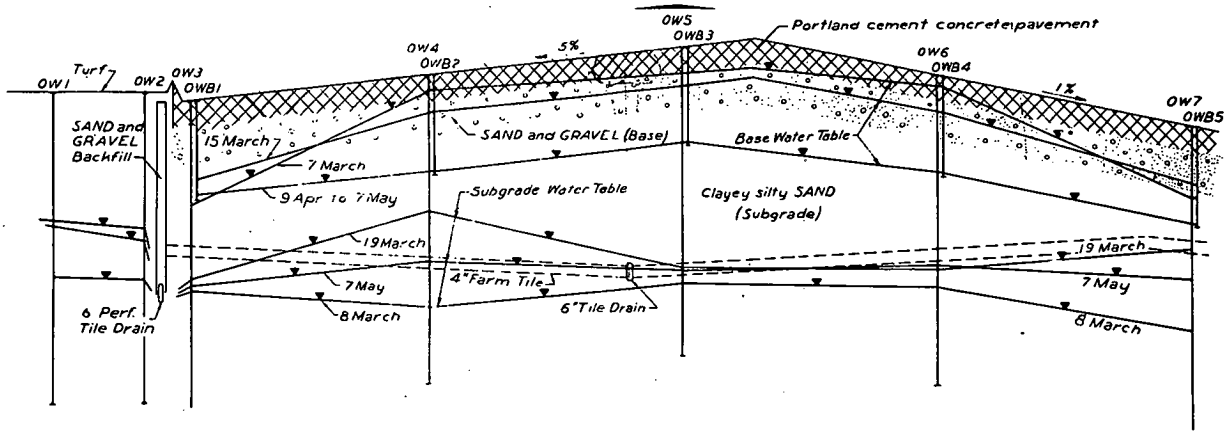


Figure 180. Apron Area - Section Through Observation Wells - Subsurface Drainage Investigation, Selfridge Field, Michigan

TABLE 59

(b) Comprehensive Sites								
Location	D (ft)	H (ft)	Slope (%)	Ho (ft)	k (f/m)	q (cfs) Computed	q (cfs) Measured	Date
TRUAX FIELD MADISON, WIS. TEST AREA "C"	400	3	1.0	5	.000058	.00001	.00628	15 March '46
TRUAX FIELD MADISON, WIS. TEST AREA "D"	75	.55	.44	.88	6**	.21122 Base	East Side	
	75	2.0	.44	2.33	.000058	.00004 Sub-base		18 March '46
						.21224 Total	.00757	
	75	.35	.44	.68	6**	.21105 Base	West Side	
	75	2.0	.44	2.33	.000058	.00004 Sub-Base		18 March '46
						.21109 Total	.0106	
SELFRIDGE FIELD MT. CLEMENS, MICH. RUNWAY TEST AREA	85	1.5	1	2.35	.036	.0199	.0107	15 March '46
(•) Subgrade Drainage Inflow (See Below)						.000031		
						.019931	.0107	

2. Correlation of Theoretical and Measured Inflow from Subgrade. Reference to Chapter 2-09h of "Draft of Recommended Changes to E.M. Chapter XXI						
$q = \frac{khc}{60}$						
Location	h (ft)	c (ft)	k (f/m)	q (cfs) Computed	q (cfs) Measured	Date
SELFRIDGE FIELD MT. CLEMENS, MICH	1.8	.85	.000015	.00031	(•) See above	
RUNWAY TEST AREA	1.3	.85	.000015	.000224	.000207	12 Dec. '45

Notes:

** Estimated from Grain Size Distribution Curve.
Coefficients of permeability from laboratory tests have been multiplied by 10 to allow for increase due to horizontal flow.

Madison, Wisconsin. Three test sections were selected. The types and thicknesses of pavement and base and type of subgrade were as follows:

	Section A	Section C	Section D
Pavement	2½-in. Bit. concrete	6-in. cement concrete	2½-in. bit. concrete
Base	8-in. crushed limestone	4-ft. sand-clay gravel (GF)	18-in. crushed limestone
Sub-base	16-in. sand gravel (GF)		24-in. sand-clay gravel (GF)
Subgrade	Gray silty clay (CL) and fine sand (SF)	Silty clay (CL) and fine sand (SF)	Gray silty clay (CL)

Observations

Section A. Contains no subsurface drains. Water was observed in the base before the freezing period and after the melting period apparently due to infiltration through and at the edges of the pavement. Water rose to within 0.2 ft. of the pavement on March 15 during the frost melting period and again on April 8 after a heavy rain. The limestone base drained during the three days following April 8 when the high water table was observed. The sub-base was completely drained on April 17. This indicates that the limestone base is satisfactorily drained by lateral flow in the sub-base.

Section C. Water rose almost to the top of the pavement on March 13 during the frost melting period, dropped until March 22 and again rose above the bottom of the pavement, then dropped approximately 30 inches by April 1 followed by another major rise and fall. The slope of the water surface indicates base drainage took place toward the side without subsurface drains as well as to the side with subsurface drains indicating flow through the sand-clay-gravel fill at the shoulder. A comparison of measured and computed maximum base course discharge for section C is shown in Table 59. The measured discharge is approximately 600 times the computed. The difference is believed due to a greater horizontal permeability than the permeability determined from laboratory test.

Section D. Water rose in the crushed limestone base during the frost melting period and after the heavy rain April 8 at which time peak discharges were recorded. The subsurface drains were apparently effective in maintaining the water level near or below the bottom of the limestone base. A comparison of theoretical and measured rates of discharge is shown in Table 59. The large discrepancy between theoretical and measured discharge quantities may be caused by the cross-section along observation wells not being typical of the test area.

Conclusions

1. The results obtained from this investigation during the fiscal year 1945-1946 in general substantiate the criteria for the design of subsurface drains given in Chapter 2, Part XIII of the Engineering Manual (Appendix A to the report).

2. Drainage of base courses on frost susceptible subgrades is required where the pavement and base thickness is not sufficient to prevent formation of ice lenses in the subgrade.

3. Based upon the field investigations, base course saturation is more likely to occur during the frost melting period but often results from infiltration of rainfall runoff through and at the edges of either bituminous or portland cement concrete pavements.

4. It is possible to determine within reasonable limits the time-degree of drainage relationship of a saturated base course on an impervious subgrade where reliable values of the coefficient of permeability in the horizontal direction, effective porosity, and height of capillary rise are known.

5. Clean pit-run gravels or crushed materials commonly used for base courses are sufficiently pervious to meet the design criterion given in Paragraph 2-08c which states that a degree of drainage of 50 percent in a base course shall be obtained in not more than ten days.

6. The inspection of drainage trench filter materials at the several field investigational sites substantiate the filter design requirements given in Paragraph 2-11 of the Engineering Manual. In no instance did detrimental movement of adjacent materials take place where the construction adhered to the design requirements.

The comprehensive report consists of 60 pages of text, 32 plates and 2 tables, and includes: Appendix A, consisting of Engineering Manual Part 13, Chapter 2, "Subsurface Drainage Facilities for Airfields" 23 pages, 10 figures; Appendix B "The Theory and Technic of Permeability Tests of Drainable Soils," 29 pages.

The Appendices to the comprehensive report are as follows:

Appendix 1 - Report on Theoretical Analysis of Drainage of Base Courses by Louis A. Pipes. 57 pages.

Appendix 2 - Report on Viscous Fluid Model Tests. 36 pages, 20 plates.

Appendix 3 - Report of Field Investigations. 69 pages, 70 plates, 13 tables.

Appendix 4 - Full scale Field Drainage Tests. 22 pages, 52 plates.

Altogether this constitutes a rather lengthy report. This abstract is intended only to convey a few of the principal findings and indicate the nature of the material contained in the report. A complete copy of the report is on file in our library and is available upon request on a 30 day loan basis.

Design for Base Drainage - The Corps of Engineers following completion of its subsurface-drainage investigation /1946-10 (abstracted in the preceding paragraphs), arrived at a method of design of base-course drainage /1946-19. The method provides means for computing (1) the rate of discharge of a base of given dimensions and properties and (2) the period of time required to obtain 50 percent drainage. The method is based on the assumptions that the base is saturated, no inflow occurs during drainage, the subgrade is impervious, and water can outflow freely from the base into the drain trench.

The Manual /1946-19 suggests the following equation be used for determining maximum rate of discharge for a saturated base course of the dimensions shown in Figure 181.

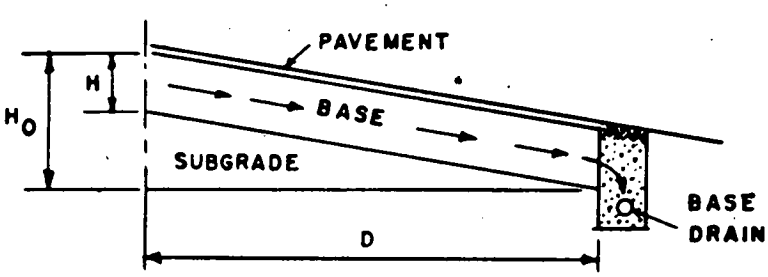


Figure 181. Design of Base Course Drainage

Where:

$q = kH \frac{H_o}{D_{60}}$

k is horizontal coefficient of permeability in feet per minute.

H, H_o and D are dimensions in feet shown on Figure 181.

q is the peak discharge quantity in cu. ft. per sec. per lineal ft. of drain.

Base-course design is based on the criterion that a degree of drainage /20 of 50 percent in the base course shall be obtained in 10 days or less.

Degree of Drainage - The degree of drainage is defined as the ratio, expressed as a percent, of the water drained in a given time to the toal amount of water that is possible to drain in a given material. Following is a simplified formula which may be used to determine time required to obtain a drainage of 50 percent.

$$t = \frac{D^2}{2kH_o} \times 10^{-5}$$

Where:

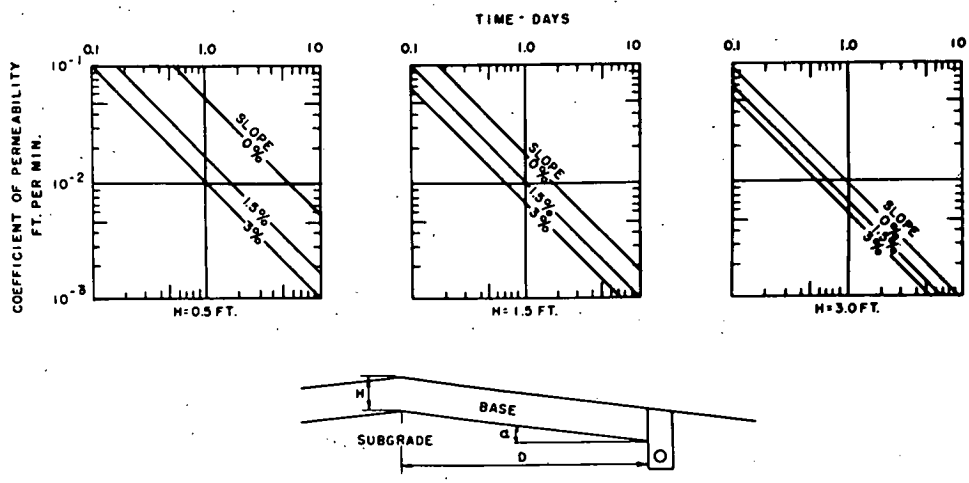
t is the time in days for 50 percent drainage

D and H_o are dimensions as shown in Figure 181 in ft.

k is the average coefficient of permeability in ft. per min. as determined by laboratory tests upon remolded samples of base material.

/20 The ratio, expressed as percent, of the amount of water drained in a given time to the total amount of water that it is possible to drain from a given material.

The time required for 50 percent drainage for bases of 6, 18, and 36 in. in thickness and having slopes of 0, 1.5, and 3 percent and base lengths of 75 ft. are shown in Figure 182.



D = 75 FT.
H = VARIABLE
s = TAN α
$n_e = 0.1$

NOTE: FOR EFFECTIVE POROSITY $n_e' \gtrless 0.1$, MULTIPLY TIME BY FACTOR $10n_e'$

COEFFICIENT OF PERMEABILITY IS AVERAGE VALUE DETERMINED BY LABORATORY TESTS ON REMOLDED SAMPLES.

Figure 182. Time for 50 Percent Drainage of Base Course (After Corps of Engineers)

Another formula may be used when the coefficient of permeability k is known for a direction parallel to the surface and the effective porosity is known.

Where:

$$t = \frac{n_e D^2}{2880 k H_o}$$

t is time in days
n_e is the effective porosity of the soil.
D and H_o are dimensions as shown in Figure 181, in feet.
k coefficient of permeability of the soil in a horizontal direction in ft. per min.

The following example, based on an assumed section as shown in Figure 182 and the values listed below, illustrates the application of the preceding formula.

$n_e = 0.1$
$D = 75$ ft.
$H_o = 3.625$
$k = 1 \times 10^{-2}$ ft. per min.

Then:

$$t = \frac{0.1 \times 75 \times 75}{2880 \times 0.01 \times 3.625} = 5.4 \text{ days}$$

Estimated average values of coefficient of permeability of sand and gravel bases are given in Table 60.

Slag and crushed rock (with few fines) generally has a coefficient of permeability in excess of 1 ft. per min.

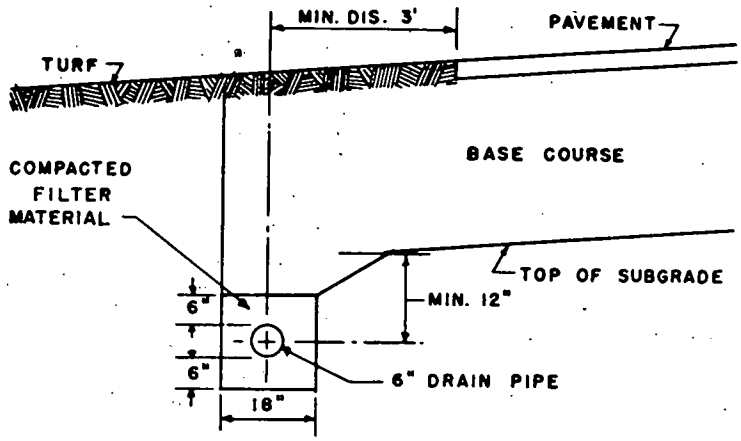
When computation shows the time required for 50 percent drainage to exceed 10 days, the spacing of the drains is decreased or a more pervious base material or a greater thickness of material is used.

Base drainage as described in the Engineering Manual /1946-19 usually "consists of subsurface drain pipes laid parallel and adjacent to pavement edges with pervious material joining the base with the drain." Figure 183 shows typical installations using different filter materials.

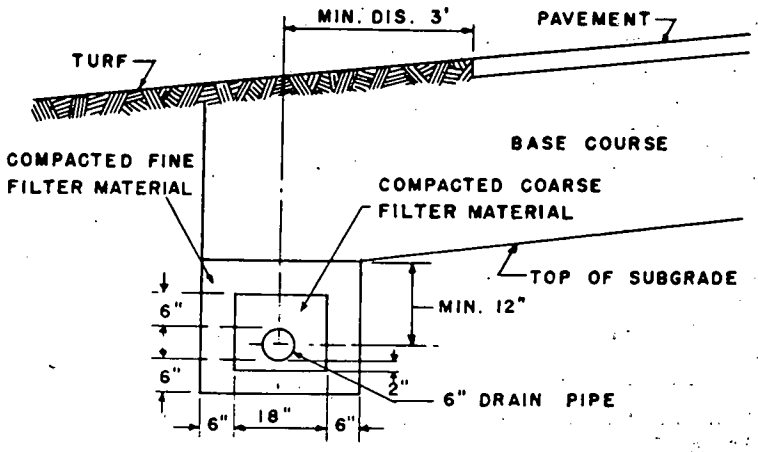
TABLE 60

Estimated Average Values of Coefficient of Permeability /21 of Sand and Gravel Bases. (After War Department 1946-19)

Percent by weight passing 200 mesh sieve	Coefficient of permability (ft. per min.)
3	10^{-1}
5	10^{-2}
10	10^{-3}
15	10^{-4}
25	10^{-5}



(A) ONE GRADATION OF FILTER MATERIAL



(B) TWO GRADATIONS OF FILTER MATERIAL

Figure 183. Typical Details of Base Drain Installations (After Corps of Engineers)

/21 Appendix A to reference 1946-19 provides a table for facilitating the estimating of the coefficient k for sand and gravel base. Design values are based on results of laboratory tests.

The Engineering Manual requires base drainage where frost action occurs in frost-susceptible /22 soils and where ground water rises to the bottom of the base course, and at locations where inundation may occur where the subgrade is relatively impervious. The following table gives values of permeability (of subgrade) suggested as a guide to where base drainage may be required where inundation occurs.

TABLE 61

Base Drainage Required if Subgrade Coefficient of Permeability is Smaller than the Following Ft. Per Min.

Dept. to Ground Water (ft.)	Coefficient of Permeability
Less than 8	1×10^{-5}
Less than 25	1×10^{-6}
More than 25	1×10^{-7}

Base drainage may also be required at the low point of longitudinal grades in excess of 2 percent except where k has a value of 1×10^{-3} or greater.

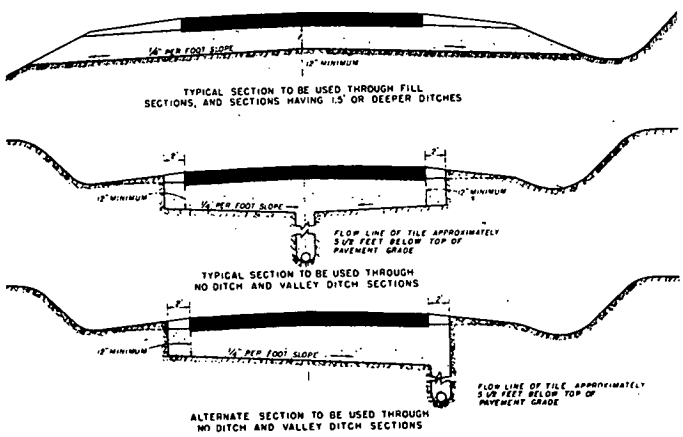


Figure 184. Typical Cross Sections for Sand Subbase (After Michigan Soil Manual)

Typical sections of granular bases and methods of drainage used in Michigan /1946-16 are shown in Figure 184. The Manual /1946-16 states, "Sand or gravel subbases are desirable where grades may not be raised and where fills may be constructed of high capillary soils.... To be successful the subbases must be drained. This is accomplished in the majority of cases by constructing them continuous from ditch to ditch on crowned earth grades". Otherwise they are drained as indicated in Figure 184, with adequate subgrade slope toward the drain.

Capillary Cut-Off Courses - It was shown early in this review that movement of water to the zone of freezing was due either to thermal differences or to differences in hydrostatic pressures, and that for practical purposes the major part of the flow was due to capillarity. The uses of courses of sand or gravel having very low capillarity to cut off the flow have been mentioned in several instances previously in this review. A number of investigators have proposed the use of capillary cut-off courses consisting of relatively impervious courses to prevent increase in soil water in the zone of freezing.

Griffin /1888-2 was granted a patent (No. 382,153) on the use of a course to limit the amount of water which enters the subgrade. He prescribed first removing earth 6 to 16 in. in depth to an even surface, then moistening and rolling the soil. He then suggested construction of drains on each side of the road as indicated in Figure 185 "to take out the surplus water from the roadbed and thus prevent the road heaving by frost." After bringing the road to the elevation indicated by the number 3 in Figure 185, he suggests placing a felt or any other water proof material, then placing a layer of cement concrete which in turn is covered with tar to form a "water-tight layer."

Lowell /1918-6 held that there is a marked difference in moisture distribution under a pavement, the area under the center of the pavement having low moisture content areas under the edges having maximum moisture content. He believed that since soil on freezing expands in proportion to its moisture content the difference in moisture distribution (Figure 186 causes unequal

/22 Based on the manual's definition of frost susceptibility.

pavement distortion. He suggested that, as a means of preventing the occurrence of non-uniform moisture distribution of the nature he described, a vertical cut-off wall be constructed under the edges of the pavement. He indicated the "old style" 36-in. deep curbing with drain tile laid outside was the nearest approach made at that time but suggested that a 3/8-in. thickness of "continuous jointed or lapped sheets of bituminous material" be used to form underground walls 4 to 5 ft. deep along each side of the pavement with a 6-in. drain tile laid "on a good grade" just outside the base of the walls. (see Fig. 187).

Taber /1929-2 found that a relatively thin layer of coarse sand will stop upward movement of water by capillarity and thus prevent formation of ice layers. Casagrande /1938-5 suggested both water proof courses and courses having very low capillarity. For courses of coarse materials (gravel and chippings) of the material, and such courses should be underlain with a filter course to prevent infiltration of fine-grained soil. Sourwine /1939-12 was granted a patent which provided for an "earthen subgrade foundation which shall resist the entrance of water in capillary form and by so doing shall limit the occurrence of ground freezing in such earthen foundations." His patent also included the use of admixtures which lowered the freezing point for soils. A brief review of the patent has been given under "Admixtures". Mamanina /1944-2 reported that the rise of capillary moisture was completely prevented by the use of coarse sand 0.09 to 0.02 in. (about 60-mesh to 270-mesh material) and that relatively thin layers (1.5-in.) give appreciable effect if the material was cleaned carefully. Williams /1945-5 held that both depth of freezing and winter thaws were greater under the pavement than under shoulders, and that a frost partition existed. Precipitation and melting snow caused a state of saturation above the frost partition. Williams suggested sealing the shoulders as indicated in Figure 188 to prevent surface runoff water from entering the subgrade.

Beskow /1938-11, /1947-12 illustrated the use of capillary "cut-off" courses in Sweden. One method is illustrated in alternative No. 2 in Figure 156 in which the layer of coarse sand acts as an "isolation layer," breaking the capillary connection with overlying backfill of frost susceptible soil. Figure 189 illustrates the use of a capillary cut-off course which also acts as a drainage course. Beskow illustrates in Figure 191 examples of how not to construct layers to cut off capillary water.

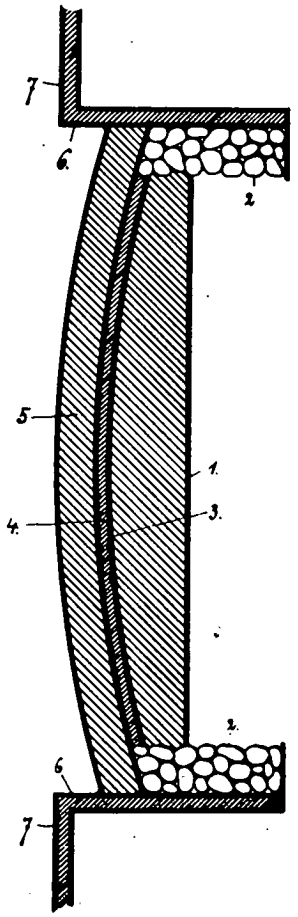


Figure 185
Griffin's Patented Method of Paving Streets and Roads
(U.S. Pat. Off.)

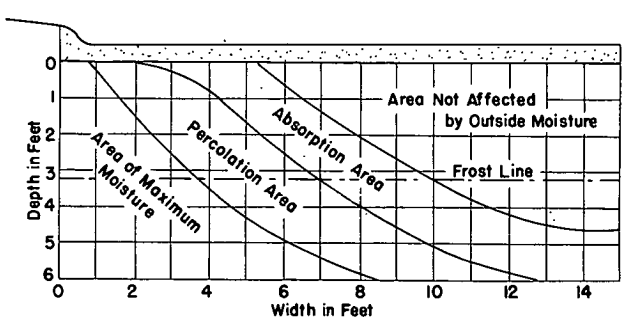


Figure 186. Distribution of Moisture Prevailing in Unprotected Clay and Loam Subgrades. (After Lowell)

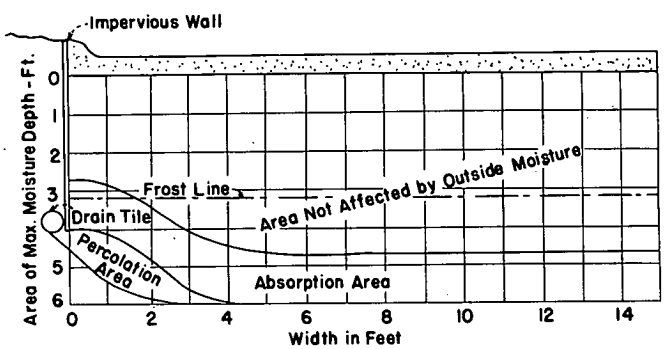


Figure 187. Effect of Impervious Wall Upon Distribution of Moisture in Subgrades. (After Lowell)

Surface Drainage - A few writers have indicated that surface drainage can serve to aid in reducing damage due to frost. Harrison /1918-2 suggested that ditches should be deep enough below the bottom of the pavement so there will be no leakage from ditches to the subgrade.

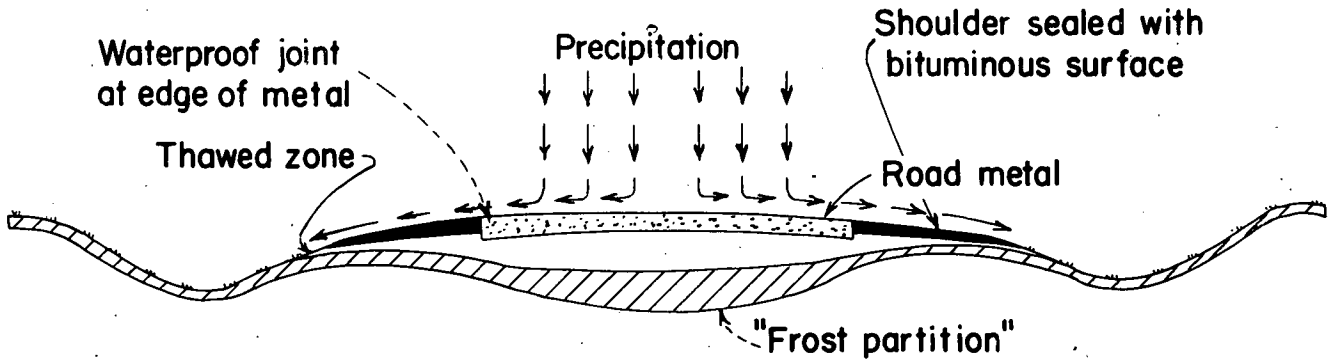


Figure 188
A third remedial measure. Capping of shoulders with bituminous treatment carries precipitation to roadside ditches, and prevents occurrence of excessive saturation of upper thawed zone. (After Williams)

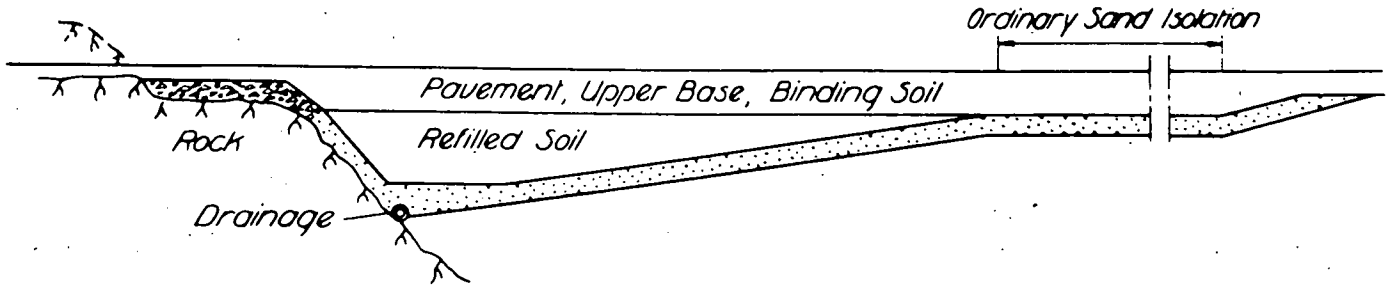


Figure 189
Use of "sand-wedge" construction, for smoothening transition between different frost heaving road sections (avoiding abrupt ledges on surface of frozen road). Figure represents a soil cut (strongly frost-heaving) with rock base. Deep transition wedge between rock and frost-heaving soil; also the ordinary sand insulation layer in the soil cut ending with a short wedge. (After Beskow)

Erlenback /1937-3 reported that drainage by side ditches or channels proved entirely ineffective in preventing frost damage. L. Casagrande /1938-5 also held that "road ditches do not prevent damage by frost, and...deep drainage only does so in very exceptional cases." The exceptional cases refer to relatively permeable soils. His later report /1940-8 restated that side ditches cannot prevent frost damage. Smith /1948-28 brought out that the most common and most disastrous type of freeze damage in the Amarillo area in Texas occurs near the pavement edges where surface water (some from melting snow) enters the base. He found the damage small where adequate shoulder slope was used. He suggested that "a minimum shoulder slope of 1-in. per ft. should be used in designing crown width or so-called feather edge base sections...that at least a seal coat should cover the shoulder slope and thus provide water proofing." He stated that a side slope not flatter than 7 to 1 with 6 to 1 the usual slope and a minimum ditch depth of 1.5 ft. will provide better drainage for the melting snow.

Design to Limit or to Prevent Freezing

Insulating Courses - Beskow /1938-11 reported the use of insulating courses of moss, straw, or needle brushwood on gravel roads in Sweden. The manner of placing the materials is indicated in Figure 190. Beskow also illustrates in Figure 191 how not to construct insulation layers in trench-type construction.

Later Beskow /1947-12 reported the use of insulating materials for railroads and rigid pavements (P. C. concrete) on strongly frost-heaving ground. To obtain a backfill with great resistance to frost in order to limit the depth of excavation Scandinavian engineers have used wet moor. Frost penetration in wet moor is about half that in moist sand. Beskow suggested that

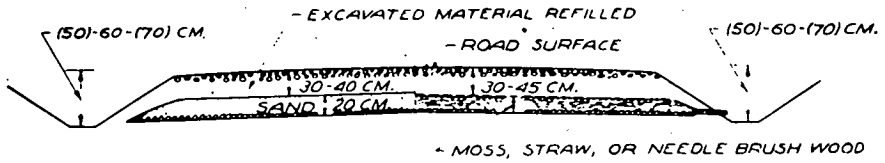


Figure 190. Insulation against Frost Damage. Figure shows excavation filled with original frost-heaving soil above the insulation layer. This is cheap construction and satisfactory for gravel roads of minor importance. On more important highways where a bituminous surface is used, a more satisfactory soil or a stone foundation is used above the insulation layer. (After Beskow)

when used as fill under flexible pavements, the minimum thickness of base and pavement should be 0.5 m. (1.64 ft. or 19.7 in.). A 30 cm. (11.8 in.) layer of moor down to a depth of 0.8 m. (31.5 in.) would prevent freezing as long as would a sand layer to a depth of 1.1 m. (43.3 in.). If then the moor is covered by a 20 cm. (7.9 in.) layer of very porous slag, the frost resistance of the combination with a depth of only 0.8 m. (31.5 in.) would be equal to that of a sand fill of 1.3 to 1.5 meters (51.2 to 59.1 in). A graphical example showing the effect of moor is given in Figure 192. The sand fill, if deep, has greatest frost resistance if the bottom part is kept wet to provide the greatest frost "storage capacity."

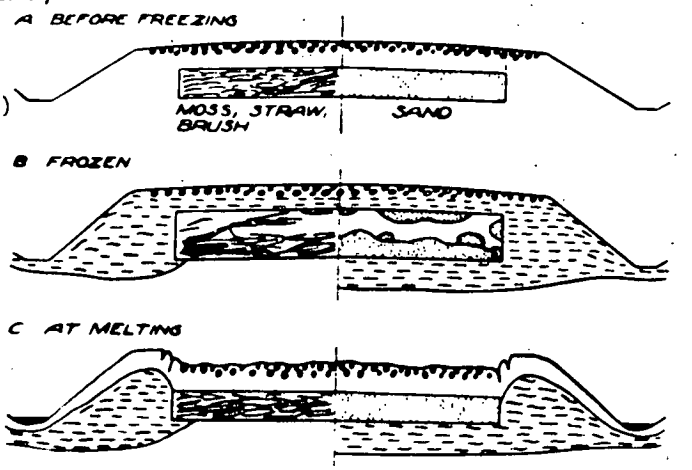


Figure 191. How Not to Construct Insulation Layer. (After Beskow)

Beskow reported that Norwegian railroads were using hard, molded blocks of moor 1 by 0.5 m. and up to 0.5 m. thick placed in uniform layers. He reports some settlement due to consolidation of the moor. That settlement ranges from 50 percent for Sphagnum moor fill to 20 percent or less for Sphagnum moor blocks. Skaven-Haug /1951-45 gave much added detail regarding the Norwegian practices. They developed methods for computing the penetration of frost into soil. He expressed the resistance to frost, R, in a layer deposited at a given depth under other materials by the equation:

$$R = y \frac{d^2}{2}\left[\frac{1}{\lambda} + \frac{2}{d}\left(\frac{1}{\alpha} + \frac{do}{\lambda}\right)\right] \qquad \text{Denomination h deg. C.}$$

The total resistance to frost ΣR of the combined layers that lie upon each other and freeze completely through in the course of the winter is equal to the amount of frost F. The symbols of the equations have the following meanings:

d - thickness of layer in meters.
q - cold-storing capacity of material in K. Cal. per cu. m.
λ - heat conductivity of material in k cal. per m. h. deg. C.
α - heat transmission constant between surface and air in k. cal. per sq. m. h. deg. C.
$\Sigma \frac{do}{\lambda_o}$ - resistance to penetration of heat in top layer in sq. m. h. deg. C. per k. cal.

In Table 62, below, heat constants for the most commonly used materials are given, based on measured water content along the line in the course of the winter.

TABLE 62

Substance	V Percent Water by volume	k cal[q] per cu. m.	λ k cal per m. h deg. C.
Hard-packed snow	50		0.5
Ballast stone	8	7800	0.57
Ballast gravel	13	12300	0.80
Cinders (incl. ash)	20	17400	0.40
Peat	85	70700	1.05

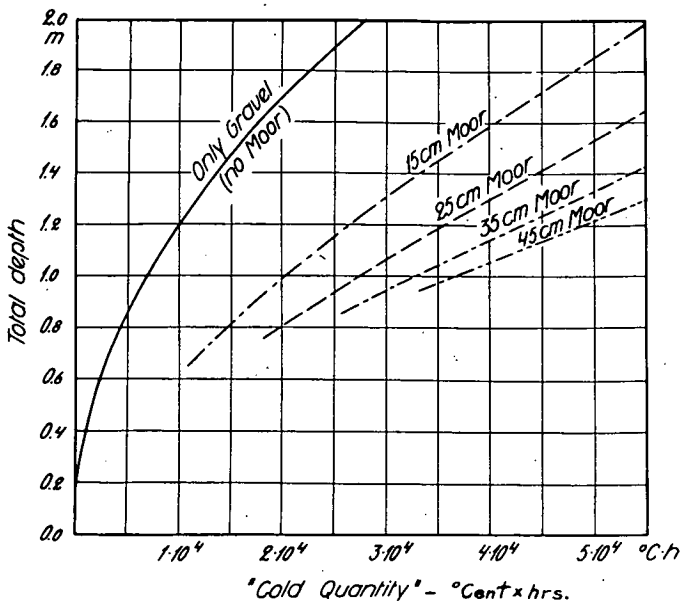


Figure 192. (Redrawn from Brudal 1945). Frost safe excavation depth for gravel back fill on a layer of wet moor. Minimum depth to top of moor layer = 0.5 m, calculated for 4 different dimensions of moor layer. Material constants:

Material	Gravel	Moor
Water conent F (vol %)	7	70
Heat conductivity, λ (cal/cm.sec. deg. C.....	0.0014	0.0025
Frost storing capacity, Q (cal/cm³).....	7	58

Note: No reduction for heat conduction from below. In reality the necessary depths are somewhat smaller. (After Beskow)

Compressed peat consists of baled sphagnum peat 1 m. long, ½ m. wide, and ranging in thickness from 0.3 to 0.5 m. The blocks are laid as indicated in Figure 195.

Figure 193 shows the necessary depth of replacement (d) for different materials depending on the amount of frost. This is based on the assumption of stone ballast 0.5 meter deep which after freezing is covered with a layer of hard packed snow of 0.06 m. (2.4 in.) in thickness. Figure 194 shows the depth of replacement when there are 0.20 m. (7.8 in.) of pressed peat at the bottom of the trench, and the remainder of the trench is filled with gravel, stone or cinders. It may be seen that to obtain the greatest resistance to frost, the top layer of ballast should be as dry as possible and the bottom layer as wet as possible (and have the greatest cold-storing capacity).

Computations show that for a cold quantity of 33000 h. deg. C. the road bed may be built of a combination of 0.50 m. broken stone ballast over 0.87 m. cinders or of a combination of 0.50 m. broken stone ballast over 0.38 m. compressed peat.

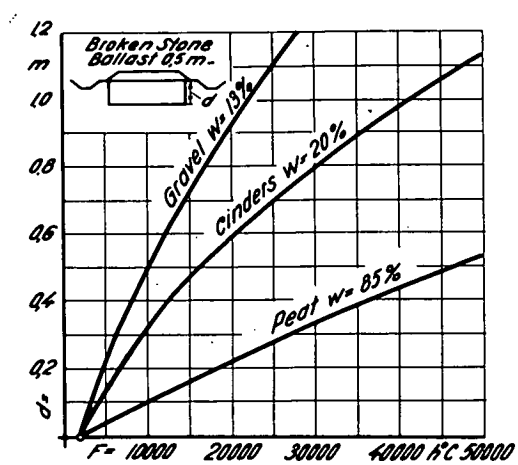


Figure 193
Frost Resistance
(After Skauven-Haug)

During the winter of 1945-46 observations were made of frost penetration on five sections of railway. The results for one section are shown in Figure 196. The amount of frost in cumulative deg. C. hours is shown in the upper half of the figure. The middle graph shows the calculated and observed depths of penetration through the ballast (which showed an increasing water content from 10.5 to 18.9 as winter progressed) and the peat mat which had a water content of 86 percent. The lower graph shows that the frost heaving in the ballast layer was about 2 mm. and in the completely frozen peat 15 mm. With 86 percent water (by volume) in a peat layer 0.4 m. (15.75 in.) thick the expansion of water to ice would cause a 30 mm. (1.2 in.) upward movement. The difference may be due to the compression of the peat. Movements of 30 mm. are small and do not affect railroad traffic.

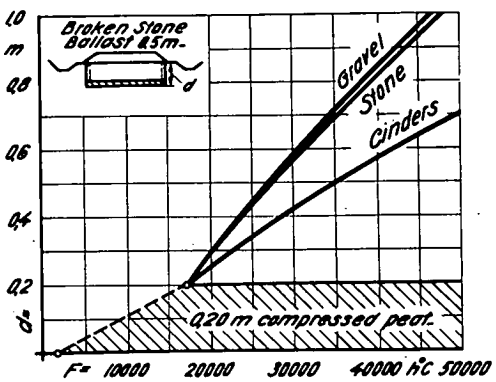


Figure 194
Frost Resistance
Capacity 33000 h deg. C.
(After Skauven-Haug)

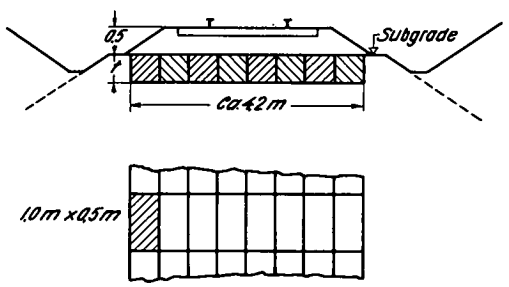


Figure 195
Methods of placing compressed peat blocks under railroad ballast (After Skauven-Haug)

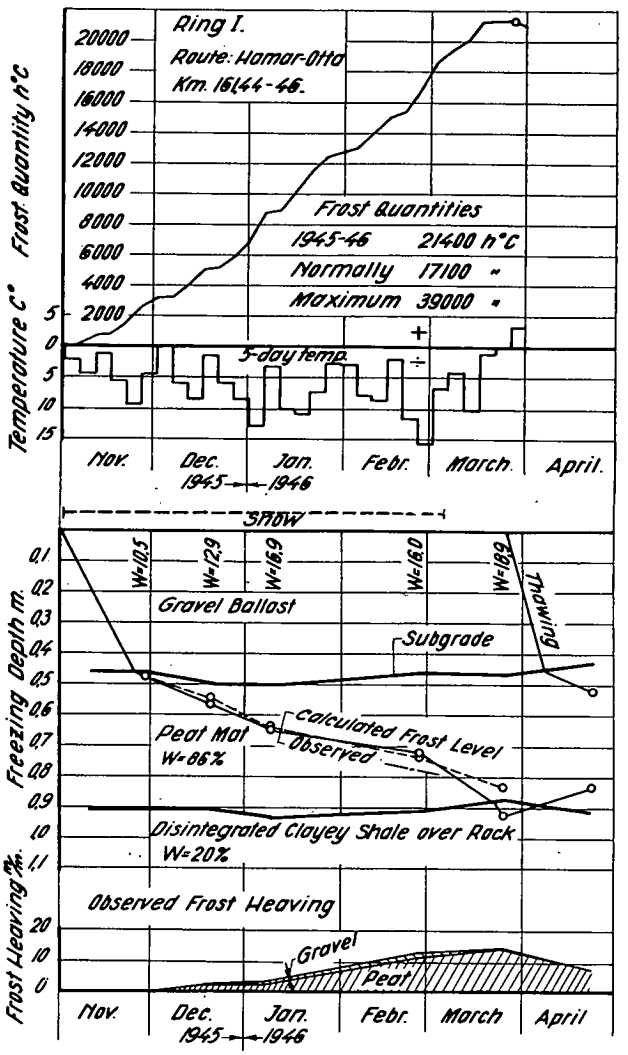


Figure 196. Observations of Frost Penetration in a section of Norwegian railway and observed heaving in peat blocks and gravel (After Skauven-Haug)

The Corps of Engineers /1947-2 concluded from its tests for thermal conductivity of base materials in the unfrozen state that the "thermal conductivity of slag and cinders is about one-half that of other base materials such as sand, sand and gravel or crushed rock. Since the depth of frost penetration, all other conditions the same, varies with the square root of the coefficient of thermal conductivity in frozen state it may be concluded that the depth of frost penetration into cinders or slag would be about two thirds of that into sand, sand and gravel, crushed rock. This conclusion is contingent upon cinder or slag having approximately the same ratio of thermal conductivity in the frozen state as in the unfrozen state to that of sand, sand and gravel or crushed rock." The Engineering Manual /1946-5 provides for a reduction in the combined thickness of base and pavement when an insulating material is used. Provision is made that "in the case of slag or cinders 4 in. may be substituted for every 6 in. of sand, gravel or crushed rock." That provision does

not hold when design is based on a reduction in strength of subgrade. An example of design in which the insulating value of slag or cinders is considered is given in a preceding paragraph under "Design Methods."

Use of Admixtures - The use of admixtures has been covered previously in this review.

Protection of Slopes

Although there is a vast amount of literature on the subject of soil erosion, the writings, with one exception, contain few data on the effectiveness of frost as a soil-loosening agent to facilitate erosion or to cause sloughing or sliding of surfaces of slopes. That exception concerns the effect of frost as a factor in causing soil flow in arctic areas. The reviewer found one article which gives information on treatments to prevent soil movement on slopes, and that is the work of Lane /1948-24 on "Treatment of Frost Sloughing Slopes." No doubt additional information could be obtained if a search were made of work done by horticulturists in the planting of slopes with shrubs to prevent frost sloughing.

Lane described the usual form of frost slough in which the top 6 to 30 in. of surface soil moves down the slope, filling any ditch present. He cites cases in central New York where such flows have covered highways. The reviewer has seen several flows of that nature occur during the frost melting period in the central west. The movement usually takes place when the soil has thawed to only part of its full frozen depth. The underlying cause is the heaving and the large increase in soil water accompanying heaving which on thawing leaves the soil in a loose, saturated condition and having very low shearing strength.

Lane showed that a silt having a dry weight per cu. ft. of 100 and a water content of 25 percent would, if it heaved 40 percent (increased in length of a cylinder of soil from unfrozen to frozen state), have a dry weight of 72 pcf. and a water content of 50 percent. He cited nature's methods of protecting slopes by bonding the surface to underlying soil with deep roots: covering with an insulating layer of vegetable matter or creating a more pervious surface soil. He also described successful methods including (1) the use of cinder blankets 6 to 12 in. thick (one was 3 ft. thick) by New England railroads; (2) the use of 12 in. of sand plus 6 in. of topsoil on a New Hampshire road cut slope through varved silt; (3) the use of 24 in. of gravel plus 6 in. of topsoil on the Keene, N. H., airport; (4) the use of 24 in. of sand and 3 in. of topsoil on the Rutland, Vermont, airport; and (5) the use of a 24 in. sand blanket on 1 to $1\frac{1}{2}$ slopes on a Vermont road. From those experiences he concluded that a pervious blanket was considered a satisfactory treatment for frost sloughing. Unless there is also a problem of seepage, the addition of underdrains appears unnecessary. He considered the cinder fill slightly superior because of its lower thermal conductivity.

Hursh /1948-7, in discussing the planting of slopes in the United States, says that repeated freezings and thawings bring about the formation of ice crystals, the principal cause of surface instability of bare soils on slopes. He held that "alternate freezing and thawing of exposed soil can occur 50 to 75 times during a single winter. Freezing and thawing is most severe on moist south-facing slopes where it will account for the erosion of one foot of soil in a single winter on a 1 to 1 slope clay bank."

He found that ice crystals (needle ice) will raise fragments of soil which settle farther down slope or roll into the ditch and that uniform and polished slopes are particularly subject to this type of frost action.

Design Methods for Structures

Design engineers in areas when frost penetrates deeply and where soil heaving occurs are well aware of protective measures which need to be taken for structures. No effort has been made toward a complete review of frost action with respect to structures. The literature cited is intended merely to call the problem to mind so those who are not familiar with it may make a more complete study. While any type of structure not founded below frost (and even those founded below frost depth should the soil adfreeze to the sides of the structure) may be susceptible to movement, only retaining walls, piers, and buildings are considered here.

Retaining Walls - The American Air Force Aviation Manual /1945-9 states that "heaving produced by frost action in soils used to backfill retaining walls will gradually force them out of line." The Manual suggests that silty and clayey soils should be avoided for backfilling purposes and that if clean granular fill material is available it should preferably be placed

in a wedge as indicated in Figure 197c. If the supply is limited it should be placed as indicated in Figure 197b but the thickness should be no less than the expected frost penetration. In any case, weep holes should be provided for drainage.

Terzaghi and Peck /1948-12 showed illustrations for draining backfill to prevent formation of ice layers by means of a blanket of pervious material over the original ground slope. The backfill along the inside toe of the slope below ground level (on the exposed side) is made of impervious material. Tile drains are provided for removing any water intercepted and collected by the pervious slope blanket.

