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Contents

Frost Considerations in Highway Pavement Design: Eastern United States

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•AN up-to-date assessment of knowledge pertaining to frost action in soils was presented in "Highway Pavement Design in Frost Areas, A Symposium: Part 1. Basic Considerations" (HRB Bull. 225, 1959). The papers covered such interrelated factors as soil type, moisture, and temperature penetration, and also the mechanism of frost heaving.

Various state highway departments, as well as other agencies responsible for design and maintenance of pavement structures, have accumulated a vast wealth of empirical knowledge which they use to design pavements to withstand the effects of frost as well as other environmental factors and use.

This paper includes a summary and discussion of the design practices of 16 highway departments in the Eastern United States (Zone 1). The area involved extends from the Atlantic coastline inland for 200 to 400 miles, from Maine to Georgia. These states offer an extremely wide diversity of climate, soil and other factors that influence the severity of frost action. As a matter of fact, such states as New York, New Hampshire and Maine have significant variations in freezing conditions from south to north or from the coastline inland within their boundaries. The following paragraphs describe in a general way the type of surface soils that are prevalent in the area and also present information on freezing air temperatures and rainfall.

SURFACE SOILS IN ZONE 1

Figure 1 has been adapted from a soils map of the United States prepared by Donald J. Belcher. For the purpose of this paper, the surface soils have been grouped into four major groups: glacial, coastal plain, residual, and non-soil areas.

Glacial

The predominant soil type in the glaciated area is glacial till. The character of glacial till varies widely and is directly related to its parent material. In the crystalline bedrock areas of New England, till is generally a heterogeneous well-graded soil with all grain sizes from clay to large boulders. The material can, however, range from a boulder clay to a slightly plastic silty till. Where the parent materials are interbedded shales or sandstones, the derived till is generally a clayey sand or gravel with few cobbles and boulders. Clayey sand or clayey gravel tills are found overlying the red sedimentary shales and sandstones of Connecticut and Newark Triassic troughs. They also overlie large areas in central New York and northwestern Pennsylvania. Although the glacial till soils have a high bearing capacity for the support of buildings and bridges, they range from moderate to high in frost susceptibility.

Waterlain granular soils are the next most common soil type found in glaciated areas. These materials were deposited by meltwater streams flowing from the main glacial terminus or from detached and melting blocks of ice. They generally consist of stratified silts, sands and gravels.

Although deposits of glacial clay, both fresh water and marine, have attained notoriety because of construction problems they engender, they are actually the least abundant of the glacial deposits. Most fresh water clay deposits are confined to areas once occupied by a few large glacial lakes, notably Lake Hartford and Lake Albany.

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