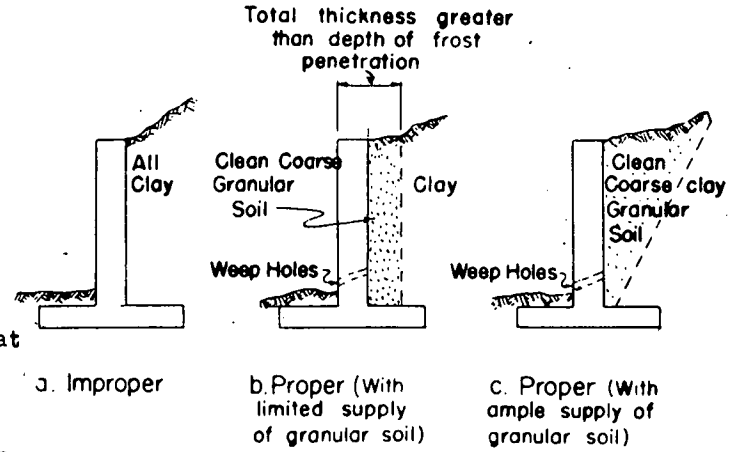


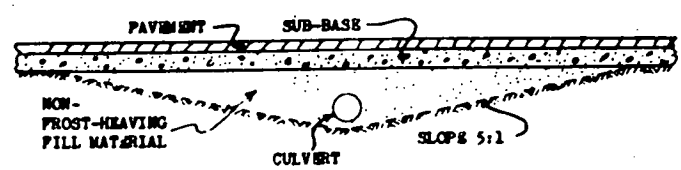
Figure 197. Backfilling around retaining Wall. (After AAF Manual)

Piers - It has been mentioned previously that heavy piers and abutments have been heaved out of position due to the action of frost where the depth of frost exceeded the depth of the pier. McCready /1923-6 showed that it was possible for a pier to be lifted due to adfreezing of and heaving of the adajcent soil. McCready did not offer a design solution for similar cases but it is evident that founding to below depth of frost does not always constitute satisfactory design. It may be necessary to backfill adjacent to the pier with granular material and provide adequate drainage.

Culverts - Keene /1951-43 brings out that a special case of frost action under pavements is due to what he terms "chimney action" in culverts, especially in cross-road pipe culverts of large diameter. Cold air circulates through the pipe, unless blocked with snow or full of water, and causes frost penetration into the earth around the pipe. This type of frost penetration will result in vertical and horizontal heaving if the earth is a frost-heaving type and sufficient moisture is available.

An example of harmful chimney action is in a central Connecticut city where a dual-lane expressway crosses a wet area for about one mile. The 9 in. concrete pavement and 12 in. gravel subbase are on a low fill of alluvial silt; top of pavement is 4 to 7 ft. above ground. The ground is swampy, with water table at or near the ground surface. Elevations taken on 100 points on the pavement revealed that the ordinary heave here was ½ in. to 1½ in., but at the four 36 in. and 48 in. cross culverts, heaves were 1 in. or 1½ in. greater. The pavement was not reinforced, because of the steel shortage, and large transverse cracks developed at and near these culverts because of the non-uniform heaving. At the numerous 12 in. cross culverts, extending only one half the width of the expressway, heaves were about ½ in. greater than normal and cracks developed over and near them also.

A good remedy for chimney action is to place non-frost-heaving fill around the pipe for a thickness equal to the frost penetration around the pipe. Above the fill would be the normal embankment material. If this amount is difficult to determine, an easier method is to fill with the good material up to subgrade and for some distance each side of the culvert, as in Figure 198. If the culvert is far below the pavement, this method would be expensive. In such cases, no remedy would be necessary, except in unusual cases, as the heaving around the pipe would be diffused over a wide area by upward arching in the embankment and largely absorbed by compressive strains in the embankment.



(Note: Foundation Conditions Under Pipe and Adjacent to Pipe Should be Uniform to Make Settlements Uniform (If Any))

Figure 198. Method of Eliminating Serious Differential Frost Heaving Due to Chimney Action in Culver. (After Keene)

A working rule in Connecticut is to use one of the above treatments if the embankment will be a bad frost-heaving soil and if top of pipe will be less than 4 or 5 ft. below top of pavement and flow line is less than 4 ft. above ground-water table.

Chimney action at box culverts is not a problem as the roof is usually 12 in. or more

in thickness and more than 5 ft. above water table. Pervious material placed for several feet beyond the walls serves to prevent capillary rise from below.

Frost Safe Foundations for Buildings - Beskow /1947-12 suggests that the "first rule" is to extend the foundation to a "frost-safe depth." However, even if the footing is deep below frost line heaving may occur due to adfreezing (see piers) if soil moisture and freezing conditions are severe. For vertical foundations Beskow recommends the use of a coarse sand or fine gravel to a distance of 25-30 cm. (about 10-12 in.) out from the wall. If the footing is trapezoidal in shape, heaving due to adfreezing is not likely to occur.

Foundations for Cold Storage Warehouses - Several authors have described the effect of freezing of soil under cold storage warehouses. Cooling and Ward /1948-32 suggest that "The simplest way to avoid heaving under cold stores, where the ground conditions are favorable for frost heaving, is to build the structure above ground with a free air-space beneath." He cites instances of -6 to -10 C. where a 6-in. slab of baked cork was not sufficient to prevent the frost from penetrating several feet in the ground in 3 or 5 years. In one of the cases mentioned, where such freezing occurred, a low-temperature electrical heating grid was placed just below the 6 in. of cork insulation. About 0.15 watt per sq. ft. of energy was used. The input was controlled by relays operated by thermocouples embedded in the soil so that the temperature was maintained just above freezing.

Ruckli /1948-26 calculated the insulation necessary to maintain an above-freezing temperature under a cold storage basement. He also computed the floor heating necessary to prevent penetration of freezing temperature (0 C.) into the ground. Ruckli's report listed eight references on the subject.

CONSTRUCTION AND MAINTENANCE PRACTICES RELATIVE TO FROST ACTION

Much of the information given previously or later in this review has practical value and can be applied in construction and maintenance of roads and airfields. A few items which have been overlooked thus far, or which pertain directly to construction or maintenance, are given here.

Subsurface Drainage (With or Without Excavation and Backfill)

Arnold /1917-5 made a study of 96 failed areas totaling 16 mi. on bituminous macadam in New York (Wyoming County). Fifty-seven were on fill, 17 in cut and 26 at grade points. He found the cause to be water entering the pervious bottom course in cuts at the top of grades and flowing downhill to fill sections and suggested that a stone-filled V-shaped trench pointing uphill and placed at the end of every clay cut will prevent breaks in such locations. He also suggested placing weeps through shoulders at 50-ft. intervals.

White /1928-2 also reported good success with the French-type drains. He marked boil areas in the spring. Later when they had become stable he cut trenches 4 ft. wide by 30 in. deep and filled them to within 8 in. of the surface with "one man-size common field stone, placing any smaller stones...on top." He then added 12 to 15 in. of gravel and also provided a 6-in. tile outlet. Regarding the performance of the drains, White stated, "Although the spring of 1927 showed severe frost boils in other areas, no location where drains had been installed failed." Henton /1928-6 and Buetow /1929-3 cite similar good results from installation of drains in Wisconsin.

Lang /1930-4 described the use of a steam jet through heaved frozen areas to provide vertical drainage. He found it was not permanently successful. Lang emphasized the futility of placing drains above the water table in soils having high capillary lift. He showed a center drain construction which "successfully cured a very bad section of frost boils which occurred in a silty soil." The depth of the drain ranged from 3½ to 5 ft. Coarse gravel backfill was used. Later, Motl /1931-2 described several types of frost-boil cures based on the principles of excavation and backfill with pervious material and outlet drains. He stated, "The designs vary in top width of trench from 22 ft. to only 2½ ft. while the extreme depth is kept at from 2½ to 3 ft. Briefly described, the various types can be grouped about as follows: (a) longitudinal center line trench 2½ to 3 ft. wide and 2½ to 3 ft. deep, backfilled with porous material and having outlet trenches at suitable intervals; (b) same as (a) except that longitudinal and outlet tile or pipe drains are used near bottom of trench; (c) longitudinal center line V-trench 10 to 20 ft. wide on top and 2½ to 3 ft. deep at center, backfilled with

porous material and having outlet drains at low points of profile; (d) same as (c) except provided with open subdrains as in (b); (c) longitudinal flat trench 22 ft. wide on top, 2½ ft.deep at the sides and 2 ft. deep along the center line provided with side tile longitudinal drains and metal pipe outlets."

Although frost penetration in Minnesota reaches a depth of 5 ft., Motl chose to place drains at a 30-in. depth, because (1) frost boil trouble was critical when thawing reached about 2 ft.; (2) greater depth of installation made costs excessive; (3) outlets exposed to air are preferable to deep tile connections and can be obtained readily if depth is held to 30 in.; (4) the added time for thawing to penetrate to a deep drain may mean the difference between failure and success. He found that tile drains laid at 42 to 54-in. depths in herringbone arrangement under coarse gravel were unsuccessful in many locations.

Gould /1931-7 (Washtenaw Co., Michigan) installed over 6 mi. of drains 10 in. wide and 4 ft. deep in the center of the road; 2½ in. maximum size stone and crushed gravel were used as backfill, and 4-in. tile drains were used to remove the water. The method was said to be successful in preventing frost boils.

Williams /1945-5 presented his theory on the cause of spring break-up as being due to infiltration of surface water (and its reaction) above a frost partition under the pavement. He suggested an effective method of releasing excess water is to make openings through the frost partition with a steam jet. Another method is to dig holes with a post-hole auger and backfill with a mixture of crushed gravel and 25 lb. of calcium chloride to assure results lasting several years. Williams illustrates this method in Figure 199.

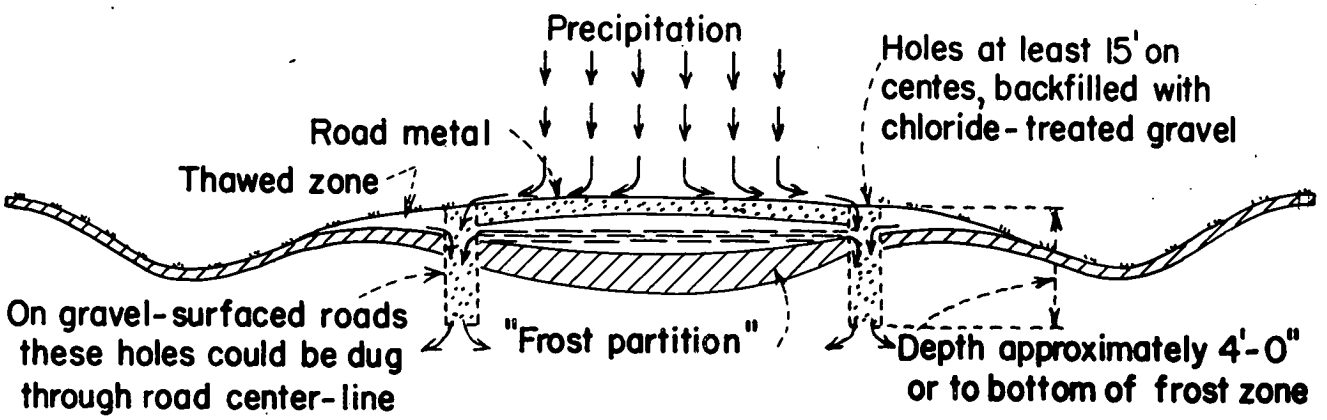


Figure 199. Subterranean drainage one remedial measure. Here the subgrade has been provided with pits of chloride-treated gravel, to allow percolation of excess free water through "frost partition". (After Williams)

Fuller /1951-44 states that the most severe heaving occurs in cut sections, and that the causes are too much water and not enough drainage. The ditches may be too shallow or too narrow or both, or drainage may be blocked by snow banks forcing water to be held on the road. Fuller held that broken and cracked pavements "totalling countless miles in extent will attest the frost damage which has resulted from this one condition alone."

Unexposed rock ledges also tend to prevent drainage. These should be undercut by a deeper ditch or drained by a tile under-drain. Fuller also explains that an excessive growth of underbrush along backslopes of narrow cuts sometimes is a source of heaves. He also stresses proper marking of outlets for subsurface drains, keeping the ends raised so they will not clog. Blocked farm entrances are listed as a cause of heaves.

Fuller also stresses the need for keeping concrete and macadam pavements well sealed, keeping shoulders well sloped so they will drain, and if shoulders tend to be rutted by traffic, to widen the pavement with a bituminous mix.

Use of Chloride Salts

The use of chloride salts as a maintenance procedure for correcting frost heaves and frost boils has been reported on several occasions. A project committee of the Highway Research Board made several experimental installations and reported on the effectiveness of the use of calcium chloride for alleviating frost heave.

Miller and Smith /1934-2 reported on a method in which 1-ft. diameter by 2-ft. deep holes were dug at 4 to 8 ft. centers in a heaving and frost boil area on a gravel road. Water-proof paper was placed on the bottoms of the holes. The holes were backfilled with a mixture of pea gravel and calcium chloride. A second method consisted of pumping a solution of calcium chloride into the road through holes in which well points were inserted. They believed the first method more satisfactory and reported excellent results by its use. Later they observed spring break-up and undertook similar experiments on additional sections. Again, they found the method gave satisfactory results which lasted for at least two seasons. The results were reported unsatisfactory on one project where the holes were only 15 to 24 in. deep.

Engineering and Contract Record /1942-13 reported temporary benefit through the use of calcium chloride or salt placed in holes.

Nunn /1942-10 reported that calcium chloride and hay cover were used to prevent freezing of the subgrade; 1½ lb. of calcium chloride per sq. yd. was applied and worked into the subgrade to a depth of 2 in. This was covered with 9 in. of loose depth of marsh grass hay. The subgrade did not freeze although air temperatures reached 10 F.

Calcium Chloride Association News /1944-3 reports on experience in six counties in Minnesota where calcium chloride treatments (generally similar to those described above) were used. All six counties reported less spring breakup on calcium chloride treated roads.

Roads and Streets /1944-7 suggested that a mixture of 66 percent pea gravel and 34 percent powdered or flake calcium chloride may be placed in pockets at intervals of 4 ft. in silt, 5 ft. in clay and 6 ft. in graded mixtures. Holes should be 20 to 30 in. deep and 8 to 12 in. in diameter. They also suggested another method in which the diameter of the hole is reduced to 2 to 4 in.

Lang /1944-10, then chairman of a Highway Research Board Committee on Treatment of Subgrade Soils with Calcium Chloride to Prevent Detrimental Frost Action reported on installations of maintenance test sections in Michigan, Minnesota, and Indiana in which calcium chloride was placed in test holes made through concrete pavements into the subgrade. Woods /1947-1, /1948-19 reported on the activities of the Highway Research Board Committee in its maintenance experiments, using calcium chloride to prevent the recurrence of frost heaves in four states, which treated bases and subgrades by introducing calcium chloride in holes through the surface. Minnesota concluded from data obtained that the treatments appeared to be ineffective. Indiana concluded that the chloride did not prevent but may have minimized the amount of heaving. Data on ground-water movement indicated a transverse flow through the cut studied which may have removed some chemicals from the soil. Massachusetts concluded that "while there was a reduction in the heaving due to the use of calcium chloride, there was surface damage resulting from the chloride treatment." Michigan studied four locations and concluded, in part, that calcium chloride "has not been effective in reducing frost heaving...the desired migration has not taken place or...a greater part of the material has leached away." Thus, the results of the field experiments were erratic. The indications were that adverse water conditions were responsible for removing at least some of the chloride.

The work of Williams /1945-5 has been described above under "Subsurface Drainage."

Thomas /1943-6 reported the use of calcium chloride to accelerate thawing of frozen subgrades. Applications of 2 to 6 psy. of calcium chloride were used. The most effective use was obtained when sufficient heat was applied to allow quick dissolving of the flake chloride. On one project an application of 6 psy. was effective to a depth of 15 in.; in another project 2 psy. thawed stabilized gravel to a depth of 6 in. although the temperature was 0 F.

Smith /1951-40 groups the chemical treatment of soils or soil aggregates for frost control by means of calcium chloride into two classifications according to the type of problem encountered: (1) treatment to permit the drainage of water trapped above frozen ground and (2) treatment to minimize frost action and controlling the loss of subgrade support during the spring melting period.

He writes that a soil containing a 10 percent solution will have a freezing point well below 23 F. A solution of calcium chloride does not freeze solid at its freezing point. Rather, a few crystals are formed which are composed of nearly pure water. The extra chloride is added to the remaining solution making it more concentrated so a 10 percent solution does not freeze solid until its temperature reaches -1.5 F.

Smith explains that as the frost in the ground begins to melt from above and below there is an intermediate layer of frozen soil through which free water cannot drain and that a method of using vertical drains backfilled with gravel and calcium chloride has been successful in draining the water through the frozen zone. That method consists of drilling 7 holes with diameters at 5-ft. intervals with a power drill, filling each hole with clean sandy gravel to which is added 3⅓ gals. of solution composed of 100 lbs. of calcium chloride and 30 gal. of water. The gravel is consolidated with a vibrator. The cost of completed work was reported about $1 per hole.

A similar method has been used in Ionia County, Michigan, except that holes were bored at the edge of the roadway on 15-ft. centers.

Smith also describes the use of calcium chloride to prevent soil freezing and the subsequent reduction in bearing capacity during the melting period. Permanency tests of calcium chloride-treated aggregate base showed that after 5 to 10 yr. one third to one half of the chemical originally placed still remained. Almost all the loss occurred during the first 5 yr.

Recommended range of depth of treatment is 6 to 12 in. Recommended quantity is 1 percent by weight of the soil, for minimum temperatures of zero or below and ½ percent for minimum temperature of zero or above. Assuming compacted soil to have a density of 100 lbs. per cu. ft. these quantities represent the following amount on a sq.-yd. basis:

Percent of Treatment	Depth of Treatment in.	Pounds of Calcium Chloride per Sq. Yd.
½	6	2.25
½	12	4.5
1	6	4.5
1	12	9.0

The above treatment is considered as an added factor of safety in the design of flexible bases, in that bases designed for sufficient load-carrying capacity may fail entirely or be damaged severely due to the critical loss of bearing value during the short period of the melting action. The value of the chemical in the subbase has a dual purpose as a compactive aid and as a frost-action control.

Field projects and laboratory research, sponsored by the Calcium Chloride Association, are now under investigation in measuring the loss of load-bearing capacity due to frost action and the effective control of calcium chloride treatments, which should be of value for consideration in the design of flexible bases in frost-affected areas.

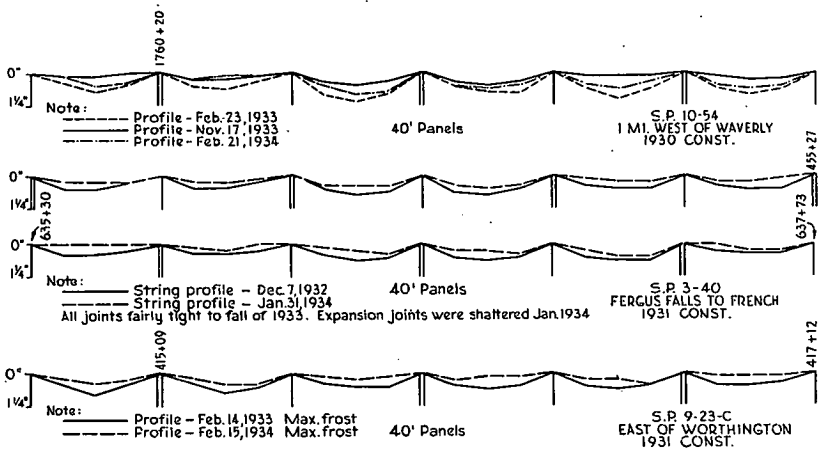


Figure 200. Effect of Special Joint Maintenance

Maintenance of Paved Surfaces

Minnesota /1945-8 reports in January, 1933, 160 mi. of distorted concrete pavements were recorded, but mileage dropped to 111 in Jan. 1934. The decrease was attributed to special joint sealing during the fall months. The report states that where pavement joints were sealed in the fall months, "warping was reduced an appreciable amount; but in all instances where this maintenace was omitted in the fall, warping was as severe as during the preceding winter." The report presented Figure 200 to illustrate representative sections on 3 projects where joints were

maintained in the fall "before frost action." It may be seen that objectionable high joints have been reduced. Figure 201 illustrates representative sections of a pavement which did not have fall joint maintenance. It may be seen that the joint raised to approximately the same heights as in the preceding winter.

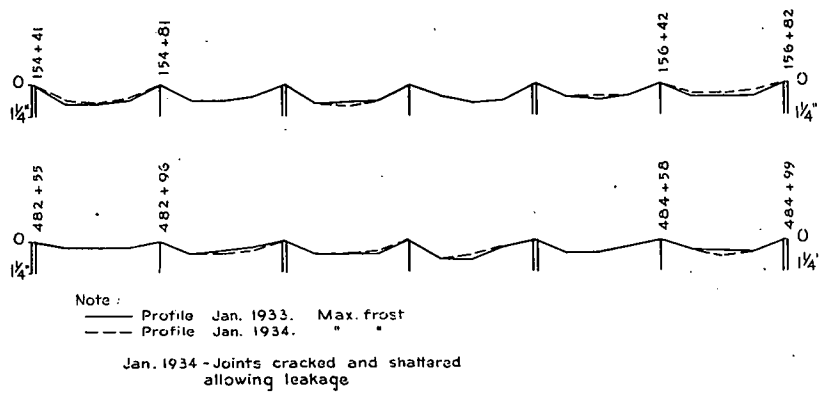


Figure 201. Effect of No Fall Joint Maintenance
S.P. 8-50-2 Crookston to 16 Mi. Corner 1930 Const.

Goeltz /1948-15 in discussing snow removal on secondary roads held that leaving a crust of snow on the road "will keep frost from sinking so deeply into the roadbed, and that consequently breakup damage will be lessened when the frost goes out the following spring."

Smith /1948-28 reported that the condition of the surface of flexible pavements in the Amarillo area had an influence on the amount of damage due to freezing. Cracks in the surface caused by dry-weather shrinkage permitted water to enter, which caused reduction in bearing capacity on freezing and thawing. He reported that the most common "and by far the most disastrous type" of freeze damage occurs near the outside edge of the traffic lane, due to freezing and thawing of moisture entering the subgrade at the edges of the pavement. He held that one of the sources of the trouble was water absorbed from melting snow piled adjacent to the pavement edge. He associated the major damage with trench-type base construction (as compared with full-width bases) and believed adequate shoulder slopes minimize the damage.

Load Restriction During and Following the Spring Thaw Period

Data on laws governing seasonal load restrictions, and practices on enforcement by several organizations of such restrictions are being assembled. These include Subcommittee No. 2 of the Maintenance Committee of the American Association of State Highway Officials; the National Highway Users Conference; Project Committee No. 7 on Load Carrying Capacity of Roads as Affected by Frost Action of the Department of Maintenance of the Highway Research Board; and the staff of the Highway Research Board. Because much information is being assembled by these groups, no effort is made here to summarize the published literature on the subject.

PERMAFROST

Perennially-frozen ground, which has recently been termed permafrost, denotes ground which remains frozen throughout the year and from year to year. It underlies an active zone subject to seasonal freezing and thawing in a like manner to the freezing and thawing in the North Temperate zone. According to the Corps of Engineers /1946-7, permafrost may be continuous and may or may not contain ground ice; it may exist as islands in unfrozen ground or as layered permafrost. Typical sections through ground containing permafrost are illustrated in Figure 202.

The same basic concepts which apply to soil freezing in a temperate climate apply to soil freezing in an arctic climate. However, soil freezing in arctic climes is more intense, and its manifestations stand out more clearly. For that reason, the reviewer believes a brief review of permafrost literature is of value to all engineers interested in soil freezing. Space permits review of only a few of the available writings. However a fairly comprehensive list of publications pertaining to permafrost is given in the bibliography.

Structure of Permafrost - The visible effect of freezing of the active surface layer in the region of permafrost depends upon the type of soil, water conditions, and terrain. The type of structure most widely described in writings is that which has a polygonal outline. That form of structure may be associated with the presence of ground ice wedges. Tyrrell /1904-1 described wedges of clear ice 6 in. to 3 ft. or more in thickness but did not associate them with polygonal surface structure. They ranged in length up to 1000 ft. and in width from 50 to 200 ft. Similar formations were described by Maddren /1907-1 and Moffit /1913-3.

Leffingwell /1915-4, /1919-1, and Taber /1943-4 described wedge-shaped segregations of ground ice which ranged up to 8 ft. in width and up to 30 ft. in depth. Paterson /1940-17, Washburn /1947-19, and Troll /1944-12 described frost cracks without visible ice. Both ground ice wedges and the frost cracks were associated with distinctive polygonal patterns with "cells" ranging up to tens of feet in diameter. Leffingwell believed the ice wedges resulted from the filling of cracks with water and freezing, the development of thick ice wedges requiring many repetitions of the cycle. Taber held that the growth of the ice mass itself exerted an expansion force. Leffingwell /1919-1 presented several sketches showing the nature of ice wedges and polygonal structure.

Frost /1951-31 defines two general types of perennially frozen ground: dry frozen and detrimentally frozen. "Dry frozen refers to a condition in elastic materials in which the mass is rendered solid by the freezing of interstitial water. In the normal and unfrozen state such soils would be well-drained internally. Ice lenses, ice wedges, or ground-ice areas usually are lacking and such soils can experience thaw without severe settlement. ...Such soils are usually confined to granular areas situated high topographically and having what would normally be a low ground-water table. Typical soil types are those associated with sand dunes or high sand terraces.

Detrimentally Frozen Materials

"This type of permafrost includes: fine-textured soils which contain a large percentage of ice in their mass in the form of crystals, small lenses, or small wedges; soil masses which have been so arranged by segregation of ice and soil particles that they form polygonal blocks of varying size and types; materials situated low topographically and having large masses of ground ice as an integral part of their mass; and large masses of ground ice. In general, the most detrimental permafrost situations may occur on nearly all common land-form types, provided that soil textures are fine and that the topographic situation is somewhat depressed, very slightly sloping, or exceedingly flat. Thus, such land forms as broad flat plains (level or slightly sloping), valley fill, transition zones, low colluvial slopes, lake beds, backwater flood plains, and others of a similar nature which contain fine-textured soils can be expected to contain detrimental permafrost."

Frost adds "In arctic and subarctic regions polygons indicate detrimentally frozen ground, with the one exception being a particular type found in a gravel-soil outwash area. Polygons are geometric configurations which are presumably formed by an adjustment of the frozen earth mass, just beneath the surface, to stresses from seasonal expansion and contraction which are brought about by temperature variations.

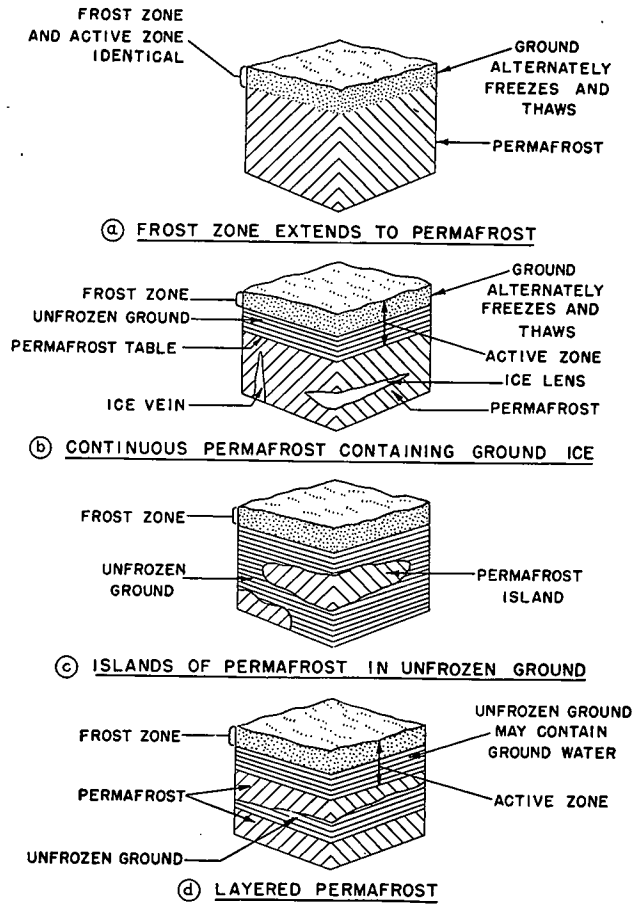


Figure 202. Typical Sections Through Ground Containing Permafrost. (After Corps of Engineers)

"The two most important types which are associated with permafrost are (a) those with depressed centers, or pans, which are enclosed by raised dykes or perimeters and (b) those with raised centers and depressed perimeters as outlining channels. Polygons of both types vary in size from 15 to 20 ft.....to perhaps 200 ft. across the polygon. The number of sides varies from four to six with five-sided figures being the most common."

Perennially-frozen ground may or may not include ground ice. Leffingwell /1919-1 describes the following types of ground ice: (1) grains of clear ice, the largest an inch in diameter, mixed with earth; (2) thin undulating sheets or ribbons of ice alternating with thin beds of earth; (3) heavy horizontal beds of clear ice; (4) heavy beds of ice alternating with beds of earth; (5) heavy deposits of ice with isolated earth inclusions; and (6) a network of

vertical wedges of ice surrounding polygonal bodies of earth. The polygonal type is said to be the most widespread in occurrence along the Arctic Coast of Alaska.

Figure 203. Ice Wedge In Detrimental Permafrost. (After Frost)

Figure 204. Ice Lenses In Detrimental Permafrost. (After Frost)

Nelson /1949-14 found that perennially frozen ground occurred in all types of soil along the Alaska Highway. He found very few soils, however, which contained an appreciable amount of clay (3 samples in 100). He found soil type to be of importance because of the "variation in type and extent of permafrost...found in different soils." Although the reviewer has found few data showing the effect of soil type, he concludes from the general statements in the literature that silts offer the greatest problems in the permafrost region. An example of an ice wedge in detrimental permafrost is illustrated in Figure 203. Ice lenses in detrimental permafrost are illustrated in Figure 204. Airphotos showing examples of raised center polygons and depressed center polygons may be seen in Figures 205 and 206 respectively.

Soil polygons are also described by Hobbs /1913-2, Högbom /1914-2, Huxley and Odell /1924-5, Nordensjold /1928-11, and Antevs /1932-7.

Terminology - The literature is at variance regarding proper terminology to describe permafrost. Perhaps the best recent discussion of terms which pertain to permafrost is given by Bryan /1948-1 and the discussions of Bryan's work. Bryan proposes a new terminology for permafrost and suggests cryopedology being derived from the Greek "Kryos" (ice cold) pedon (ground or soil) and logos (knowledge). The surface layer he would call congeliturbate from "congelare" (to freeze) and "Turbare" (to stir up) and pergelisol from "per" (through) "gelare" (to freeze) and "solum" (soil). The characteristic parts of perennially frozen soil suggested by Bryan are shown in Figure 207.

In a discussion of Bryan's suggested terminology Taber suggests the use of the term perennially frozen soil. Black takes exception to cryopedology and also pergelisol, mollisol, and tabetisol. Zeuner would prefer the word tjaele used by Scandinavian and German scientists for frozen soil. Lewis prefers to avoid technical jargon wherever possible while Edelman and Tavernier agree on the term cryopedology yet they regret that the "already existent and generally accepted term tjaele has not found its way into the new American investigations."

Occurrence of Permafrost

Geographical Distribution - Nikiforoff /1928-7 wrote that permafrost covers an area of about 3 million sq. mi. in Asia alone. The area is bounded on the north by the Arctic ocean, although it has been found at several places at the bottom of the Arctic Ocean. He found the southern boundary irregular but did construct a map showing its distribution in Asia or in North America. Cressey /1939-7 after making field studies in Siberia and reviewing Russian literature, held that the permafrost area was 750,000 sq. mi. greater in extent than the figure given by Nikiforoff. He found that the area of permafrost does not correspond to the glaciated area, except possibly for still buried masses of Pleistocene ice. A more recent estimate by Wilson /1948-17 is that permafrost covers one fifth of the earth's surface and is found in 80 percent of Alaska, 50 percent of Canada and practically all of Siberia.

Climatic Distribution - Leffingwell /1919-1 held that the line limiting the distribution will "follow closely the isothermic line for 1 deg. C." He added, however, that the presence of lakes, flowing surface or subsurface water, amount of snow cover, and other factors have considerable influence on distribution of permanently frozen ground. The War Department also suggests that perennially-frozen ground will occur where the mean annual temperature is at freezing, provided the winters are long and cold and the summers are short, dry, and cool and the annual precipitation is low. Taber /1943-4 held that development of perennially frozen ground requires a mean annual temperature not higher than about 26 F.

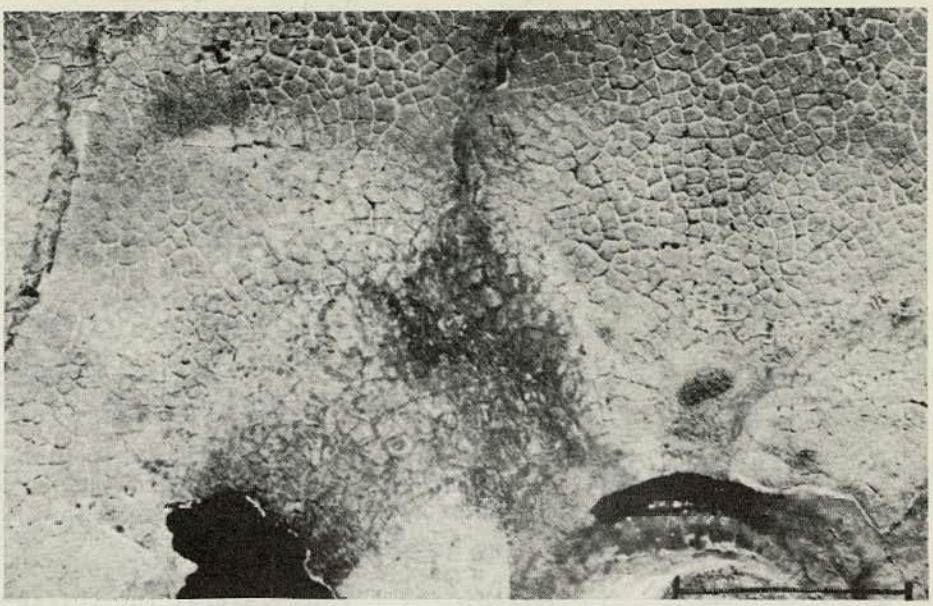


Figure 205
Airphoto of Raised Center Polygons. (After Frost)

Cressy /1939-7 believed the required average annual air temperature need not be freezing and stated that "where the average annual air temperatures are from 1 to 2 C., frozen ground typically extends to a depth of 20 meters."

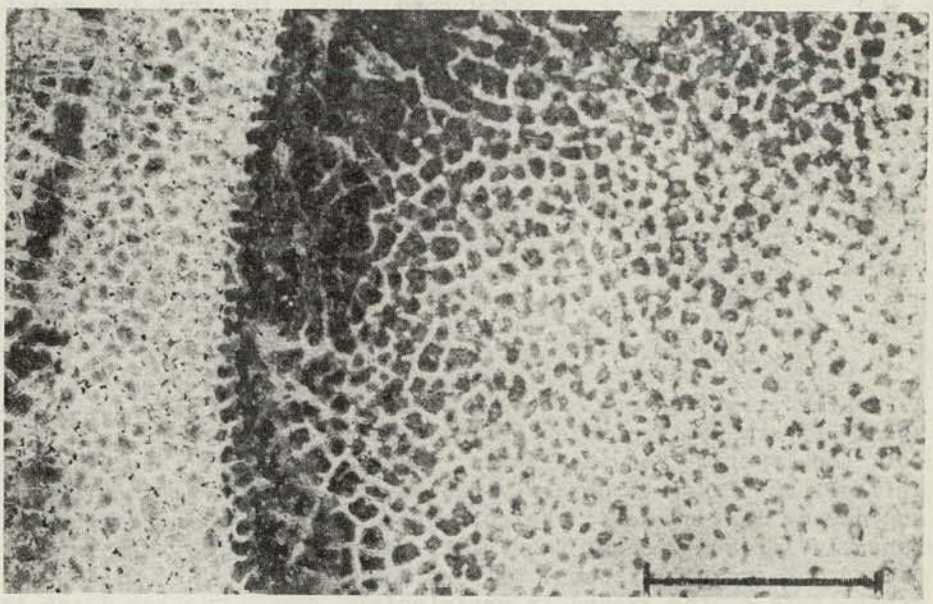


Figure 206.
Airphoto of Depressed Center Polygons. (After Frost)

Several writers have attempted to determine whether the present state of development of permafrost is a statistical one or whether the southern boundary is extending or receding. Nikiforoff /1928-7, in his writing on Siberian permafrost, ventured the probability that its southern boundary has moved northwards because it has been discovered "in at least 16 places along the southern boundary". The surface of the permafrost lies at a greater depth from the ground surface than it does further north. He cites as an example an instance where a layer of permafrost 70 ft. thick was found with its surface 103 ft. below ground near its southern limits. Nelson /1949-14 states that extensive permafrost deposits occur as far south as the Sikanni Chief River in British Columbia, and isolated pockets can be found as far south as Athabaska, in northern Alberta.

Soil Temperature in Permafrost - Perhaps the most widely discussed set of temperature readings taken in permafrost are those obtained from the Schergin shaft at Yakutsk, Siberia. The "shaft", a well dug to obtain drinking water, was dug by hand during 1827-1837 to a depth of 382 ft. Temperatures were observed by Middendorf in 1844 and 1845. The temperature data he obtained are given in Table 63. Nikiforoff /1928-7 expresses some doubt that the temperatures are representative as they were observed 8 to 17 years after the well was dug.

TABLE 63

Temperatures in Permafrost (Schergin Shaft)
(After Nikiforoff /1928-7)

Depth (ft.)	Temperature (deg. C.)
7	-11.2
15	-10.2
20	-10.2
50	- 8.3
100	- 6.6
200	- 4.8
300	- 3.9
382	- 3.0

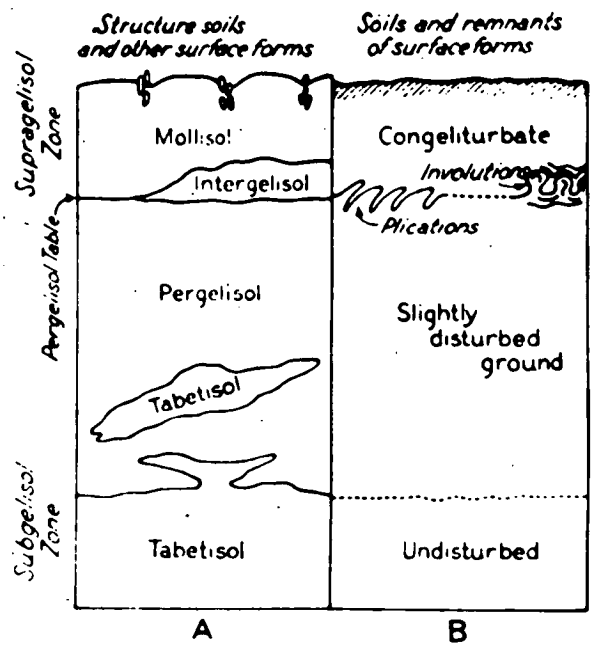


Figure 207
A. Characteristic parts of the ground in areas of permanently frozen ground: Mollisol, the layer melted each year: Intergelisol, the layer temporarily frozen during one or more summers: Pergelisol: "permanently" frozen ground: Tabetisol, layer of body of ground unfrozen. B. Characteristic parts of the ground in periglacial areas: Congeliturbate: material stirred up and usually moved down slope by freezing and thawing of the Mollisol: Plications and Involutions when present show movement. (After Bryan)

Depth of Permafrost - Leffingwell /1919-1 believed the depth of permafrost at Yakutsk, Siberia, to be about 650 ft. He reported that in the Koyukuk region of Alaska the full depth of frozen ground was not reached at 365 ft. Cressy /1939-7 held that at the time the greatest known depth (890 ft.) was at Arnderma, Siberia. Wilson /1948-17 reported that where found in continuous layers, the thickness ranges up to about 1500 ft.

Depth of Active Layer - The depth of summer thaw depends on the climatic conditions, the type and amount of cover, water conditions, etc., and may range between wide limits. An example for a specific region is given by Black and Barksdale /1949-1, who found for the Point Barrow region in Northern Alaska that "summer thaw penetrates from 6 to a maximum of about 30 in. The average summer thaw is about 8-20 in."

Frost /1951-31 stated with reference to the Arctic that "in areas where a thick moss carpet exists the active layer is very shallow -- ice can be found beneath the moss in many instances even as late in the season as September. In southern parts of permafrost regions the active zone is much thicker, since soil temperatures in the permafrost are warmer and the duration of summer heat is longer than in the Arctic. In well-drained granular soils which are situated high topographically the active zone is much deeper than in the areas of fine textured soils."

Surface Icing Associated with Permafrost - Icing (sometimes called glaciering or water boils) is a mass of surface ice formed by successive freezing of sheets of water, which may seep from the ground or come from a spring, small stream, or river. It occurs especially as a sheet or field of ice but may occur as a mound, or irregular crusts on natural or excavation slopes. It may occur in small streams and completely clog culverts. Although not a soil-frost phenomenon, it is related to freezing of the ground and sometimes occurs in the United States. The work of Eager and Pryor /1945-3 on Ice Formation on the Alaska Highway gives an excellent description of the surface icing phenomenon. The following summary is taken wholly from their work. Examples of icing are shown in Figures 208 and 209. In some instances icing has been known to cover a continuous stretch of road a mile or more in length.

Factors associated with icing - Climate and topography are the two major factors which cause icing. Other factors which have a bearing on the formation of icing are (1) the degree and extent of ground cover, (2) subsurface drainage conditions, (3) soil type, (4) road location, and (5) the occurrence of permafrost.

Climate - Icing is most apt to occur in a climate having short, wet summers and long, cold, dry winters, which is characteristic of the climate along a major portion of the Alaska highway. The mean annual temperature ranges from a high of 35 deg. F. at Fort St. John, B.C. to 30 at Fort Nelson, 28 at Watson Lake, Y.T., to a low of about 26 F. at Fairbanks, Alaska. The most severe icing occurred between Fort Nelson and the Alaska-Yukon Territory Boundary under a mean annual temperature range from 27 to 30 F.

Figure 208. Icing in a Waterway at Culvert Inlet - Painted post in lower right of photo locates culvert inlet. (After Bureau of Public Roads)

The average mean monthly temperature along the Alaska Highway is indicated in Figure 210. An interesting fact observed from this chart is that the lowest mean monthly temperatures in two winters differed by 20 F. and that the greatest variation in mean monthly temperatures occurs during the winter season.

Figure 209. Outlet of 6 ft. by 6 Ft. Wood Box Culvert Completely Plugged by Ice. (After Bureau of Public Roads.

During the 1943-44 observations of icing on the Alaska Highway the minimum temperature was reached in January. The minor icings were most frequent in December, but the major icings were most active during January and February. The icing activity did not begin to recede until about a month after the occurrence of the minimum mean monthly temperature.

Topography, location, and source of water - Topography and source of water are important in the production of icing. Rough and mountainous terrain of the nature indicated in Figures 210 and 211 forms "typical icing country."

Sources of water flow most likely to remain active during the winter and cause road icing are: (1) small streams or seeps draining muskeg areas, (2) springs in porous or jointed strata, and, (3) subsurface water flow forced to the surface by artificial conditions. From Dawson Creek to Whitehorse, during the winter of 1943-1944, springs, seepages, and small streams accounted for 16, 38, and 41 percent of the major icings, respectively, or a total of 95 percent of the total major icings. Examples of icing in waterways are shown in Figures 208 and 209.

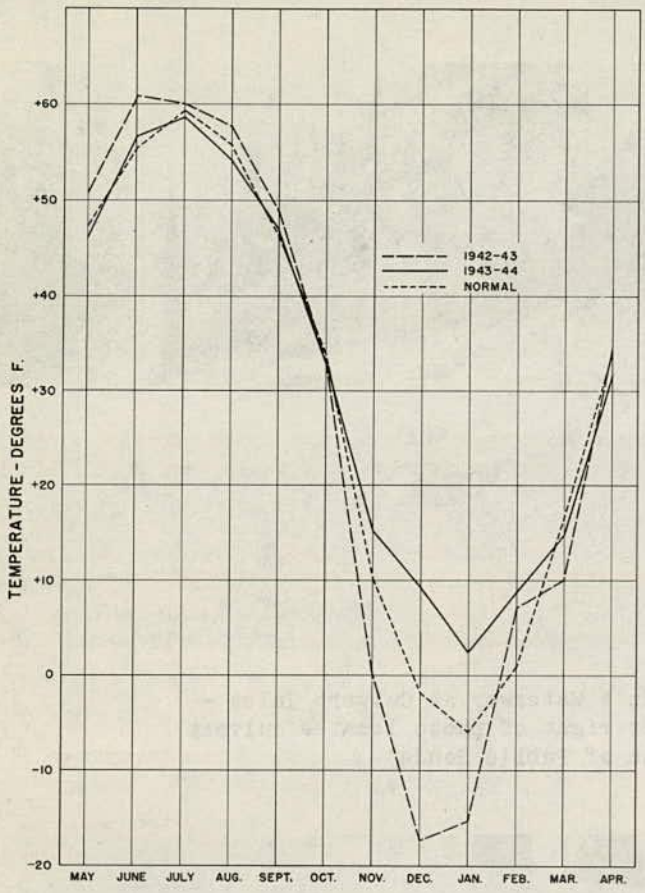


Figure 210. Mean Monthly Temperatures on the Alaska Highway, based on records at Fort St. John, Fort Nelson, Watson Lake, Whitehorse, Northway, Tanacross, Big Delta, and Fairbanks. (After Public Roads)

The natural topography or the locations most frequently associated with icing are:

a. Wide, shallow exposed channels, glacial streams or alluvial fans.
b. Slopes at or near the foot of a mountain or hill or on a hillside where springs and seeps will be intercepted in road construction.
c. Locations immediately downstream from poorly drained areas, such as muskeg swamps, which due to heavy vegetative cover are apt to continue draining throughout the winter.
d. Any sudden reduction in natural channel gradient (of a channel not otherwise subject to icing) which tends to either force subsurface water to the surface or to cause debris to be deposited, impeding drainage.

Forty-nine percent of the icings during the winter of 1943-1944 formed where subsurface flow emerged naturally near the foot of a hill or mountain. Approximately two thirds of the major icings occurred on slopes up to 15 deg.

Culverts may be constructed to reduce the stream gradient and cause icing. Figure 209 shows an example of a wood box culvert completely filled with ice.

Elevation was found to be a factor in producing icing. The greatest frequency of icings occurred above the 2100-ft. elevation, where waterflows intercepted by the road were numerous and temperature fluctuations conducive to icing were greater.

Effect of exposure - Ground Cover - Moss, vegetal debris, and fallen timber form a good insulating medium. A thickness of 12 to 18 in. of moss, low bushes and grasses, when densely matted will preserve underlying permafrost during the summer and prevent water from freezing near the roots of vegetation in winter.

Figure 211. Looking Eastward along the North side of Pickhandle Lake (Mile 1160.5). Showing Typical Icing Country (After Bureau of Public Roads)

Seventy-two percent of the icings on the Alaska Highway occurred in areas of heavy (12 to 18-in.) ground cover and 25 percent occurred in areas of medium (6 to 12-in.) cover for a total of 97 percent of the icings in areas where ground cover served to collect water, protect it from freezing, and release it slowly as seepage so ice would form where it emerged in open areas.

Snow cover - Heat is conducted through moist soil from 4 to 20 times as fast as through snow, the rate depending on the compactness, water content and type of soil and the age and compactness of the snow. A deep snow cover serves as an excellent protection against freezing of water as it emerges into otherwise unprotected areas. A close correlation was found between depth of snow cover and icing. An example of this was between Whitehorse and Mile 1,163 where icings occurred at regular intervals where the depth of snow was less than 15 in. but did not occur where the snow cover exceeded 15 in.

Soil type - Seeps and underground movement of water were most frequently associated with pervious soils; 45 percent of the 1943-44 icings was associated with sand, gravel, and rock materials, and a total of 68 percent were found in areas of pervious materials.

Permafrost - Of interest from the icing studies was the fact that it was evident from observations along the highway that the mean annual temperature must not exceed 28 F. if permafrost is to be maintained.

Where permafrost occurs one of the factors which causes icing is freezing of the active surface layer down to permafrost under the road because of its greater exposure to freezing than the soil under natural cover. This frequently forms an ice dam under the road, sealing normal seepage channels, forcing the water out on the high side of the road and leaving all drainage to surface drainage through culverts. The culverts, being exposed, become filled with ice, forcing the water over the road causing icing.

The ratio between percentage of highway constructed over permafrost and the percentage of icings in such areas indicates a greater tendency for icing to occur in permafrost areas, because of its tendency to limit the downward movement of water.

From Whitehorse, Yukon Territory, to Big Delta, Alaska, 28 percent of the road was constructed over ground classed as permanently frozen at the time of construction.

Sixty-eight percent of all major icings between the two points were located in areas of permafrost.

Methods to Prevent or Reduce Icing on New Construction - The following rules have been set down from the experience on the Alaska Highway as being of value in preventing or reducing icing:

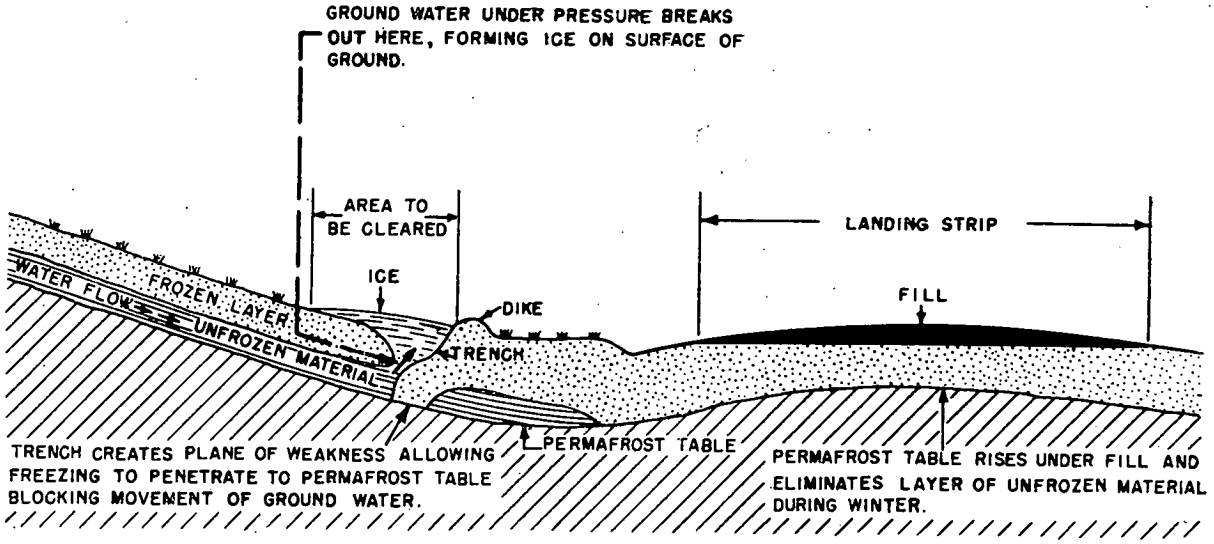


Figure 212. Method of Creating Induced Field of Surface Ice.
(After Corps of Engineers)

1. Construct drainage ditches for ice control by making them as deep and narrow as conditions permit. Then considerable protection is obtained from close high banks and from vegetation growing on them.

2. Place culverts at frequent intervals on sections subject to icing. Culverts should be deep in the original ground and double the size normally necessary where no icing occurs.

3. Construct a high grade line.

4. Where feasible, induce icing at some distance on the uphillside of the road by stripping so ice dams occur. This can be done where:

(A) The flow of water is light near the surface, and the storage area is large. (A flow of 7 gals. per min. will cover more than an acre of ground to a depth of 1 ft. in a months time.)

(B) Conditions will not merely divert the flow around the ice formations and cause icing at new locations.

(C) Snow cover can be removed if it tends to prevent the expected induced icing.

5. Construct diversion ditches.

6. Construct subsurface drains if they can be prevented from freezing from the surface or from underlying permafrost.

Figure 213. Induced Icing Area Across Water Course Uphill From Roadway. (After Bureau of Public Roads)

A diagram illustrating a method of creating an induced field of surface ice is shown in Figure 212.

An example of where an area was cleared to increase exposure and a dike built to impound the ice in an induced icing is shown in Figure 213.

Frost Mound, Ice Mound, Pingo, Ice Formation, Soil Blister and Ice Cavern - These terms have all been applied to mounds which occur in the Arctic. These mounds have been seen in several areas by different observers.

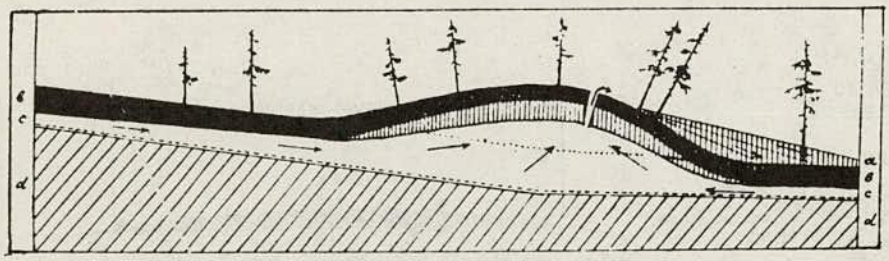


Figure 214
Cross-Section of a Soil Blister
a, accumulation of clear ice, both on the surface and under the raised frozen surface soil; b, frozen soil; c, water in liquid form; d, ever-frozen subsoil.
(After Nikiforoff)

Figure 215
Oblique Photo of a Pingo
(After Frost)

Leffingwell /1919-1 described "gravel mounds (Pleistocene and Recent)" observed in the Northern Alaskan coastal plain, some 30 mounds in a relatively small area. The mounds were generally less than 40 ft. high, but some were 50 to 60 ft. in height, and one measured 230 ft. high. The general aspect of some mounds was that of upheaved ground with a split in the middle. Leffingwell includes descriptions of other mounds seen by several other observers.

Leffingwell believed that the mounds were the result of hydraulic pressure set up when surface freezing progressed downward sufficiently in a sloping coastal plain composed largely of water-permeable materials.

Nikiforoff /1928-7 described ice-lined "soil blisters" which sometimes reached heights of 20 ft. He held that they were caused by expansion of water under pressure and found between the upper frozen layer and the underlying permafrost. When punctured, the water escapes and leaves an ice cavern. A sketch showing a cross section of a soil blister and indicating its mode of formation is shown in Figure 214.

Porsild /1938-7 also described two types of mounds and called the mounds "Pingos." The first was similar to that described by Leffingwell /1919-1 . Porsild also described a second type of pingo much larger than the first. These pingos, which occurred in the McKenzie delta and in low lake-field country, are up to about 50 ft. high and have a diameter up to 575 ft. He suggests that Leffingwell's theory of hydraulic pressure cannot be applied to the second type of pingo. He believes they were formed by local upheaval due to expansion of the progressive downward freezing of water or semi-fluid mound or silt enclosed by the bed rock and the frost surface soil, much in the way the cork in a bottle filled with water is pushed up by the expansion of water on freezing. He shows a photo of a pingo near Tuktuayaktok on the Arctic Sea coast of the Mackenzie delta showing the irregular rupture of the summit. The pingo was 134 ft. high.

Studies by Frost /1951-31 led him to classify pingos in three types: (a) the true pingos, or frost mounds, which are believed to be the result of upheavel of the earth's crust from pressure of ice from beneath the surface; (b) the isolated dome-shaped terrace remnants, which are the result of peculiarities of Arctic erosion and dissection which rendered the mounds conical in shape; and (c) those mounds formed by upward flow of water and/or soils to the surface through an orifice of some type.

Sharp /1949-9 also described the occurrence of mounds in the Yukon territory ranging in size from a foot or two high up to 1500 sq. ft. in area.

Effect of Snow Cover on Permafrost - Nikiforoff /1928-7 showed that even comparatively small depth of snow cover had a marked effect on the temperature of frozen ground. The effect may be seen from Table 64.

TABLE 64

Effect of Depth of Snow Cover on Temperature of Soil at Bomnak, Siberia
(After Nikiforoff)

Years	Month	Depth of Snow Cover (cm)	Mean T of Air (deg. C.)	Mean T. of Soil at 1.5 meters (deg. C.)
1912-13-15	February	17	-22.2	-4.3
1914-16-17 18-19	February	30	-23.6	-2.1

Roberts /1903-1 reported that there is no perceptible thawing due to the warmth of the earth until the snow cover of at least 25 in. existed and the specific gravity of the snow had reached 0.25.

Effect of Permafrost on Topography

The Development of Thaw Lakes - Permafrost and associated soil movements due to freezing and heaving development of frost polygons or the formation of frost mounds and later thawing, accompanied by the formation of small lakes or movements due to settlements or to soil flow, combine to show a marked effect on topography in permafrost areas.

The formation of frost mounds has been seen to occur so frequently as to give the horizon an undulating appearance. When those mounds melt and collapse, they often form small lakes surrounded by rings of earth. The frost polygons, although they never develop to the extent that they show large differences in elevation, do mark the landscape, so it is easily recognized.

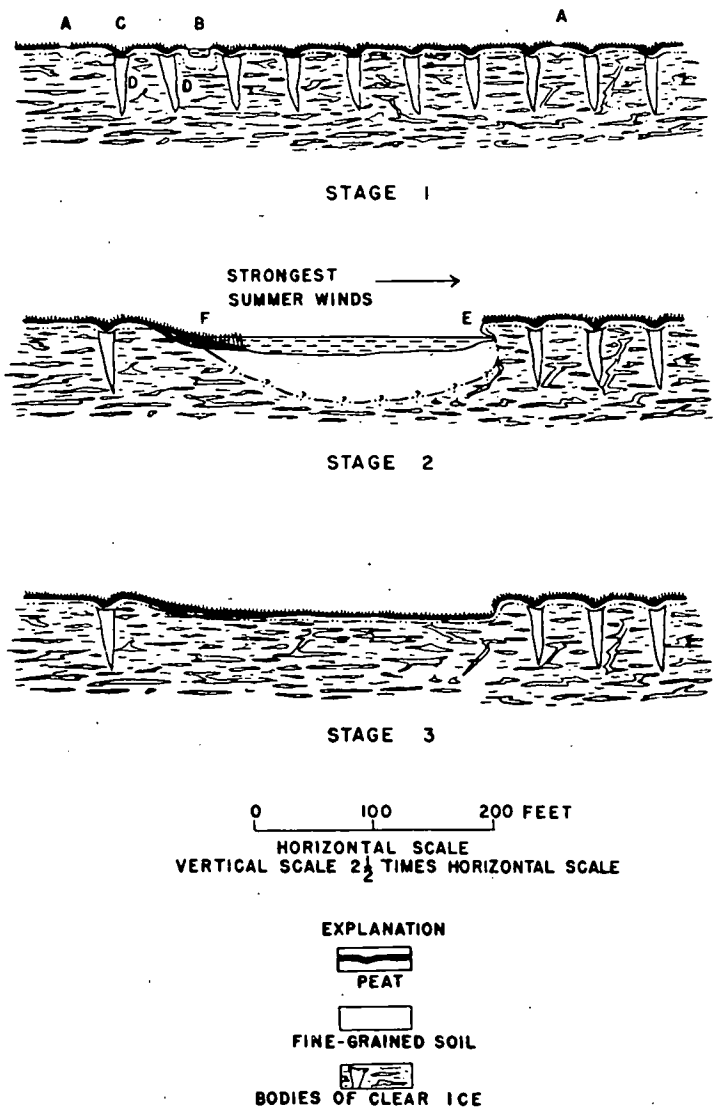


Figure 216. Diagrammatic cross sections illustrating the origin and development of a typical thaw lake. Stage 1 shows the initiation of areas and deep thaw beneath frost boils (A) and beneath a small pool (B). Note shallow trenches (C) over buried ice wedges (D). In stage 2 the small pool has grown by thawing and caving into a thaw lake, which is migrating in the direction of strongest summer winds. On the windward side (E) active caving is in progress, but on the leeward side (F) the lake is being filled with peat-forming vegetation. In stage 3 the lake has been drained and new perennially frozen ground has formed beneath the bottom. (After Hopkins)

Hopkins /1949-2 describes thaw depressions (from subsidence following thawing of permafrost); thaw lakes (which occupy thaw depressions and cave-ins); and thaw sinks (closed depressions with subterranean drainage believed to have originated as thaw lakes). Those occur in the Imuruk Lake area of Alaska north of Nome. He ascribes the origin of the thaw lakes to subsidence following thawing of perennially frozen silty ground in regions containing large quantities of clear ice. The process is indicated in the diagrammatic cross sections shown in Figure 216. The thaw lakes seldom exceed 1000 ft. in diameter.

Hopkins also describes thaw sinks, which are the result of drainage of thaw lakes in areas where the bottom of thaw lakes contact material which will permit subterranean drainage.

Soil Flow and its Effect on Topography - The effect of freezing and thawing on the unstability of slopes and the creep or flow of soils has long been observed in the North Temperate Zone and more recently in arctic areas, where it is often associated with the presence of permafrost. Kerr /1881-1 and later Davison /1889-2 studied the phenomenon experimentally, causing soil to move on a slope when thawed after freezing. Their studies showed that movement took place by a normal (to incline of plane) rise during freezing and vertical settlement during thawing.

Anderson /1906-2 refers to the work of Geike /1894-1 and held that the sluffing of slopes was due to the melting from top downward of soil high in accumulated water. Anderson termed the process solifluction (derived from solum "soil," and fluere, "to flow"). He believed solifluction to be one of the chief agents of destruction in regions having a "subglacial" climate with heavy snowfall melting in summer.

Eakin /1916-2 took a somewhat broader view than the earlier observers, and brought the forces of frost action, the position of the water table and thaw effects more clearly into the picture. He held that the position of the water table near the surface "minimizes differential heave and gives a dominant expression to horizontal thrust." Eakin also found that vegetation influenced soil flow, a thin covering of vegetation "favors stronger movement, a thick covering retards it."

Ekblaw /1918-8, in writing about northern Greenland, stated that nivation is closely associated with solifluction, (Nivation is the process by which quiescent neve (snow ice) effects the change of land forms.) Because the snow melts slowly most of the water seeps into the ground softening it and, aided by freezing and thawing, results in solifluction causing the development of terranes and solifluction slopes described by Eakin /1916-2.

Taber /1930-9 and Mullis /1930-8 both were aware of the effect of freezing and thawing on the mobility of soils. Mullis cited instances of soil flow, and showed an example of soil movement on hillsides.

Sharpe /1938-1, in his work on landslides, recognized five types of soil movement and included solifluction as one of the dominant forces causing soil movements in cold climates. He stated that the type of soil flow indicated by solifluction is not limited to cold climates, but its usage has come to be associated with frost action and cold climate phenomena. He held that cold climates have the following features which favor solifluction: (1) A soil with sparse or no vegetative cover, (2) A continual supply of water (from melting ice and snow) and (3) Deeply frozen or perennially frozen soil which, when it thaws only a few feet, prevents vertical drainage.

Troll /1944-12 took a broad view on soil movements due to frost and included not only earth flow over a perennially or seasonally frozen substratum but also sorting of the debris on level ground (for example, the formation of stone rings described later) and the movements caused by short periodic or daily freezing and thawing of surface layers.

Budel /1944-13, in his comments on the developmemt of land forms in Europe, opposed the views of many that the land forms in unglaciated areas have been shaped largely by stream action. He argues that the low temperatures of the glacial ages promoted other powerful geomorphic processes and that the surficial features produced by those processes still dominate the landscape because of small changes during the post glacial period. He held that a large portion of the debris was accumulated by means of solifluction.

Sigafoos and Hopkins /1951-29 studied the instability of soil on slopes in the Seward Peninsula region of Alaska and described certain microrelief features, soil terraces, lobate terraces, soil lobes, tundra /23 mudflows, and stone stripes directly attributable to this instability. The soil mantles encountered here showed considerable textural variation but all were noted to be particularly wet even in elevated hillside position.

The downslope movement was attributed either to creep or viscous flow. The process of creep is shown in Figure 217.

Sigafoos and Hopkins state further that soils are involved in creeps primarily during the autumn freezing cycle. "Wet soils are distended upon freezing, owing to the segregation of lenses of clear ice. ...Soil movement downslope is greater in the surface layer than at the base of the seasonally thawed layer because the surface, on freezing, is distended more and thus moves farther horizontally from its original position. Vertical subsidence upon thawing consequently moves the surface material farther downslope than the underlying layers."

Two rates of viscous flow are defined, slow and rapid, the former being productive of the terrace forms, the latter of mudflows. Viscous flow is most conspicuous in the spring and early summer when the soil may be so wet that it is practically a suspension. "Creep and viscous flow rarely occur separately, but one process or the other generally predominates in the movement of given bodies of soil."

Local climatic variations were qualitatively correlated with the intensities of the creep and viscous flow movements. It was found that the "optimum climatic conditions...are found in climates with large fluctuations between winter and summer temperatures and in climates with frequent summer frosts and winter thaws." Variation in intensity of movement with soil texture was also noted, the soils with a high proportion of silt being most unstable.

/23 Tundra is defined here as those areas in high latitude in which timber is lacking and the ground bears a partial or complete cover of sedges, willows, mosses, etc.

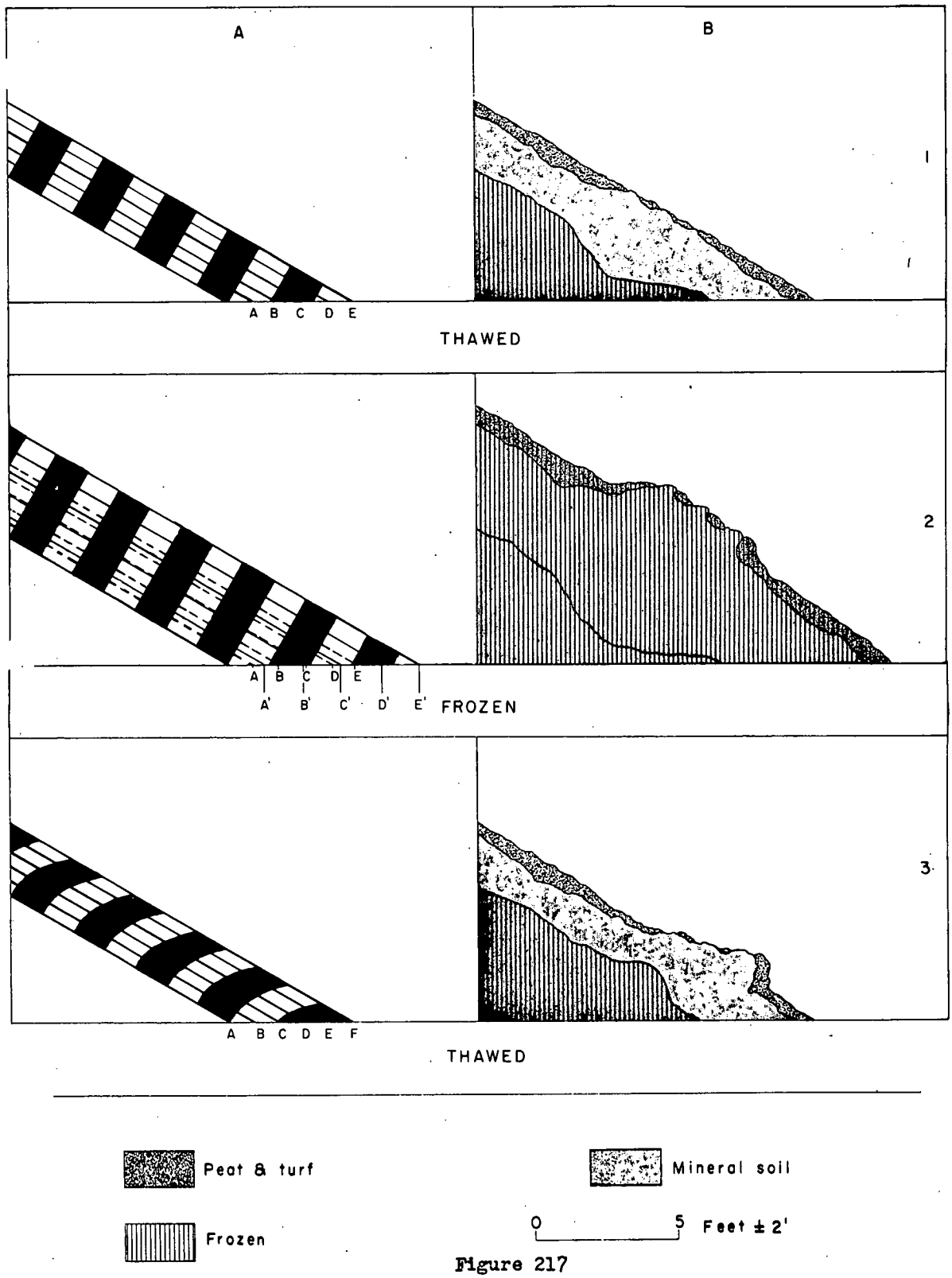


Figure 217

Diagrammatic illustration of creep mechanism and terrace formation. (A) Theoretical mechanism of creep. Wide black bars represent theoretical columns of soil whose distension upon freezing is shown by thin lines parallel to slope. Upon thawing, columns bend downslope, owing to vertical subsidence of soil. (B) Illustration of terrace formation by creep, horizontal thrusting, and viscous flow. Soil moves downslope farther than vertical subsidence would carry it, as it is pushed by frost thrusting, and it flows. (After Sigafoos and Hopkins)

Sigafoos and Hopkins identified and classified soil terraces, lobate terraces, soil lobes, and tundra mudflows as micro-relief features resulting from the instability of soil on slopes subjected to intensive frost action. They gave detail description of the topographic expression and position, the development process, the soil and substratum, vegetation, water table and perennially frozen ground the character during periods of thaw, the appearance on aerial photographs, and the construction characteristics.

The many sloping terraces, sloping benches, or altiplanation terraces in the arctic are described by Frost /1951-31. "They vary in size from narrow dissected bench-remnants situated high on the side slopes of a valley to vast tilted surfaces of several square miles in area in large valleys.... The processes creating such features are believed to be peculiar types of solifluction, since they contain accumulations of loose rock materials which appear to have been moved by some types of flow." See also Smith /1939-18.

Those interested in the subject of soil flow may find additional material on the subject in the following references:

Belcher /1855-1, pp. 306-9; Prestwich /1892-3, pp 263, 322-8; Geike /1894-1, pp. 389, 723; Högbom /1914-2, pp. 259-60; and Lotze /1932-9, pp. 267-9.

Physical Properties of Permafrost - Hardy /1946-2 conducted consolidation tests on frozen samples of soil from the permafrost zone using loads up to 10 tons per sq. ft. The loading tests were made at -0.5 C. No thawing or appreciable deformation resulted from the loading. He stated, "The result could have been deduced beforehand from the physical properties of ice." His studies indicated a foundation on frozen sand would not be appreciably affected if the sand thaws. However, for silts, the settlement might be equal to the depth of thaw if conditions were such that the soil would flow. The minimum settlement would equal the reduction in volume on consolidation of the thawed specimen.

Hardy found no ice banding in soils falling within a band ranging from about 82 to 98 percent finer than 1 mm. on the coarse and to 0 to 3 percent finer than 0.01 mm. on the fine end. However silts having from about 7 to 27 percent finer than 0.02 mm. diameter showed ice banding. That was also true of a well-graded gravel having about 6 percent finer than 0.02 mm.

Problems in Construction of Roads and Airfields in Permafrost Areas - The first problem which concerns the engineer who must build roads and airfields in the Arctic is the recognition of the presence of permafrost, and, where it occurs, the nature of the permafrost. One method of locating and classifying permafrost involves the use of air photos.

The current state of advancement of the efforts to identify permafrost features from airphotos is indicated in part by the papers of Frost /1950-11, /1951-31, Frost and Mintzer /1949-27, Frost, Hittle, and Woods /1948-31, /1948-53, and Nelson /1949-14.

It is believed that the use of airphotos to identify soil textures and permafrost conditions in arctic and sub-arctic regions vastly facilitates site selection and development of adequate structural designs. Such identification is possible through understanding of the "pattern" of the surface features as it appears on airphoto prints. The more important elements comprising the various patterns are the nature of land forms, topographic position, drainage pattern, erosion features, gully characteristics, vegetation, color tone, and others.

Knowing the location of permafrost, the second part of the problem involves knowledge of how to construct roads and airfields at minimum cost. A natural first reaction of the engineer is to build very heavy strong structures to counteract the great forces brought-about by the freezing or thawing soil. He must learn that the easiest and most economical approach is to use "passive" methods which make permafrost work for him. This means that designs must be based on insulation and preservation of the permafrost and control of the location of the permafrost table and of its effect on drainage and the structural properties of the soil.

Frost /1951-31 states, "Structures placed on permafrost of the dry frozen type usually do not experience serious damage from the resulting thaw. Airfields, runways, roads, and buildings have been placed on frozen sands and gravels and have performed satisfactorily not only following construction but throughout the critical break-up seasons as well. Thereare, however, often local areas containing detrimental permafrost which lie within the bounds of dry frost and which should be avoided in engineering construction. Such areas are common in land forms associated with water deposition. Those structures placed on detrimentally frozen material

experience severe distress almost immediately following construction."

The Engineering Manual /1946-7 suggests the following approach to the problem of designing a road pavement in permafrost:

(1) Determine the typical thawing index by plotting the accumulative deg.-days above freezing for a particular site using the greatest number of years' observations of air temperatures available from authoritative sources. An example of a thawing index curve is shown in Figure 218.

(2) The thawing index indicated for the site given in the example is 2,710.

(3) Use Figure 219 to determine the estimated depth of thawing, which is 9.2 ft. in the example.

(4) Thus, to satisfy design at the site (9 ft.) of base material is required to prevent the permafrost from melting.

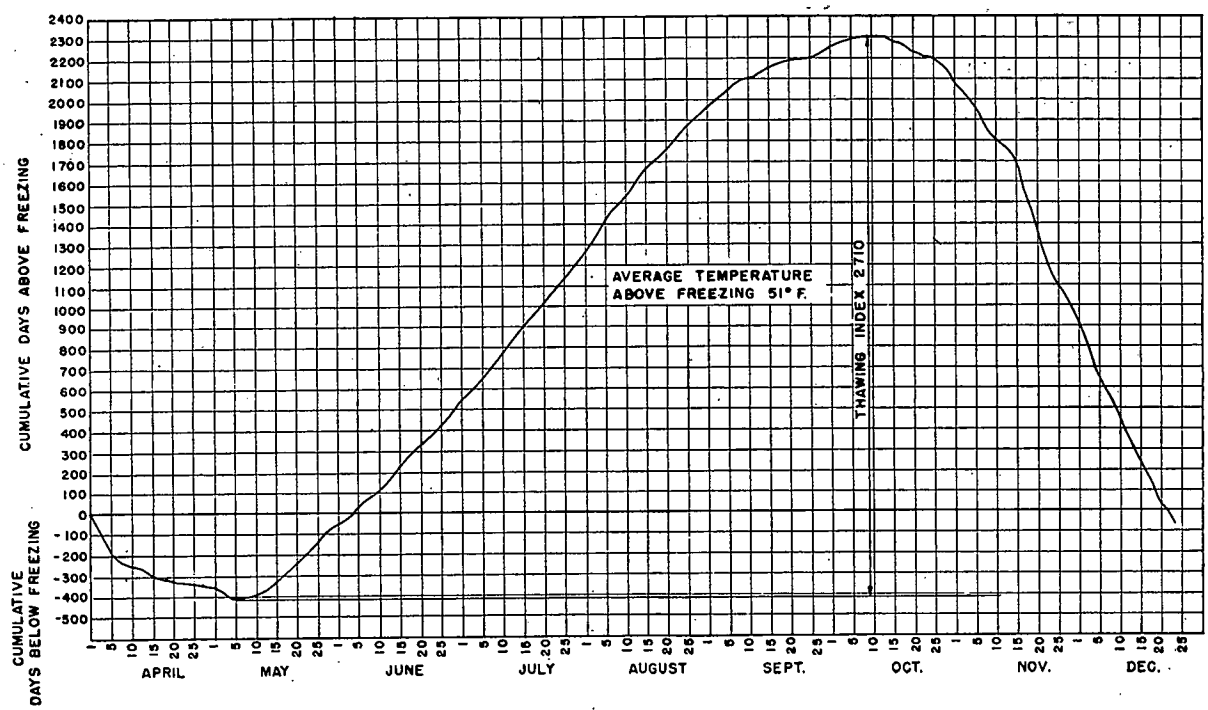


Figure 218
Curve for Cumulative Degree Days Above Freezing (Thawing Index)
(After Corps of Engineers).

The first layer of an insulation course should be fine-grained granular soils which meet the requirements for filter materials.

Muller's review /1945-4 of Russian studies brings out the importance of keeping in mind that the permafrost table will rise beneath the fill and may extend into it if the height of the fill is in excess of the thickness of the active layer. Also seasonal thawing will penetrate deeper on the south side.

The movement of water in an area traversed by a fill may also be affected by the fill, especially if it runs at right angles to the direction of flow, as is indicated in Figure 220. The effect of berms in preserving the frozen condition beneath a fill and in preventing the thawing of underground ice is illustrated in Figure 221.

Road cuts present even greater difficulties, and Muller suggests that the only solution appears to be a replacement of the unstable soil by material which is not affected detrimentally by frost, whether permafrost or seasonal freezing and thawing.

The problem of surface icing is a serious one in some permafrost areas rich in underground water. Surface icing has been described in previous paragraphs. One form of solution is that of artifically inducing a field of ice, which has been illustrated previously.

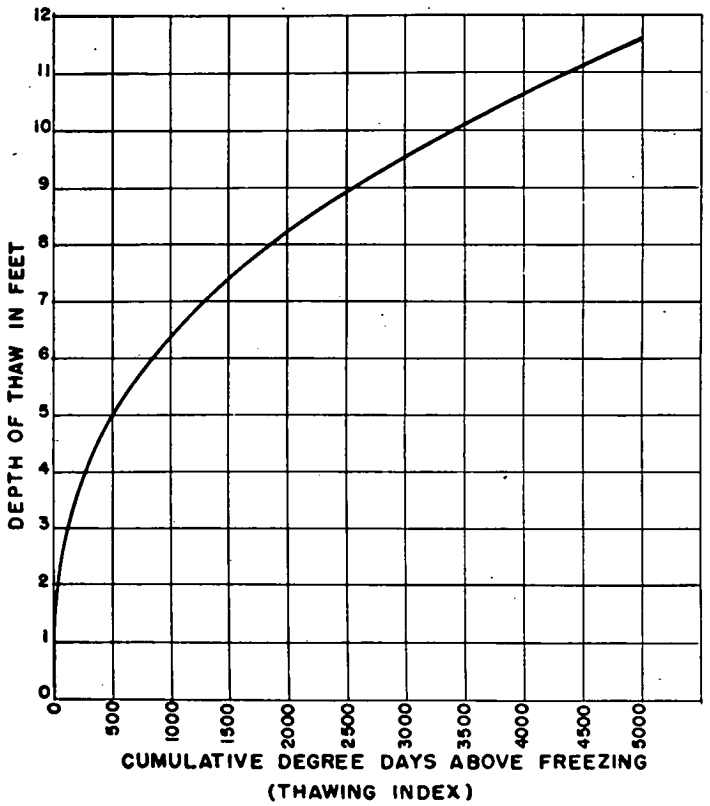


Figure 219. Tentative Curve for Determining Depth of Thaw for Pavements. (After Corps of Engineers)

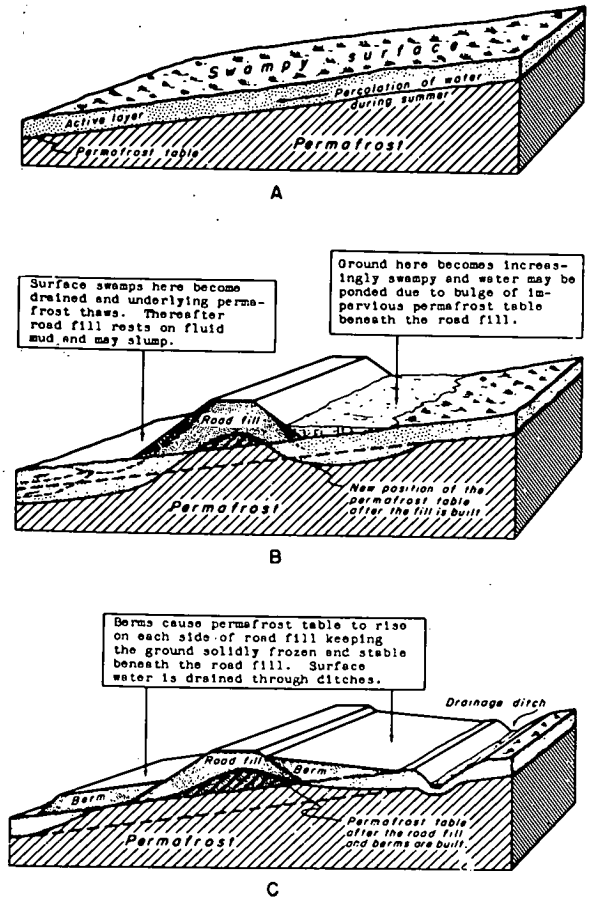


Figure 220. Influence of Road Construction on Permafrost Table. (After Muller)

The above paragraphs by no means tell the whole story. They are intended merely to introduce the problem and to indicate its complexity and the need for careful and thorough study of the nature of permafrost and the various solutions which have been worked out or which are now being investigated. Investigations in progress have been stated in part by Carlson /1948-37, and Nelson /1949-14.

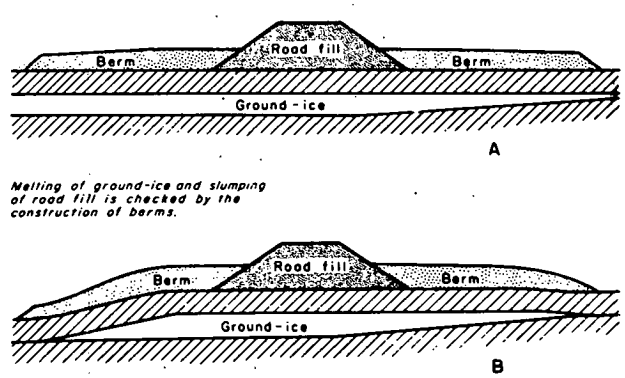


Figure 221. Effect of berms in preserving the frozen condition beneath a road fill. (After Bykov and Kapterev)

Problems in Constructing Foundations for Bridges, and Buildings and Constructing Services in Permafrost Areas - Foundations for structures and proper handling of utilities presents difficult problems in Arctic areas where permafrost occurs. Space does not permit even a superficial treatment of all phases of handling construction in permafrost. The reader if interested may consult the Corps of Engineers Manual /1946-7, Muller /1947-14, Lewin /1948-4, Wilson /1948-17, Carlson /1948-37, and Nelson /1949-14 for a brief introduction into the problems involved.

Evidences of Permafrost in the United States - Field studies by geologists during the last 50 years have, when taken as a whole, brought out a concept that a zone of indefinite width marginal to the continental ice sheets existed and was characterized by intense frost phenomena. Recently this zone has been termed the "periglacial" zone. There occur, in this zone, numerous "fossil" forms which indicate the presence of intense frost action, such as is now found in the

arctic regions where permafrost occurs. This brings out the possibility of permafrost having existed in a large part of northern United States.

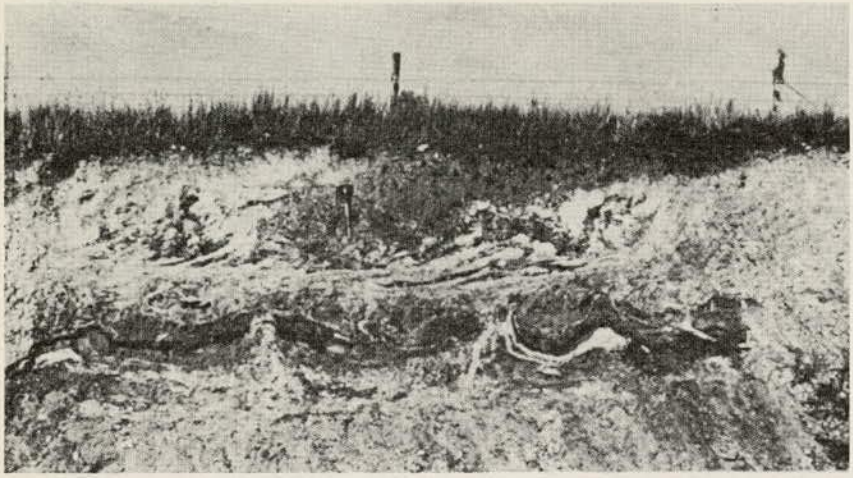


Figure 222. Involutions in Soil Profile in Central Montana. (after Schafer)

While of no great practical significance to road builders, knowledge of its occurrence and the ability to identify some of the "fossil" forms is of interest to engineers working with soils.

Smith /1949-18 discussed 13 types of features which have been interpreted as indicative of former periglacial conditions. Those which are most generally applicable in areas associated with continential ice sheets are: casts of ground-ice wedges, involutions, and solifluction deposits (including block accumulations.

Schafer /1949-5 describes the existence of wedge structures in central Montana which consist of wedge-shaped vertical fissures in weathered bed-rock, which he holds were opened by ice veins developed when the ground became perennially frozen. He also described involutions (local deformation and interpenetration in stratified materials) in central Montana and found they are common throughout the area and occur in a variety of forms. One example is shown in Figure 222. They are believed to have developed by differential freezing and growth of ground ice in a seasonally thawed zone overlying permafrost. Horberg /1949-3 describes the existence of a possible fossil ice wedge in Bureau County (North Central) Illinois.

Several authors have described the existence of stone rings and stone stripes in the United States while numerous authors have described similar surface features presently found in arctic regions. Sharpe /1938-1 gives an excellent review of the literature (and bibliography) on stone rings. He found that there is during intense frost action a sorting action which takes place in boulders or gravelly soils which produces a central area of fine debris. Sharpe describes stone stripes which occur adjacent to U.S. 97 about 12 mi. north of Satus pass in South-central Washington at an elevation of less than 3,000 ft. Sharpe illustrated the relationship between stone rings and solifluction in the development of stone stripes as indicated in Figure 223.

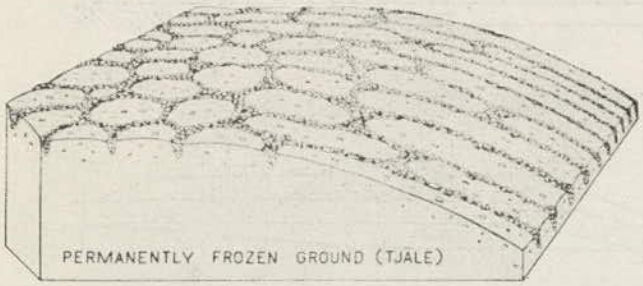


Figure 223. Frost action and solifluction. Stone-rings on the flat surface merge into stone-stripes on the slope. (After Sharpe)

Eakin /1916-2 was one of the earlier authors who studied intense frost action in the Arctic and who offered an explanation of the processes in the development of stone rings. His explanation is well illustrated in Figure 224.

For further study of those phenomena the reader is directed to the following sources: Hobbs /1913-2, pp. 282-4; Högbom /1914-2, pp. 309-18; Hawkes /1924-2, p. 509; Huxley and Odell /1924-5, pp. 208, 224-5; Nordenskjold /1928-11, p. 53; Antevs /1932-7, pp. 49-58; Denny /1936-11, p. 177.

Fossil ground-ice wedges or loam-filled cracks have been described from European areas by Kessler /1925-3, Lotze /1932-9, Soergel /1936-15, Selzer /1936-16, Paterson /1940-17, Weinberger /1944-17, and from Alaska by Taber /1943-4. Such casts as described by those authors are considered positive evidence of periglacial conditions comparable to present day arctic conditions. They could have formed only in association with perennially frozen ground.

There are many other interesting formations which develop in areas of intense frost action and also under normal freezing in the United States. Some of the latter are described later in this review.

Origin of Permafrost - The origin of permafrost has long been a subject of controversy, some believing it is a relic of the ice ages and others believing that it more nearly reflects the results of present climatic conditions.

Russell /1889-1 reports the studies of Thompson, who made extensive calculations of the rate of heat flow from the earth under existing climatic conditions and concluded that "even the deepest ice stratum reported in the Arctic might have resulted from a mean annual temperature no lower than now exists in Alaska."

Leffingwell /1919-1 presented three possibilities: (1) that the present thickness is the result of successive additions of frozen soil, (2) that it is the result of a colder Pliestocene climate, and (3) the statement of Woodward given by Russell.

Nikiforoff /1928-7 was of the opinion that each hypothesis is partly correct and partly incorrect. He questioned that if the glacial hypothesis were true how could the great ice sheets melt while the permanently frozen ground remains. He felt there was no question but that the present climate is favorable to the existence of permafrost. He stated that studies have been made to determine if the southern boundaries are receding or extending, but the question remains unanswered. It has been discovered "in at least 16 places along the southern boundary" that the surface of permafrost lies at a greater depth below the surface than it does farther north. In one place in Siberia a layer 70 ft. thick was found with its surface 103 ft. below the ground near the southern limit of permafrost.

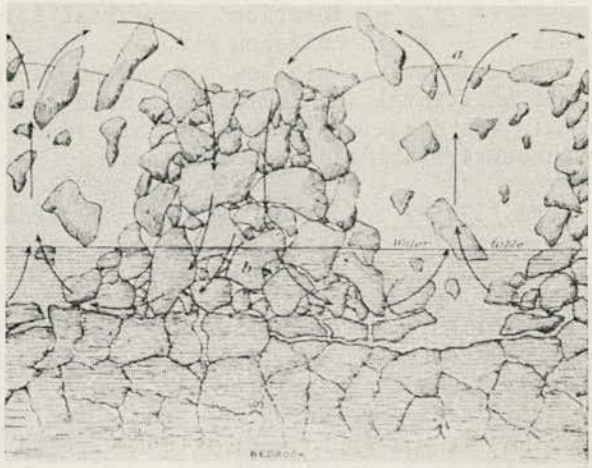


Figure 224. Diagram illustrating segregation of coarse and fine detritus and development of frost-heaved mounds by solifluction. (After Eakin)

Cressey /1939-7 points out the fact that the temperature of the lower depths is lower than near the surface and maintains this indicates ancient origin, while the presence of remains of mammoths points to more recent freezing.

The Corps of Engineers /1946-7 states that "permanently frozen ground may be expected where the mean annual temperature is below freezing in a climate with long, cold winters, short, dry and relatively cool summers, and small precipitation during all seasons."

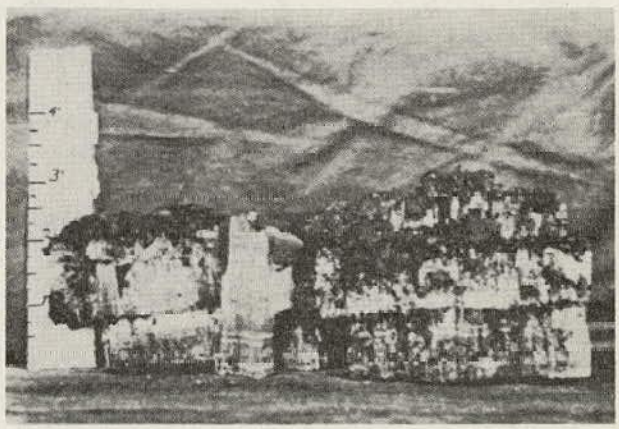


Figure 225. Examples of layered needle ice (sometimes called hoar frost) which grew at or slightly below the ground surface. (After Roberts)

Hardy /1946-2 states that "permafrost is a relic of the last ice age" but "can readily be caused to form where it does not ordinarily occur" and cites cases where permafrost 28 ft. deep formed under a pile of boulders from mining operations and 25 ft. deep under a building in the latitude of Edmonton.

Muller /1947-14, in his extensive studies of permafrost, believed it could originate if the mean temperature were about freezing.

HOAR FROST

The growing of ice crystals, the migration of water as it moves to the growing crystal, and the forces developed during the development of ice crystals to form ice whether in soil or simply as ice on water, may be detected in many different forms. Hoar frost, or needle ice,

is one of those forms. Hoar frost as is described here is not to be confused with the silvery white deposit of ice needles on objects above ground, formed by direct condensation at temperatures below freezing due to nocturnal radiation. Rather it is needle ice which forms on or slightly below the surface of wet soils during spring and fall nights when the air temperature falls below freezing but when the ground is unfrozen. It may occur as numerous separate or small groups of needles or as a mass or layer of needle ice on the surface.

Roberts /1903-1 described hoar frost formed in the Puget Sound region when the air temperature fell to 24 F. on unfrozen ground. During the first night's frost a layer of ice 5/8 in. thick formed immediately below the ground surface. The climate continued favorable for several days, and additional ice layers continued to form for several successive nights. Examples of the needle ice described by Roberts are shown in Figure 225. The three strata of ice represent three successive night's accumulation.

Abbe /1905-2 described similar ice forms and added that they are best developed in sheltered spots on a gravelly soil. He brought out that the ice columns are caused first by the capillary movement of the water to the surface.

Several European investigators, beginning with Hessilman /1907-3, Hogbom /1914-2, Hamberg /1915-6, Johansson /1916-6, Kokkonen /1926-3, and Beskow /1935-1, described this form of soil ice. Their descriptions differ in regard to the type of soil necessary to promote growth of needle ice. Hesselman described needle ice on sod; Hamberg found the ice in clay. Johannson, Kokonen, and Beskow found that it can develop on all mineral soils from sand to clay but found that the fine sands, fine silts, and very silty clays are most susceptible. Beskow gives an excellent description and illustrates different forms of hoar frost.

Bouyoucos and McCool /1928-5 found that a slow and gradual reduction in temperature, but not below about 20 F., causes the film of water at the surface to freeze and draw water from the underlying soil, which grows into long needle-like crystals.

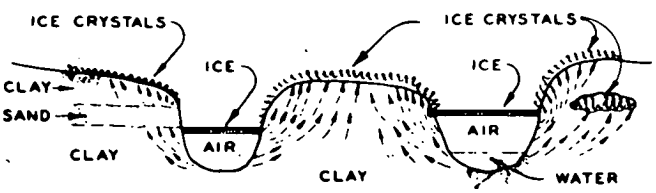


Figure 226. Illustrating Water Movements, Frost and Ice Formations on a Wet Rutted Clay Surface. (After Bureau of Public Roads)

Mullis /1930-8 held that the conditions essential for producing crystals of maximum length are found where the intensity of cold is such that the capillary movement is rapid enough to prevent freezing of the soil to any appreciable depth below the surface.

Mullis illustrated (see Fig. 226) that one-way needle ice forms by associating it with air-filled spaces under ice sheets, the water from ruts and pockets in the soil rising through the soil to the surface to form ice crystals.

Ruckli /1943-2, Troll /1944-12, and Hursh /1948-7 also described this type of fibrocrystalline ice. Troll preferred the name "brush-ice." Troll and Hursh both stressed it as an important agent in erosion and sloughing of soil on bare slopes, particularly on moist, south-facing slopes.

APPENDIX A

THERMAL PROPERTIES OF MATERIALS

This Appendix gives definitions of terms relating to heat transfer; presents limited data pertaining to the thermal properties of water, ice and snow, soil and other materials, and presents a partial review of literature on heat flow in soils.

Definitions

Specific Heat - c_1, of any substance is defined as the calories of heat required to raise one gram one deg. C.

Volumetric Heat Capacity - C, is defined as the heat in Btu. required to raise one cu. ft. of soil one deg. F.

British Thermal Unit - Btu, is the heat required to raise the temperature of water one deg. F. at its maximum density. It is equal to 252 calories.

Latent Heat of Fusion - L, is the quantity of heat required to change unit mass of ice to water with no temperature change.

Thermal Conductivity - k, is the quantity of heat that will pass a unit area of unit thickness in unit time under a unit temperature gradient.

The following units used in studies of heat transfer are expressed as follows:

$$\text{specific heat} = \frac{\text{calories}}{\text{grams X deg. C.}}$$

$$\text{conductivity} = \frac{\text{calories}}{\text{cm. X deg. C. X seconds}}$$

$$\text{diffusivity} = \frac{\text{conductivity}}{\text{specific heat X density}} = \frac{\text{cm}^2}{\text{sec.}}$$

In the English system density is expressed as lbs. per cu. ft.
Conductivity is expressed as:

$$\frac{\text{Btu.}}{\text{Ft. X deg. F. X hour;}}$$

and diffusivity as

$$\frac{\text{sq. ft.}}{\text{hour}}$$

Thermal Properties of Water

Effect of Pressure on the Freezing Point of Water

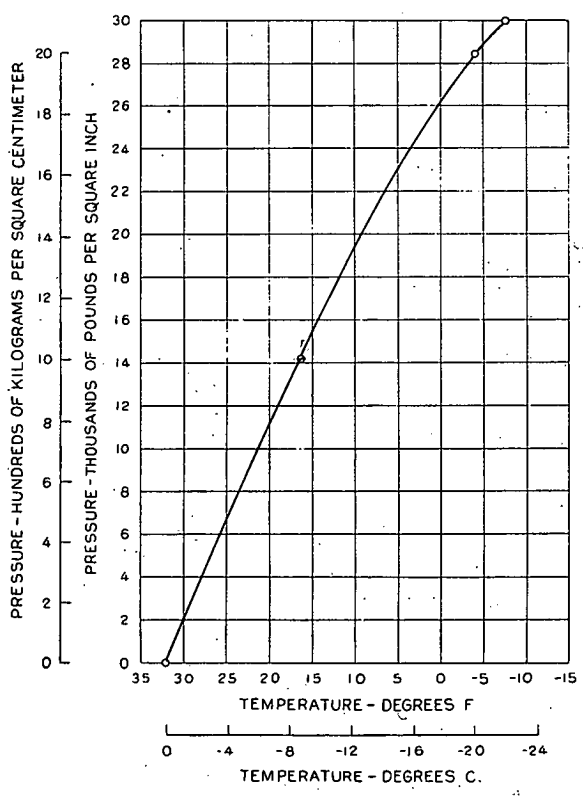


Figure 227. Relation Between the Freezing Point of Water and the Pressure Under Which the Water Exists. (Data from Smithsonian Physical Tables 1921)

TABLE 65

Density and Volume of Water
-10 to +250 C.

The mass of one cubic centimeter at 4 C. is taken as unity

(From Smithsonian Tables)

Temp. C.	Density.	Volume.	Temp. C.	Density.	Volume.
—10°	0.99815	1.00186	**+35°**	0.99406	1.00598
—9	843	157	36	371	633
—8	869	131	37	336	669
—7	892	108	38	300	706
—6	912	088	39	263	743
5	0.99930	1.00070	**40**	0.99225	1.00782
—4	945	055	41	187	821
—3	958	042	42	147	861
—2	970	031	43	107	901
—1	979	021	44	066	943
+0	0.99987	1.00013	**45**	0.99025	1.00985
1	993	007	46	0.98982	1.01028
2	997	003	47	940	072
3	999	001	48	896	116
4	1.00000	1.00000	49	852	162
5	0.99999	1.00001	**50**	0.98807	1.01207
6	997	003	51	762	254
7	993	007	52	715	301
8	988	012	53	669	349
9	981	019	54	621	398
10	0.99973	1.00027	**55**	0.98573	1.01448
11	963	037	60	324	705
12	952	048	65	059	979
13	940	060	70	0.97781	1.02270
14	927	073	75	489	576
15	0.99913	1.00087	**80**	0.97183	1.02899
16	897	103	85	0.96865	1.03237
17	880	120	90	534	590
18	862	138	95	192	959
19	843	157	100	0.95838	1.04343
20	0.99823	1.00177	**110**	0.9510	1.0515
21	802	198	120	.9434	1.0601
22	780	220	130	.9352	1.0693
23	757	244	140	.9264	1.0794
24	733	268	150	.9173	1.0902
25	0.99708	1.00293	**160**	0.9075	1.1019
26	682	320	170	.8973	1.1145
27	655	347	180	.8866	1.1279
28	627	375	190	.8750	1.1429
29	598	404	200	.8628	1.1590
30	0.99568	1.00434	**210**	0.850	1.177
31	537	465	220	.837	1.195
32	506	497	230	.823	1.215
33	473	530	240	.809	1.236
34	440	563	250	.794	1.259

From -10 C. to 0 C. the values are due to means from Pierre, Weidner, and Rosetti; from 0 C to 41 C., to Chappuis 42 C. to 100 C., to Thiesen; 110 C. to 250 C., to means from the works of Ramsey, Young, Waterston, and Hirn.

TABLE 66

Lowering of Freezing Points by Salt in Solution

In the first column is given the number of gram-molecules (anhydrous) dissolved in 1000 grams of water; the second contains the molecular lowering of the freezing point; the freezing point is therefore the product of these two columns. After the chemical formula is given the molecular weight, then a reference number.

(From Smithsonian Tables)

g. mol. / 1000 g. H_2O	Molecular Lowering.	g. mol. / 1000 g. H_2O	Molecular Lowering.	g. mol. / 1000 g. H_2O	Molecular Lowering.	g. mol. / 1000 g. H_2O	Molecular Lowering.
$Pb(NO_3)_2$, 331.0: 1, 2.		0.0500	3.47°	0.4978	2.02°	$MgCl_2$, 95.26: 6, 14.	
0.000362	5.5°	.1000	3.42	.8112	2.01	0.0100	5.1°
.001204	5.30	.2000	3.32	1.5233	2.28	.0500	4.98
.002805	5.17	.500	3.26	$BaCl_2$, 208.3: 3, 6, 13.		.1500	4.96
.005570	4.97	1.000	3.14	0.00200	5.5°	.3000	5.186
.01737	4.69	$LiNO_3$, 69.07: 9.		.00498	5.2	.6099	5.69
5015	2.99	0.0398	3.4°	.0100	5.0	KCl, 74.60: 9, 17-19.	
$Ba(NO_3)_2$, 261.5: 1.		.1671	3.35	.0200	4.95	0.02910	3.54°
0.000383	5.6°	.4728	3.35	.04805	4.80	.05845	3.40
.001259	5.28	1.0164	3.49	.100	4.69	.112	3.43
.002681	5.23	$Al_2(SO_4)_3$, 342.4: 10.		.200	4.66	.3139	3.41
.005422	5.13	0.0131	5.6°	.500	4.82	.476	3.37
.008352	5.04	.0261	4.9	.586	5.03	1.000	3.286
$Cd(NO_3)_2$, 236.5: 3.		.0543	4.5	.750	5.21	1.989	3.25
0.00298	5.4°	.1086	4.03	$CdCl_2$, 183.3: 3, 14.		3.209	3.25
.00689	5.25	.217	3.83	0.00299	5.0°	NaCl, 58.50: 3, 20, 12, 16.	
.01997	5.18	$CdSO_4$, 208.5: 1, 11.		.00690	4.8	0.00309	3.7°
.04873	5.15	0.000704	3.35°	.0200	4.64	.01000	3.67
$AgNO_3$, 167.0: 4, 5.		.002685	3.05	.0541	4.11	.0221	3.55
0.1506	3.32°	.01151	2.69	.0818	3.93	.04949	3.51
.5001	2.96	.03120	2.42	.214	3.39	.1081	3.48
.8645	2.87	.1473	2.13	.429	3.03	.2325	3.42
1.749	2.27	.4129	1.80	.858	2.71	.4293	3.37
2.953	1.85	.7501	1.76	1.072	2.75	.700	3.43
3.856	1.64	1.253	1.86	$CuCl_2$, 134.5: 9		NH_4Cl, 53.52: 6, 15.	
0.0560	3.82	K_2SO_4, 174.4: 3, 5, 6, 10, 12.		0.0350	4.9°	0.0100	3.6°
.1401	3.58	0.00200	5.4°	.1337	4.81	.0200	3.56
.3490	3.28	.00398	5.3	.3380	4.92	.0350	3.50
KNO_3, 101.9: 6, 7.		.00865	4.9	.7149	5.32	.1000	3.43
0.0100	3.5	.0200	4.76	$CoCl_2$, 129.9: 9.		.2000	3.390
.0200	3.5	.0500	4.60	0.0276	5.0°	.4000	3.393
.0500	3.41	.1000	4.32	.1094	4.9	.7000	3.41
.100	3.31	.200	4.07	.2369	5.03	LiCl, 42.48: 9, 15	
.200	3.19	.454	3.87	.4399	5.30	0.00302	3.7°
.500	3.08	$CuSO_4$, 159.7: 1, 4, 11.		.538	5.5	.0455	3.5
.750	2.94	0.000286	3.3°	$CaCl_2$, 111.0: 5, 13-16.		.09052	3.53
1.000	2.81	.000843	3.15	0.0100	5.1°	.2474	3.50
$NaNO_3$, 85.09: 2, 6, 7	2.66	.002279	3.03	.05028	4.85	.5012	3.01
0.0100	3.6°	.006670	2.70	.1006	4.79	.7939	3.71
.0250	3.46	.01463	2.59	.5077	5.33	$BaBr_2$, 297.3: 14	
.0500	3.44	.1051	2.28	.946	5.3	0.100	5.1°
.2000	3.345	.2074	1.95	2.432	8.2	.150	4.9
.500	3.24	.4043	1.84	3.469	11.5	.200	5.00
.5015	3.30	.8898	1.76	3.829	14.4	.500	5.18
1.000	3.15	$MgSO_4$, 120.4: 1, 4, 11.		0.0478	5.2	$AlBr_3$, 267.0: 9.	
1.0030	3.03	0.000675	3.29	.153	4.91	0.0078	1.4°
NH_4NO_3, 80.11: 6, 8.		.002381	3.10	.331	5.15	.0559	1.2
0.0100	3.6°	.01263	2.72	.612	5.47	.1071	1.07
.0250	3.50	.0580	2.65	.998	6.34	.4355	1.07
		.2104	2.23				

1 Hausrath, Ann. Phys. 9, 1902.
2 Leblanc-Noyes, Z. Phys. Ch. 6, 1890.
3 Jones, Z. Phys. Ch. 11, 1893.
4 Raoult, Z. Phys. Ch. 2, 1888.
5 Arrhenius, Z. Phys. Ch. 2, 1888.
6 Loomis, Wied. Ann. 57, 1896.
7 Jones, Am. Chem. J. 27, 1902.
8 Jones-Caldwell, Am. Chem. J. 25, 1901.
9 Biltz, Z. Phys. Ch. 40, 1902.
10 Jones-Mackay, Am. Chem. J. 19, 1897.
11 Kahlenberg, J. Phys. Ch. 4, 1901.
12 Abegg, Z. Phys. Ch. 20, 1896.
13 Jones-Getman, Am. Ch. J. 27, 1902.
14 Jones-Chambers, Am. Ch. J. 23, 1900
15 Loomis, Wied. Ann. 60, 1897
16 Roozeboom, Z. Phys. Ch. 4, 1889.
17 Raoult, Z. Phys. Ch. 27, 1898.
18 Roloff, Z. Phys. Ch. 18, 1895.
19 Kistiakowsky, Z. Phys. Ch. 8, 1890.
20 Loomis, Wied. Ann. 51, 1894

Compiled from Landolt-Börnstein-Meyerhoffer's Physikalisch-chemische Tabellen.

TABLE 67

Specific Heat of Water
(From Smithsonian Tables)

Temperature C.	Barnes	Rowland	Barnes-Regnault	Temperature C.	Barnes	Barnes-Regnault
-5	1.0155	-	-	60	0.9988	0.9994
0	1.0091	1.0070	1.0094	65	.9994	1.0004
+5	1.0050	1.0039	1.0053	70	1.0001	1.0015
10	1.0020	1.0016	1.0023	80	1.0014	1.0042
15	1.0000	1.0000	1.0003	90	1.0028	1.0070
20	0.9987	.9991	0.9990	100	1.0043	1.0101
25	.9978	.9989	.9981	120	-	1.0162
30	.9973	.9990	.9976	140	-	1.0223
35	.9971	.9997	.9974	160	-	1.0285
40	.9971	1.0006	.9974	180	-	1.0348
45	.9973	1.0018	.9976	200	-	1.0410
50	.9977	1.0031	.9980	220	-	1.0476
55	.9982	1.0045	.9985	——	-	-

TABLE 68

Relative Humidity, Vapor Pressure and Dry Temperatures
(From Smithsonian Tables)

Vapor Pressure, mm.	Air Temperatures, dry bulb, ° Centigrade.																			
	20°	21°	22°	23°	24°	25°	26°	27°	28°	29°	30°	31°	32°	33°	34°	35°	36°	37°	38°	39°
1	6	5	5	5	5	4	4	4	4	3	3	3	3	3	3	3	2	2	2	2
2	12	11	10	10	9	8	8	8	7	7	6	6	6	5	5	5	5	4	4	4
3	17	16	15	14	14	13	12	11	11	10	10	9	9	8	8	7	7	6	6	6
4	23	22	20	19	18	17	16	15	14	13	13	12	11	11	10	10	9	9	8	8
5	29	27	25	24	23	21	20	19	18	17	16	15	14	13	13	12	11	11	10	10
6	34	32	31	29	27	26	24	23	21	20	19	18	17	16	15	14	14	13	12	12
7	40	38	36	34	32	30	28	26	25	24	22	21	20	19	18	17	16	15	14	13
8	46	43	41	38	36	34	32	30	29	27	25	24	23	21	20	19	18	17	16	15
9	52	49	46	43	41	38	36	34	32	30	29	27	25	24	23	22	20	19	18	17
10	57	54	51	48	45	43	40	38	36	34	32	30	28	27	25	24	23	21	20	19
11	63	60	56	53	50	47	44	42	39	37	35	33	31	29	28	26	25	24	22	21
12	69	65	61	58	54	51	48	45	43	40	38	36	34	32	30	29	27	26	24	23
13	75	70	66	62	59	55	52	49	46	44	41	39	37	35	33	31	29	28	26	25
14	80	76	71	67	63	60	56	53	50	47	44	42	40	37	35	33	32	30	28	27
15	86	81	76	72	68	64	60	57	53	50	48	45	42	40	38	36	34	32	30	29
16	92	87	82	77	72	68	64	60	57	54	51	48	45	43	41	38	36	34	32	31
17	98	92	87	81	77	72	68	64	61	57	54	51	48	45	43	41	38	36	34	33
18	-	97	92	86	81	77	72	68	64	60	57	54	51	48	46	43	41	39	37	35
19	-	-	97	91	86	81	76	72	68	64	60	57	54	51	48	45	43	41	39	37
20	-	-	-	96	90	85	80	76	71	67	63	60	57	53	51	48	45	43	41	38
21	-	-	-	-	95	89	84	79	75	71	67	63	59	56	53	50	48	45	43	40
22	-	-	-	-	100	94	88	83	78	74	70	66	62	59	56	53	50	47	45	42
23	-	-	-	-	-	98	92	87	82	77	73	69	65	62	58	55	52	49	47	44
24	-	-	-	-	-	-	96	91	85	81	76	72	68	64	61	57	54	51	49	46
25	-	-	-	-	-	-	100	94	89	84	79	75	71	67	63	60	56	54	51	48
26	-	-	-	-	-	-	-	98	93	87	83	78	74	70	66	62	59	56	53	50
27	-	-	-	-	-	-	-	-	96	91	86	81	76	72	68	65	61	58	55	52
28	-	-	-	-	-	-	-	-	100	94	89	84	79	75	71	67	63	60	57	54
29	-	-	-	-	-	-	-	-	-	97	92	87	82	78	73	69	65	62	59	56
30	-	-	-	-	-	-	-	-	-	-	95	90	85	80	76	72	68	64	61	58
31	-	-	-	-	-	-	-	-	-	-	98	93	88	83	78	74	70	66	63	60
32	-	-	-	-	-	-	-	-	-	-	-	96	91	86	81	77	72	69	65	62
33	-	-	-	-	-	-	-	-	-	-	-	99	93	88	84	79	75	71	67	63
34	-	-	-	-	-	-	-	-	-	-	-	-	96	91	86	81	77	73	69	65
35	-	-	-	-	-	-	-	-	-	-	-	-	99	94	89	84	79	75	71	67
36	-	-	-	-	-	-	-	-	-	-	-	-	-	96	91	86	81	77	73	69
37	-	-	-	-	-	-	-	-	-	-	-	-	-	99	94	89	84	79	75	71
38	-	-	-	-	-	-	-	-	-	-	-	-	-	-	96	91	86	81	77	73
39	-	-	-	-	-	-	-	-	-	-	-	-	-	-	99	93	88	83	79	75
40	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	96	90	86	81	77
41	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	98	93	88	83	79
42	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	100	95	90	85	81
43	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	97	92	87	83
44	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	99	94	89	84
45	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	96	91	86
46	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	99	93	88
47	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	95	90
48	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	97	92
49	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	99	94
50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	96
51	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	98
52	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	100
53	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
54	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

SMITHSONIAN TABLES.

TABLE 69

Thermal Conductivity of Water and Salt Solutions
(From Smithsonian Tables)

Substance	°C	k_t	Authority	Solution in water.	Density	°C	k_t	Authority
Water	0	0.00150	Goldschmidt,'11	$CuSO_4$	1.160	4.4	0.00118	H.F. Weber
	11	.00147		KCl	1.026	13.	.00116	Graetz
	25	.00136	Lees,'98	NaCl	1.178	4.4	.00115	
	20	.00143		NaCl	--	26.3	.00135	H.F. Weber
			Milner, Chattock,'98	H_2SO_4	1.054	20.5	.00126	
				"	1.180	21.	.00130	Chree
				$ZnSO_4$	1.134	4.5	.00118	
				"	1.136	4.5	.00115	H.F. Weber

Thermal Properties of Ice and Snow

The following properties of ice and snow were taken from Barnes /1928-1

TABLE 70

Properties of Ice

Density	.91676	
Heat of Fusion	79.67	
Latent heat of vaporization		
slow rate (av.)	700	Calories
fast rate (av.)	608	Calories
Specific heat	0.49	
Thermal conductivity	0.0050-0.00517	
Coefficient of cubical expansion	0.000160	
Vapor pressure		
Compressive strength		
(parallel to crystals) @ 28 deg. F.	300 psi	
@ 14 deg. F.	693 psi	
(rate of loading of 500 lb. per sec.) @ 2 deg. F.	811 psi	
Shear strength - St. Lawrence ice		
In a direction parallel to walls of crystals	1800 psi	
In a direction parallel to basal planes	1050 psi	

TABLE 71

Properties of Snow

Density	Varies widely 0.045-0.105
Thermal conductivity	0.00028-.00045

Table 72 gives thermal data on snow taken from Ingersoll and Koepp /1924-1.

TABLE 72

Material	Diffusivity	Specific Heat	Density grams per cc.	Density pcf.	Conductivity
Snow, densely packed	0.0041	0.50	0.54	33.7	0.0011

Thermal Properties of Soils

The soil mass is made up of soil, air, and water. The water may be in the frozen or unfrozen state. If in the frozen state it may exist as mass ice or as ice crystals occupying only a part of the spaces between grains. Because the water in fine-grained soils may range from loosely held pore water to "tightly bound" water in film phase, some of the water may exist in the frozen form while some may exist in the unfrozen state due to a difference in the freezing points of the different phases of soil water. Thus the composition of the soil grains, the proportion of soil water, and the density of the soil mass, each has an influence on the thermal properties of soils.

Several investigators have made determinations of the thermal properties of soils in the dry state. One reason for making determinations in this state has been the difficulty of determining experimentally the conductivity of moist soils because of the movement of moisture in the direction of heat transfer during the test, making it difficult to maintain uniform moisture content. However, some investigators have made determinations on moist soils and have presented data on soil type (texture) and condition (moisture content and density). Data presented here are from only a few of the principal investigators. Additional sources of information are given for those who wish to extend their studies beyond the scope of this review.

Patten /1909-1 was one of the early investigators to bring out that the thermal conductivity of soil constituents varied but little from one soil to another. The work of Smith in determining thermal conductivity on dry disturbed samples /1939-11 and on dry undisturbed samples /1942-8, the work of Smith and Byers /1938-6, and the work of Baver /1940-2 confirmed previous findings. The work of those investigators in the determination of thermal properties has shifted the emphasis from composition to porosity and moisture content.

Tests were made at Harvard University for the Corps of Engineers /1945-17, /1947-2 to determine the thermal conductivity of various cohesionless materials and asphaltic bituminous concrete. The grain size distribution curves of the materials tested are shown in Figure 228.

The materials used are described as follows:

(1) Sand (Lowell Sand) consisted of a cohesionless, siliceous sand from a glacial outwash deposit at South Lowell, Massachusetts.

(2) Crushed Rock (Winchester Crushed Rock) consisted of a fine-grained quartz diorite obtained from quarry at Winchester, Mass.

(3) Slag (Mystic Slag) consisted of basic residue from blast furnace located at Everett, Massachusetts

(4) Cinders (Somerville Cinders) consisted of commercial grade cinders obtained locally as a result of burning bituminous coal.

(5) Sand and Gravel (Bangor Sand and Gravel) consisted of a well graded sand and gravel of glacial origin obtained from Bangor, Me.

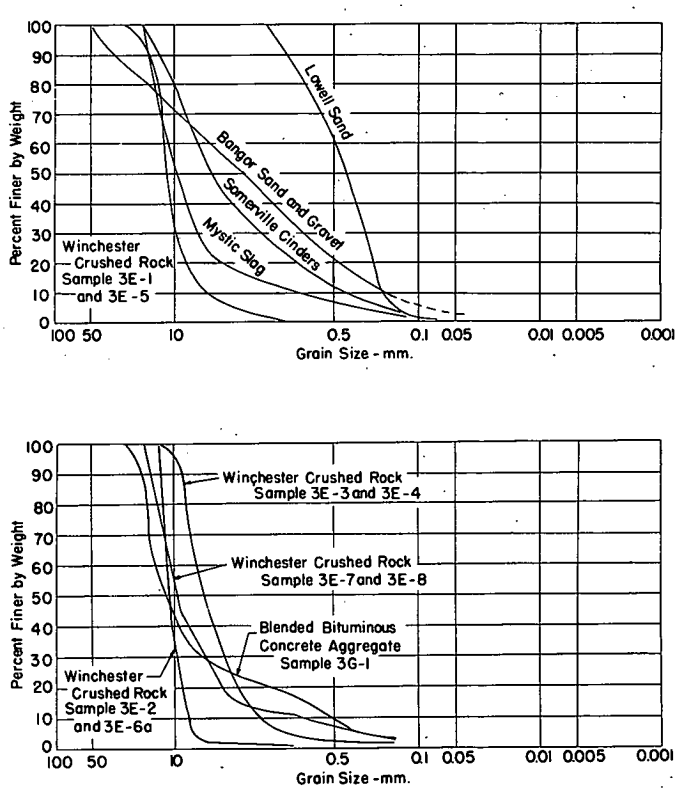


Figure 228. Gradation of Base Materials (After Corps of Engineers)

(6) <u>Blended Bituminous Concrete Aggregate</u> consisted of locally processed aggregates of sand and partially crushed gravel obtained from bins at plant at Cambridge, Massachusetts

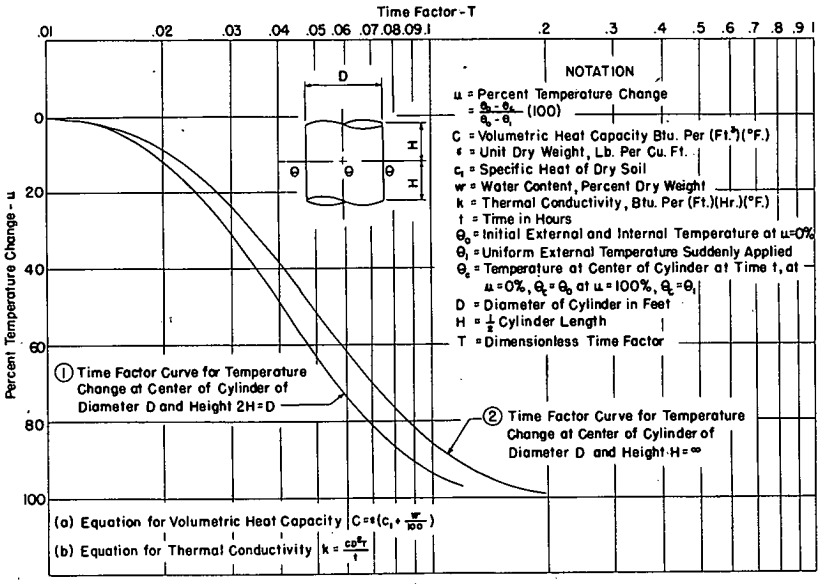


Figure 229
Time Factor Curves for Temperature Change
At Center of a Cylinder
(After Corps of Engineers)

(7) <u>Asphaltic Bituminous Concrete</u> consisted of the blanded bituminous-concrete aggregate and 4.5 percent bitumen.

Time factor curves for temperature change at the center of a cylinder and typical time curves (thermal conductivity determinations) are shown in Figures 229 and 230.

Data from the thermal conductivity tests are summarized in Table 73.

The relationships between thermal conductivity and water content of the various base materials are shown in Figure 231. It should be noted that slag and cinders showed markedly lower values of thermal conductivities than did crushed rock and sand at comparable water contents.

TABLE 73

SUMMARY OF TEST DATA

Series No.	Laboratory Sample No.	Material	Unit Dry Weight Lbs/cu.ft. γ	Water Content Percent Dry Weight w	Specific Gravity G	Specific Heat(1) Dry Soil Btu/(lb)(deg F) c	Volumetric Heat Capacity Total Sample Btu/(ft³)(deg F) C	Thermal Conductivity Btu/(ft)(hr)(deg F) k	REMARKS
3A	3A-4	Lowell Sand (well-graded medium to coarse sand) (2)	105.0	0.2	2.66	0.20	21.2	0.224	(1) Assumed (2) Minimum dry density 92.9 lbs/cu.ft. Max. dry density 110.9 lb/cu.ft. (3) Sample not properly sealed some water leaked into sample during test. (4) Test results are not consistent with results of other tests. (5) Average — w = 14.2% top of sample w = 27.3% bottom of sample (6) Non-uniform water content (7) Percent Bitumen 4.5%
	3A-4a		105.0	0.2	2.66	0.20	21.2	0.224	
	3A-5(3)		101.0	0.2	2.66	0.20	20.4	0.201	
	3A-6c		106.5	16.4	2.66	0.20	38.8	1.22	
	3A-7		101.0	20.9	2.66	0.20	41.3	1.19	
	3A-8		103.0	4.5	2.66	0.20	25.3	0.855	
	3A-9		83.5	4.9	2.66	0.20	20.8	0.559	
	3A-10(3)		84.5	2.3	2.66	0.20	18.8	0.399	
	3A-11(3)		91.1	1.9	2.66	0.20	19.9	0.418	
	3A-12		109.0	2.2	2.66	0.20	24.3	0.693	
	3A-13		103.0	2.0	2.66	0.20	22.7	0.566	
	3A-15		89.3	2.1	2.66	0.20	19.7	0.552	
	3A-16		105.0	5.1	2.66	0.20	26.4	0.925	
	3A-17		90.8	2.1	2.66	0.20	20.1	0.520	
3B	3B-1	Bangor Sand and Gravel	127.0	3.4	2.70	0.20	29.8	1.06	
	3B-2		131.5	1.1	2.70	0.20	27.7	0.801	
	3B-3		127.0	9.3	2.70	0.20	36.3	1.34	
3C	3C-1	Somerville Cinders (1-inch maximum)	60.9	20.7(5)	1.28	0.18	23.6	0.420	
	3C-2		60.0	36.6	1.28	0.18	32.8	0.550	
	3C-3		60.8	21.2(6)	1.28	0.18	23.9	0.422	
	3C-4		61.7	11.3	1.28	0.18	18.1	0.354	
3D	3D-1	Mystic Slag (1½-inch maximum)	79.1	9.1	2.18	0.17	17.5	0.224	
	3D-2(4)		81.2	33.5	2.18	0.17	40.9	0.658	
3E	3E-1	Winchester Crushed Trap Rock (3/4-in. maximum)	99.2	1.9	2.91	0.20	21.7	0.416	
	3E-2		100.0	2.1	2.91	0.20	22.1	0.441	
	3E-3		98.5	4.4	2.91	0.20	23.6	0.480	
	3E-4		98.5	27.2	2.91	0.20	46.5	1.01	
	3E-5(4)		99.3	28.4	2.91	0.20	48.0	2.76	
	3E-6a(4)		100.0	27.7	2.91	0.20	47.7	2.20	
	3E-7		102.0	2.5	2.91	0.20	23.0	0.441	
	3E-8		102.0	26.7	2.91	0.20	47.7	1.76	
3F	3F-1	Asphaltic Bituminous Concrete	150.0(7)	0.0	2.60	0.20	30.3	0.820	
	3F-2		150.0(7)	0.0	2.60	0.20	30.3	0.815	
3G	3G-1(3)	Blended Bituminous Concrete Aggregate	133.5	0.0	2.81	0.20	26.7	0.372	

Method of Computing Thermal Conductivity - The equation used for computing the thermal conductivity from the data obtained, together with the nomenclature, is contained on Figure 229. This equation (b) was derived for the following assumptions:

(1) The temperature of the exterior boundary of the soil is equal to the temperature of the liquid bath into which the container is immersed.

(2) The range of temperature change during a test was either above or below the freezing point of water, hence latent heat of fusion was not a factor.

Computation for Thermal Conductivity- From the test data the thermal conductivity k, may be computed if the volumetric heat capacity is known. It may be assumed that the volumetric heat capacity can be computed for a given soil using equation (a) Figure 229. This equation is based upon the assumption that the volumetric heat capacity is equal to the sum of the volumetric heat capacities of the dry soil and of the water present in the soil. The value for the specific heat, c_1, of dry soil is a variable depending upon the mineral and chemical constituents of the soil. Reference to the tabulations of specific heat for various minerals, rocks, and dry soils based upon tests by various investigators will show that the specific heat of dry soil, minerals, and rocks varies within narrow limits and that a value of approximately 0.2 Btu per lb. (deg. F) is a good average value. The value of the specific heat of water is 1 Btu per lb. (deg. F.) and of ice is 0.5 Btu per lb. (deg. F.). Hence, using equation (a), the volumetric heat capacity of each test may be computed using the assumed value for specific heat of dry soil shown in Table 73, and the determined water content and density. Substituting the computed value for volumetric heat capacity, the thermal conductivity, k, may be computed using equation (b). An example of the determination of thermal conductivity k is as follows:

Equation $k = \frac{C\ D^2 T}{t}$

Test Data from Table 73, Sample 3A-B are used:

$C_1 = 0.20$
$\gamma = 103$ lbs. per cu. ft.
$w = 4.5$ percent

$$C = \gamma\left(C_1 + \frac{w}{100}\right) = 103\left(0.20 + \frac{4.5}{100}\right) = 25.3 \text{ BTU per } (ft^3)(\text{Deg. F.})$$

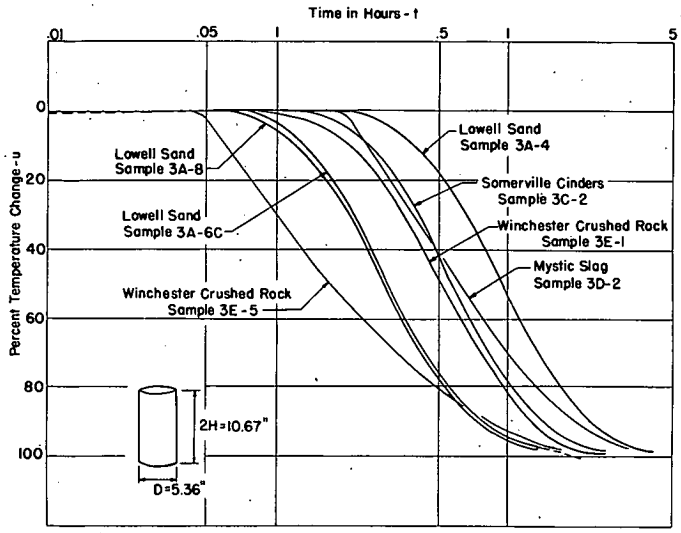


Figure 230
Typical Time Curves
Thermal Conductivity Determinations

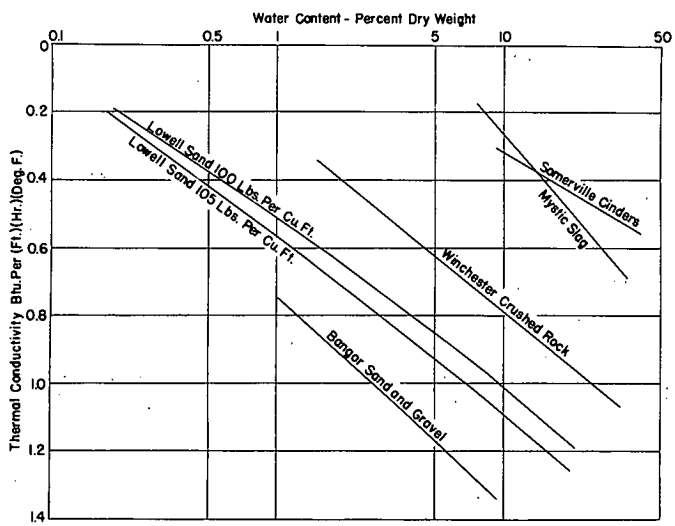


Figure 231
Thermal Conductivity vs Water Content
of Various Base Materials

From Fig. 230

$D^2 = (5.36'')^2 = (0.446')^2 = 0.1995$ Ft.^{2}

FOR μ = 50 percent. t = 0.295 hours.

(50% temperature change is arbitrarily taken. Any value of μ on the straight portion of the curve may be used).

From Fig. 229 Curve (2)

FOR μ = 50%, T = 0.05

Substituting in equation

$$k = \frac{C\,D^2\,T}{t} = \frac{(2.53)\ (0.1995)\ (0.05)}{(2.95)}$$

$$k = 0.855 \text{ Btu. per (Ft.)(Hr.)(Deg. F.)}$$

Patten was also one of the first to make comprehensive studies of the thermal properties of soils through a wide range of moisture contents. Patten's data were initially published in tabular form. The Corps of Engineers, in its frost investigation /1945-17 summarized Patten's work in tabular and graphical form.

Ingersoll and Koepp determined the thermal values for three different materials. Their results are shown in Table 74.

TABLE 74

THERMAL PROPERTIES OF SOIL AS DETERMINED BY INGERSOLL AND KOEPP /1924-1

Material	Percent Moisture Content	Diffusivity	Specific Heat	Density g.per cc.	Density pcf.	Conductivity k
Quartz sand medium, fine, dry	0	0.0020	0.19	1.65	103.0	0.00063
Same with water	8.3	0.0034	0.24	1.75	109.2	0.0014
Sandy clay	15.0	0.0038	0.33	1.78	111.1	0.0022
Calcareous earth, water saturated	43.0	0.0019	0.53	1.67	104.3	0.0017

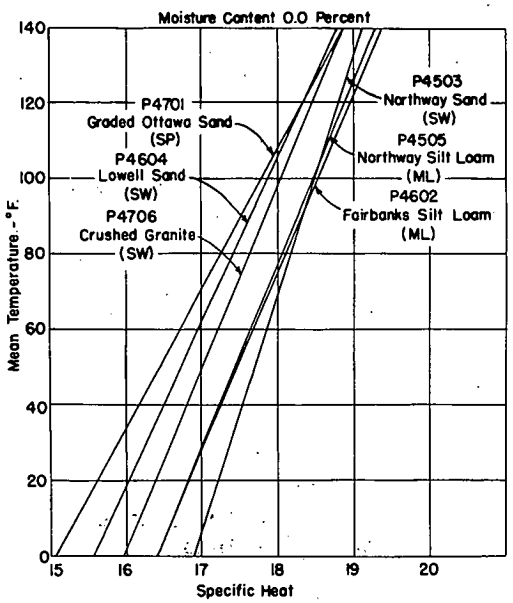


Figure 232. Variation of Specific Heat With Mean Temperature (Moisture Content 0.0%) (After Kersten)

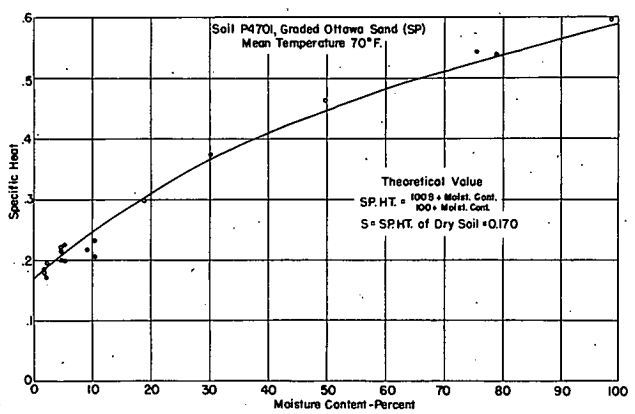


Figure 233. Relationship of Specific Heat of Soil Water Mixtures and Moisture Content (After Kersten)

Kersten /1948-33, /1948-41, and /1949-16 carried on extensive studies of the thermal properties of soils for the Corps of Engineers. He tested 19 soils, five of which were sands and gravels, six were of fine-grained texture ranging from sandy loam to clay, seven were minerals or crushed rocks and one was an organic soil. In addition, tests were made on a bituminous paving mixture and several insulating materials. Kersten's extensive studies brought out the effects of temperature, moisture content, density, particle size, and shape and mineral composition on the thermal conductivity, specific heat, and diffusivity of the materials tested.

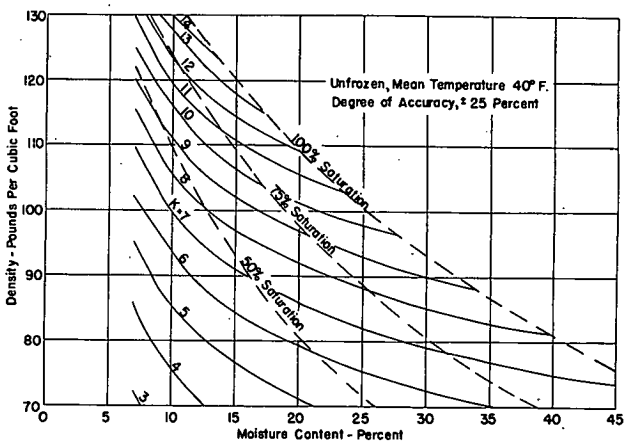


Figure 234. Diagram of Thermal Conductivity for Silt and Clay Soils. (After Kersten)

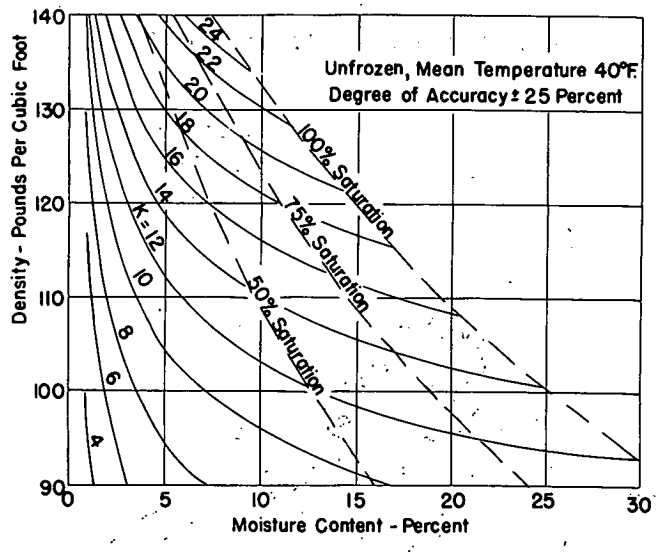


Figure 235. Diagram of Thermal Conductivity for Sandy Soils. (After Kersten)

Space does not permit the presentation of data showing those relationships for all the materials tested. The relationship between mean temperature and specific heat for six soils and the relationship between specific heat and the moisture content for graded Ottawa sand are shown in Figures 232 and 233 respectively. The effect of the combined factors of density and moisture content on the thermal conductivities of soils is shown in the diagrams in Figures 234, 235, 236 and 237. The relationship between thermal conductivity and mean temperature of the bituminous paving mixture is shown in Figure 238.

Diffusivity - Kersten made no tests for diffusivity. However, the values may be calculated for dry soils by the formula:

$$h^2 = \frac{0.08333k}{(\text{specific heat})\ (\text{density})} \qquad \text{(a)}$$

in which h^2 is the diffusivity in feet squared per hour.

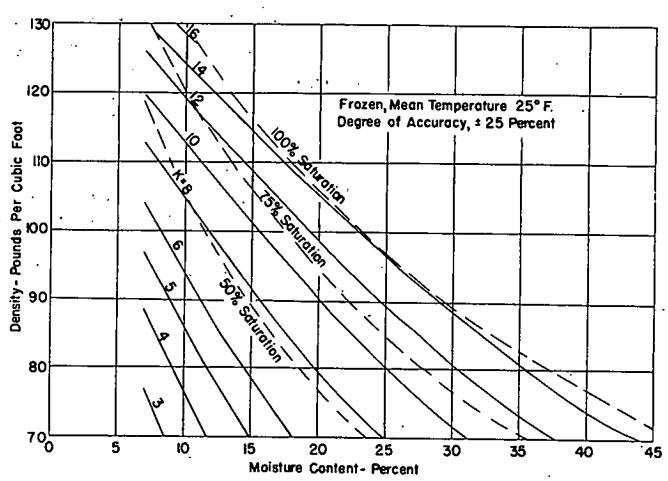


Figure 236. Diagram of Thermal Conductivity for Silt and Clay Soils. (After Kersten)

For moist, unfrozen soils the diffusivity equation may be written:

$$h^2 = \frac{0.08333k}{\text{dry density (spec. heat soil} + \frac{\text{moisture content}}{100})} \qquad \text{(b)}$$

For frozen soils the equation is

$$h^2 = \frac{0.08333k}{\text{dry density (spec. heat soil} + \frac{\text{moisture content}}{100} \times 0.5)} \qquad \text{(c)}$$

The value 0.5 is the approximate specific heat of ice.

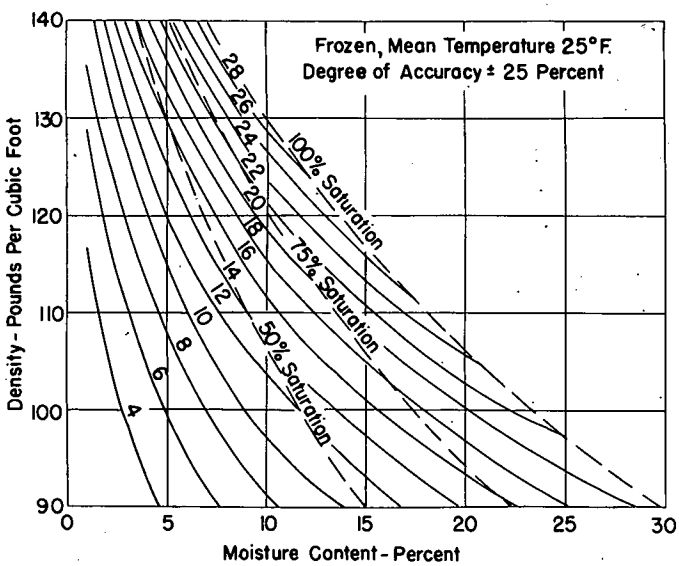


Figure 237. Diagram of Thermal Conductivity for Sandy Soils. (After Kersten)

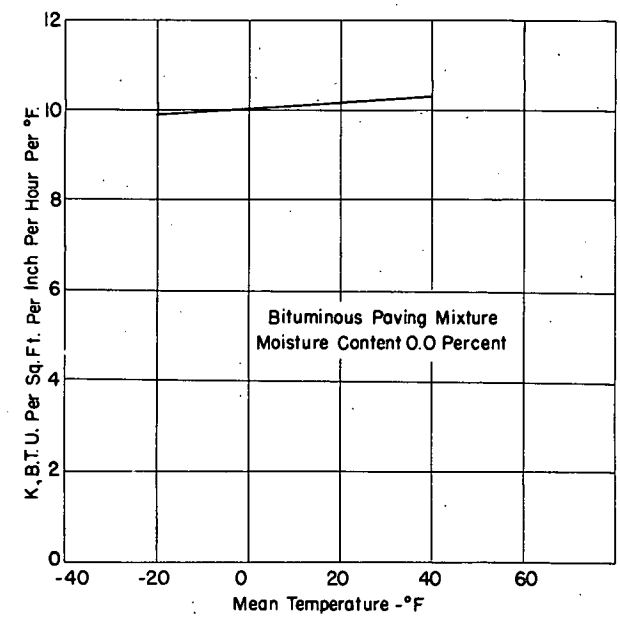


Figure 238. Variation of Thermal Conductivity with Mean Temperature of Test. (After Kersten)

The effect of such items as temperature, moisture content, and density may be predicted by a study of the above formulas.

Effect of temperature on diffusivity - An inspection of the formula (a) given above shows that a decrease in temperature will cause almost no change in diffusivity. For most soils, an inspection of formula (b) shows there would be a decrease. The value k changes very little with temperature at below freezing temperatures. Variations in specific heat are small, hence changes in diffusivity would be small.

However, when the temperature changes from above to below freezing the diffusivity may change markedly, especially at the higher soil moisture contents. For example, if a frozen soil has a conductivity 20 percent greater than unfrozen soil and the specific heat is only 0.5 the denominator of the equation becomes less. At a moisture content of 15 percent the diffusivity of a soil might increase 60 percent in being frozen, whereas, if the soil were dry, there would be no change.

Kersten showed that doubling the moisture content of a soil gave an increase of 30 to 40 percent in conductivity. Assuming a specific heat of 0.17 a change of moisture from 2.5 to 5 percent would increase the denominator of formula (b) by only 16 percent; a change of 5 to 10 percent moisture by 23 percent and from 10 to 20 percent by 37 percent. Thus at low or moderate moisture contents increasing the moisture content produces a marked increase in diffusivity. That is not true for high moisture contents.

Since the specific heat of ice is only about half that of water, an increase in moisture content markedly increases diffusivity of frozen soils.

Effect of density on diffusivity - Kersten showed that the thermal conductivity coefficient at 40 deg. increases about 2.8 percent for each lb. per cu. ft. increase in density for all moisture contents. Inspection of formulas (a) and (b) shows that increasing the density will cause a slight increase in diffusivity.

Dr. Terzaghi prepared for the Corps of Engineers /1949-23 a diagram showing the relationship between thermal conductivity for sand and clay under various conditions of density, porosity, and degree of saturation. The diagram is based on Kersten's work. It shows limiting values of thermal conductivity for frozen and unfrozen clay and sand. It may be seen that as the porosity approaches 100 percent the conductivity of a saturated soil approaches that of water and, if frozen, approaches the value for ice. The diagram is shown in Figure 238.

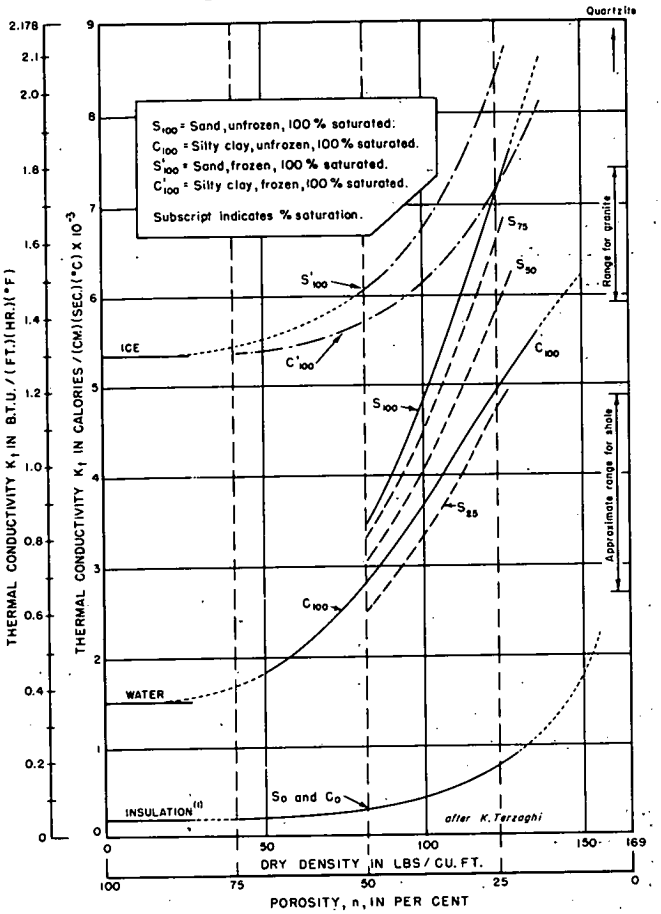
Effect of mineral composition and particle size and shape - Mineral composition affects diffusivity about as it effects conductivity since the specific heats are about the same. For example, the conductivities of trap rock and quartz may differ by 100 percent or more and hence their diffusivities may differ. Particle size and shape - may produce as much as 50 percent difference in conductivity in quartz samples and a like difference in diffusivity.

Additional references cited for information on the thermal properties of soils are: Keen /1931-1; Beskow /1935-1; Smith and Byers /1938-6; Smith /1939-11, /1942-8; Shannon /1945-1.

Thermal Properties of Minerals

The following tabulation assembled by the Corps of Engineers /1949-23 presents value of specific heat of various soils and minerals.

MATERIAL	SPECIFIC HEAT BTU per(LB)(Deg. F.)
Asbestos	0.195
Basalt	0.20
Calcapar	0.20
Cement	0.20
Chalk	0.214
Clay, dry	0.22
Cinders	0.18
Dolomite	0.222
Gneiss	0.18
Granite	0.192
Hornblende	0.195
Humus	0.44
Kaolin	0.224
Marble	0.21
Mica	0.206
Quartz	0.188
Salt, rock	0.21
Sand	0.195
Sandstone	0.22
Serpentine	0.25
Talc	0.209



(1) Such as dry asbestos or cotton.

Figure 239. Thermal Conductivity Versus Density, Porosity, and Saturation of Frozen and Unfrozen Soils. (After Corps of Engineers)

The following tabulations are from the Smithsonian tables

TABLE 75

SPECIFIC HEAT OF VARIOUS SOLIDS

Solid	Temperature C	Specific Heat
glass, normal thermometer 16	19-100	.1988
glass, French hard thermometer		.1869
glass crown	10-50	.161
glass flint	10-50	.117
ice	-80	.350
ice	-40	.434
ice	-20	.465
ice	0	.487
india rubber (para)	?-100	.481
mica	20	.10
paraffin	-20- + 3	.3768
paraffin	-19- + 20	.5251
paraffin	0-20	.6939
paraffin	35-40	.622
paraffin fluid	60-63	.712
woods	20	.327

TABLE 76

Thermal Conductivity of Organic Liquids

Substance	C*	k_t	Refer.	Substance	C*	k_t	Refer.	Substance	C*	k_t	Refer.
acetic acid	9-15	.03472	1	carbon disulphide	0	.03387	3	oils; olive	-	.03395	4
alcohols; methyl	11	.0352	2	chloroform	9-15	.03288	1	oils; castor	-	.03425	4
alcohols; ethyl	11	.0346	2	ether	9-15	.03303	1	toluene	0	.03349	3
alcohols; amyl	0	.03345	3	glycerine	25	.0368	2	vaseline	25	.0344	2
aniline	0	.03434	-	oils; petroleum	13	.03355	5	xylene	0	.03343	3
benzene	9-15	.03333	1	oils; turpentine	13	.03325	5				

References: (1) H. F. Weber; (2) Lees; (3) Goldschmidt; (4) Wachsmuth; (5) Graetz
*Temperature in degrees C.

TABLE 77

Thermal Conductivity of Various Insulators

k_t is the heat in gram-calories flowing in 1 sec. through a plate 1 cm. thick per sq. cm. for 1 deg. C. drop in temperature

Substance	Density	C	k_t	Substance	k_t	Authority
asbestos fiber	0.201	500	.00019	asbestos paper	0.00043	Lees-Chorlton
		100	.00016	blotting paper	.00015	
85% magnesia asbestos	.216	500	.00017	portland cement	.00071	
cotton	.021	100	.000111	cork, t, 0°C	.0007	Forbes
cotton	.101	100	.000071	chalk	.0020	H,L,D
Eiderdown	.0021	150	.00015	ebonite, t, 49°	.00037	
Eiderdown	.109	150	.000046	glass, mean	.002	Various
Lampblack, Cabot number 5	.193	100	.000074	ice	.0057	Neumann
		500	.000107	leather, cow-hide	.00042	Lees-Chorlton
quartz, mesh 200	1.05	500	.00024	leather, chamois	.00015	
Poplox, popped Na_2SiO_3	0.093	200	.000091	linen	.00021	
		500	.000160	silk	.000095	
Wool fibers	.015	100	.000118	caen stone, limestone	.0043	H,L,D
Wool fibers	.054	100	.000085	free stone, sandstone	.0021	
Wool fibers	.192	100	.000054			

Left-hand half of table from Randolph, Tr. Am. Electroch. Soc.XXI, p. 550, 1912; k_t (Randolph's values) is mean conductivity between given temperature and about 10 deg. C. Note effect of compression (density). The following are from Barratt Proc. Phys. Soc., London, 27, 81, 1914.

Substance	Density	k_t at 20 C	k_t at 100 C	Substance	Density	k_t at 20 C	k_t at 100 C
brick, fire	1.73	.00110	.00109	boxwood	0.90	.00036	.00041
carbon, gas	1.42	.0085	.0095	greenheart	1.08	.00112	.00110
ebonite	1.19	.00014	.00013	lignumvitae	1.16	.00060	.00072
fiber, red	1.29	.00112	.00119	mahogany	0.55	.00051	.00060
glass, soda	2.59	.00172	.00182	oak	0.65	.00058	.00061
silica, fused	2.17	.00237	.00255	whitewood	0.58	.00041	.00045

The following values are from unpublished data furnished by C. E. Skinner of the Westinghouse Co. Pittsburgh, Pa. They give the mean conductivity in gram-calories per sec. per cm.cube per °C when the mean temperature of the cube is that stated in the table. Resistance in thermal ohms (watts/inch^2/inch/°C) = $\frac{1}{10.6}$ Conductivity.

Substance	Grams per cu.cm.	Conductivity 100 C	200 C	300 C	400 C	500 C	Safe temp.
air-cell asbestos	0.232	0.00034	0.00043	0.00050	—	—	320
cork, ground	.168	.00015	.00019	-	—	—	180
diatomit	.326	.00028	.00032	.00037	0.00042	0.00046	600
infusorial earth, natural	.506	.00034	.00032	.00040	—	--	
infusorial earth'd pressed blocks	.321	.00030	.00029	.00033	.00036	—	400
magnesium carbonate	.450	.00023	.00025	.00025	—	--	300
vitribestos	.362	.00049	.00066	.00079	.00090	.00102	600

TABLE 78

Thermal Conductivity of Gases

The conductivity of gases, $k_t = \frac{1}{4}(9\gamma - 5)\mu C_v$, where γ is the ratio of the specific heats, C_p/C_v and μ is the viscosity coefficient (Jeans, Dynamical Theory of Gases, 1916). Theoretically k_t should be independent of the density and has been found to be so by Kundt and Warburg and others within a wide range of pressure below one atm. It increases with the temperature.

Gas	Temp C	k_t	Ref.	Gas	Temp C	k_t	Ref	Gas	Temp C.	k_t	Ref
air*	-191	0.0000180	1	CO_2	100	0.0000496	1	Hg	203	0.0000185	3
air	0	0.0000566	1	C_2H_4	0	0.0000395	2	N_2	-191	0.0000183	1
air	100	0.0000719	1	He	-193	0.000146	1	"	0	0.0000568	1
argon	-183	0.0000142	1	He	0	0.000344	4	"	100	0.0000718	1
argon	0	0.0000388	1	He	100	0.000398	1	O_2	-191	0.0000172	1
argon	100	0.0000509	1	H_2	-192	0.000133	1	"	0	0.0000570	1
CO	0	0.0000542	1	H_2	0	0.000416	4	"	100	0.0000743	1
CO_2	- 78	0.0000219	1	H_2	100	0.000499	1	NO	8	0.000046	2
CO_2	0	0.0000332	1	CH_4	0	0.0000720	4	N_2O	0	0.0000353	4

References: (1) Eucken, Phys. Z. 12, 1911; (2) Winkelmann, 1875; (3) Schwarze, 1903; (4) Weber, 1917

*Air: $k_0 = 5.22(10^{-5})$cal.cm.$^{-1}$ sec.$^{-1}$deg. C^{-1}; 5.74 at 22°; temp. coef. = .0029; Hercus-Laby, Pr. R. Soc. A_{95},100, 1919.

TABLE 79

Diffusivities

The diffusivity of a substance = $h^2 = k/cp$, where k is the conductivity for heat, c the specific heat and p the density (Kelvin). The values are mostly for room temperatures, about 18 deg. C.

Material	Diffusivity	Material	Diffusivity
aluminum	0.826	coal	0.002
antimony	0.139	concrete (cinder)	0.0032
bismuth	0.0678	concrete (stone)	0.0058
brass (yellow)	0.339	concrete (light slag)	0.006
cadmium	0.467	cork (ground)	0.0017
copper	1.133	ebonite	0.0010
gold	1.182	glass (ordinary)	0.0057
iron (wrought, also mild steel)	0.173	granite	0.0155
iron (cast, also 1% carbon steel)	0.121	ice	0.0112
lead	0.237	limestone	0.0092
magnesium	0.883	marble (white)	0.0090
mercury	0.0327	paraffin	0.00098
nickel	0.152	rock material (earth aver.)	0.0118
palladium	0.240	rock material (crystal rocks)	0.0064
platinum	0.243	sandstone	0.0133
silver	1.737	snow (fresh)	0.0033
tin	0.407	soil (clay or sand, slightly damp)	0.005
zinc	0.402	soil (very dry)	0.0031
air	0.179	water	0.0014
asbestos (loose)	0.0035	wood (pine, cross grain)	0.00068
brick (average fire)	0.0074	wood (pine with grain)	0.0023
brick (average building)	0.0050		

Taken from An Introduction to the Mathematical Theory of Heat Conduction, Ingersoll and Zobel, 1913

The Flow of Heat and Its Relation to Frost Action

The freezing of soils takes place when sufficient heat is removed from the soil to the air and, as freezing progresses, from the unfrozen soil to the frozen soil and thence to the air. The rate at which soil freezes depends on the difference between the air and soil temperatures, and on the thermal properties of the soil. It has been shown under Thermal Properties of Soils how the nature of the soil (composition, grain and shape, and grading) and the state of the soil mass (moisture content, density, and temperature) influence the flow of heat in soils. It has also been shown how the thermal constants may be used (assuming linear temperature gradients and a state of steady flow), to compute quantities of heat transfer and the heat quantity-time relationships for given sets of conditions.

The flow of heat from the air to the soil or from the soil to the air under natural conditions is seldom in the steady state except for periods of relatively short duration in the winter under cloudy skies and near constant day and night temperature. However during winter it often approaches sufficiently close to state of steady flow that the method can be used to make determinations. In the summertime and during most of the year in the tropics, heat flow is more apt to be a variable state of flow.

State of Steady Flow

Several writers, including Preston /1904-2, Patten /1909-1 and Baver /1940-2 use the case of a metal bar as a means of illustrating the flow of heat partly because it has been used in experiments to study heat flow. Assume a metal bar 100 cm. in length with one end in contact with a heat source which will produce a steady flow of heat with the hot end at 100 C. and the other at 0 C. (Figure 240). The steady flow of heat is indicated by the temperature gradient of line 1. Thus the same quantity of heat will pass across any plane parallel to the two surfaces per second after steady flow has been established.

The heat flow is directly proportional to the temperature difference ($\theta_1 - \theta_2$) of the faces. For layers of the same material but of different thickness whose faces have identical temperature difference ($\theta_1 - \theta_2$) the flow is inversely proportional to the thickness.

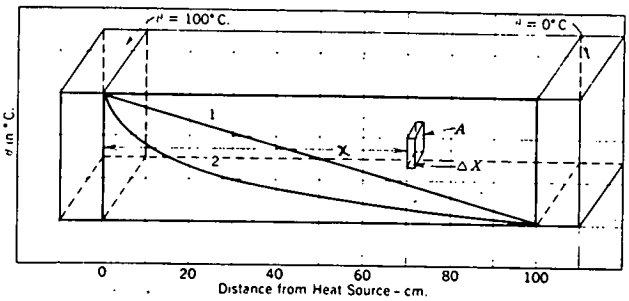


Figure 240
Diagram illustrating the flow of heat through soils

The quantity of heat Q which flows through an area A of the layer in a time t, is proportional to A and also to t. That is,

$$Q = k\,A\,t\,\frac{(\theta_1 - \theta_2)}{x}$$

where k is the coefficient of thermal conductivity.

Assume the bar to be soil; that soil is isotropic (having the same properties in all directions); and that the flow across the element from warm to cold is proportional to the temperature gradient.

If the area of element is A, its distance from the heated end x cm. its thickness Δx, and the temperature of the element, the flow of heat through A per unit of time is

$$-kA\,\frac{d\theta}{dx} \qquad (1)$$

where dx is the temperature gradient and k the coefficient of thermal conductivity. The temperature of the element decreases as x increases thus the negative sign.

The temperature of A at $x + \Delta x$ distance from the heat source is lower than at x and equals.

$$\theta - \frac{d\theta}{dx}\,\Delta x \qquad (2)$$

Multiplying $\frac{d\theta}{dx}$ the rate at which the temperature decreases as x increases, by Δx the distance over which the temperature decrease occurs, gives the total decrease in temperature from x to $x + \Delta x$. Thus the heat flow out through A at $x + \Delta x$ is

$$- kA \frac{d\theta}{dx} \left(\theta - \frac{d\theta}{dx} \Delta x \right) \qquad (3)$$

Less heat leaves the element A than enters it, therefore the temperature at $x + \Delta x$ is lower than at x. Thus subtracting (3) from (1) and simplifying leaves:

$$- kA \frac{d^2\theta}{dx^2} \Delta x \qquad (4)$$

which gives the relationships for the bar illustrated in Figure 240.

Before the heat flow reaches the steady state (line 1) the temperature gradient is indicated by curve 2 in which the amount of heat which leaves one side of element A is less than enters. In other words the element must first be heated before it will allow heat to pass through it. The quantity of heat necessary to raise the temperature of the element will depend on the heat capacity C (also called thermal capacity or volumetric heat capacity) of the volume of soil involved and will equal

$$AC \frac{d\theta}{dt} \qquad (5)$$

where C is the volumetric heat capacity (density X specific heat).

If no heat is lost transversely, then equations (4) and (5) are equal.

$$Ak \left(\frac{d^2\theta}{dx^2} \right) \Delta x = AC \left(\frac{d\theta}{dt} \right) \Delta x \qquad (6)$$

or

$$\frac{d\theta}{dt} = \frac{k}{C} \left(\frac{d^2\theta}{dx^2} \right)$$

The term $\frac{k}{C}$ is termed the diffusivity of the soil

Equation (6) is the fundamental equation of heat conduction in soil

(The explanation of the mathematical theory may be found in many articles and text books. Among them are Preston /1904-2, Patten /1909-1, Keen /1931-1, Baver /1940-2, and Ruckli /1943-2)

Variable State of Flow - Several writers, especially in fields related to agronomy, have presented data and theoretical analyses of the propogation of daily and seasonal waves of temperature in soil. The theoretical analyses have, in the main, been applications of Fourier's equation to compute temperatures in soil from known air temperatures and heat conductivity constants for the soil, or to compute diffusivity of the soil from temperature data observed in experiments. Ingersoll and Zoble /1913-4, Preston /1919-2, and Keen /1931-1 presented general solutions to Fourier's equation which satisfied the boundary conditions for simple harmonic variations of temperature applied to the soil surface, a condition which would exist if the temperature-time relation at the surface were a sine curve. Keen /1931-1 brought out that "this condition is approximately realized in practice, especially in the tropics."

Factors Affecting the Flow of Heat in Soils - The influence of soil moisture, density, and soil temperature as well as composition, grain size and grading of soils on specific heat, thermal conductivity, volumetric heat capacity and thermal diffusivity have been presented previously in this review.

Some General Effects of Wind, Rain and Snow on Soil Temperatures

Effect of Rainfall - The effect of moisture on soil temperature can be determined in terms of broad comparisons. For a given density, dry soil has poor thermal contact between soil grains and thus has a low conductivity and the temperature changes rapidly with depth. Wet soil has better thermal contact through moisture films and thus better conductivity (that is, a greater quantity of heat will pass through a given depth of wet soil than dry soil under comparable conditions), but the specific heat of the moist soil is so much greater that the actual change in temperature is small. In other words, it has a higher heat storage capacity.

The marked effect of soil moisture on heat flow and soil temperatures and the effect of temperature differences on moisture movements suggest that drought, rain, snow, and wind may

modify the heat flow either in the warming or the cooling period. From the point of view of frost action, the most significant factor is the effect of rain-water percolation through the soil both because it carries heat with it and because it affects the moisture content and thus the conductivity and diffusivity of the soil.

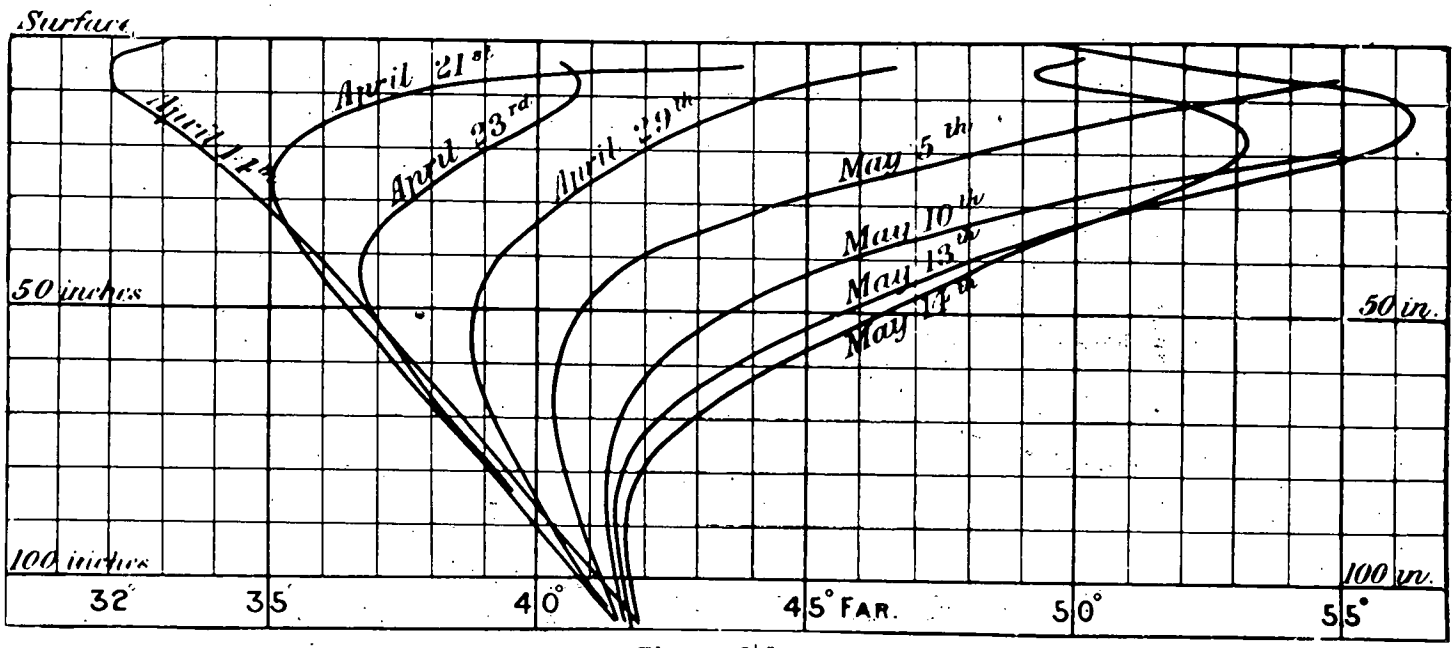


Figure 241
Curves Showing the Variation of Temperature with Depth
at Various Dates. (After Callendar)

Callendar /1895-1 and Callendar and McCleod /1896-1 made some observations which throw light on the effect of rainfall on the thermal diffusivity of the soil. They used a graphical method somewhat similar to that used by Patten /1909-1. Depth-temperature curves were drawn for days ending certain periods as indicated in Figure 241. The area between any two of these curves when multiplied by the volumetric heat capacity C gives the total quantity of heat absorbed per unit of area of stratum of the soil between the dates and depths for which the data represented by the curves were taken. The quantity of heat per unit area which passes by conduction into any stratum of the soil at a depth x at any time is equal to the product of the thermal conductivity k and the mean temperature gradient $\frac{d\theta}{dt}$ at the depth x throughout the interval of time considered. The quantity of heat absorbed by a stratum of soil between the depths x_1 and x_2 is equal to

$$k\left(\frac{d\theta}{dx_1} - \frac{d\theta}{dx_2}\right)$$

The mean value of $\frac{d\theta}{dx}$, the temperature gradient at any depth for any interval of time, can be found by drawing the tangents to the temperature-depth curves. By equating the two expressions for the quantity of heat absorbed, the value of the diffusivity $\frac{k}{C}$ is found. The process is equivalent to a graphic integration of the differential equation (6) which is the fundamental equation for heat flow.

Callendar and McCleod's results are summarized graphically in Figure 242. Figure 242 shows the change in the diffusivity for different periods during the year. It also shows the average daily rainfall for identical periods. During one 18-hr. period (not shown on the graph) when the rainfall was high and a large amount of cold water percolated through the soil in a short time the diffusivity reached the exceptionally high value of 0.323 on the basis of the computations which is 200 times greater than that due to conductivity alone. The low value of 0.0016 which was obtained during February when the ground surface was frozen and covered with snow indicates that diffusivity rate may be due to pure conduction. The diagram indicates a seasonal variation but perhaps more nearly a rainfall and, therefore, also a soil moisture variation.

Effect of Winds - Franklin /1920-2 and Keen /1931-1 found that the direct heating or cooling effects of warm or cold wind is slight, but that the indirect effect is greater due to the evaporation which takes place.

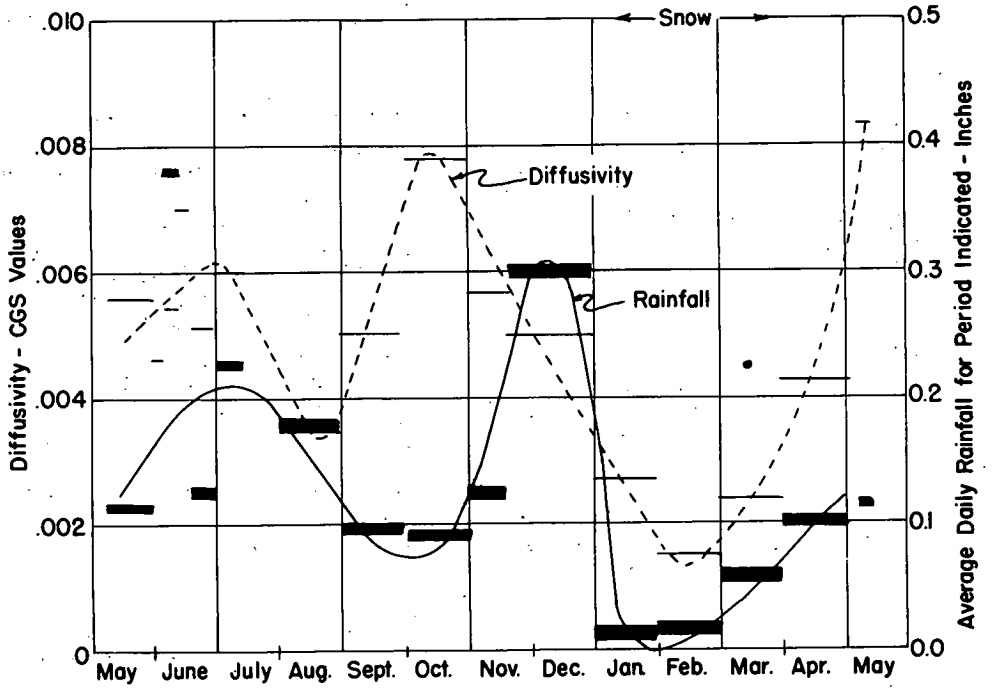


Figure 242
Diffusivity of Soil at Montreal Between 20 and 100 in. depth.
(After Callendar and McCleod)

Effect of Snow - The effect of snow as an insulating agent in preventing heat flow has been brought out earlier in the review.

BIBLIOGRAPHY

1765-1. Runeberg, E. O. "Anmarkningar om Några Forandringar pa Jord-ytan i Allmanhet och under de Kalla Climat i Synnerhet." Kungl. Vetensk. Akad. Handl., pp. 81-115, (see especially pp. 92-96), 1765.

1845-1. Forbes, James D. "Account of Some Experiments on the Temperature of the Earth at Different Depths, and in Different Soils, near Edinburgh." 1846. Transactions, Royal Society of Edinburgh, Vol. XVI, (1845-49).

1848-1. Middendorff, A. T. Von. "Sibirische Reise." Band 1, Theil 1, p. 169, 1848.

1850-1. Lecomte, J. "Observations on a Remarkable Exudation of Ice from the Stems of Vegetables, and on a Singular Protrusion of Ice Columns from Certain Kinds of Earth During Frosty Weather." Proceedings, American Association for the Advancement of Science, Vol. 3, pp. 20-34, 1850.

1853-1. Lavelle, J. Paris, C. R. Acad. Sci., 36, 493, 1853.

1854-1. Volger, G. H. Otto. "Uber die Volumveranderungen, Welche durch die Krystallisation hervorgerufen werden." Annalen der Physik und Chemie, Vierte Reiche (herausgegben zu Berlin Von J. C. Poggendorf), Band XCIII, pp. 232-237, 1854.

1855-1. Belcher, E. "The Last of the Arctic Voyages." Vol. 1, pp. 306-309. London: Lovell Reeve, 1855.

1860-1. Thompson, W. "On the Reduction of Underground Temperatures." Transactions, Royal Society of Edinburgh, Vol. 22, p. 410, 1860.

1860-2. Everett, J. D. "On a Method of Reducing Observations of Underground Temperatures." Transactions, Royal Society of Edinburgh, Vol. 22, p. 430, 1860.

1881-1. Kerr, W. C. "On the Action of Frost in the Arrangement of Superficial Earthy Material." American Journal of Science, Third Series, Vol. 21, pp. 345-358, May, 1881.

1888-1. Lehmann, O. Molekularphysik, 1, (342) Leipzig, 1888.

1888-2. Griffin, P. "Method of Paving Streets and Roads." U. S. Patent No. 382,153, May 1, 1888.

1888-3. Davison, C. "Note on the Movement of Scree-Material." Geological Society Quarterly Journal, (London), Vol. 44, pp. 232-238, 1888.

1889-1. Russell, I. C. "Surface Geology of Alaska." Geological Society of America Bulletin, Vol. 1, pp. 99-162, 1889.

1889-2. Davison, C. "On the Creeping of the Soil Cap Through the Action of Frost." Geological Magazine, New Ser., Decade 3, Vol. 6, pp. 255-261, 1889.

1889-3. Yachevskiy, . Über den ewigen Eisboden in Sibirien. Izvestiya Imperatorskago Russkago Geograficheskago Obshchestva. News of the Imperial Russian Geographical Society, (St. Petersburgh), Tom 25, 1889, p. 341. (In Russian).

1890-1. Gilbert, G. K. "Lake Bonneville." U. S. Geological Survey, Monograph No. 1, 1890, 438 pp.

1890-2. Ekholm, Nils. Observations faites au Cap Thordsen, Spitzberg, par l'expédition suédoise publiées par L'Académie Roy. des Sciences de Suède, Tom 1:3. Observations météorologiques - Expl. int. des regions polaires 1882-1883. Stockholm, 1890.

1892-1. Prestwich, J. "The Raised Beaches, and Head or Rubble - Drift, of the South of England." Geological Society Quarterly Journal, (London), Vol. 48, pp. 263, 322-328, 1892.

1893-1. Hazen, A. "Some Physical Properties of Sands and Gravels." Massachusetts State Board of Health, 24th Annual Report, (1892), p. 551, 1893.

1894-1. Geike, J. "The Great Ice Age." 3rd Edition, pp. 387-389, 722-723. London: Edward Stanford, 1894.

1894-2. Loughridge, . "Investigations in Soil Physics: The Capillary Rise of Water in Soils." California Exp. Sta., Annual Report, pp. 91-100, 1892-1894.

1894-3. Oldham, R. D. Notes on ground-ice. Natural Science, Vol. 4, No. 28, 1894, p. 450-52.

1895-1. Callendar, H. L. "Preliminary Results of Observations of Soil Temperatures with Electrical Resistance Thermometers Made at the McDonald Physics Building, McGill University, Montreal." Transactions, Royal Society of Canada, Second Ser., Vol. 1, Sec. 3, pp. 63-84, May, 1895.

1896-1. Callendar, H. L. and McCleod, C. H. "Observations of Soil Temperatures with Electrical Resistance Thermometers." Proceedings and Transactions, Royal Society of Canada, Second Ser., Vol. 2, Sec. 3, pp. 109-126, May, 1896.

1897-1. Holmquist, P. J. "Uber Mechanische Storingungen und Chemische Umsetzungen in dem Banderthon Schwedens." Bulletin, Geol. Inst., Uppsala, Vol. 3, No. 12, 1897.

1897-2. Briggs, L. J. "The Mechanics of Soil Moisture." U. S. Department of Agriculture, Bureau of Soils, Bulletin 10, 1897.

1899-1. King, F. H. "Principles and Conditions of the Movements of Ground Water." U. S. Geological Survey, Nineteenth Annual Report, 1897-1898, pp. 59-294, 1899.

1900-1. Buckley, E. R. "Ice Ramparts." Transactions, Wisconsin Academy of Sciences, Arts and Letters, Vol. 13, Pt. 1, pp. 141-157, 1900.

1902-1. Briggs, L. J. and Lapham, M. H. "The Capillary Movement of Water in Dry and Moist Soils." U. S. Dept. of Agriculture, Bureau of Soils, Bul. No. 19, pp. 19-30, 1902.

1902-2. Fries, Th. M. Några ord om rutmarken (Polygonboden) på Spetsbergen Och Beeren Island. Geologiska Föreningens i Stockholm Förhandlingar, Bd. 24, 1902, p. 370-72.

1902-3. Collier, Arthur J. "A Reconnaissance of the Northwestern Portion of Seward Peninsula, Alaska." U. S. Geological Survey, Professional Paper, #2, 1902.

1903-1. Roberts, M. "Notes on the Action of Frost on Soil." Journal of Geology, Vol. 11, No. 3, pp. 314-317, April-May, 1903.

1903-2. Wulff, Th. Observations botaniques faites au Spitzberg. (In Missions scientifiques pour la Mesure d'un arc de meridien au Spitzberg. Entreprises en 1899-1902 sous les auspices des gouvernements suédois et russe. Mission suédoise, Tom 2, Section 10, 1903, iv, 64 p., illus., tables, diagrs.)

1904-1. Tyrrell, J. B. "Crystosphenes or Buried Sheets of Ice in the Tundra of Northern America." Journal of Geology, Vol. 12, No. 3, p. 234, April-May, 1904.

1904-2. Preston, T. "The Theory of Heat." 2nd Ed., pp. 625-667. N. Y.: MacMillan and Co., 1904.

1904-3. Burr, W. H., Hering, R., and Freeman, J. R. "Report of the Commission on Additional Water Supply for the City of New York." pp. 603-613, 1904.

1904-4. Geer, Gerard de. Om skridjord och rutmark på Spetsbergen. Referat af meddelande. Geologiska Föreningens i Stockholm Förhandlingar, Bd. 26, 1904, p. 465-66.

1905-1. Becker, G. F. and Day, A. L. "The Linear Force of Growing Crystals." Proceedings, Washington Academy of Sciences, Vol. 7, pp. 283-288, July, 1905.

1905-2. Abbe, C. "Ice Columns in Gravelly Soil." U. S. Dept. of Agriculture, Weather Bureau, Monthly Weather Review, Vol. 33, No. 4, pp. 157-158, April, 1905.

1905-3. Atterberg, A. "Die Rationelle Klassifikation der Sande und Kiese." Chem. Zeitung, Vol. 29, Pt. 1, p. 196, 1905.

1906-1. Barnes, H. T. "Ice Formation." 1st Ed., N. Y.: John Wiley and Sons, 1906. 260 pp.

1906-2. Anderson, J. G. "Solifluction, A Component of Subaerial Denudation." Journal of Geology, Vol. 14, No. 2, pp. 91-112, Feb.-March, 1906.

1906-3. Hilgard, E. W. "Soils, their Formation, Properties, Composition, and Relation to Climate and Plant Growth in Humid and Arid Regions." N. Y.: MacMillan and Co., 1906.

1906-4. Campbell, M. R. "Nautral Mounds." Journal of Geology, Vol. 14, Nov.-Dec., 1906, p. 708-717.

1907-1. Maddren, A. G. "Smithsonian Exploration in Alaska in 1904." Smithsonian Miscellaneous Collections, Vol. 49, No. 1741, pp. 42-44, 1907.

1907-2. Buckingham, E. "Studies on the Movement of Soil Moisture." U. S. Dept. of Agriculture, Bureau of Soils, Bul. No. 38, 1907, 61 pp.

1907-3. Hesselman, H. "Studier över Skogsvaxt a Mossar." Skogsvardfören, Tidskrift, 1907.

1907-4. Kuhle, W. "The Annual Fluctuations of Soil Temperature in Different Climates." Diss. Univ. Berlin. p. 68.

1907-5. Collier, Arthur J., Hass, Frank L., Smith, Phillip S., and Brooks, Alfred H. "The Gold Placers of Parts of Seward Peninsula, Alaska, Including the Nome, Council, Kougarok, Port Clarence, and Goodhope Precincts." U. S. Geological Survey, Bulletin 328, 1907.

1909-1. Patten, H. E. "Heat Transference in Soils," U. S. Dept. of Agriculture, Bureau of Soils, Bul. No. 59, 1909. 54 pp.

1909-2. Resvoll-Dieset, Hanna. Lidt om Spitsbergens plantevekst. Det Norske Geografiske Selskabs Aarbog, 20, 1908-09, (pub) 1909, p. 9-17, illus.

1909-3. Resvoll-Holmsen, Hanna. Om jordbundsstrukturer i polarlandene og planternes forhold til dem. Nyt Mag. for Naturvidenskaberne, Bd. 47, Hefte 4, 1909, p. 289-96.

1909-4. Holmsen, Gunnar. Beretning om en geologisk ekspedition til Spitsbergen 1909. Bergens Museums Aarbog, Nr. 9, 1911, 76 p.

1910-1. Moore, W. L. "Descriptive Meteorology." Pp. 107-114. N. Y.: D. Appleton and Co., 1910. 344 pp.

1910-2. Stefansson, V. "Underground Ice in Northern Alaska." Bulletin of the Geographical Society, Vol. 42, No. 5, pp. 337-345, May, 1910.

1910-3. Rabot, Charles. Découverte de gisements de glace fossile à Nouvelle Zemble et au Spitzberg. La Géographie, 1910, p. 282-.

1910-4. Hobbs, William H. Soil stripes in cold humid regions, and a kindred phenomenon. Report of the Michigan Academy of Science, (No. 12), 1910, p. 51-53, illus.

1910-5. Högbom, Bertil. Einige Illustrationen zu den geologischen Wirkungen des Frostes auf Spitzbergen. Bulletin of the Geological Institution of the University of Upsala, Vol. 9, 1908-09, (pub) 1910, p. 41-59.

1910-6. Rusanov, V. A. Sur les lambeaux de glace

fossile en Nouvelle Zemble. Comptes Rendus Hebdomadaires des Séances de l'Académie des Sciences (Paris), Tom. 150, 1910.

1910-7. Smith, Philip S. "Geology and Mineral Resources of the Solomon and Casadepage Precincts Seward Peninsula, Alaska." U. S. Geological Survey, Bulletin 433, 1910.

1911-1. Watson, W. "Change of Volume During Fusion, and Latent Heat of Fusion." A Textbook of Physics, Paragraphs 209 and 211, pp. 236-239. N. Y.: Longmans, Green and Co., 1911.

1911-2. Ule, W. Glazialer Karree- Oder Polygonboden. Zeitschrift der Gesellschaft für Erdkunde zu Berlin, 1911, p. 262-63.

1911-3. Drygalski, Erich von. Spitzbergens Landformen und ihre Vereisung. Abhandlungen der Kgl. Bayerischen Akademie der Wissenschaften. Math.-Phys. Kl., Bd. 25, Abh. 7, 1911, 61 p.

1911-4. Drygalski, Erich von. Spitabergens Vereisung. (In Miethe, A. and Hergesell, H. Mit Zeppelin nach Spitzbergen. Bilder von der Studienreise der deutschen arktischen Zeppelin-Expedition, 1911, p. 177-84, illus.)

1912-1. Bergström, E. En märklig form av rutmark från barrskogsregion i Lappland. Geologiska Foreningens i Stockholm, Förhandlingar, Bd. 34, 1912, p. 335-42.

1912-2. Holmsen, Gunnar. Spitsbergens jordbundsis og de bidrag dens undersøkelse har kunnet gi til forstaaelsen av de i arktiske land optraedende varige isleier i jorden. Det Norske Geografiske Selskaps Aarbok, 24, 1912-13, (pub) 1913, p. 1-150, illus., tables, diagrs.

1912-3. Seest, B. W. Ueber den ewigen Eisboden und die Wasserversorgung der Umgegend der Station Sjelowo der Armurbahn. Meteorologicheskiy Vestnik. Meteorological Messenger, 1912.

1912-4. Sapper, K. Über Fliesserde und Strukturboden auf Spitzbergen Zeitschrift der Gesellschaft für Erdkunde zu Berlin, Nr. 4, 1912, p. 259-70.

1912-5. Meinardus, Wilhelm. Beobachtungen über Detritussortierung und Strukturboden auf Spitzbergen. Zeitschrift der Gesellschaft für Erdkunde zu Berlin, Nr. 4, 1912, p. 250-59.

1912-6. Penck, A. Diskussion über Bodenfluss. Der Polygonboden in Spitzbergen. Zeitshcrift der Gesellschaft für Erdkunde zu Berlin, Nr. 4, 1912, p. 244-46.

1912-7. Miethe, A. Diskussion über Bodenfluss. Über Karreebodenformen auf Spitzbergen. Zeitschrift der Gesellschaft für Erdkunde zu Berlin, Nr. 4, 1912, p. 241-44.

1912-8. Nikiforoff, Constantin. The dynamic processes in the soil of ever-frozen land. Pedology, Vol. 2, 1912, p. 50-74.

1912-9. Moffit, Fred H. "Headwater Regions of Gulkana and Susitna Rivers, Alaska." U. S. Geological Survey, Bulletin 498, 1912.

1913-1. Bouyoucos, G. J. "An Investigation of Soil Temperatures and Some of the Factors Influencing It." Michigan Agricultural College, Exp. Sta., Technical Bul. No. 17, Feb., 1913. 196 pp.

1913-2. Hobbs, W. H. "Soil Flow." Bulletin of the American Geographical Society, Vol. 45, No. 4, pp. 281-284, April, 1913.

1913-3. Moffit, F. H. "Geology of the Nome and Grand Central Quadrangles-Alaska." U. S. Geological Survey, Bul. No. 553, pp. 53-54, 1913.

1913-4. Ingersoll, L. R., and Zobel, O. J. "An Introduction to the Mathematical Theory of Heat Conduction." Boston: Ginn and Co., 1913.

1913-5. Holmsen, Gunnar. Om jordlags langsomme glidning, solifuktien Det Norske Geografiske Selskaps Aarbok, 25, 1913-14, (pub) 1915, p. 25-41, illus. diagr.

1913-6. Frödin, J. Geografiska studier i. St. Lule älvs källområde. Sveriges Geologiska Undersokning, Avhandlingar och Uppsatser, Serie C, No. 257, Arsbok 7, No. 4, 1913.

1913-7. Sapper, K. Erdfliessen und Strukturboden in polaren und sub-polaren Gebieten. Geologische Rundschau, Bd. 4, Heft 2, 1913, p. 103-15.

1913-8. Smith, Phillip S. "The Noatak-Kobuk Region, Alaska." U. S. Geological Survey, Bulletin 536, 1913.

1914-1. Johansson, Simon. "Die Festigkeit der Bodenarten bei Verschiedenem Wassergehalt." Sveriges Geologiska Undersokning, Ser. C., No. 256, Yearbook 7, 1913.

1914-2. Högbom, B. "Uber die Geologische Bedeutung Des Frostes." Bulletin, Geol. Inst., Upsala, Vol. 12, 1913-1914.

1914-3. Thoroddsen, Th. An account of the physical geography of Iceland with special reference to the plant life. In Kolderup-Rosenvinge, and Warming, The botany of Iceland, Vol. 1, 1914, p. 187-343. (See p. 257-65).

1914-4. Leiviska, I. Fossiles Eis in einem fluvioglazialen Hügel unweit von Åbo. Zeitschrift fur Gletscherkunde, für Eiszeitforschung und Geschichte des Klimas, Bd. 8, 1914, p. 209-25.

1915-1. Bouyoucos, G. J. "Effect of Temperature on Movement of Water Vapor and Capillary Moisture in Soils." Journal of Agricultural Research, Vol. 5, No. 4, pp. 141-172, Oct. 25, 1915.

1915-2. Bouyoucos, G. J. and McCool, M. M. "The Freezing Point Method as a New Means of Measuring the Concentration of the Soil solution Directly in the Soil." Michigan Agricultural College, Exp. Sta., Technical Bulletin No. 24, Dec., 1915. 44 pp.

1915-3. Tarr, R. S. and Engelin, O. D. "Experimental Studies of Ice with Reference to Glacier Structure and Motion." Zeitschrift fur Gletscherkunde,

Band 9, Heft 2, p. 91, 1915.

1915-4. Leffingwell, E. de K. "Ground Ice Wedges the Dominant Form of Ground Ice on the North Coast of Alaska." Journal of Geology, Vol. 23, pp. 635-654, 1915.

1915-5. Lauchli, E. "Ground Temperature Variations Decrease with Depth." Engineering News, Vol. 74, No. 11, pp. 510-511, Sept. 9, 1915.

1915-6. Hamberg, Axel. Zur Kenntnis der Vorgänge im Erdboden beim Gefrieren und Auftauen sowie Bemerkungen über die erste Kristallisation des Eises im Wasser. Geologiska Föreningens i Stockholm Förhandlingar, Bd. 37, Hafte 5, 1915, p. 583-619.

1915-7. Leather, J. W. "Soil Temperatures." Mem. of Dept. of Agr. Ind. Chem. Series, Vol. 4, pp. 18-84.

1915-8. Bouyoucos, G. J. "Effect of Temperature on Some of the Most Important Physical Processes in Soils." Mich. Agr. Exp. Sta., Tech. Bulletin #22, pp. 1-63.

1915-9. Capps, S. R. "The Willow Creek District." U. S. Geological Survey, Bulletin 607, 1915.

1916-1. Rambaut, A. A. "Results of Meteorological Observations." Also Underground Temperatures in Twelve Years, 1898-1910. Radcliffe Observations, Radcliffe Observatory, Oxford, Vol. 51, pp. 103-215, London: Oxford University Press, 1916.

1916-2. Eakin, H. M. "The Yukon-Koyukuk Region, Alaska." U. S. Geological Survey, Bul. No. 631, pp. 76-82, 1916.

1916-3. Bouyoucos, G. J. "Soil Temperature." Michigan Agricultural College, Exp. Sta., Technical Bul. No. 26, Jan., 1916. 133 pp.

1916-4. Taber, S. "The Growth of Crystals under External Pressure." American Journal of Science, Fourth Ser., Vol. 41, No. 246, pp. 532-556, June, 1916.

1916-5. Bouyoucos, G. J. and McCool, M. M. "Further Studies on the Freezing Point Lowering of Soils." Michigan Agricultural College, Exp. Sta., Technical Bul. No. 31, Nov., 1916. 51 pp.

1916-6. Johansson, S. "Agrogeologisk under sökning av Ultuna Egendom." Sveriges Geologiska Undersokning, Ser. C, No. 271, Yearbook 9, pp. 73-74, 1915.

1916-7. Douville, Robert. Sols polygonaux ou reticules. Géographie, Tom. 31, No. 4, 1916-17, p. 241-51.

1917-1. Bouyoucos, G. J. "Measurement of the Inactive, or Unfree Moisture in the Soil by Means of the Dilatometer Method." Journal of Agricultural Research, Vol. 8, No. 6, pp. 195-217, Feb. 5, 1917.

1917-2. Taber, S. "Pressure Phenomena Accompanying the Growth of Crystals." Proceedings, National Academy of Sciences, Vol. 3, No. 4, pp. 297-302, April, 1917.

1917-3. Gilkey, H. J. "Freezing Ground Acts Like Hydraulic Jack." Engineering News-Record, Vol. 79, No. 8, pp. 360-361, Aug. 23, 1917.

1917-4. Bouyoucos, G. J. "Classification and Measurement of the Different Forms of Water in Soil by Means of the Dilatometer Method." Michigan Agricultural College, Exp. Sta., Technical Bul. No. 36, Sept., 1917. 48 pp.

1917-5. Arnold, F. P. "Frost Breaks in Macadam Roads Due to Inadequate Drainage." Engineering News-Record, Vol. 79, No. 20, p. 933, Nov. 15, 1917.

1917-6. McCool, M. M. and Wheeting, L. C. "Movement of Soluble Salts Through Soils." Journal of Agricultural Research, Vol. 11, No. 11, pp. 531-547, Dec. 10, 1917.

1917-7. Stoll, O. Zur Entstehung des Strukturbodens in polaren Gebieten. Veröffentlichungen des Deutschen Observatoriums Ebeltofthafen, Spitzbergen, Heft, 7, 1917, 14 p.

1917-8. Kindle, E. M. Some factors affecting the development of mud cracks. Journal of Geology, Vol. 25, 1917, p. 134-49.

1918-1. Taber, S. "Ice Forming in Clay Soils will Lift Surface Weights." Engineering News-Record, Vol. 80, No. 6, pp. 262-263, Feb. 7, 1918.

1918-2. Harrison, J. L. "First Frost is Never Responsible for Cracked Concrete Roadways." Engineering News-Record, Vol. 80, No. 9, pp. 418-420, Feb. 28, 1918.

1918-3. Wilson, J. "Frost Action on Rigid Pavements." Engineering News-Record, Vol. 80, No. 13, p. 626, March 28, 1918.

1918-4. Wyckoff, L. B. "Some Observations on Effect of Frost in Raising Weight." Engineering News-Record, Vol. 80, No. 13, pp. 627-628, March 28, 1918.

1918-5. Norton, C. D. "Water Expansion in Ground Cause of Heaving in Winter." Engineering News-Record, Vol. 80, No. 22, p. 1058, May 30, 1918.

1918-6. Lowell, J. W. "An Investigation of the Vertical Movements of Concrete Pavements." Proceedings, American Concrete Institute, Vol. 14, pp. 366-373, 1918.

1918-7. Engineering News-Record. "Impervious Bituminous Wall Suggested to Present Seepage Under Paving." Engineering News-Record, Vol. 81, No. 5, pp. 226-228, Aug. 1, 1918.

1918-8. Ekblaw, W. E. "The Importance of Nivation as an Erosive Factor and of Soil Flow as a Transporting Agency in Northern Greenland." Proceedings, National Academy of Sciences, Vol. 4, No. 9, pp. 288-293, Sept. 15, 1918.

1918-9. Frödin, J. Über das Verhältnis zwischen Vegetation und Erdfliessen in den alpinen Regionen des schwedischen Lappland. Lunds Universitetets Arsskrift, N. F., Avd. 2, Bd. 14:2, Nr. 24, 1918, 32 pp.

1918-10. Taber, S. "Surface Heaving Caused by Segregation of Water Forming Ice Crystals." Engineering News-Record, Vol. 81, No. 15, pp. 683-684, Oct. 10, 1918.

1918-11. Palladin, V. I. "Plant Physiology." English Edition. Edited by B. E. Livingston, Philadelphia, Pa., p. 134, 1918.

1918-12. Eakin, H. M. "The Cosna-Nowitna Region, Alaska." U. S. Geology Survey Bulletin, 667, pp. 1-54, 1918.

1918-13. Harrington, George L. "The Anvik-Andreafski Region, Alaska." U. S. Geological Survey, Bulletin 683, 1918.

1919-1. Leffingwell, E. DeK. "The Canning River Region - Northern Alaska." U. S. Geological Survey, Professional Paper No. 109, pp. 179-243, 1919.

1919-2. Preston, T. "Heat." 3rd Edition. London: MacMillan and Co., 1919.

1919-3. Blanchard, A. H. "American Highway Engineers Handbook." Pp. 450, 451, 923. N. Y.: John Wiley and Sons, 1919.

1919-4. Blanck, E. Ein Beitrag zur Kenntnis arktischer Böden insbesondere Spitzbergens. Chemie der Erde, Bd. 1, 1919, p. 421-76.

1919-5. Macdougal, D. T. "Soil Temperature Survey of U. S. and Canada." Carnegie Inst., Washington. Year Book, Vol. 18, pp. 57-102.

1919-6. Capps, Stephen R. "The Kantishna Region, Alaska." U. S. Geological Survey, Bulletin 687, 1919.

1920-1. Bouyoucos, G. J. "Degree of Temperature to which Soils can be Cooled Without Freezing." Journal of Agricultural Research, Vol. 20, No. 4, pp. 267-269, Nov. 15, 1920.

1920-2. Franklin, T. B. "The Effect of Weather Changes on Soil Temperatures." Proceedings of Royal Society, (Edin.) 40 pp. 56-79, 1919-20.

1920-3. Werenskiold, W. Spitsbergens fysiske geografi. Naturen, Aarg. 44, 1920, p. 209-42.

1920-4. Nansen, Fridtjof. En ferd til Spitsbergen. Kristiania, 1920, 283 p.

1920-5. Allen, J. R. "Theory of Heat Losses From Pipes Buried in the Ground." American Society of Heating and Ventilating Engineers, Vol. 26, P. 335.

1921-1. Bridgeman, P. W. "Effect of Pressure on the Freezing Point of Water." Smithsonian Physical Tables, Smithsonian Miscellaneous Collections, Vol. 71, p. 200, 1921.

1921-2. Smithsonian Physical Tables. "Density and Volume of Water." Smithsonian Physical Tables, Smithsonian Miscellaneous Collections, Vol. 71, p. 120, 1921.

1921-3. Bouyoucos, G. J. "A New Classification of Soil Moisture." Soil Science, Vol. 11, No. 1, pp. 33-47, Jan., 1921.

1921-4. Keen, B. A. and Russell, E. J. "The Factors Determining Soil Temperature." Journal of Agricultural Science, Vol. 11, Pt. 3, pp. 211-239, July, 1921.

1921-5. Bouyoucos, G. J. "The Concentration of the Soil Solution Around the Soil Particles." Soil Science, Vol. 11, No. , pp. 131-138, 1921.

1921-6. Köppen, W. Bodeneis und Eisboden. Meteorologische Zeitschrift, 1921.

1921-7. Waldbaur, H. Schuttglättung und Steinstrome im Oberengadin. Petermanns Geographische Mitteilungen Bd. 67, 1921, p. 195.

1921-8. Trevor-Battye, Aubyn. A Theory of the origin of surface-polygons in polar lands. Geographical Journal, Vol. 58, No. 4, 1921, p. 306-09, illus. (Includes note by H. W. Feilden, p. 308-09).

1922-1. Shanklin, G. B. "The Effects of Moisture on the Thermal Conductivity of Soils." Transactions, American Institute of Electrical Engineers, Vol. 41, pp. 94-101, 1922.

1922-2. Kincer, J. B. "Precipitation and Humidity." Atlas of American Agriculture, Pt. 2 (Climate), Sec. A (Precipitation and Humidity), Advance Sheets No. 5, March 15, 1922.

1922-3. Bouyoucos, G. J. and McCool, M. M. "A Study of the Causes of Frost Occurrence in Muck Soils." Soil Science, Vol. 14, No. 5, pp. 383-389, Nov. 1922.

1922-4. Glazebrook, R. "A Dictionary of Applied Physics." Vol. 1 - Mechanics, Engineering, Heat. Pp. 467-470. London: MacMillan and Co., 1922.

1922-5. Illinois Division of Highways. "Frost Action and Uniformity of Subgrade Support." An Unpublished Report of Studies of Frost Action on the Bates Experimental Test Road. 1922.

1922-6. Illinois Division of Highways. "Slab and Subgrade Movement Due to Frost Action." An Unpublished Report of Studies of Frost Action on the Bates Experimental Test Road. April 1922.

1922-7. Ule, W. Polygonaler Strukturboden auf dem Hochland von Norwegan. Petermanns Geographische Mitteilungen, Bd. 68, 1922, p. 247-48.

1922-8. Werenskiold, W. Frozen earth in Spitsbergen. Geofysiske Publikationer, Vol. 2, No. 10, 1922, 10 p., diagrs.

1922-9. Werenskiold, W. "Telaskott." On upheaval of the ground by freezing. Naturen, Aarg. 46, 1922, p. 54-56.

1922-10. Nansen, Fridtjof. Spitzbergen. Dritte Auflage. Leipzig, F. A. Brockhaus, 1922, 327 p. See pp. 107-21.

1923-1. Meinzer, O. E. "The Occurrence of Ground Water in the United States." U. S. Geological

vey, Water Supply Paper, No. 489, pp. 30-63, 1923.

1923-2. McColloch, J. W. and Hayes, W. P. "Soil Temperature and Its Influence on White Grub Activities." Ecology, Vol. 4, No. 1, pp. 29-36, Jan., 1923.

1923-3. Bouyoucos, G. J. "Movement of Soil Moisture from Small Capillaries to the Large Capillaries of the Soil Upon Freezing." Journal of Agricultural Research, Vol. 24, No. 5, pp. 427-431, May 5, 1923.

1923-4. Smirnoff, A. "Soil Temperatures and Cable Rating." Electrical World, Vol. 82, No. 9, pp. 438-439, Sept. 1, 1923.

1923-5. Illinois Division of Highways. "Unpublished Data from the Bates Experimental Test Road." 1923.

1923-6. McCready, L. D. "Action of Frost in Heaving Concrete Piers." Engineering News-Record, Vol. 91, No. 9, p. 360, August 30, 1923.

1924-1. Hawkes, L. "Frost Action in Superficial Deposits, Iceland." Geological Magazine (London), Vol. 61, No. 725, pp. 509-513, Nov., 1924.

1924-2. Ingersoll, L. R. and Koepp, O. A. "Thermal Diffusivity and Conductivity of Some Soil Materials." Physical Review, Vol. 24, No. 1, pp. 92-93. July, 1924.

1924-3. Illinois Division of Highways. "Investigation of Slab and Subgrade Movements Under Low Temperatures." An Unpublished Report of Studies of Frost Action in Highways in Three Northern and Central Illinois Districts. July, 1924.

1924-4. Older, C. "Highway Research in Illinois." Proceedings, American Society of Civil Engineers, Vol. [illegible], No. 2, pp. 175-217, Feb., 1924.

1924-5. Huxley, J. S. and Odell, N. E. "Notes on Surface Markings in Spitzbergen." Geog. Journal, Vol. 63, pp. 207-229, 1924.

1924-6. Visher, S. S. "Climatic Laws." N. Y.: John Wiley and Sons, 1924, 96 pp.

1924-7. Pohle, R. von. Frostboden in Asien und Europa. Petermanns Geographische Mitteilungen, Bd. 70, 1924, p. 86-88.

1924-8. Transactions. "Soil Temperatures in Clays of the Lake Agassiz Basin." Royal Society of Canada, Vol. 4, pp. 9-30.

1924-9. Martie, J. B. Jr., and Harrington, G. L. "The Ruby-Kuskokwim Region, Alaska." U. S. Geological Survey, Bulletin 754, 1924.

1925-1. Wintermeyer, A. M. "Percentage of Water Freezable in Soils." Public Roads, Vol. 5, No. 12, pp. 5-8, Feb. 1925.

1925-2. Terzaghi, C. "Principles of Soil Mechanics." Engineering News-Record, Vol. 95, pp. 742-746, 796-800, 1925.

1925-3. Kessler, P. Das eiszeitliche Klima und Seine geologischen wirkungen im nicht vereisten gebiet. Stuttgard, 210 pages, 1925.

1925-4. Pohle, R. von. Frostboden in Asien und Europa. Petermanns Geographische Mitteilungen, Bd. 71, 1925, p. 167-169.

1925-5. Low, A. R. Instability of viscous fluid motion. Nature, Vol. 115, 1925, p. 299-300.

1925-6. Huxley, Julian S. Les "sols polygonaux" et l'évolution des phénomènes de dénudation dans les pays arctiques. Annales de Géographie, 34 An., 1925, p. 60- (Reprint occupies 3 p.)

1925-7. Grigor'Yev, A. A. Zur Geomorphologie der Bolschesemelskaja Tundra. Zeitschrift der Gesellschaft für Erdkunde zu Berlin, 1925, pp. 117-26.

1926-1. Highland, S. G. "Study of Year Round Soil Temperatures." Journal, American Water Works Association, Vol. 16, No. 3, pp. 342-354, Sept., 1926.

1926-2. Smith, A. "A Contribution to the Study of Interrelations Between the Temperature of the Soil and of the Atmosphere and a New Type of Thermometer for such Study." Soil Science, Vol. 22, No. 6, pp. 447-457, Dec., 1926.

1926-3. Kokkonen, P. "Beobachtungen uber die Struktur des Bodenfrostes." Acta Forestalia Fennica, No. 30, Helsingfors, 1926.

1926-4. Gripp, Karl. Über Frost und Strukturboden auf Spitzbergen. Zeitschrift der Gesellschaft für Erdkunde zu Berlin, 1926, p. 351-54.

1926-5. Högbom, Bertil. Beobachtungen aus Nordschweden über den Frost als geologischer Faktor. Bulletin Geological Institution of the University of Upsala, Vol. 20, 1926, p. 243-80, illus., maps, diagrs.

1926-6. Tsi-Tung Li. "Soil Temperature as Influenced by Forest Cover." Yale University School of Forestry, Bulletin No. 18.

1927-1. Harger, W. G., and Bonney, E. A. "Handbook for Highway Engineers." Pp. 367-368. N. Y.: McGraw-Hill Book Co., 1927.

1927-2. Dillman, G. C. "Causes and Cure of Frost Boils." Proceedings, Highway Research Board, Vol. 7, pp. 291-295, 1927.

1927-3. Lebedeff, A. F. "Methods of Determining the Maximum Molecular Moisture Holding Capacity of Soils." Proceedings, First International Congress, Soil Science, Vol. 1, pp. 551-563, 1927.

1927-4. Report of Committee on Maintenance. W. H. Root, Chairman, Pp. 269-297, Vol. 7, Proceedings, Highway Research Board, (1927). Discussion, Pp. 298-301.

1927-5. Department of Highways, Ohio. Displacement of Pavement by Frost." Proceedings, Highway Research Board, Vol. 6, p. 157, 1927.

1927-6. Elton, Charles S. The nature and origin of soil-polygons in Spitsbergen. Quarterly Journal Geological Society, Vol. 83, Part 1, 1927, p. 163-94, illus., diagrs.

1927-7. Shostakovich, V. B. Der ewig gefrorene Boden Sibiriens. Zeitschrift der Gesellschaft für Erdkunde zu Berlin, 1927, p. 394-427.

1927-8. Gripp, Karl. Bodenfrost und Erdfliessen in Spitzbergen. Jahresbericht der Schlesischen Gesellschaft für vaterlandische Kultur, 99. 1927, p. 42.

1927-9. Gripp, Karl. Beiträge zur Geologie von Spitzbergen. Abhandlungen des Naturwissenschaftlichen Vereins zu Hamburg, Bd. 21, Heft 3, 1927, p. 1-66, illus., diagrs.

1927-10. Thorne, Gerald. "Report of Soil Temperatures at Various Depths 1923-25 at Salem, Utah." Journal of Agr. Research, Vol. 34, p. 267.

1928-1. Barnes, H. T. "Ice Engineering." Montreal. Renouf Publishing Co., 1928. 364 pp.

1928-2. White, A. C. "Frost Boils and Their Elimination." Public Works Magazine, Vol. 59, No. 1, pp. 10-11, Jan., 1928.

1928-3. Harrington, E. L. "Soil Temperatures in Saskatchewan." Soil Science, Vol. 25, No. 3, pp. 183-194, March, 1928.

1928-4. Engineering News-Record Survey. "Heavy Frost Damage to Roads Follows Open Winter." Engineering News-Record, Vol. 100, No. 17, pp. 668-669, April 26, 1928.

1928-5. Bouyoucos, G. J. and McCool, M. M. "The Correct Explanation for the Heaving of Soils, Plants and Pavements." Journal of the American Society of Agronomy, Vol. 20, No. 5, pp. 480-491, May, 1928.

1928-6. Henton, J. T. "Modern Subdrainage Proves Successful Cure for Frost Boils." Highway Magazine, Vol. 19, No. 12, pp. 316-318, Dec. 1928.

1928-7. Nikiforoff, C. "The Perpetually Frozen Subsoil of Siberia." Soil Science, Vol. 26, No. 1, pp. 61-82, July, 1928.

1928-8. Kincer, J. B. "Temperature, Sunshine and Wind." Atlas of American Agriculture, Pt. 2 (Climate), Sec. B (Sunshine and Wind), Advance Sheets No. 7, Nov., 1928.

1928-9. Beskow, G. "Snoplogningens Betydelse for Vagarnas Tjalforhallanden." Svenska Vagforeningens Tidskrift, H. 1, 1928.

1928-10. Zunker, F. "Uber das Schwinden und Quellen der Boden und ein neues Bodenuntersuchungaverfahren." Der Kulturtechniker, pp. 527-554, 1928.

1928-11. Nordenskjold, O. "The Geography of the Polar Regions." American Geographical Society, Spec. Pub. No. 8, pp. 52-57, 1928.

1928-12. Blanck, E., and Rieser, A. Mit einem Beitrag von H. Mortensen. Die wissenschaftlichen Ergebnisse einer bodenkundlichen Forschungsreise nach Spitsbergen im Sommer 1926. Chemie der Erde, Bd. 3, 1928, p. 588-698.

1928-13. Bjørlykke, K. O. Bodenprofile aus Svalbard. Soil Research (Bodenkundliche Forschungen). Beiheft zu den Mitteilungen der Internationalen Bodenkundlichen Gesellschaft, Bodenkundliche Forschungen, Vol. 1, No. 2, 1928, p. 96-108.

1928-14. Mal'Chenko, Ye. V. Permafrost in eastern Siberia and Yakutiia. Trudy Kommissii po Izucheniyu Yakutskoy ASSR. Transactions of the Commission for the Study of Yakutskaya A.S.S.R. (Leningrad) No. 11, 1928, p. 150-76; 223-35.

1928-15. Mal'Chenko, Ye. V. Standig gefrorener Boden in Ostsibirien und Jakutien. Materially Kimissii po Izucheniyu Yakutskoy ASSR. Material of the Commission for the Study of Yakutskaya ASSR (Leningrad), Tom 2, 1928, p. 273- (In Russian).

1928-16. Kinzl, H. Beobachtungen uber Strukturboden in den Ostalpen. Petermanns Geographische Mitteilungen, Bd. 74, 1928, p. 261-65.

1928-17. Gladtsyn, I. N. Kamennyye mnogougol'niki. Rocks polygons. Izvestiya Gosudarstvennogo Russkogo Geograficheskogo Obshchestva. News of the State Russian Geographical Society. (Leningrad), Tom 60, Nos. 1/2, 1928.

1928-18. Horn, Gunnar, and Orvin, Anders K. Geology of Bear Island with special reference to the coal deposits, and with an account of the history of the Island. Skrifter om Svalbard og Ishavet, Nr. 15, 1928, xii, 152 p., illus., maps, tables, diagrs.

1928-19. Rawlins, E. L. and Johnson, T. W. "Comparisons of Ground Temperatures at Different Depths and Temperature Fluctuations of Atmosphere." Gas Age, Vol. 61, pp. 583-4.

1928-20. Ellis, J. H. and Shafer, W. "The Influence of Nitrogen Content on the Temperature of Red River Valley Soils." Scientific Agriculture, Vol. 9, p. 231.

1929-1. Smith, A. "Daily and Seasonal Air and Soil Temperatures at Davis, California." Hilgardia, Vol. 4, No. 3, pp. 77-112, May, 1929.

1929-2. Taber, S. "Frost Heaving." Journal of Geology, Vol. 37, No. 5, pp. 428-461, July-Aug., 1929.

1929-3. Buetow, W. C. "Causes and Control of Damaging Frost Action in Shoulders and Subgrades." Proceedings, 8th Annual Asphalt Paving Conference, The Asphalt Association and Association of Asphalt Paving Technologists, pp. 146-151, 1929.

1929-4. Smith, A. "Comparisons of Daytime and Nighttime Soil and Air Temperatures." Hilgardia, Vol. 4, No. 10, pp. 241-272, Dec., 1929.

1929-5. Eno, F. H. "The Influence of Climate on the Building, Maintenance and Use of Roads in the United States." Proceedings, Highway Research Board, Vol. 9, pp. 216-223, 1929.

1929-6. Beskow, G. "Tjalproblemets Grundfragor." Statens Vaginstitut, Meddelande No. 13, Svenska Vagforeningens Tidskrift, No. 1, 1929.

1929-7. Beskow, G. "Draneringens Betydelse for Vagarnas Tjalforhallanden." Statens Vaginstitut, Meddelande No. 15, Svenska Vagforeningens Tidskrift,

No. 3, 1929.

1929-8. Terzaghi, C. "The Science of Foundations - Its Present and Future." Transactions, American Society of Civil Engineers, Vol. 93, pp. 291-293, 1929.

1929-9. Nature. (London) "The Permanently Frozen Soils of Russia." Nature, (London), Vol. 123, No. 3106, pp. 741-742, May 11, 1929.

1929-10. Terzaghi, Chas. "The Science of Foundations." (The Effect of Freezing on Foundations). Transactions, Am. Soc. of C. E., Vol. 93, pp. 291-293, 1929.

1929-11. Sumgin, M. I. Ueber die ewige Gefrornis im Boden. Zeitschrift der Gesellschaft für Erdkunde zu Berlin, 1929.

1929-12. Salomon-Calvi, Wilhelm. Arktische Bodenformen in der Alpen Sitzungsberichte der Heidelberger Akademie der Wissenschaften (Math-Natur. Kl.), Jahrg. 1929, Abh. 5, p. 1-31.

1929-13. Gripp, Karl. Glaciologische und geologische Ergenbnisse der Hamburgischen Spitzbergen-Expedition 1927. Abhandlungen des Naturwissenschaftlichen Vereins zu Hamburg, Bd. 22, Heft 2-4, 1929, p. 145-251, illus., maps, tables, diagrs.

1929-14. Krekeler, F. Fossile Strukturboden aus der Umgebung von Giessen und Wiesbaden. Zeitschrift der Deutschen Geologischen Gesellschaft, Bd. 81, 1929, p. 462.

1929-15. Sarason, S. D. "Application of Aerial Mapping to Highway Construction." Proceedings, Highway Research Board, 9th Annual Meeting, December, 1929.

1930-1. U. S. Weather Bureau. "Climatic Summary of the United States." Bulletin W., Sec. 1 to 105, 1930.

1930-2. Meyer, E. V. "Conference Report on Porous (Cellular) Concrete." First International Congress for Concrete and Reinforced Concrete, Leige, Vol. 2, Question VI-9, pp. 139-144, 1930. (In French with English summary).

1930-3. Johnston, W. A. "Frozen Ground in the Glaciated Parts of Northern Canada." Transactions, Royal Society of Canada, Third Ser., Vol. 24, Sec. 3, pp. 31-40, 1930.

1930-4. Lang, F. C. "Discussion--Subgrade and Pavement Bases." Proceedings, 27th Annual Convention, American Road Builders Association, pp. 508-516, 1930.

1930-5. Sourwine, J. A. "A Method of Analysis of Data on Frost Occurrence for Use in Highway Design." Public Roads, Vol. 11, No. 3, pp. 51-60, May, 1930.

1930-6. Engineering News-Record. "Mitigating Frost Action on Road Surfaces." Engineering News-Record, Vol. 104, pp. 1021-1023, June 19, 1930.

1930-7. Taber, S. "The Mechanics of Frost Heaving." Journal of Geology, Vol. 38, No. 4, pp. 303-317, May-June, 1930.

1930-8. Mullis, I. B. "Illustrations of Frost and Ice Phenomena." Public Roads, Vol. 11, No. 4, pp. 61-79, June, 1930.

1930-9. Taber, S. "Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements." Public Roads, Vol. 11, No. 6, pp. 113-132, Aug., 1930.

1930-10. Burton, V. R. and Benkelman, A. C. "The Relation of Certain Frost Phenomena to the Subgrade." Proceedings, Highway Research Board, Vol. 10, pp. 259-275, 1930.

1930-11. Beskow, G. "De Geologiska Faktorer nas Betydelse for Vagarnas Tjalforhallanden." Statens Vaginstitut, Meddelande No. 24, Svenska Vagforeningens Tidskrift, No. 1, 1930.

1930-12. Beskow, G. "Om Vagarnas Barighet vid Vattenovermattning." Statens Vaginstitut, Meddelande No. 24, Svenska Vagforeningens Tidskrift, No. 2, 1930.

1930-13. Beskow, G. "Om Jordarternas Kapillaritet. En ny Metod for Bestamning av Kapillarkraften." (On the Capillarity of Soils. A new Method for Determining the Capillary Pressure). With English Summary, Statens Vaginstitut, Meddelande No. 25, 1929.

1930-14. Beskow, G. "Om Isoleringsatgarder mot Tjalskott ock Tjalskjutning." Statens Vaginstitut, Meddelande No. 26, Svenska Vagforeningens Tidskrift, No. 3, 1930.

1930-15. Beskow, G. "Inverkan av Klorkalcium och Sulfitlut pa Vagarnas Barighetsforhallanden." Statens Vaginstitut, Meddelande No. 27, 1930.

1930-16. American Road Builders Association. "Subgrades and Pavement Bases: Report of General Committee." Proceedings, American Road Builders Association, 27th Annual Convention, pp. 457-505, 1930.

1930-17. Zunker, F. "Das Verhalten des Bodens zum Wasser." I: Blanck, Handbuch der Bodenlehre, Band VI, pp. 66-220, 1930.

1930-18. Benkelman, A. C. and Olmstead, Frank R. "A New Theory of Frost Heaving." Proceedings, Highway Research Board, Vol. 11, pp. 152-165. Discussion pp. 165-177.

1930-19. Engineering News-Record. "Highway Progress and Investigations." Engineering News-Record, Vol. 105, No. 25, pp. 978-981, December 18, 1930.

1930-20. Satow, Gretel. Das Bodeneis in der Arktis. Tatsachen und Hypothesen. Aus dem Archiv der Deutschen Seewarte, Bd. 49, Nr. 5, 1930, 46 p., illus. maps.

1930-21. Nieland, H. Über Erscheinungen des Bodenfrostes und Auftaubodens in Westgrönland. Zeitschrift für Gletscherkunde, fur Eiszeitforschung und Geschichte des Klimas, Bd. 18, 1930, p. 346-51.

1930-22. Meinardus, Wilhelm. Arktische Boden. (In Blanck, E. Handbuch der Bodenlehre, Berlin, 1930, Bd. 3, pp. 27-96.

1930-23. Koloskov, P. I. On the question of heat

melioration in the areas of permafrost and deep winter freezing of soil. Materially Komissii po Izucheniyu Yestestvennykh Proizvoditel'nykh Sil SSSR. Commission for the Study of Natural Productive Forces of the USSR (Moscow, Leningrad), No. 80, 1930, p. 201-31.

1930-24. Kalitin, N. N. The role of actinometry in the solution of permafrost problems. Materially Komissii po Izucheniyu Yestestvennykh Proizvoditel' nykh Sil SSSR. Material of the Commission for the Study of Natural Productive Forces of the USSR. (Moscow, Leningrad), No. 80, 1930, p. 157-76.

1930-25. Gripp, Karl. Gletscher und Bodenfrost, rezent und diluvial. (Vorlaufige Mitteilung). Vortr. Kolloquium Arktis, Greifswald Mai 1930. Geologische Rundschau, Bd. 21, 1930, p. 351-52.

1930-26. Gregory, J. W. On frost stone-packing on the shores of Loch Lomond. Quarterly Journal of the Geological Society of London, Vol. 86, Nos. 144-45, 1930.

1930-27. Gregory, J. W. Stone polygons beside Loch Lomond. Geographical Journal, Vol. 76, No. 5, 1930, p. 415-18, illus.

1930-28. Beskow, G. Erdfliessen und Strukturboden der Hochgebirge im Lichte der Frosthebung. Geologiska Föreningens i Stockholm Förhandlingar, Bd. 52, Häfte 4, 1930, p. 622-38.

1930-29. Behlen, H. Eine neue Theorie der Struktur- (Stein-ring - Steinnetz-, oder Brodel-) Bodens unter besonderer Beruchsichti-gung von Spitsbergen. Zeitschrift der Deutschen Geologischen Gesellschaft. Bd. 82, 1930, p. 635-39.

1931-1. Keen, B. A. "Soil Temperature." The Physical Properties of Soil, Chap. 9, pp. 295-333. London: Longmans, Green and Co., 1931.

1931-2. Motl, C. L. "Curing Minnesota Frost Boils by Drains." Engineering News-Record, Vol. 106, No. 7, pp. 270-272, Feb. 12, 1931.

1931-3. Burton, V. R. and Benkelman, A. C. "Frost Action in Silt Soils Defined by Field and Cold Room Studies." Engineering News-Record, Vol. 106, No. 7, pp. 266-270, Feb. 12, 1931.

1931-4. Burton, V. R. "Application of Soil Science to Highway Engineering." Roads and Streets, Vol. 71, No. 5, pp. 163-165, May, 1931.

1931-5. Burton, V. R. "Application of Soil Science to Highway Engineering." Proceedings: 17th Annual Conference on Highway Engineering, University of Michigan, Official Publication, Vol. 32, No. 70, pp. 77-88, May 30, 1931.

1931-6. Hogentogler, C. A., Wintermeyer, A. M., and Willis, E. A. "Subgrade Soil Constants, Their Significance, and Their Application in Practice." Public Roads, Vol. 12, No's. 4 and 5, pp. 89-108, 116-144, June and July, 1931.

1931-7. Gould, C. B. "Tile Drains Solve Frost Boil Problem." Roads and Streets, Vol. 71, No. 7, pp. 257-258, July, 1931.

1931-8. Benkelman, A. C. "Studies of Frost Heaves in Michigan." Roads and Streets, Vol. 71, No. 7, pp. 272-276, July 1931.

1931-9. Watkins, W. I. and Aaron, H. "The Soil Profile and Subgrade Survey." Public Roads, Vol. 12, No. 7, pp. 51-64, Sept., 1931.

1931-10. Gilboy, G. "Soil Mechanics Research." Proceedings, American Society of Civil Engineers, Vol. 57, No. 8, pp. 1185-1186, Oct., 1931.

1931-11. Benkelman, A. C. and Olmstead, F. R. "A New Theory of Frost Heaving." Proceedings, Highway Research Board, Vol. 11, Pt. I, pp. 152-165, 1931.

1931-12. Watkins, W. I. "Discussion on Frost Heaving." Abstracted. Proceedings: Highway Research Board, Vol. 11, Pt. 1, pp. 165-168, 1931.

1931-13. Taber, S. "Discussion on Frost Heaving." (Abstracted). Proceedings, Highway Research Board, Vol. 11, Pt. 1, pp. 173-177, 1931.

1931-14. Casagrande, A. "Discussion on Frost Heaving." Proceedings, Highway Research Board, Vol. 11, Pt. 1, pp. 168-172, 1931.

1931-15. Ducker, A. "Untersuchungen uber die Frostgefahrlicheit nicht Bindiger Boden." (Investigations Concerning the Danger of Frost Heave in Noncohesive Soils). Forschungsarbeiten aus dem Strassenwessen, Vol. 17, Berlin: Volk und Reich Verlag, 1931.

1931-16. Bykov, B. I. Vechnaya merzlota i stroitel' stvo Igarki Permafrost and the construction of Igarka. Za Industrilizatsiyu Sovetskogo Vostoka. For the Industrialisation of the Soviet East (Moscow), Sbornik 1934, p. 42-79. Extent of permafrost in Igarka area; lessions learned in laying foundations of buildings there, 1931-32. Copy in the Library of Congress.

1931-17. Gignoux, M. Les sols polygonaux dans les Alpes et la genèse des sols polaires. Annales de Géographie, 40 An., 1931, p. 610-19.

1931-18. Poser, Hans. Beiträge zur Kenntnis der arktischen Bodenformen. Geologische Rundschau, Bd. 22, 1931, p. 200-331.

1931-19. Seidenfaden, Gunnar. Moving soil and vegetation in East Greenland. A preliminary report. Meddelelser om Grønland, Bd. 87, Nr. 2, 1931, 21 p. illus., map, tables, diagr.

1931-20. Ahlmann, Hans W: son. Polygonal markings. Geografiska Annaler, Bd. 17, 1935, p. 7-19, illus., table, diagrs. Scientific results of the Swedish-Norwegian Arctic Expedition in the summer of 1931, Part 12, 2; Spitsbergen.

1931-21. Moffit, Fred H. "The Slana District, Upper Copper River Region, Alaska." U. S. Geological Survey, Bulletin 824-B, 1931.

1932-1. West, E. S. "The Effect of Soil Mulch on

Soil Temperature." Journal of the Council for Scientific and Industrial Research (Australia), Vol. 5, pp. 236-246, 1932.

1932-2. Smith, A. "Seasonal Subsoil Temperature Variations." Journal of Agricultural Research, Vol. 44, No. 5, pp. 421-428, March 1, 1932.

1932-3. Beskow, G. "Nyars Undersokningar over Grusvagbanornas Kornstorlekssammansattning." Svenska Vagforeningens Tidskrift, No. 3, pp. 192-207, 1932.

1932-4. Jung, E. "Weitere Beitrag zur Aggregierenden Einwirkung des Frostes auf den Erdboden." Zeitschrift, F. Pflanzenenahrung, Dungung u. Bodenschichten. Einfuhrung in die Geophysik II, pp. 169-290. Berlin: J. Springer, 1932.

1932-5. Mattson, S. "The Laws of Colloidal Behaviour: VIII. Forms and Functions of Water." Soil Science, Vol. 33, No. 4, pp. 301-322, 1932.

1932-6. Michigan State Highway Department. "Subgrade Soil Exhibit." 29th Annual Convention and Exposition, American Road Builders Association, Detroit, Michigan, January, 1932.

1932-7. Antevs, E. "Alpine Zone of Mount Washington Range." Auburn, Maine: Merrill and Webber Co., 1932, 118 pp.

1932-8. Nichols, D. A. "Solifluction and Other Features in Northern Canada Shown by Photographs from the Air." Transactions, Royal Society of Canada, Sec. 4, pp. 267-269, 1932.

1932-9. Lotze, F. "Uber Schichtaufrichtungen an Kluffen." Deutsch. Geol. Gesell. Zeitsch, Vol. 84, pp. 67-77, 1932.

1932-10. Weickmann, L. Beobachtungen von Strukturböden wahrend der Polarfahrt des "Graf Zeppelin," Juli 1931. Berichte über die Verhandlungen der Sachsischen Akademie der Wissenschaften zu Leipzig Math.-Phys. Klasse, Bd. 84, 1932, p. 121-22.

1932-11. Bahr, Adolph. Frostgestauchte Böden in westlichen Schleswig-Holstein. Zeitschrift der Deutschen Geologischen Gesellschaft, Bd. 84, 1932, p. 29.

1932-12. Wolansky, Dora Strukturboden und arktische Bodenformen. Geologische Rundschau, Bd. 23, 1932, p. 132-40.

1933-1. Fowle, E. F. "Densities and Melting and Boiling Points of Inorganic Compounds." Smithsonian Physical Tables, Smithsonian Miscellaneous Collections, Vol. 88, pp. 255-256, 1933.

1933-2. Ewdokimow-Rokotowsky, M. I. "Bauten im Ewiggefrorenen Boden in Siberien" (Construction in the Perpetually Frozen Soil in Siberia). Der Bauingenieur, Vol. 14, Heft 15/16, pp. 224-226, April, 1933. (In German).

1933-3. Beskow, G. "Praktiska Metoder att Bedoma Markens Tjalfarlighetagrad." Vagmastaren, No. 2, 1933.

1933-4. Beskow, G. "Markundersokningar for Vagandamal i U. S. A." Svenska Vagforeningens Tidskrift, No. 3, pp. 198-206, 1933.

1933-5. Beskow, G. "Svenska Jordartsundersokningar for Vagandamal." Svenska Vagforeningens Tidskrift, No. 5, pp. 351-364, 1933.

193306. Beskow, G. "Tjalens Betydelse for Vagbelaggningar." Svenska Statens Vaginstitut, Meddelande No. 41, 1933.

1933-7. Mattson, S. and Falconer, J. G. "The Laws of Colloidal Behaviour: XIII Osmotic Inhibition." Soil Science, Vol. 36, No. 4, pp. 317-327, 1933.

1933-8. Zunker, F. "Die Durchlassigkeit des Bodens." Transactions, 6th Com. Int. Soc., Science, (Groningen) Vol. B. pp. 18-43, 1943.

1933-9. Steche, Hans. Beitrage zur Frage der Strukturboden. Berichte der Mathematisch-Physischen Klasse der Sachsischen Akademie der Wissenschaften zu Leipzig, Bd. 85, 1933, p. 193-272, illus., tables, diags.

1933-10. Poser, Hans. Das Problem des Strukturbodens. Geologische Rundschau, Bd. 24, 1933, p. 105-21.

1934-1. Kimball, D. A., Ruhnke, G. N., and Glover, M. P. "A Comparison of Temperatures in Air and at Various Depths in a Light, Sandy Soil in Southern Ontario." Scientific Agriculture (Canada), Vol. 14, No. 7, pp. 353-359, March, 1934.

1934-2. Miller, H. H. and Smith, D. N. "Methods for Prevention of Road Failures Due to Frost." Roads and Streets, Vol. 77, No. 6, pp. 219-221. June, 1934.

1934-3. Aaron, H. "Frost Heave in Highways and Its Prevention." Public Roads, Vol. 15, No. 1, pp. 10-25, March, 1934.

1934-4. Paradis, A. "Foundations and Protection Against Frost Heaving." Canadian Engineer, Vol. 67, No. 16, pp. 21-24, Oct. 16, 1934.

1934-5. Thomson, W. A. "Soil Temperatures at Winnepeg, Manitoba," Scientific Agriculture (Canada), Vol. 15, No. 4, pp. 209-217, Dec., 1934.

1934-6. Wilsdon, B. H. "Problems of Porous Bodies and Their Behaviour as Building Materials." Journal of the Society of Chemical Industry (London), Vol. 53, pp. 397T-402T, Dec. 28, 1934.

1934-7. Beskow, G. "Uppfrysning och Frostbranna. Dersa Beroende av Jordgrund, Vattenforhallanden och Snotacke." Svenskt Land No. 2, pp. 32-34, 1934.

1934-8. Beskow, G. "Tjalbildningen i Vagarna och Atgarder mot dess Skadegorelse. Tekniskekonomiska utredningar Rorande Vagvasendet." Del. I: Vagar. Statens off. utredningar 27, 1934. Svensks Vaginstitut. Maddelande No. 44, pp. 292-346, 1934.

1934-9. Beskow, G. "Grusvagbanors Sammansattning." Svenska Vaginstitut, Meddelande No. 44, pp. 365-388, 1934.

1934-10. Eno, F. H. "Some Effects of Soil, Water and

Climate upon the Construction, Life, and Maintenance of Highways." Ohio State University, Eng. Exp. Sta. Bul. No. 85, pp. 57-60, Nov., 1934.

1934-11. Breuil, Henri. De l'importance de la solifluxion dans l'étude des terrains quaternaires du nord de la France et des pays voisins. Revue de Géographie Physique et de Geologie Dynamique. Vol. 7, 1934, p. 269-322.

1934-12. Dücker, Alfred. Fossile Bodenfrosterscheinungen (Brödelboden in Schleswig-Holstein.) Zeitschrift "Die Heimat," 1934, p. 235-

1934-13. Gripp, K., and Simon, W. G. Die experimentelle Darstellung des Brodelbodens. Die Naturwissenschaften, Bd. 22, 1934, p. 8-10.

1934-14. Hollingsworth, S. E. Some solifluction phenomena in the northern part of the Lake District. Proceedings of the Geologists' Association, Vol. 45, Part 2, 1934, p. 167-88.

1934-15. Orvin, Anders K. Geology of the Kings Bay region, Spitsbergen, with special reference to the coal deposits. Skrifter om Svalbard og Ishavet, Nr. 57, 1934, (ii), 195 p. illus., maps, tables, diagrs.

1934-16. Polunin, Nicholas. The vegetation of Akpatok Island. Part 1, Journal of Ecology, Vol. 22, 1934, p. 337-95. (See p. 354).

1934-17. Poser, Hans. Bemerkungen zum Strukturbodenproblem. Zentralblatt für Mineralogie, Geologie und Paläontologie, Abt. 13, No. 1, 1934, pp. 39-45.

1934-18. Svetozarov, I. The hydrogeology of permafrost regions, based on investigation in the area of the town Yakutsk. Problemy Sovetskoy Geologii. Problems of Soviet Geology. (Moscow, Leningrad), No. 10, 1934, p. 119-32.

1934-19. Fleming, W. L. S. and Gatty, O. Geomorphology: surface markings. Geographical Journal, Vol. 84, No. 2, 1934, p. 119-20. The Oxford University Expedition to Spitsbergen, 1933, Appendix D.

1934-20. Porsild, A. E. "Earth Mounds in Unglaciated Arctic Northwestern America." Geographical Review, Vol. 24, No. 4, 1934.

1935-1. Beskow, G. "Tjälbildningen och Tjällyftningen." (Soil Freezing and Frost Heaving; with Special Reference to Highways and Railroads). Statens Väginstitut, Meddelande 48, Sveriges Geologiska Undersökning, Ser. C, No. 375, Stockholm, 1935. 242 pp.

1935-2. Mattimore, G. S. and Rahn, G. A. "Research on Concrete Disintegration." Proceedings, American Society for Testing Materials, Vol. 35, Pt. 2, Technical Papers, pp. 410-420, 1935.

1935-3. Lang, F. C. "Soil Science Applied to Flexible Surfaces." Better Roads, Vol. 5, No. 2, pp. 20-27, Feb., 1935.

1935-4. Engineering News-Record. "Ice Pressure Determinations in Clay Soils." Engineering News-Record, Vol. 115, No. 4, p. 127, July 25, 1935.

1935-5. Miller, H. H. and Smith, D. N. "Prevention of Frost Heaving in Roads." Roads and Streets, Vol. 78, No. 8, pp. 273-274, Aug., 1935.

1935-6. Clark, K. A. "Some Examples of Frost Boils Occurring on Alberta Highways." Canadian Engineer, Vol. 69, No. 11, pp. 7-10, Sept. 10, 1935.

1935-7. Bretz, J. Harlen. Physiographic studies in East Greenland. (In Boyd, Louise A.: The fiord region of East Greenland. American Geographical Society, Special Publication No. 18, 1935, p. 159-266, illus., table.)

1935-8. Datskiy, N. G. Swelling of ground along the railroads under permafrost conditions. Trudy Kommssii po Izucheniyu Vechnoy Merzloty. Transactions of the Commission for the Study of Permanently Frozen Soil. (Moscow, Leningrad), Tom 4, 1935, p. 171-87.

1935-9. Sørensen, Thorvald. Bodenformen und Planzendecke in Nordostgrönland (Beiträge zur Theorie der polaren Bodenversetzungen auf Grund von Beobachtungen über deren Einfluss auf die Vegetation in Nordostgrönland). Meddelelser om Grønland, Bd. 93, Nr. 4, 1935, 69 p., tables, diagrs.

1935-10. Steche, Hans. Beiträge zur Frage der Strukturböden. Neues Jahrbuch für Mineralogie, Geologie und Paläontologie, Ref. 2, 1935, p. 61.

1935-11. Suslov, C. P. Auto-truck road under permafrost conditions in the Enisei forest-tundra. Trudy Kommissii po Izucheniyu Vechnoy Merzloty. Transactions of the Commission for the Study of Permanently Frozen Soil. (Moscow, Leningrad), Tom 4, 1935, p. 95-145.

1936-1. Allen, Harold and Johnson, A. W. "The Results of Tests to Determine the Expansive Properties of Soils." Proceedings, Highway Research Board, Vol. 16, pp. 220-233, 1936.

1936-2. Public Works Magazine. "Soil Freezing and Frost Heaving." Public Works Magazine, Vol. 67, No. 2, pp. 41-42, Feb., 1936.

1936-3. Engineering News-Record. "Frost Penetration Studies at Portland, Maine." Engineering News-Record, Vol. 116, No. 15, p. 537, April 9, 1936.

1936-4. Mail, G. A. "Soil Temperatures at Bozeman, Montana, During Subzero Weather." Science, Vol. 83, No. 2163, p. 574, June 12, 1936.

1936-5. Morton, J. O. "The Application of Soil Mechanics to Highway Foundation Engineering." Proceedings, International Conference on Soil Mechanics and Foundation Engineering, Harvard University, Vol. 1, pp. 243-247, June 22-26, 1936.

1936-6. Morton, J. O. "Soil Profile Field Methods in New Hampshire." Proceedings, International Conference on Soil Mechanics and Foundation Engineering, Harvard University, Vol. 1, p. 247, June 22-26, 1936.

1936-7. Jürgenson, L. "Field Test for Identification

of Soil Capable of Frost Heaving (English Summary). Proceedings, International Conference on Soil Mechanics and Foundation Engineering, Harvard University, Vol. 2, pp. 171-173, June 22-26, 1936.

1936-9. Mackintosh, A. "Progress Report of an Investigation of Frost Action in Soils." Proceedings, International Conference on Soil Mechanics and Foundation Engineering, Harvard University, Vol. 2, pp. 260-262, June 22-26, 1936.

1936-10. Fuller, H. U. "Frost Penetration as Affected by Weather and Snow Conditions." Journal, New England Water Works Association, Vol. 50, No. 3, pp. 299-301, Sept., 1936.

1936-11. Denny, C. S. "Periglacial Phenomena in Southern Connecticut." American Journal of Science, Vol. 37, pp. 322-342, 1936.

1936-12. Hay, T. "Stone Stripes." Geographical Journal, Vol. 87, pp. 47-50, 1936.

1936-13. Grawe, O. R. "Ice as an Agent of Rock Weathering." Journal of Geology, Vol. 44, pp. 173-182, 1936.

1936-14. Kogler, A. and Scheidig, H. "Leussink: Beitrage zur Frostfrage im Strassenbau." (Contributions to the Frost Question in Highway Construction). "Bodenmechanik und neuzeitlicher strassenbau" (Soil Mechanics and Modern Highway Construction). Schriftenreihe der Strasse, Issue 3, pp. 32-41. Berlin: Volk und Reich Verlag, 1936.

1936-15. Soergel, W. "Diluviale Eiskeile" Deutsch. Geol. Gesell, Zeitsch. Vol. 88, pp. 223-247, 1936.

1936-16. Selzer, G. "Diluviale Losskeilnetze aus der Umgebung Gottingens," Geol. Rundsch, Vol. 27, pp. 275-293, 1936.

1936-17. Poser, Hans. Talstudien aus Westspitzbergen und Ostgrönland. Zeitschrift für Gletscherkunde, für Eiszeitforschung und Geschichte des Klimas, Bd. 24, 1936, pp. 43-98.

1936-18. Mel'nikov, P. I. Nasosnaya stantsiya v g. Yakutske i yeye vliyanie na rezhim vechnoy merzloty. Pumping station in the town of Yakutsk and its influence on the régime of permanently frozen soil. Merzlotovedenie. Study of Permanently Frozen Soil (Moscow), Vol. 2, No. 1, 1947, pp. 58-68. Study of effect of this heated building on surrounding soil, 1936-45. Copy in Foreign Office Research Department, London.

1936-19. Tsytovich, N. A. Principles of construction and design of foundations erected on permafrost. Trudy 1-oy Geologo-Razvedoch-noy Konferentsii Glavsevmorputi. Transactions of the First Geological Prospecting Conference of the Chief Administration of the Northern Sea Route (Moscow, Leningrad), No. 3, 1936, pp. 78-101.

1936-20. Edelman, C. H., and others. Über spätpleistozäne und frühholozäne kryoturbate Ablagerungen in den östlichen Niederlanden. By C. H. Edelman, F. Florschutz, and J. Jeswiet. Verhandelingen van het Geologisch-Mijnbouwkundig Genootschap voor Nederland en Kolonien, Geologische series, Tom 11, No. 4, 1936, pp. 301-36.

1936-21. Lavrova, M. A. Über das Vorfinden von Dauerfrostboden im Gebiet der Woltschja- und Montsche- Tundra auf der Halbinsel Kola. Neues Jahrbuch für Mineralogie, Geologie und Paläontologie, Ref. 2, 1936, pp. 102-03.

1936-22. Moore, Charles B. Air temperature and frost penetration. Journal Maine Water Utilities Association, Vol. 12, No. 2, 1936, pp. 46-48.

1936-23. Sumgin, M. I. Zur Frage des Dauerfrostbodens in Torfhü eln auf der Halbinsel Kola. Neues Jahrbuch für Mineralogie, Geologie und Paläontologie, Ref. 2, 1936, pp. 100-02.

1936-24. Bouyoucos, G. J. The Dilatometer Method as an Indirect Means of Determining the Permanent Wilting Point of Soils. Soil Science, Vol. 42, pp. 217-223, 1936.

1937-1. Bodman, G. B. and Day, P. R. "Thermoelectric Method of Determining the Freezing Points of Soils." Proceedings, Soil Science Society of America, Vol. 2, pp. 65-71, 1937.

1937-2. Gottstein, E. "Fundamentals on Frost Damage to Roads, Its Causes and Prevention." (Grundsätzliches über Frostschäden an Strassen, ihre Ursachen und ihre Verhütung), Berlin, 1937 (Volk and Reich Verlag), p. 19, Fig. 28. Road Abstracts, Vol. 5, No. 2, Abst. No. 74, p. 21, April, 1938.

1937-3. Erlenbach, L. "Recent Observations and Research on Frost Damage in East Prussia." Strasse, 1937, 5 (2), 42-6. Road Abstracts, Vol. 5, No. 3, Abst. No. 139, pp. 39-40, May, 1938.

1937-4. Semiakin, F. M. and Mikhalev, P. F. "Formation of Periodic Ice Streaks During Soil Freezing." Comptes Rendus (Doklady) de l'Academie des Sciences, U.S.S.R., 1937, 17 (8) 405-7. (In English). Science Abstracts, 1938, 41 (484), 356. Road Abstracts, Vol. 5, No. 7, Abst. No. 388, p. 110, Oct., 1938.

1937-5. Mabee, W. C. "Lessons from the Winter of 1935-36." Journal, American Water Works Association, Vol. 29, No. 1, pp. 7-8, Jan., 1937.

1937-6. Ivie, J. O. and Richards, L. A. "A Meter for Recording Slow Liquid Flow." Review of Scientific Instruments, Vol. 8, No. 3, New Ser., pp. 86-89, March, 1937.

1937-7. Emrey, D. J. "Subgrade Soils, Their Analysis and Drainage." Canadian Engineer, Vol. 72, No. 12, pp. 5-9, March 23, 1937. (Also No's. 13 and 15, March 30 and April 13, 1937).

1937-8. Lang, F. C. "Combating Frost and Drainage Problems." Proceedings, 23rd Annual Road School, Purdue University, Engineering Bul., Vol. 21, No. 3, (Extension Ser. No. 39), pp. 46-56, May, 1937.

1937-9. McClain, C. W. "Combating Frost and Drainage Problems." Proceedings, 23rd Annual Road School, Purdue University, Engineering Bul., Vol. 2, No. 3, (Extension Ser. No. 39), pp. 57-58, May, 1937.

1937-10. Engineering and Contract Record. "Extent of Frost Penetration in Soils." Engineering and Contract Record, Vol. 50, No. 89, p. 11, Sept. 8, 1937.

1937-11. Lewis, M. R. "Rate of Flow of Capillary Moisture." U. S. Dept. of Agricultural, Technical Bul. No. 579, Oct., 1937. 29 pp.

1937-12. Keil, K. "How Can We Judge the Liability of Stratified Rocks to Frost Damage." Strassenbau, 1937, 28 (9), pp. 109-115. Road Abstracts, Vol. 4, No. 8, Abst. No. 348, pp. 121-122, Nov., 1937.

1937-13. Armco Culvert Manufacturers Association. "Handbook of Culvert and Drainage Practice." Pp. 41, 67, 111, 305-308, 331. Chicago: R. R. Donnelley & Sons, 1937.

1937-14. Hogentogler, C. A. "Engineering Properties of Soil." Pp. 127-158, 332. N. Y.: McGraw-Hill Book Co., 1937.

1937-15. Mozley, A. "Frozen Ground in the Sub-Arctic Region and its Biological Significance." Scottish Geographical Magazine, Vol. 53, No. 4, pp. 266-270, July, 1937.

1937-16. Dücker, A. Über Strukturboden im Riesengebirge. Ein Beitrag zum Bodenfrost- und Lössproblem. Zeitschrift der Deutschen Geologischen Gesellschaft, Bd. 89, 1937, p. 113-29.

1937-17. Bilibin, Yu. A. Active and passive permafrost. Izvestiya Gosudarstvennogo Geograficheskogo Obshchestva. News of the State Geographical Society, (Leningrad), Tom 69, No. 3, 1937, pp. 409-11.

1937-18. Lukashev, K. I. Granulometric composition of ground in the permafrost area. Yezhegodnik Leningradskogo Universiteta No. 16, Seriya Geologicheskaya, Pochvovedcheskaya i Geograficheskaya, No. 3. Leningrad University Annual, No. 16, Geological, Soil and Geographical Series, No. 4, 1937, pp. 170-184.

1937-19. Obruchev, S. V. Soliflyuktsionnyye (nagornyye) terrasy i ikh genezis na osnovanii rabot v Chukotskom Kraye. Solifluctional (upland) terraces and their genesis, on the basis of work in the Chukotka area. Problemy Arktiki. Problems of the Arctic. (Leningrad), No. 3, 1937, p. 27-48; No. 4, 1937, pp. 57-83.

1937-20. Sumgin, M. I. Vechnaya merzlota pochvy v predelakh SSSR. 2-e dopolnennoye izdanie. Permanently frozen soil within the confines of the USSR, 2nd augmented edition. Moscow, Leningrad, Izdatel'stvo Akademii Nauk SSSR. Publishing House of the Academy of Sciences of the USSR, 1937, 379 pp., illus., maps, diagrs., 24 cm. A number of additions based on recent work has been made to 1st edition (1927); but work is still a geophysical monograph and contains no practical engineering information. Copy in Library of Congress.

1937-21. Presnyakov, Ye. A. Certain forms of relief due to permafrost. Izvestiya Gosudarstvennogo Geograficheskogo Obshchestva. News of the State Geographical Society. (Leningrad), Tom 69, No. 1, 1937.

1937-22. Tsytovich, N. A. and Sumgin, M. I. Osnovaniya mekhaniki merzlykh gruntov. Basis of the mechanics of frozen soils. Moscow, Leningrad, Izdatel'stvo Akademii Nauk SSSR. Publishing House of the Academy of Sciences of the USSR, 1937. 432 pp., tables, 25 cm. Behaviour of permanently frozen ground under action of natural forces and of load placed on it; practical measures to be taken when digging foundations. Copy in Library of Congress, Washington.

1937-23. Ruedy, R. "Soil Temperatures in Canada," National Research Council of Canada. Ottawa, Canada.

1937-24. Missouri State Highway Commission. Report on a Study of Concrete Pavement Distortion in Missouri Investigational Project 33-2, Pp. 37-38, July, 1937.

1937-25. Texas Highway Department. Compilation of Data on the Guadalupe County Research Project. Parts I - IV, 1933-1937.

1938-1. Sharpe, C. F. S. "Landslides and Related Phenomena." A Thesis. N. Y.: Columbia University, 1938. 137 pp.

1938-2. Casagrande, L. "Experimental Road Sections: The Drainage of Foundations." Strassee, 1938, 5 (16), 529. Road Abstracts, Vol. 6, No. 1, Abst. No. 20, p. 5, Jan., 1939. (See also Road Abstracts, Vol. 5, No. 5, Abst. No. 273, pp. 73-74, Aug., 1938.)

1938-3. Keil, K. "Results of a Frost Action Study on a Stretch of Motor Road." Strassenbau, 1938, 29 (5), pp. 67-71; (6) pp. 94-97; (8) pp. 126-129. Road Abstracts, Vol. 5, No. 5, Abst. No. 273, pp. 73-74. Aug., 1938.

1938-4. Wilford, H. D., Downey, B. R., Briggs, G. F., Hogentogler, C. A., Jr., Knight, J. A., Ellenan, J.A., and Burggraf, F. "Uses of Calcium Chloride in Road Stabilization." Proceedings, Highway Research Board, Vol. 18, Pt. 2, pp. 209-256, 1938.

1938-5. Casagrande, L. "Effects of Frost in Soil." Permanent International Association of Road Congresses, First and Second Sec. Combined, 8th Congress, The Hague, 6th Question, pp. 10-11, 1938.

1938-6. Smith, W. O. and Byers, H. G. "The Thermal Conductivity of Dry Soils of Certain of the Great Soil Groups." Proceedings, Soil Science Society of America, Vol. 3, pp. 13-19, 1938.

1938-7. Porsild, A. E. "Earth Mounds in Unglaciated Arctic Northwestern America." Geographical Review, Vol. 28, No. 1, pp. 46-58, Jan., 1938.

1938-8. Heating and Ventilating. "Frost Penetration in Ground." Heating and Ventilating, Vol. 35, No. 6, pp. 53-56, June, 1938.

1938-9. Winn, H. F. "Frost Action in Stabilized Soil Mixtures." Proceedings, Highway Research Board, Vol. 18, Pt. 1, pp. 264-290, 1938.

1938-10. Morton, J. O., Tremper, B., Stokstad, O. L. and Casagrande, L. "Prevention of Detrimental Frost Heave." Proceedings, Highwav Research Board, Vol. 18, Pt. 2, pp. 356-365, 1938.

1938-11. Beskow, G. "Prevention of Detrimental Frost Heave in Sweden." Proceedings, Highway Research Board, Vol. 18, Pt. 2, pp. 366-370, 1938.

1938-12. Endell, K., Loos, W., Meischeider, H., Berg, V. "On the Relations Between the Behaviour of Clay Minerals with Respect to Water and the Physical Properties of Cohesive Soils." Deutschen Forschungsgesellschaft fur Bodenmechanic, Heft 5. Berlin: Julius Springer, 1938.

1938-13. Housel, W. S. "Applied Soil Mechanics, Pt. 1: Soil as an Engineering Material." Civil Engineering Department, University of Michigan, pp. 91-100, 1938.

1938-14. Woods, K. B., and Litehiser, R. R. "Soil Mechanics Applied to Highway Engineering in Ohio." Ohio State Eng. Exp. Sta., Bulletin No. 99, (Eng. Ser., Vol. 7, No. 2, July 1938).

1938-15. Shvetsov, P. F. Permafrost and engineering-geological conditions of the Anadyr' region. Trudy Gorno-Geologicheskogo Upravleniya Glavnogo Upravleniya Severnogo Morskogo Puti. Transactions of the Mining and Geological Administration of the Chief Administration of the Northern Sea Route. (Moscow, Leningrad), 1938, p. 1-78.

1938-16. Lukashev, K. I. The region of permafrost. Leningrad University, 1938, 187 p.

1938-17. Glazov, N. V. Method of studying the degradation of permafrost. Trudy Komiteta po Vechnoy Merzloty. Transactions of the Committee on Permanently Frozen Soil (Moscow, Leningrad), Tom 6, 1938, p. 155-61.

1938-18. Baranov, I. Ya. Mapping of areas of permafrost. Trudy Komiteta po Vechnoy Merzloty. Transactions of the Committee on Permanently Frozen Soil, (Moscow, Leningrad), Tom 6, 1938, pp. 107-25.

1939-1. Ducker, A. "Untersuchungen über die Frostgefährlichen Eigenschaften Nichtbindiger Böden" (Investigations on the Frost Danger Element of Cohesionless Soils). Forschungsarbeiten aus dem Strassenwesen, Band 17, 1939. 79 pp. (In German).

1939-2. Endell, K., Loos, W., and Breth, H. "Zusammenhang Zwischen Kolloidchemischen Sowie Bodenphysikalischen Kenziffern Bindiger Böden und Frostwirking." Forschungsarbeiten aus dem Strassenwesen, Band 16, 1939. 55 pp. (In German). Relationship between colloid chemicals as well as soil-physico-chemical indices of cohesive soils and frost action.

1939-3. Rowat, R. M. "Control of Frost Heave." Proceedings, Highway Research Board, Vol. 19, pp. 464-466, 1939.

1939-4. Ravn, H. H. "Frost Damage on Roads" Dansk Vejtidsskr. 1939, 16 (4), pp. 142-148. Road Abstracts, Vol. 7, No. 3, Abst. No. 81, pp. 37-38, April, 1940.

1939-5. Gardner, D. L. and Wright, G. C. "Frost Heave Measurements for 1938-1939." A thesis in partial fulfillment for the Degree of Bachelor of Science, University of New Hampshire, 1939. 45 pp.

1939-6. Goodell, B. C. "Soil Boring Tool for Frost Depth Determination." Journal of Forestry, Vol. 37, No. 6, pp. 457-459, June, 1939.

1939-7. Cressey, G. B. "Frozen Ground in Siberia." Journal of Geology, Vol. 47, No. 5, pp. 472-488, July-Aug., 1939.

1939-8. McLeod, N. W. "Soil Science Applied to Subgrade and Base Course Design." Canadian Engineer, Vol. 77, No. 5, pp. 5-6, with discussion on pp. 52 and 54, Aug., 1939.

1939-9. Keil, K. "Results of Frost Action on Experimental Road." Public Works Magazine, Vol. 70, No. 9, pp. 49-50, Sept. 1939.

1939-10. Garneau, J. B. Discussion of "Soil Stabilization and Prevention of Frost Heaves." Canadian Engineer, Vol. 77, No. 12, pp. 52-54, Sept., 1939.

1939-11. Smith, W. O. "Thermal Conductivities in Moist Soils." Proceedings, Soil Science Society of America, Vol. 4, pp. 32-40, 1939.

1939-12. Sourwine, J. A. "Art of Preparing an Earthen Foundation." U. S. Patent No. 2, 162, 185, June 13, 1939.

1939-13. Clay Products Association. "Highway Drainage." 1st Ed. P. 10. Chicago: The Association, 1939.

1939-14. Gill, A. F., and Thomas, J. F. J. "Some Observations Regarding Frost Action." Proceedings, Highway Research Board, Vol. 19, p. 281, 1939.

1939-15. Romanovsky, Vsevolod. Toubillons dans les boues épaisses. Application aux terrains polygonaux. Comptes Rendus Hebdomadaires des Séances de l'Académie des Sciences (Paris), Tom. 208, 1939.

1939-16. Romanovsky, Vsevolod. Application de la théorie convective aux terrains polygonaux. Revue de Géographie Physique et de Géologie Dynamique, Vol. 12, fasc. 2, 1939, pp. 315-27.

1939-17. Kushev, S. L. Morphology and genesis of hilly swamps and their geographical distribution. Trudy Komiteta po Vechnoy Merzloty. Transactions of the Committee on Permanently Frozen Soil, (Moscow, Leningrad), Tom 8, 1939, pp. 119-61.

1939-18. Smith, Phillip S. "Areal Geology of Alaska." U. S. Geology Survey, Professional Paper No. 192, 1939.

1939-19. Lobeck, A. E. "Geomorphology." McGraw-Hill Book Co., 1939.

1940-1. Slesser, Charles. "Field Studies of Migration of Calcium Chloride and Sodium Chloride in Soil." Thesis. Purdue University, 1940.

1940-2. Baver, L. D. "Soil Physics." Pp. 153-155 and 275-289. N. Y.: John Wiley and Sons, 1940. 370 pp.

1940-3. Atkinson, H. B. and Bay, C. E. "Some Factors Affecting Frost Penetration." Transactions, American Geophysical Union, Pt. 3, pp. 935-947, 1940. Discussion by K. C. Roberts, P. 949.

1940-4. Ravn, H. H. "The Origin of Frost Damage and Methods of Combating It." Dansk Vejtidsskr, 1940, 17 (2), pp. 40-52, (3) 90-97, (5), 151-171. Road

Abstracts, Vol. 14, No. 4, Abst. No. 262, p. 48, April 1, 1947. Highway Research Abstracts, No. 141, p. 6, May, 1947.

1940-5. Osterberg, J. O. "A Survey of the Frost Heaving Problem." Civil Engineering, Vol. 10, No. 2, pp. 100-102, Feb., 1940.

1940-6. Woods, K. B. "Frost Action." Aims and activities of the Joint Highway Research Project. Purdue University, Engineering Bul., Vol. 24, No. 2, (Research Ser. No. 71, Highway Research Bul. No. 3), pp. 35-37, 41-43, 48-49, March, 1940.

1940-7. Winn, H. F. and Rutledge, P. C. "Frost Action in Highway Bases and Subgrades." Purdue University, Engineering Bul., Vol. 24, No. 2 (Research Ser. No. 73, Highway Research Bul. No. 4), May, 1940. 104 pp.

1940-8. Casagrande, L. "What has the Frost Damage of Last Winter Taught Us." Strasse, Vol. 7, Nos. 9-10, pp. 193-201, 1940. Road Abstracts, Vol. 8, No. 11, Abst. No. 343, p. 133, Oct. 14, 1941. Highway Research Abstracts, No. 86, pp. 7-8, Jan., 1942.

1940-9. Fuller, H. U. "Studies of Frost Penetration." Journal, New England Water Works Association, Vol. 54, No. 3, pp. 275-281, Sept., 1940.

1940-10. Winn, H. F. "Frost Action on Highway Subgrades and Bases." Proceedings, Purdue Conference on Soil Mechanics and its Applications, Purdue University, Symposium on Frost Action, pp. 444-457, Sept. 2-6, 1940.

1940-11. Lang, F. C. Discussion on "Frost Action in Highway Subgrades and Bases." Proceedings, Purdue Conference on Soil Mechanics and its Applications, Purdue University, Symposium on Frost Action, pp. 457-460. Sept. 2-6, 1940.

1940-12. Skelton, R. R. "Field Observations of Frost Heave." Proceedings, Purdue Conference on Soil Mechanics and its Applications, Purdue University, Symposium on Frost Action, pp. 460-470, Sept. 2-6, 1940.

1940-13. Bennett, E. F. "Frost Action in Soils." Proceedings, Purdue Conference on Soil Mechanics and its Applications, Purdue University, Symposium on Frost Action, pp. 471-473, Sept. 2-6, 1940.

1940-14. Belcher, D. J. "Subgrade Soil Temperatures." Proceedings, Purdue Conference on Soil Mechanics and its Applications, Purdue University, Symposium on Frost Action, pp. 474-482, Sept. 2-6, 1940.

1940-15. Pratt, F. R. "Depth of Frost under Pavement." Engineering News-Record, Vol. 125, No. 25, p. 67, Dec. 19, 1940.

1940-16. Dorsey, N. E. "Properties of Ordinary Water Substances." American Chemical Society, Monograph Ser. 81, p. 473, 1940.

1940-17. Paterson, T. T. "The Effects of Frost Action and Solifluction around Baffin Bay and in the Cambridge District: Geological Society Quarterly Journal, (London) Vol. 96, No. 381, pp. 99-130, April, 1940. Discussion by S. E. Hollingworth, pp. 128-129.

1940-18. Ducker, A. "Frosteinwirkung auf bindige Boden." (Effect of Frost on Cohesive Soils). Strassenbaujahrbuch 1931-1940, p. 111. Berlin: Volk und Reich Verlag, 1940.

1940-19. Plummer, F. L. and Dore, S. M. "Soil Mechanics and Foundations." Pp. 131-140, 353-357. N. Y.: Pitman Publishing Corp., 1940.

1940-20. Urquhart, L. C. "Civil Engineering Handbook." 2nd Ed. Pp. 627, 840. N. Y.: McGraw-Hill Book Co., 1940.

1940-21. Denny, C. S. "Stone-Rings on New Hampshire Mountains." American Journal of Science, Vol. 238 No. 6, pp. 432-438, June, 1940.

1940-22. O'Rourke, C. E. "General Engineering Handbook." 2nd Ed., pp. 836, 1913, 1914. N. Y.: McGraw-Hill Book Co., 1940.

1940-23. Stull, R. T., and Johnson, P. V. "Some Properties of the Pore System in Bricks and Their Relation to Frost Action." Journal of Research, National Bureau of Standards Research Paper 1349, Vol. 25, pp. 711-730, December, 1940.

1940-24. Enenshteyn, B. S. Application of electroprospecting by direct current on perpetually frozen ground in the Igarka region. Comptes Rendus (Doklady) de l'Academie des Sciences de l'URSS (Moscow), Vol. 26, No. 4, 1940, pp. 338-41. Results of experiments made in 1939; possibility established of determining thickness of thawed and frozen layers by this method. In English. Copy in Philosophical Library, Cambridge.

1940-25. Bykov, N. I., and Kapterev, P. N. Vechnaia merzlota i stroitel' stvo na nei. Permafrost and construction on it. Moscow, 1940, 372 pp.

1940-26. Capps, S. R. Geology of the Alaska Railroad region. U. S. Geological Survey, Bulletin 907, 1940.

1940-27. Dines, H. G. and others. The mapping of head deposits. Geological Magazine, Vol. 77, 1940, p. 198-226.

1940-28. Goldthwait, R. P. Geology of the Presidential Range. New Hampshire Academy of Sciences, Bulletin 1, 1940, 43 pp.

1940-29. Jahn, Alfred. Research on the structure and temperature of the soils in western Greenland. Bulletin de l'Académie Polonaise des Sciences et des Lettres, Chasse des Sciences, Mathématiques et Naturelles, Serie A: Sciences Mathématiques, No. sommaire, 1940-46, (pub. 1948), pp. 50-59.

1940-30. Lukashev, K. I. Hugelbildung als Erscheinung von Spannung in den Böden in Verbindung mit dem Dauerfrostbodens. Neues Jahrbuch für Mineralogie, Geologie und Paläontologie, Ref. 2, 1940, p. 438-39.

1940-31. Romanovsky, Vsevolod. Une application des tourbillons convectifs: formation des sols polygonaux. Journal de Physique det le Radium: Ser. 8,

Tom. 1, Nr. 8, 1940.

1940-32. Viluksela, M. Viimeaikaisista Toimenpiteista Routimishaittojen Vahentamiseksi Rautateilla. Teknillinen Aikakauslehti (Helsinki), Vol. 30, No. 2, February 1940, p. 22-25. Report on damage caused by ground frost to railways of northern countries and on methods adopted to counteract this. English abstract p. 25.

1941-1. Schaible, L. "Frost Damage. Bautechnik, 1941, 19 (49), pp. 529-31; (50/51), pp. 541-3. Road Abstracts, Vol. 10, No. 3, Abst. No. 163, pp. 37-38, March, 1943. Highway Research Abstracts, No. 99, p. 7, April, 1943.

1941-2. Heje, K. "Studies of Frost Heaving Carried out at the Norwegian Technical College and their Economic Results." Meddelande Veidirekt Norsk Ing. Foren, 1941, (6), 65-71. Road Abstracts, Vol. 9, No. 6, Abst. No. 217, p. 67, May, 1942.

1941-3. U. S. Department of Agriculture. "Climate and Man." Yearbook of Agriculture, Pt. 5, Climatic Data, with Special Reference to Agriculture in the U. S., pp. 664-1210, 1941.

1941-4. Belotelkin, D. T. "Soil Freezing and Forest Cover." Transactions, American Geophysical Union, Pt. 1, pp. 173-175, 1941.

1941-5. Thompson, W. T. "A Method of Measuring Thermal Diffusivity and Conductivity of Stone and Concrete." Kansas State College, Bulletin, Vol. 25, No. 3, (Eng. Exp. Sta. Bul. 40), May 1, 1941.

1941-6. Edlefsen, N. E. and Bodman, G. B. "Field Measurements of Water Movement Through a Silt Loam Soil." Journal of the American Society of Agronomy, Vol. 33, No. 8, pp. 713-731, 1941.

1941-7. Lang, F. C. "Temperature and Moisture Variations in Concrete Pavements." Proceedings, Highway Research Board, Vol. 21, pp. 260-271, 1941.

1941-8. Endell, K. "Expansivity of Clays in Foundation Soil and Its Importance in Construction Work." Die Bautechnik, Vol. 19, No. 19, pp. 201-209, May 2, 1941.

1941-9. Grim, R. E. "The Clay Minerals in Soils and Their Significance." Illinois Geological Survey, Circ. No. 65, pp. 11-12, 1941.

1941-10. Krynine, D. P. "Soil Mechanics, Its Principles and Structural Applications." 1st Ed., pp. 77-84. N. Y.: McGraw-Hill Book Co., 1941.

1941-11. Solntzev, D. I., and Sorkov, V. S. "Electrochemical Stabilization as Means of Preventing Ground Failure in Railroads." Conference on Stabilization of Ground and Rock, Research Institute of Communications and Construction, Peoples Commissariat of Communications, Academy of Sciences, U.S.S.R. pp. 67-79, Moscow, 1941. (Translated from the Russian by V. P. Sokoloff, August, 1946. Copy of translation available in Highway Research Board Library).

1941-12. Russell, M. B., and Spangler, M. G. "The Energy Concept of Soil Moisture and Mechanics of Unsaturated Flow." Proceedings, Highway Research Board, Vol. 21, pp. 435-449, 1941. Discussion on Soil Water Phenomena by Bert Meyers, pp. 451-452, and C. A. Hogentogler and E. S. Barber, pp. 452-466.

1941-13. Ivies, R. L. Vegetation indications of solifluction. Journal of Geomorphology, Vol. 4, No. 2, 1941, pp. 128-32.

1941-14. Orvin, Anders K. Hvordan opstar jordbunnsis? Norsk Geografisk Tidsskrift, Bd. 8, Heft 8, 1941, pp. 294-306. (English summary).

1941-15. Romanovsky, Vsevolod. Application du critérium de Lord Rayleigh à la formation des tourbillons convectifs dans les sols polygonaux du Spitzberg. Comptes Rendus Hebdomadaires des Séances de l'Academie des Sciences (Paris), Tom. 213, 1941, pp. 877-78.

1941-16. Wedmore, E. B. Thermal conductivity of soil. Quarterly Journal Royal Meteorological Society, Vol. 67, No. 288, 1941, p. 38.

1941-17. Bliss, D. E., Moore, D. C. and Bream, C. E. "Air and Soil Temperatures in a California Date Garden." Soil Science, Vol. LIII, pp. 55-64, 1941.

1942-1. Slate, F. O. "Use of Calcium Chloride in Subgrade Soils for Frost Prevention." Proceedings, Highway Research Board, Vol. 22, pp. 422-441, 1942.

1942-2. Mersman, W. A., Berrgren, W. P., and Boelter, L. M. K. "The Conduction of Heat in Composite Infinite Solids." University of California, Publications in Engineering, Vol. 5, No. 1, 1942. 22 pp.

1942-3. Schaible, L. "The Repair of Frost Damage." Strasse, 1942, 9(17/18), pp. 173-6. Road Abstracts, Vol. 12, No. 1, Abst. No. 28, p. 6, Jan. 2, 1945. Highway Research Abstracts, No. 118, p. 4, March, 1945.

1942-4. Schmid, A. "Frost Heaving in Soils and Modern Road Construction." Bauingenieur, 1942, 23 (½), 1-6. Road Abstracts, Vol. 10, No. 3, Abst. No. 111, p. 25, 1943. Highway Research Abstracts, No. 99, p. 4, April, 1943.

1942-5. Linde, S. F. "The Freezing of the Foundation Soil under Cold Storage Buildings." Warme u. Kaltetechnik, 1942, 44 (2) 17-23; (3) 34-8, Building Science Abstracts, Vol. 16 (New Ser.), No. 3, Abst. No. 198, p. 41, March, 1943.

1942-6. Geslin, H. "The Rate of Freezing in Soil and its Dependence on the Thickness of the Snow Layer." Comtes Rendus (France), 1942, 214 (3), 124-5. Building Science Abstracts, Vol. 16 (new series), No. 1, Jan., 1943. Highway Research Abstracts, No. 99, p. 8, April, 1943.

1942-7. Casagrande, L. "Lessons From Frost Damage to Roads." Roads and Bridges (Canada), Vol. 80, No. 2, p. 100, Feb., 1942.

1942-8. Smith, W. O. "The Thermal Conductivity of Dry Soil." Soil Science, Vol. 53, No. 6, pp. 435-459, June, 1942.

1942-9. Sharp, R. P. "Ground Ice Mounds in Tundra."

Geographical Review, Vol. 32, No. 3, pp. 417-423, July, 1942.

1942-10. Nunn, E. H. "Subgrade Protection and Winter Paving on Access Roads." Calcium Chloride Association News, Vol. 8, No. 4, pp. 4, 5, 10, Aug. 1, 1942.

1942-11. Ducker, A. "Ueber 'Bodenkolloide' und ihr Verhalten bie Frost" (Soil Colloids and their Behaviour when Subjected to Frost Action). Der Bauingenieur, Vol. 23, pp. 235-237, Aug. 5, 1942. (Translated by H. B. Edwards, Engineer Dept. Research Centers, U. S. Waterways Exp. Sta., Vicksburg, Miss., Dec., 1944).

1942-12. Pennsylvania Road Builder. "Preventing Detrimental Frost Heaving." Pennsylvania Road Builder, Vol. 15, No. 9, pp. 4, 15, Sept., 1942.

1942-13. Engineering and Contract Record. "Inexpensive Treatment for Frost Boils Helps Prevent Costly Pavement Repairs." Engineering and Contract Record, Vol. 55, No. 42, p. 33, Oct. 21, 1942.

1942-14. Anderson, A. B. C., Fletcher, J. E., and Edlefsen, N. E. "Soil-Moisture Conditions and Phenomena in Frozen Soils." Transactions, American Geophysical Union, Pt. 2, pp. 356-371, 1942.

1942-15. Roads and Bridges. "How Can Frost Boils Be Prevented." Roads and Bridges (Canada), Vol. 80, No. 11, pp. 26-46, Nov., 1942.

1942-16. Sharp, R. P. "Periglacial Involutions in Northeastern Illinois." Journal of Geology, Vol. 50, pp. 113-133, 1942 a.

1942-17. Hewes, L. I. "American Highway Practice." Vol. 1, pp. 183-184. N. Y.: John Wiley and Sons, 1942.

1942-18. Science News Letter. "Russia's Frozen Expanses Have Lessons for U. S." Science News Letter, Vol. 42, No. 3, p. 39, July 18, 1942.

1942-19. Keen, B. A. "Soil Physics: Theory and Practice." From Canton Lectures delivered before the Royal Society of Arts Journal of the Royal Society of Arts, Vol. XC, No. 4618, pp. 552-555, July 24, 1942.

1942-20. Cailleux, André. Les actions éoliennes périglaciaires en Europe. Memoires de la Société Geologique de France, No. 41, 1942, pp. 1-176.

1942-21. Sharp, Robert P. Mudflow levees. Journal of Geomorphology, Vol. 5, No. 3, 1942, pp. 222-27.

1942-22. Sharpe, C. F. Stewart, and Dosih, Earl F. The relation of soil creep to earth flow in the Appalachian Plateau. Journal of Geomorphology, Vol. 5, No. 4, 1942, pp. 312-24.

1942-23. Eardley, A. J. "Aerial Photographs: Their Use and Interpretation." Harper, 1942.

1942-24. Romanovsky, Vsevolod. Sur la découverte au Spitzberg, de deux gisements de la glace fossile. Bulletin de l'Association des Géographes Francais, Nos. 146-147, 1942, pp. 55-56.

1942-25. Romanovsky, Vsevolod, and Cailleux, André. Sols polygonaux et fentes de desiccation. Bulletin de la Société Géologique de France, Ser. 5, Tom 12, 1942.

1942-26. Romanovsky, Vsevolod. Observations sur les sols polygonaux des Alpes francaises. Comptes Rendus Sommaires des Séances de la Société Géologique de France, No. 14, 1942.

1942-27. Orvin, Anders K. Om dannelse av strukturmark. Norsk Geografisk Tidsskrift, Bd. 9, Hefte 3, 1942, pp. 105-23, illus., diagrs. (English summary).

1942-28. Gatty, O., and others. Some types of polygonal surface markings in Spitsbergen. By O. Gatty, W. L. S. Fleming and J. M. Edmonds. American Journal of Science, Vol. 240, February 1942, p. 81-92.

1942-29. Sharp, R. P. (1942) Soil structures in the Saint Elias Range, Yukon Territory: Jour. Geomorphology, Vol. 5, pp. 274-301.

1942-30. Griffiths, E. "The Measurement of the Thermal Conductivity of Materials Used in Building Construction." Inst. Heat. Vent. Engr. Journal, Vol. X, No. 112, pp. 106-138, 1942.

1942-31. Dalquest, Walter W. and Scheffer, Victor B. "The Origin of the Mima Mounds of Western Washington." Journal of Geology, Vol. 50, No. 1, January-February, 1942, pp. 68-84.

1943-1. Berggren, W. P. "Prediction of Temperature Distribution in Frozen Soils." Transactions, American Geophysical Union, Pt. 3, pp. 71-77, 1943. Included are: Analysis of an idealized system, solution of examples and special adaptations of the analysis.

1943-2. Ruckli, R. "Die Frostgefahrlichkeit des Strassenuntergrundes." (Frost Damage in Highway Subgrades). Strasse und Verkehr, Vol. 29, No. 19, pp. 311-318; No. 20, pp. 332-9; No. 21, pp. 343-353; No. 22, pp. 357-64; No. 24, pp. 388-96; No. 25, pp. 397-403, Sept. 17 - Dec. 10, 1943. (Translated by H. B. Edwards, Engineer Dept. Research Centers, U. S. Waterways Exp. Sta., Vicksburg, Miss., Aug., 1944).

1943-3. Moos, A., von. "The Part Played by the Foundation in Road Failures." Strasse u. Verkehr, 1943, 29 (1) 1-7; (2), 21-9. Road Abstracts, Vol. 12, No. 3, Abst. No. 118, p. 25, March, 1945.

1943-4. Taber, S. "Perennially Frozen Ground in Alaska: Its Origin and History." Bulletin of the Geological Society of America, Vol. 54, No. 10, pp. 1433-1548, Oct. 1, 1943.

1943-5. Belcher, D. J., Gregg, L. E., and Woods, K. B. "The Formation, Distribution and Engineering Characteristics of Soils." Purdue University, Engineering Bul., (Research Ser. No. 87, Highway Research Bul. No. 10), pp. 125-132, Jan., 1943.

1943-6. Thomas, B. P. "Thawing Frozen Subgrades." Calcium Chloride Association News, Vol. 9, No. 1, pp. 8, 10, Feb. 1, 1943.

1943-7. Slate, F. O. "Use of Calcium Chloride in Subgrade Soils for Frost Prevention." Calcium Chloride Association News, Vol. 9, No. 1, p. 5, Feb. 1, 1943.

1943-8. Roads and Bridges. "Mechanics of Frost Heaving." Roads and Bridges (Canada), Vol. 81, No. 5, pp. 84-85, May, 1943.

1943-9. Slesser, Charles. "The Migration and Effect on Frost Heave of Calcium Chloride and Sodium Chloride in Soil." Purdue University, Engineering Bul., Vol. 27, No. 4, (Research Ser. No. 89, Highway Research Bul. No. 11), July, 1943, 168 pp.

1943-10. Taber, S. "Some Problems of Road Construction and Maintenance in Alaska." Public Roads, Vol. 23, No. 9, pp. 247-251, July-Aug.-Sept., 1943.

1943-11. Arndt, W. J. "Temperature Changes and Duration of High and Low Temperatures in a Concrete Pavement." Proceedings, Highway Research Board, Vol. 23, pp. 273-279, 1943.

1943-12. Hansen, C. C. "Conference on Subgrades and Clay Stabilized Bases." Minnesota Department of Highways, Minneapolis, Minnesota, April 27-30, 1943.

1943-13. Rozanski, G. "Stone-Centered Polygons." Journal of Geology, Vol. 51, No. 5, pp. 330-341, July-Aug., 1943.

1943-14. Terzaghi, K. "Theoretical Soil Mechanics." Pp. 297-308, 330-332. N.Y.: John Wiley and Sons, Inc., 1943.

1943-15. Winterkorn, H. F. "The Condition of Water in Porous Systems." Soil Science, Vol. 56, April 5, 1943.

1943-16. Edlefsen, N. E. and Anderson, A. B. C. "Thermodynamics of Soil Moisture." Hilgardia, Vol. 15, No. 2, pp. 31-298, 1943.

1943-17. Dege, Wilhelm. Über Auxmass und Art der Bewegung arktischer Fliesserde. Zeitschrift für Geomorphologie, Bd. 11, 1943, pp. 318-29.

1943-18. Milon, Y., and Heger, P. Les Polygons de cryoturbation des côtes de Bretagne. Comptes Rendus Sommaires des Séances de la Société Géologique de France, Tom 7, 1943.

1943-19. Romanovsky, Vsevolod. Remarques sur une communication de M.M. Milon et Heger à propos des polygons de cryoturbation des côtes de Bretagne. Comptes Rendus Sommaires des Séances de la Société Géologique de France, No. 9, 1943.

1943-20. Smith, W. O. "Thermal Transfer of Moisture in Soils." Transactions, American Geophysical Union, (1943).

1943-21. Burrill, Meredith F. "Soil Temperature in the Matanuska Valley of Alaska: Observations of 1941." American Geophysical Union Translation, Vol. 1, pp. 151-153.

1943-22. Eardley, A. J. "Aerial Photographs and the Distribution of Constructional Materials." Proceedings, Highway Research Board, 23rd Annual Meeting, November, 1943.

1943-23. Kingery, Robert. "Aerial Mapping Used in Regional Highway Planning." Proceedings, Highway Research Board, 23rd Annual Meeting, November, 1943.

1943-24. Smith, H. T. U. "Aerial Photographs and Their Applications." Appleton Century, 1943.

1943-25. Belcher, D. J. "The Engineering Significance of Soil Patterns." Proceedings, Highway Research Board, Vol. 23, 1943.

1943-26. Hittle, Jean E. "The Use of Aerial Photographs in Identifying Granular Deposits and Other Soils." Proceedings, 29th Annual Purdue Road School, March, 1943.

1943-27. Belcher, D. J. "The Development of Engineering Soil Maps." Proceedings, 29th Annual Purdue Road School, March, 1943.

1943-28. Cooke, C. Wythe. "Elliptical Bays." Journal of Geology, Vol. 51, No. 6, August-September, 1943, pp. 419-427.

1943-29. Portland Cement Association. "Observations of Procedure in Placing P. C. Concrete Pavement During Cold Weather at a Pacific Northwest Air Base." Portland Cement Association, Chicago, 1943.

1944-1. Shannon, W. L. "Frost Investigations at Dow Field, Bangor, Maine." Proceedings, Highway Research Board, Vol. 24, pp. 71-86, 1944.

1944-2. Mamanina, L. "The Effect of Insulating Layers on the Rise of Capillary Moisture in Heavy Loams." Pochvovedenie (Pedology) (Moscow), 1944, (2-3), 101-5. (In Russian with English summary). Road Abstracts, Vol. 13, No. 3, Abst. No. 173, p. 34, March 5, 1946.

1944-3. Calcium Chloride Association News. "Calcium Chloride Treated Roads Guard Against Spring Breakup." Calcium Chloride Association News, Vol. 10, No. 3, pp. 3-5, June, 1944.

1944-4. Public Works Magazine. "Preventing Spring Break-up of Roads." Public Works Magazine, Vol. 75, No. 8, p. 18, Aug., 1944.

1944-5. Engineering News-Record. "The Saga of the Greenland Bases." Engineering News-Record, Vol. 133, No. 10, pp. 96-100, Sept. 7, 1944.

1944-6. Elgar, W. H. "Soil Movements as Affecting Paved Surfaces." Surveyor (London), Vol. 103, No. 2746, pp. 427-8, Sept. 8, 1944.

1944-7. Roads and Streets. "Simple Method of Preventing Spring Breakup of Roads." Roads and Streets, Vol. 87, No. 10, p. 81, Oct., 1944.

1944-8. Collins, A. R. "The Destruction of Concrete by Frost." Journal of the Institution of Civil Engineers (London), Vol. 23, No. 1, pp. 29-41, Nov., 1944. Highway Research Abstracts, No. 116, pp. 5-7, Jan., 1945.

1944-9. Hieronymus, G. "Earth Ambient Temperatures for Loading Limits." Electrical World, Vol. 122,

No. 24, pp. 92-93, Dec. 9, 1944.

1944-10. Lang, F. C. "Progress Report of Committee on Freezeproofing Treatment of Subgrade Soils with Calcium Chloride to Prevent Detrimental Frost Action." Special Committee of the Highway Research Board, 1944, Highway Research Abstracts, No. 118, pp. 9-10, March, 1945.

1944-11. Winterkorn, H. F. and Fehrman, R. G. "The Effect of Freezing - Thawing and Wetting - Drying Cycles on the Density and Bearing Power of Five Soils." Proceedings, Soil Science Society of America, Vol. 9, pp. 248-252, 1944.

1944-12. Troll, C. "Strukturboden, Solifluktion und Frostklimate der Erde." Geologische Rundschau, Band 34, Heft 7/8, pp. 545-694.

1944-13. Budel, J. "Die Morphologischen Wirkungen des Eiszeitklimas im Gletscherfreiem Gebiet." Geologischo Rundschau, Band 34, Heft 7/8, pp. 482-519, 1944.

1944-14. Kersten, M. S. "Survey of Subgrade Moisture Conditions." Proceedings, Highway Research Board, Vol. 24, pp. 497-512, 1944.

1944-15. Lancaster, C. M. "Discussion on Survey of Subgrade Moisture Conditions." Proceedings, Highway Research Board, Vol. 24, pp. 512-513, 1944.

1944-16. Valle-Rodas, P. "Capillarity in Sands." Proceedings, Highway Research Board, Vol. 24, pp. 389-398, 1944.

1944-17. Weinberger, L. "Frostpalten und Frostrukturen in Schottern bei Leipzig." Geol. Rundsch, Vol. 34, pp. 539-544, 1944.

1944-18. Sochava, V. B. "Microrelief Forms in the Tundras near Amur." Academia Nauk, USSR, Priroda, No. 5-6, pp. 107-109, 1944. (Condensed by I. V. Poire, U. S. Geological Survey, 1949).

1944-19. Eager, William L., and Pryor, William T. "Ice Formation Along the Alaska Highway." Proceedings, Vol. 24, Highway Research Board, p. 398, 1944.

1944-20. Cooling, L. F. and Ward, W. H. Damage to cold stores due to frost-heaving. Proceedings Institute of Refrigeration, Vol. 41, 1944-45, p. 37-58.

1944-21. Fedosov, A. Ye. Prognoz osadok sooruzheniy pri ottaivanii gruntov osnovaniy. Prognosis of the settling of buildings when the ground under them thaws. Trudy Instituta Merzlotovedeniya im. V. A. Obrucheva. Transactions of the V. A. Obruchev Institute of Permanently Frozen Soil. (Moscow, Leningrad), Vol. 4, 1944, pp. 93-124. Practical engineering formulae worked out for manner and rate of settling under given loads. English summary.

1944-22. Saltykov, N. I. Fundamenty zdaniy v rayone Bol'shezemelskoy tundry. Foundations of buildings in the Bol'shezemelskaya tundra. Trudy Instituta Merzlotovedeniya im. V. A. Obrucheva. Transactions of the V. A. Obruchev Institute of Permanently Frozen Soil. (Moscow, Leningrad), Vol. 4, 1944, p. 125-204. Based on experimental construction in area; mathematical analysis of thermal processes under foundations, types of deformation in buildings caused by thaw, and advice on choice and depth of foundations. English summary.

1944-23. Tavernier, R. Phénomènes périglaciaires en Belgique. Bulletin de la Société Belge des Etudes Géographiques, Tom. 14, No. 2, 1944-45, pp. 112-33.

1944-25. Tsyplenkin, Ye. I. Vechnaya merzlota i yeye agronomicheskoye znachenie. Permanently frozen soil and its agricultural significance. Trudy Institute Merzlotovedeniya im. V. A. Obrucheva. Transactions of the V. A. Obruchev Institute of Permanently Frozen Soil. (Moscow, Leningrad), Vol. 4, 1944, pp. 230-55. Analysis of intensity of permanent frost and its reaction on different types of soil; factors affecting choice of land for agriculture. English summary.

1944-26. Yefimov, A. I. Glukoye promerzanie gruntov i rezhim nadmerzlotnykh vod pod teplymi zdaniyami. (Deep freezing of soils and the regime of water lying on permanently frozen ground under heated buildings.) Trudy Institute Merzlotovedeniya im. V. A. Obrucheva. Transactions of the V. A. Obruchev Institute of Permanently Frozen Soil, (Moscow, Leningrad), Vol. 4, 1944, p. 205-25. Experimental work on 2 building sites in Zabaykal'ya, carried out 1941-43, with a view to controlling upthrust of water under buildings. English summary.

1944-27. Winterkorn, Hans F. "Climate and Highways." Trans. of the American Geophysical Union.

1944-28. Belcher, D. J. "Identifying Land Forms and Soils by Aerial Photographs." Proceedings, 30th Annual Purdue Road School, March, 1944.

1944-29. Johnson, Douglas. "Mysterious Craters of the Carolina Coast." American Scientist, Vol. 32, No. 1, January, 1944.

1945-1. Shannon, W. L. "Prediction of Frost Penetration." Journal, New England Water Works Association, Vol. 59, No. 4, pp. 356-363, Dec., 1945.

1945-2. Corps of Engineers. "Construction of Runways, Roads and Buildings on Permanently Frozen Ground." War Dept., Technical Bul. TB 5-255-3, Jan., 1945. 64 pp.

1945-3. Eager, W. L. and Pryor, W. T. "Ice Formation on the Alaska Highway." Public Roads, Vol. 24, No. 3, pp. 55-74, Jan.-Feb.-March, 1945.

1945-4. Muller, S. W. "Permafrost or Permanently Frozen Ground and Related Engineering Problems." U. S. Geological Survey, Special Report, Strategic Engineering Study No. 62, Prepared for Military Intelligence Division, Office, Chief of Engineers, U. S. Army, 1945, 2nd printing. Ann Arbor, Mich.: J. W. Edwards, Inc., 1947. 231 pp. Reprint.

1945-5. Williams, A. M. "What Causes the Spring Break-up?" Better Roads, Vol. 15, No. 11, pp. 22-23, Nov., 1945.

1945-6. Gardner, R. "Some Effects of Freezing and Thawing on the Aggregation and Permeability of Dispersed Soils." Soil Science, Vol. 60, No. 6, pp. 437-443, Dec., 1945.

1945-7. Swanberg, J. H. "Temperature Variations in a Concrete Pavement and the Underlying Subgrade." Proceedings, Highway Research Board, Vol. 25, pp. 169-180, 1945.

1945-8. Allen, Harold. "Report of Committee on Warping of Concrete Pavements." Proceedings, Highway Research Board, Vol. 25, pp. 199-250, 1945.

1945-9. American Air Force. Aviation Engineer School. "Soil Familiarization." pp. 62-68, 1945.

1945-10. Middlebrooks, T. A. "Classification of Materials for Subgrades for Airfields and Granular Type Roads." Proceedings, Highway Research Board, Vol. 25, pp. 384-386, 1945.

1945-11. Huntting, M. T. "Geology in Highway Engineering." Transactions, American Society of Civil Engineers, Vol. 110, pp. 278-280, 1945.

1945-12. Kersten, M. S. "Subgrade Moisture Conditions Beneath Airport Pavements." Proceedings, Highway Research Board, Vol. 26, pp. 450-463, 1945.

1945-13. Stratton, J. H. "Military Airfields: A Symposium. Construction and Design Problems, Design of Drainage Facilities." Transactions, American Society of Civil Engineers, pp. 684, 764, 778, 781, 783, 1945.

1945-14. Obruchev, V. "Eternal Frost: The Frozen Soil of Northern Russia and Siberia." National Review, Vol. 124, pp. 220-227, March, 1945.

1945-15. Carter, D. "Defrosting the Frozen North: Report on Institute of Eternally Frozen Soil." Science Digest, Vol. 17, pp. 4-8, May, 1945.

1945-16. Hank, R. J. "Subgrade Soil Moisture Contents." Unpublished Data on Subgrade Moisture Contents in Texas. Submitted to the Chairman, Division I, Committee on Materials, American Association of State Highway Officials, May 18, 1945.

1945-17. Corps of Engineers. "Frost Investigation, 1944-1945. Report on Laboratory Tests on Frost Penetration and Thermal Conductivity of Cohesive Soils." Corps of Engineers, U. S. Engineer Office, Boston, Mass., (Appendix 13, to Comprehensive Report). June, 1945.

1945-18. Gardner, R. "Some Effects of Freezing and Thawing on the Aggregation and Permeability of Dispersed Soils." Soil Science, Vol. 60, pp. 437-444, December, 1945.

1945-19. Winterkorn, H. F. and Eyring, H. "Theoretical Aspects of Water Accumulation in Cohesive Subgrade Soils." Proceedings, Highway Research Board, Pp. 422-434, 1946

1945-20. Spangler, M. G. "Some Problems in Subgrade Moisture Control." Proceedings, Highway Research Board, Vol. 26, pp. 434-450, 1946.

1945-21. U.S.S.R. Akademiya Nauk. Referaty nauchno-issledovatel'skikh rabot za 1944 god. Otdelenie geologo-geograficheskikh nauk. Papers on scientific research work for 1944. Department of geological and geographical sciences. (Leningrad, Moscow, Izdatel'stvo Akademii Nauk SSSR (Publishing House of the Academy of Sciences of the U.S.S.R., 1945, 146 pp., 21½ cm. Short resume of the work of geological and geographical institutes including Obruchev Institute of Permanently Frozen Soil, together with summary of publications of their members.

1945-22. Brudal, H. "Telefrie Veger." (Frost free roads), Statens Vegvesen, Veglaboratoriet, Oslo (Norway), 1945.

1945-23. Dill, Robinson and Robinson. "Measurements of Heat Losses from Slab Floors." Report BMS 103, Building, Materials and Structures, Nat. Bureau of Standards, March, 1945.

1945-24. Hadfield, A. H. Soil Classification and Evaluation of Subgrade Supporting Power for Airfields. Proceedings, Highway Research Board, Vol. 25, pp. 386-388, 1945.

1946-1. Smith, H. W. "Frost Damage to Sealed Surfaces in Southland." Proceedings, New Zealand Institute of Engineers, Vol. 32, pp. 570-594, 1945-1946. Highway Research Abstracts, No. 138, pp. 2-4, Feb., 1947.

1946-2. Hardy, R. M. "Permanently Frozen Ground and Foundation Design, Part 1." Engineering Journal (Canada), Vol. 29, No. 1, pp. 4-7, Jan., 1946.

1946-3. D'Appolonia, E. "Permanently Frozen Ground and Foundation Design, Part 2." Engineering Journal (Canada), Vol. 29, No. 1, pp. 7-12, Jan., 1946.

1946-4. Jankins, D. S., Belcher, D. J., Gregg, L. E., and Woods, K. B. "The Origin, Distribution and Airphoto Identification of United States Soils, with Special Reference to Airport and Highway Engineering." Technical Development Report No. 52, U. S. Dept. of Commerce, Civil Aeronautics Administration, May, 1946. 202 pp. & 63 plates.

1946-5. Corps of Engineers. "Airfield Pavement Design: Frost Conditions." Ad Interim Engineering Manual for War Department Construction, Pt. 12, Chap. 4, U. S. War Dept., Corps of Engineers, Office of Chief of Engineers, July, 1946. 10 pp.

1946-6. Hardy, R. M. "Research on Frost Action in Soils." Roads and Bridges (Canada), Vol. 84, No. 9, pp. 74-77, 102, Sept., 1946.

1946-7. Corps of Engineers. "Airfield Pavement Design: Construction of Airfields on Permanently Frozen Ground." Engineering Manual for War Department Construction, Pt. 12, Chap. 7, U. S. War Dept., Corps of Engineers, Office of Chief of Engineers, Oct., 1946. 20 pp.

1946-8. Lane, K. S. and Washburn, D. E. "Capillarity Tests by Capillarimeter and by Soil Filled Tubes." Proceedings, Highway Research Board, Vol. 26, pp. 460-473, 1946.

1946-9. Colman, E. A. "The Place of Soil Moisture Meters in Hydrologic Research." Transactions, American Geophysical Union, Vol. 27, No. 6, pp. 847-853, Dec. 1946.

1946-10. Corps of Engineers. "Subsurface Drainage

Investigation 1945-1946." Corps of Engineers, U. S. Army, U. S. Engineer Office, Boston, Mass., Sept. 1946.

1946-11. Bryan, K. "Cryopedology - the Study of Frozen Ground and Intensive Frost Action with Suggestions on Nomenclature." American Journal of Science, Vol. 244, Part 2, pp. 622-642, 1946.

1946-12. Benkelman, A. C. and Olmstead, F. R. "The Cooperative Project on Structural Design of Non-Rigid Pavements." Proceedings, Highway Research Board, Vol. 26, pp. 13-25, 1946.

1946-13. Clay Sewer Pipe Association, Inc. "Clay Pipe Engineering Manual." Pp. 129-130. Columbus, O.: The Association, 1946.

1946-14. Swanberg, J. H. and Hansen, C. C. "Development of a Procedure for the Design of Flexible Bases." Proceedings, Highway Research Board, Vol. 26, pp. 44-57, 1946.

1946-15. Dementiev, A. I., and Tumel, V. F. "Civil Engineering in Frozen Soil, U.S.S.R." Canadian Geographical Journal, Vol. 32, No. 1, pp. 32-33, Jan., 1946.

1946-16. Michigan State Highway Department. "Field Manual of Soil Engineering." Rev. Ed., pp. 64, 77-82, 106-109. Lansing, Mich.: The Dept., Feb., 1946.

1946-17. Lane, A. C. "Northern Climatic Variations Affecting Geotherm Initial." Canadian Mining and Metallurgical Bulletin, No. 411, pp. 397-402, July 1946.

1946-18. Popular Science Monthly. "Revolt of the Earth; Permafrost Tests Ingenuity of Arctic Builders." Popular Science Monthly, Vol. 149, pp. 126-129, Nov., 1946.

1946-19. U. S. War Department. "Drainage and Erosion Control, Subsurface Drainage Facilities for Airfields." Engineering Manual for War Department Construction, Part 13, Chapter 2, War Department, Corps of Engineers. July, 1946.

1946-20. Krynine, D. P. "Some Experiments on Capillary Flow of Moisture Through Gravels and Silts." Proceedings, Highway Research Board, Vol. 26, pp. 474-483, 1946.

1946-21. Chekotillo, A. M. Bor'ba s naledyami. The fight against ice-mounds. Priroda (Nature), No. 1, 1946, pp. 20-28. Explanation with sketches of several ways of preventing formation of ice-mounds which would otherwise damage buildings or roads. Copy in School of Agriculture Library, Cambridge.

1946-22. Conrad, V. Polygon nets and their physical development. American Journal of Science, April, 1946.

1946-23. Andrianov, P. I. Svyazannaya voda pochv i gruntov. Soil moisture. Trudy Institute Merzlotovedeniya im. V. A. Obrucheva Transactions of the V. A. Obruchev Institute for the Study of Permanently Frozen Soil, (Moscow, Leningrad) Tom 3, 1946, p. 5-138. Summary of present (c.1940) state of knowledge on behaviour and characteristics of water content of soils, intended as basis for study of properties of permanently frozen soil. Bibliography, English summary. Copy in Science Section. Society for Cultural Relations with USSR, London.

1946-24. Dement'yev, A. I. and Tumel', V. F. Civil engineering in frozen soil, USSR. Canadian Geographical Journal, Vol. 32, No. 1, 1946, pp. 32-33. Notes on the work of the Soviet Institute for the Study of Permanently Frozen Soil (Institut Merzlotovedeniya im. V. A. Obrucheva).

1946-25. Jahn, Alfred. Badania nad struktura i temperatura gleb w zachodniej Grenlandii. Rozprawy Wydzialu Matemat.-przyrodn. Polskiej Adakemii Umiejetności. Ogólnego zbioru, Tom 72, Dzial A. (Seria 3, Tom 32), 1946, Nr. 6, (published 1948), 121 pp.

1946-26. Nörvang, A. Nogle forekomster af arktisk strukturmark (Brodelboden) bevarede i danske istids aflejringer. Danmarks Geologiske Undersøgelse, 2 2 Raekke, Nr. 74, 1946.

1946-27. Zhukov, V. F. Zemlyanye raboty pri ustroystve fundamentov i osnovaniy v oblasti vechnoy merzloty. Treatment of ground for the construction of beds and foundations in the permanently frozen soil area. Moscow, Leningrad, Izdatel'stvo Akademii Nauk SSSR Publishing House of the Academy of Sciences of the USSR, 1946, 142 pp., illus., 22 cm.

1946-28. Bryan, Kirk. Permanently frozen ground. Military Engineer, Vol. 38, No. 246, 1946, p. 168.

1946-29. Barnes, Lynn C. Permafrost: a challenge to engineers. Military Engineer, Vol. 38, No. 243, 1946, p. 9-11.

1946-30. Streiff-Becker, R. Strukturboden in den Alpen. Geographica Helvetica, 1, Bd. 1, 1946.

1946-31. U. S. Engineer Office, Boston, Massachusetts. "Frost Investigation 1945-46." Report of Studies of Base Course Treatment to prevent Frost Action, June, 1946.

1946-32. Roads and Streets. "Airfield Load Tests in Progress at Selfridge Field." Roads and Streets, Vol. 89, No. 4, pp. 94-95, May, 1946.

1946-33. U. S. Engineer Office, Detroit, Michigan. "Frost Investigation 1945-1946." Report on Frost Investigations and Traffic Tests, Selfridge Field, Michigan, June, 1946.

1946-34. White, R. G. "Installations for Noting the Water and Thermal Relationships in Soil." Agr. Eng., Vol. 37, pp. 21-25.

1946-35. Wallace, Robert E. "Cave-in or Thermokarst Lakes in the Nebesna, Chisana, and Tanana River Valley, Eastern Alaska." U. S. Geological Survey, Progress Report No. 4, Permafrost Program, 1946.

1946-36. Hittle, Jean E. "The Application of Aerial Strip Photography to Highways and Airport Engineering." Proceedings, 26th Annual Meeting, Highway Research Board, December, 1946.

1946-37. Frost, Robert E. "The Use of Aerial Maps in Soil Studies and Location of Borrow Pits."

Proceedings, Kansas Highway Engineering Conference, July 1, 1946.

1946-38. Frost, Robert E. "How Can a Highway Department Use Aerial Photographs." Proceedings, Mississippi Valley Highway Conference, March, 1949.

1946-39. Parvis, Merle. "Development of Drainage Maps from Aerial Photographs." Proceedings, 26th Annual Meeting, Highway Research Board, December, 1946.

1947-1. Woods, K. B. "Report of Committee on Frost Heave and Frost Action in Soil." Presented at the 27th Annual Meeting of the Highway Research Board, Dec., 1947.

1947-2. Corps of Engineers. "Report on Frost Investigation 1944-1945." New England Division, Corps of Engineers, U. S. War Dept., Boston, Mass., April, 1947. 66 pp. with 15 appendices.

1947-3. Bouyoucos, G. J. "A New Electrical Resistance Thermometer for Soils." Soil Science, Vol. 63, No. 4, pp. 291-298, April, 1947. Highway Research Abstracts, No. 141, p. 4, May, 1947.

1947-4. Better Roads. "Holding Down Spring Thaw Damage." Better Roads, Vol. 17, No. 4, pp. 32-34, April, 1947.

1947-5. Marsh, L. L. "Holding Down Spring Thaw Damage." Better Roads, Vol. 17, No. 5, p. 26, May, 1947.

1947-6. Bouyoucos, G. J. and Mick, A. H. "Improvements in the Plaster of Paris Absorption Block Electrical Resistance Method for Measuring Soil Moisture Under Field Conditions." Soil Science, Vol. 63, No. 6, pp. 455-465, June, 1947. Highway Research Abstracts, Vol. 17, No. 8, p. 5, Sept., 1947.

1947-7. Public Works Magazine. "Minimizing Spring Thaw Damage." Public Works Magazine, Vol. 78, No. 6, pp. 55-56, June, 1947.

1947-8. Root, W. H. "Vertical Drainage" from Highway Maintenance Performance Items - Summer 1947. An unpublished report of highway maintenance items assembled by Public Roads Administration.

1947-9. U. S. Weather Bureau. "Average and Extreme Depth of Frost Penetration." U. S. Weather Bureau. Undated maps.

1947-10. Shannon, W. L. and Wells, W. A. "Tests for Thermal Diffusivity of Granular Materials." Proceedings, American Society for Testing Materials, Vol. 47, pp. 1044-1055, 1947.

1947-11. Johnson, A. W. "Frost Action in Subgrades and Bases; Icing; and Anchor Ice. Proceedings, 28th Annual Convention, Canadian Good Roads Association, pp. 13-40, 1947. Also in Roads and Bridges (Canada), Vol. 85, No. 9, pp. 104-110, 146-150, Sept., 1947; Engineering and Contract Record. Vol. 60, No. 11, pp. 66-83, Nov., 1947.

1947-12. Beskow, G. "Soil Freezing and Frost Heaving with Special Application to Roads and Railroads." Swedish Geological Society, Ser. C, No. 375, 26th Yearbook, No. 3, 1935. With a special supplement for the English translation of progress from 1935 to 1946. Evanston, Ill.: Northwestern University, Technological Institute, Nov., 1947. (Translated by J. O. Osterberg).

1947-13. Motl, C. L. "Study of Laws and Practices Applying to Special Load Limitations to Prevent Pavement Damage." Progress Report of Subcommittee No. 2, Maintenance Committee, American Association of State Highway Officials, 1947.

1947-14. Muller, S. W. "Permafrost or Permanently Frozen Ground and Related Engineering Problems." Ann Arbor, Mich.: J. W. Edwards, Inc., 1947. 231 pp.

1947-15. Shelburne, T. E. "Subgrade Soil Practices, Report of Committee on Concrete Pavement Design." American Road Builders' Association, Technical Bul. No. 121, pp. 14-23, 1947.

1947-16. Radzikowski, H. A. "Apply Climatic Data? Naturally!" Better Roads, Vol. 17, No. 7, pp. 33-36, July, 1947.

1947-17. Hughes, A. C. "Frost Damage to Roads on a Chalk Subsoil." Surveyor (London), Vol. 106, No. 2909, pp. 581-582, Nov. 7, 1947.

1947-18. Palmer, L. A. and Thompson, J. B. "Pavement Evaluation by Loading Tests at Naval and Marine Corps Air Stations." Proceedings, Highway Research Board, Vol. 27, pp. 125-143, 1947.

1947-19. Washburn, A. L. "Reconnaissance Geology of Portions of Victoria Island and Adjacent Regions." Arctic Alaska, Geol. Soc. Am., Mem. 22, pp. 84-103, 1947.

1947-20. Beskow, G. "Scandinavian Soil Frost Research of the Past Decade." Proceedings, Highway Research Board, Vol. 27, pp. 372-383, 1947.

1947-21. Science Progress. "Permafrost" Federal Science Progress, Vol. 1, No. 4, pp. 8-11, May, 1947.

1947-22. McLeod, N. W. "Airport Runway Evaluation in Canada." Highway Research Board, Research Report, No. 4-B, p. 132, 1947.

1947-23. (See No. 1947-20) (Duplication)

1947-24. Muller, Siemon William, Comp. Permafrost, or permanently frozen ground, and related engineering problems. Ann Arbor, Mich., J. W. Edwards, Inc., 1947. x, 231 pp., illus., maps, diagrs., 22½ cm. First prepared by U. S. Geological Survey, and issued as U. S. Army, Office of the Chief of Engineers, Strategic Engineering Study No. 62, 1943; second printing, with corrections and additions, 1945; reprint of second printing, 1947.

1947-25. Cailleux, André, and Hupé, P. Présence des sols polygonaux et striés dans les Pyrénées francaises. Comptes Rendus Hebdomadaires des Séances de l'Académie des Sciences (Paris), Tom. 225, 1947.

1947-26. Cailleux, André and Michaut, Jean. Rapport sur les études cryopédologiques entreprises en haute

montagne en 1947. Paris, Centre National de la Recherche Scientifique, 1947.

1947-27. Dement'yev, A. I. O klassifikatsii vechnoy merzloty so stroitel'noy tochki zreniya. On the classification of permanently frozen soil from the construction point of view. Merzlotovedenie. Study of Permanently Frozen Soil (Moscow), Vol. 2, No. 1, 1947, pp. 55-57. Present system of classifying conditions of permanently frozen soil, with comment urging further classification, especially into "firm" and "not firm." Copy in Foreign Office Research Department, London.

1947-28. Jahn, Alfred. Przyczynek do znajomosci stozków usypiskowych w krajach polarnych (Note of the talus in the polar regions). Przegladu Geograficznego, Tom 21, 1947, 10 p. (Reprint; presumably paged separately. English summary.)

1947-29. Troll, Carl. Die formen der Solifluktion und die periglaziale Bodenabtragung. Erdkunde, Bd. 1, Lfg. 4/6, 1947, pp. 162-75.

1947-30. Berezantsev, V. G. O prochnosti merzlykh gruntov pod fundamentami sooruzheniy. On the stability of permanently frozen soil under the foundations of buildings. Merzlotovedenie. Study of Permanently Frozen Soil, (Moscow), Vol. 2, No. 1, 1947, pp. 48-54.

1947-31. Moskvitin, A. I. O sledakh merzloty i neobkhodimosti ikh raspoznavaniya. On traces of permanently frozen soil and the necessity of identifying them. Merzlotovedenie. Study of Permanently Frozen Soil, (Moscow), Vol. 2, No. 1, 1947, pp. 3-22. Existence of earlier presence of permanently frozen soil recognized by evidence of solifluction, of certain soil structures, and of remains of fossil ice wedges; examples in USSR and Germany quoted. Copy in Foreign Office Research Department, London.

1947-32. U. S. Engineer Office, Boston, Massachusetts. "Frost Investigation 1946-1947." Report of studies of base course treatment to prevent frost action. August, 1947.

1947-33. Mollard, J. D. "Airphoto Mapping of Montgomery County Soils for Engineering Purposes." Proceedings, 33rd Annual Purdue Road School, Purdue University, July, 1947.

1947-34. Cabot, Edward C. "The Northern Alaskan Coastal Plain Interpreted from Aerial Photographs." Geographical Review, Vol. 37, 1947.

1947-35. Krinitzsky, E. L. "Origin of Pimple Mounds." American Journal of Science, October, 1947, Vol. 247, pp. 706-714.

1947-36. Scheffer, V. B. "The Mystery of Mima Mounds." Scientific Monthly, Vol. 65, 1947, pp. 283-294.

1947-37. Casagrande, A. Classification and Identification of Soils, Proceedings, Am. Soc. C. E., Vol. 73, No. 6, pp. 783-810, June, 1947.

1948-1. Bryan, K. "The Study of Permanently Frozen Ground and Intensive Frost Action." Military Engineer, Vol. 40, No. 273, pp. 304-305, July, 1948.

1948-2. Calcium Chloride Association News. "News Item." Calcium Chloride Association News, Vol. 14, No. 5, pp. 10, Fall, 1948.

1948-3. Cailleux, A. and Thellier, E. "Determining the Thickness of Frozen Soil." Comptes Rendus, 1947, 224 (16), 1174-5, Building Science Abstracts, Vol. XXI, (New Series No. 3, March 1948). Highway Research Abstracts, Vol. 18, No. 8, p. 9, Sept., 1948.

1948-4. Lewin, J. D. "Essentials of Foundation Design in Permafrost." Public Works Magazine, Vol. 79, No. 3, pp. 27-30, March, 1948.

1948-5. Lewin, J. D. "Dams in Permafrost." Public Works Magazine, Vol. 79, No. 5, pp. 22-23, 32, May, 1948.

1948-6. Lewin, J. D. "Dams in Permafrost." Public Works Magazine, Vol. 79, No. 6, pp. 33-34, June, 1948.

1948-7. Hursch, C. R. "Climatic Factors Controlling Design." Highway Research Board, Roadside Development Committee Reports and Papers, 28th mtg., 1948. Published Sept. 1949.

1948-8. Johnson, A. W. "Effect of Climate on the Behaviour of Subgrade Soils." Proceedings, University of Utah, 9th Annual Highway Engineering Conference Eng. Exp. Sta. Bul. No. 38, pp. 129-166, March, 1948.

1948-9. U. S. Forest Service. "Manual of Instructions for the Use of the Fiberglas Soil-Moisture Instrument." California Forest and Range Experiment Station, Berkeley, Calif., Oct. 1947, Revised Oct., 1948.

1948-10. Missouri State Highway Commission. "Soils Manual." Pp. 130-133. Jefferson City, Mo.: The Commission, 1948.

1948-11. Motl, C. L. "Report of Committee on Load Carrying Capacity of Roads as Affected by Frost Action." Proceedings, Highway Research Board, Vol. 28, pp. 273-280, 1948.

1948-12. Terzaghi, K. and Peck, R. B. "Soil Mechanics in Engineering Practice." Pp. 120, 318, 322, 327, 408. N. Y.: John Wiley and Sons, Inc., 1948.

1948-13. Civil Aeronautics Administration. "Airport Paving." U. S. Department of Commerce, May, 1948.

1948-14. Lewin, J. D. "Essentials of Foundation Design in Permafrost." Public Works, Vol. 79, No. 2, pp. 28-30, Feb., 1948.

1948-15. Geoltz, K. K. "Spring Scuttles a 25-Year Formula." Better Roads, Vol. 18, No. 6, pp. 23-24, 46, June, 1948.

1948-16. Construction Methods. "Working Frozen Ground in the Arctic." Construction Methods, Vol. 30, No. 4, pp. 92-95, April, 1948.

1948-17. Wilson, W. K., Jr. "Problem of Permafrost." Military Engineer, Vol. 40, No. 270, pp. 162-164,

April, 1948.

1948-18. Bryan, K. "Study of Permanently Frozen Ground and Intensive Frost-Action." Military Engineer, Vol. 40, No. 273, pp. 304-308, July, 1948.

1948-19. Woods, K. B. "Report of Committee on Frost Heave and Frost Action in Soil." Highway Research Board, Bul. No. 14, pp. 1-11, Oct., 1948.

1948-20. Constructor. "Effects of Freezing on Virginia Highways." Constructor, Vol. 30, No. 11, p. 47, Nov., 1948.

1948-21. Barron, Reginald A. "Viscous Flow Tube Model." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, pp. 209-211, 1948.

1948-22. Clare, K. E. "Some Laboratory Experiments in the Waterproofing of Soils." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 230-236, 1948.

1948-23. Godskesen, O. "Security against Frost Heave Can Often Be Found in Half the Frost Free Depth." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, p. 274, 1948.

1948-24. Lane, Kenneth S. "Treatment of Frost Sloughing Slopes." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, pp. 281-283, 1948.

1948-25. Riis, Axel. "Frost Damage to Roads in Denmark." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 287-291, 1948.

1948-26. Ruckli, Robert F. X. "Heat-Flow Toward the Floor of Cold Stores, Situated in the Ground Level, and Calculation of the Insulation or the Heating System to Prevent Frost Penetration in the Ground." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 18-22, 1948.

1948-27. Ruckli, Robert F. X. "Two- and Three-Dimensional Ground Water Flow Toward the Ice-Lenses Formed in the Freezing Ground." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 282-287, 1948.

1948-28. Smith, Charles W. "Freeze Damage." Short Course in Highway Engineering, Texas A & M College, Eng. Exp. Sta., Bul. No. 105, pp. 43-49, 1948.

1948-29. Otis, Paul H. "Load Carrying Studies of New Hampshire Highways Started." New Hampshire Highways, Vol. 3, No. 1, pp. 4-5, June, 1948.

1948-30. Belcher, D. J. "The Determination of Soil Conductivity by Aerial Photographic Analysis." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 313-321, 1948.

1948-31. Woods, K. B., Hittle, J. E., and Frost, R. E. "Correlation Between Permafrost and the Soils as Indicated by Aerial Photographs." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 321-324, 1948. Also in Military Engineer, Vol. 40, No. 277, pp. 497-499, Nov., 1948.

1948-32. Cooling, L. F. and Ward, W. H. "Some Examples of Foundation Movements due to Causes other Than Structural Roads." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 162-167, 1948.

1948-33. Kersten, M. S. "Thermal Conductivity of Soils." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, pp. 12-15, 1948.

1948-34. Krynine, D. P. "Analysis of the Latest American Tests on Soil Capillarity." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, pp. 100-104, 1948.

1948-35. Kersten, M. S. "Specific Heat Tests on Soils." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, pp. 158-162, 1948.

1948-36. Kersten, M. S. "Apparatus for Measuring Thermal Conductivity of Soil." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, pp. 162-165, 1948.

1948-37. Carlson, H. "Stability of Foundations in Permanently Frozen Ground." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 4, pp. 51-62, 1948.

1948-38. Hemstock, R. A. "Engineering in Permafrost in Canada's McKenzie Valley." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 4, pp. 62-64, 1948.

1948-39. Russell, I. E., and Olinger, D. J. "Wyoming Methods for Sampling Pavement Design." Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 5, pp. 165-168, 1948.

1948-40. Belcher, D. J. "The Engineering Significance of Landforms." Highway Research Board, Bull. No. 13, pp. 9-29, 1948.

1948-41. Kersten, M. S. "Determination of Thermal Properties of Soils for Investigation of Airfield Construction in Arctic and Sub-arctic Regions." Department of the Army, Corps of Engineers, St. Paul District Engineering Experiment Station, University of Minnesota, January, 1948.

1948-42. Civil Aeronautics Administration. Standard Specifications for Construction of Airports. U. S. Department of Commerce, pp. 65-70, 131-175, January, 1948.

1948-43. Bleck, A. T. "Pavements and Influences Affecting or Determining Their Performance." An unpublished report, 95 pp. 1948. (Reference 1949-11 is a condensation of the above report).

1948-44. Corps of Engineers. "Identification and Evaluation of Alaskan Vegetation from Airphotos with Reference to Soil, Moisture, and Permafrost Conditions." June, 1948.

1948-45. Highway Research Board, "Bibliography on Frost Action in Soils." (Annotated) Bibliography No. 3, Highway Research Board, 1948.

1948-46. Carothers, H. P. "Freeze Damage in Flexible Pavements." Texas Highway Department, Road Design Circular, No. 11-48, November 15, 1948. (A special report reviewed with permission of the Texas Highway Department).

1948-47. Flint, Richard Foster. Glacial geology and geomorphology. (In Boyd, Louise A. The coast of northeast Greenland) American Geographical Society, Special Publication No. 30, 1948, pp. 91-210, illus., tables, diagrs.

1948-48. Cailleux, André. Études de cryopédologie. Paris, Centre de Documentation Universitaire, Tournier & Constans, 1948. 68 pp. illus. (Publications des Expeditions Polaires Francaises, No. 3). Comprehensive account based mainly on western European and American studies.

1948-49. Troll, Carl. Der subnivale oder periglaziale Zyklus der Denudation. Endkunde, Bd. 2, Lfg. 1/3, 1948, pp. 1-21.

1948-50. Woods, K. B. and others. Use of aerial photographs in the correlation between permafrost and soils. Military Engineer, Vol. 40, No. 277, 1948, pp. 497-99.

1948-51. Frost, Robert E. and Woods, K. B. "Airphoto Patterns of Soils of the United States." Civil Aeronautics Administration, Technical Development Report No. 85, August, 1948.

1948-52. Hicks, L. D. "Observations of Moisture Contents and Densities of Soil Type Bases and Their Subgrades." Proceedings, Highway Research Board, Vol. 28, pp. 422-432, 1948.

1948-53. Frost, Robert E., Hittle, Jean E., and Woods, K. B. "Summary and Statement of Technique Aerial Photographic Reconnaissance Investigation Frozen Soils in Territory of Alaska." Engineering Experiment Station, Purdue Univ., Appendix No. 3 of "Comprehensive Report Investigation of Airfield Construction in Arctic and Subarctic Regions, Corps of Engineers, St. Paul District, 2nd Revision, May, 1948.

1948-54. Bareither, H. D., Fleming, A. N. and Alberty, B. E. "Temperature and Heat Loss Characteristics of Concrete Floors Laid on the Ground." Board of Trade, Technical Information and Documents Unit, London.

1948-55. Baver, L. D. "Soil Physics." Chapter 8, pp. 288-310, John Wiley and Sons, New York, 1948.

1948-56. Koons, Frederick C. "The Sand Mounds of Louisiana and Texas." Scientific Monthly, Vol. 66, April, 1948, pp. 297-300.

1948-57. Pewe, Troy L. "Origin of the Mima Mounds." Scientific Monthly, Vol. 66, p. 293, April, 1948.

1948-58. Spurr, Stephen, H. "Aerial Photographs in Forestry." Renald Press Co., 1948.

1948-59. Frost, R. E., and Woods, K. B. "Aerial Photographs Used as an Engineering Evaluation of Soil Materials." Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, Rotterdam, June, 1948.

1948-60. Stoeckeler, E. G. "Identification and Evaluation of Alaskan Vegetation from Airphotos with Reference to Soil, Moisture, and Permafrost Conditions." A preliminary paper, Corps of Engineers, St. Paul District, June, 1948.

1948-61. Wallace, R. E. "Cave-in Lakes in the Nabesna, Chisana and Tanana River Valleys, Eastern Alaska." Journal of Geology, Vol. 56, No. 3, 1948.

1948-62. Taylor, D. W. "Fundamentals of Soil Mechanics." John Wiley and Sons, N. Y., 1948.

1949-1. Black, R. F. and Barksdale, W. L. "Oriented Lakes of Northern Alaska." Journal of Geology, Vol. 57, No. 2, pp. 105-118, March, 1949.

1949-2. Hopkins, D. M. "Thaw Lakes and Thaw Sinks in the Imuruk Lake Area, Seward, Peninsula, Alaska." Journal of Geology, Vol. 57, No. 2, pp. 119-131, March, 1949.

1949-3. Horberg, L. "A Possible Fossil Ice Wedge in Bureau County, Illinois." Journal of Geology, Vol. 57, No. 2, pp. 132-136, March, 1949.

1949-4. Richmond, G. M. "Stone Nets, Stone Stripes, and Soil Stripes in the Wind River Mountains, Wyoming." Journal of Geology, Vol. 57, No. 2, pp. 143-153, March, 1949.

1949-5. Schafer, J. P. Some Periglacial Features in Central Montana." Journal of Geology, Vol. 57, No. 2, pp. 154-174, March, 1949.

1949-6. Smith, H. T. U. "Periglacial Features in the Driftless Area of Southern Wisconsin." Journal of Geology, Vol. 57, No. 2, pp. 196-215, March, 1949.

1949-7. Slater, C. S. and Hopp, H. "The Action of Frost on the Water Stability of Soils." Journal of Agricultural Research, Vol. 78, No. 10, pp. 341-346, May 15, 1949.

1949-8. Shelburne, T. E. and Maner, A. W. "Analysis of Spring Break-up Data in Virginia." Highway Research Board, Bul. No. 20, pp. 1-20, Sept., 1949.

1949-9. Highway Research Board. "Thickness of Flexible Pavement for Highways and Airports." Current Road Problems, No. 8-R, 1949.

1949-10. Sloane, R. L. "Results of Some Tests on Freeze and Thaw of Highly Compacted Clay Soils." Presented at the 10th Annual Highway Engineering Conference, University of Utah, March, 1949.

1949-11. Bleck, A. T. "Pavements and Their Influences Affecting or Determining Their Performance." Highway Research Board, Bul. No. 20, Sept., 1949.

1949-12. Minnesota Highway Department. "Load Carrying Capacity of Roads as Affected by Frost Action." Minnesota Progress Report to Project Committee No. 7, Highway Research Board. An unpublished report, June 24, 1949

1949-13. Kersten, M. S. "Thermal Properties of Soils." University of Minnesota, Institute of Technology Bul. No. 28, 221 pp., June 1, 1949.

1949-14. Nelson, H. E. "Military Road Construction in Foreign Theaters." Special Publication, Highway Research Board, 1948.

1949-15. Corps of Engineers. "Laboratory Research for the Determination of the Thermal Properties of Soils." Third Report, June, 1949.

1949-16. Kersten, M. S. "Thermal Properties of Soils." Bulletin 28, University of Minnesota, Engineering Experiment Station, Vol. 52, No. 21, June 1, 1949.

1949-17. Pewe, T. L. "Preliminary Report of Permafrost Investigations in the Dunbar Area." Alaska U. S. Geological Survey, Circular 42, February, 1949.

1949-18. Smith, H. T. U. "Physical Effects of Pleistocene Climatic Changes in Non-glaciated areas: Eolian Phenomena, Frost Action, and Stream Terracing." Bulletin Geol. Society of America, Vol. 60, pp. 1485-1516, September, 1949.

1949-19. McDonald, C. H. "Investigation of a Simple Method of Identifying Base Course Material Subject to Frost Action." Proceedings, Highway Research Board, Vol. 29, pp. 392-402, 1949.

1949-20. Wisconsin State Highway Commission. "Design Standards and Policy for Rural and Urban Construction," January, 1949.

1949-21. Lund, O. L. "Performance of Two Stabilized Soil Base Courses in Nebraska." An unpublished report submitted to the Highway Research Board, Committee on Frost Heave and Frost Action in Soil, July, 1949.

1949-22. Dr. Leopold Jezek. "Frost Action on Highways." Research on frost action on highways in construction and in service. Method of mapping and statistical survey of the frost action. Silnicni Obzor, pp. 161-166, 1949.

1949-23. Corps of Engineers. "Addendum No. 1, 1945-1947, Report on Frost Investigation 1944-45." New England Division, Frost Effects Laboratory, October, 1949.

1949-24. Croney, D. "Some Cases of Frost Damage to Roads." Road Research, Road Note No. 8, Department of Scientific and Industrial Research, Road Research Laboratory, London, 1949.

1949-25. Lundqvist, G. The orientation of the block material in certain species of flow earth. Points of view on method. (In Glaciers and climate. Geomorphological essays dedicated to Hans W:son Ahlmann, 14 November, 1949. Stockholm, Svenska Sallskapet för Antropologi och Geografi, 1949, pp. 334-47. Also in Geografiska Annaler, Häfte 1-2, 1949.

1949-26. Jenness, John L. Permafrost in Canada. Origin and distribution of permanently frozen ground, with special reference to Canada. Arctic, Vol. 2, No. 1, 1949, pp. 13-27, illus., maps (facing p. 76), diagrs.

1949-27. Frost, Robert L., and Mintzer, Olin W. "Influence of Topographic Position in Airphoto Identification of Permafrost." Highway Research Board, Bul. No. 28, 1950.

1949-28. Legget, R. F. and Peckover, F. L. "Soil Temperature Studies." Proceedings, Highway Research Board, Vol. 29, pp. 434-444, 1949.

1949-29. Coleman, J. D. "Soil Thermodynamics and Road Engineering." Nature, Vol. 163, No. 4134, pp. 143-145, January 22, 1949.

1949-30. Callendar, H. L. "Observations of Soil Temperature with Electrical Resistance Thermometers." Trans. of Royal Society of Canada, Second Series, Vol. 3, pp. 31-49, (1897-98).

1949-31. Algren, A. B. "Ground Temperatures as Affected by Weather Conditions." Heating, Piping and Air Conditioning, pp. 111-116, June, 1949.

1949-32. Langbein, Walter B. "Computing Soil Temperatures." Trans. of the American Geophysical Union, Vol. 30, No. 4, pp. 543-547, August, 1949.

1949-33. Allcut, E. A. "Thermal Conductivity Tests and Results." School of Engineering Research, Faculty of Applied Science and Engineering, University of Toronto, Bulletin No. 182, 1949.

1949-34. Misener, A. D. "Temperature Gradients in the Canadian Shield." Canadian Mining and Metallurgical Bulletin, Vol. 42, No. 446, pp. 280-287, June, 1949.

1949-35. Price, W. A. "Pocket Gophers as Architects of Mima Mounds in the Western United States." Texas Journal of Science, Vol. 1, 1949, pp. 1-17.

1949-36. McCullough, Charles R. "An Engineering Soil Map of Indiana Prepared from Airphotos." Proceedings, 35th Annual Purdue Road School, April, 1949.

1949-37. Frost, Robert E. "Prospecting for Engineering Materials Using Aerial Photographs." A paper prepared for the Pan American Engineering Conference, June, 1949.

1949-38. Troll, Carl. "Strukturboden, Solifluktion, und Frostklimate der Erde." Significance in the Pliestocene Chronology of Alaska. Journal of Geology, Vol. 57, No. 2, March, 1949.

1949-39. Wahrhaftig, Clyde. "The Frost Moved Rubbles of Jumbo Dome and Their Significance in the Pliestocene Chronology of Alaska." Journal of Geology, Vol. 57, No. 2, March, 1949.

1949-40. Judson, Sheldon. "Rock-fragment Slopes Caused by Past Frost Action in the Jura Mountains (AIN) France." Journal of Geology, Vol. 57, No. 2, March, 1949.

1950-1. Ekse, M. "Wet Silt Under Base Course Damages Alaska's Glenn Highway." Civil Engineering, Vol. 20, No. 1, pp. 42-43, January, 1950.

1950-2. Highway Research Board. "Load Carrying Capacity of Roads as Affectea by Frost Action."

Research Report No. 10-D, Highway Research Board, May, 1950.

1950-3. Godskesen, O. "Frost Often Penetrates to Depths Greater than Those at Which Frost Damage is Caused." Medd. Vegdirect., 1946 (8), 105-6 Road **Abstracts, Vol. 27, No. 8, August 1950 (England) Highway Research Abstracts, October, 1950.**

1950-4. Ruckli, R. "Der Frost Im Baugrund" (Frost in Soil) Wein Springer-Verlag. 279 pp., 112 illustrations. 1950.

1950-5. Hardy, R. M. "Construction Problems in Silty Soils." The Engineering Journal, Vol. 33, No. 9, pp. 775-779, September, 1950.

1950-6. Meese, R. H. "The Mechanics of Frost Action in Highways." The Trend in Engineering, Univ. of Washington, Vol. 2, No. 3, pp. 28-30, July, 1950.

1950-7. Practical Refrigeration Engineer. "Frost Heaving Damages Freezer Floors." pp. 41-44, July, 1950.

1950-8. Barentsen, P. "De Werking Van de Vorst in De Bodem." Polytechnisch, Tydschrift (Netherlands) 5e Jaargang, No. 39-40, October 3, 1950.

1950-9. Herrmann, E. M., Plate, R. B., and Sinclair, W. P. "Automatic Control of Thermal Conductivity Apparatus." A.S.T.M. Bulletin No. 170, pp. 69-74, December, 1950.

1950-10. Gemant, A. "The Thermal Conductivity of Soils." Journal of Applied Physics, Vol. 21, No. 8, pp. 750-752, August, 1950.

1950-11. Frost, Robert E. "Evaluation of Soils and Permafrost Conditions in the Territory of Alaska by Means of Aerial Photographs." Engineering Experiment Station, Purdue University for the Corps of Engineers, Department of the Army, St. Paul District, June, 1950.

1950-12. Hooper, F. C., and Lepper, F. R. "Transient Heat Flow Apparatus for the Determination of Thermal Conductivities." Presented at the Semi-Annual Meeting of the American Society of Heating and Ventilating Engineers, June, 1950.

1950-13. Lambe, T. W. "Capillary Phenomena in Cohesionless Soils." Proceedings, Am. Soc. C. E., Vol. 76, Separate No. 4, February, 1950.

1950-14. Washburn, A. L. "Patterned Ground," Revue Canadienne De Geographie Vol. 4, Nos. 3, 4, pp. 5-54, July-October, 1950.

1951-1. Hopkins, D. M. and Sigafoos, R. S. (in press) Frost action and vegetation patterns on Seward Peninsula, Alaska: U. S. Geol. Survey Bul., 974-C. 1951.

1951-2. Sigafoos, R. S. (in press) Some effects of congeliturbation upon the development of vegetation, Seward Peninsula, Alaska: Ecological Mon. 1951.

1951-3 to 1951-10. These reference numbers not used.

1951-11. Orvedal, A. C., Kubota, J., and Smith, H. M. "Major Soil Profiles and Their Relationship with Climate," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-12. Siple, P. A. "Climate Aspects of Frost Heave and Related Ground Frost Phenomena," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-13. Gregg, L. E. "Areal Distribution of Climates Related to Ground Freezing," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-14. Crawford, Carl B. "Soil Temperature - A Review of the Literature," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-15. Yoder, E. J., and Lowrie, Charles R. "Some Field Measurements of Soil Temperature in Indiana," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-16. Misener, A. D. "An Absolute Method of Determining Thermal Conductivity and Diffusivity of Soils," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-17. Hooper, F. C. "The Thermal Conductivity Probe," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-18. Bouyoucos, G. J. "Methods for Measuring the Moisture Content of Soils Under Field Conditions," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-19. Aldous, William M. and Lawton, Warren L. "The Measurement of Soil Moisture and Temperature by Heat Diffusion Type Moisture Cell," Symposium on Frost Heave and Frost Action, Highway Research Board, Special Publication, 1951.

1951-20. Belcher, D. J. "The Measurement of Soil Moisture and Density by Neutron and Gamma Ray Scattering," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-21. Hveem, F. N. "Water in Highway Subgrades and Foundations," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-22. Riise, T. B. "Soil Moisture Under Concrete Pavement of an Airport," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-23. Spangler, M. G. and Wei Te Pien. "Distribution of Capillary Moisture at Equilibrium in Stratified Soil," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-24. Redus, J. F. and Foster, C. R. "Moisture Conditions Under Flexible Airfield Pavements,"

Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-25. Touloukian, Y. S., Bottorf, J. D., and Harsen, Thor. "Heat Transfer and Temperature Distribution in Soils for Transient Heat Flow Due to Cylindrical Sources and Sinks," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-26. Kersten, Miles S. "Thermal Properties of Soils," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-27. Grim, Ralph E. "Relation of Frost Action to the Clay Mineral Composition of Soil Materials," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-28. Siple, P. A. "A Theory of Ice Blocked Drainage as a Principal Factor in Frost Heave, Slump and Solifluction," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-29. Sigafoos, R. S., and Hopkins, D. M. "Soil Instability on Slopes in Regions of Perennially Frozen Ground," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-30. Carlson, Harry. "Calculation of Depth of Thaw in Frozen Ground," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-31. Frost, Robert E. "Interpretation of Permafrost Features from Airphotos," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-32. Haley, James F., and Kaplar, Chester W. "Cold Room Studies of Frost Action in Soils," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-33. Otis, Paul S. "The Nature and Extent of Damage to New Hampshire Highways from Frost Action," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-34. Livingston, R. E. "Frost Action and Spring Break-up in Colorado," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-35. Stokstad, O. L. "Frost Action in Michigan," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-36. Croney, D. "Frost Damage to Roads in Great Britain," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-37. Lawrence, T. B. "Load Carrying Capacity of Roads Affected by Frost Action," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-38. Rogers, F. C. and Nikola, H. C. "Frost Action Studies of Thirty Soils in New Jersey," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-39. Linnell, K. A., and Haley, J. F. "Investigation of the Effect of Frost Action on Pavement Supporting Capacity," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-40. Smith, H. R. "Calcium Chloride Treatment of Frost Action," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-41. Stokstad, O. L. "Controlling the Effects of Frost Action in Michigan," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-42. Lund, O. L. "Remedies and Treatments for the Frost Problem in Nebraska," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-43. Keene, Philip. "Remedies and Treatments by Connecticut Highway Department," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-44. Fuller, Earl I. "The Influence of Frost on Highway Foundations," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-45. Skaven-Haug, Sv. "The Norvegian State Railways' Measures Against Frost Heaving," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-46. Shelburne, T. E. "Results of a Questionnaire on Remedies and Treatments," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-47. Olmstead, Frank. "Results of a Questionnaire on Needed Research," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-48. Johnson, A. W. "Needed Research Pertaining to Frost Action and Related Phenomena," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, 1951.

1951-49. Kersten, M. S. and Cox, A. E. "The Effects of Temperature on the Bearing Value of Frozen Soils," Highway Research Board, Bul. No. 40, 1951.

1951-50. Housel, W. S. "Misconceptions in the Use of Surface Tension in Capillarity," Proceedings, Highway Research Board, Vol. 30, 1950.

1951-51. Corps of Engineers, Engineering Manual for Military Construction, Part 12, Chapter 4, April, 1951.

1951-52. Hall, John. "Frost Penetration in Montana Soils," Presented at the Spring Meeting of the Montana Section of the American Waterworks Association, Helena, Montana.

1951-53. Motl, C. L. "Report of Committee on Load Carrying Capacity of Roads as Affected by Frost Action," Highway Research Board, Bul. No. 40, 1951.

1951-54. Kersten, M. S. "Report of Studies of Subgrade Moisture Conditions." The University of Tennessee Record, Vol. 54, No. 4, pp. 24-47, July, 1951.

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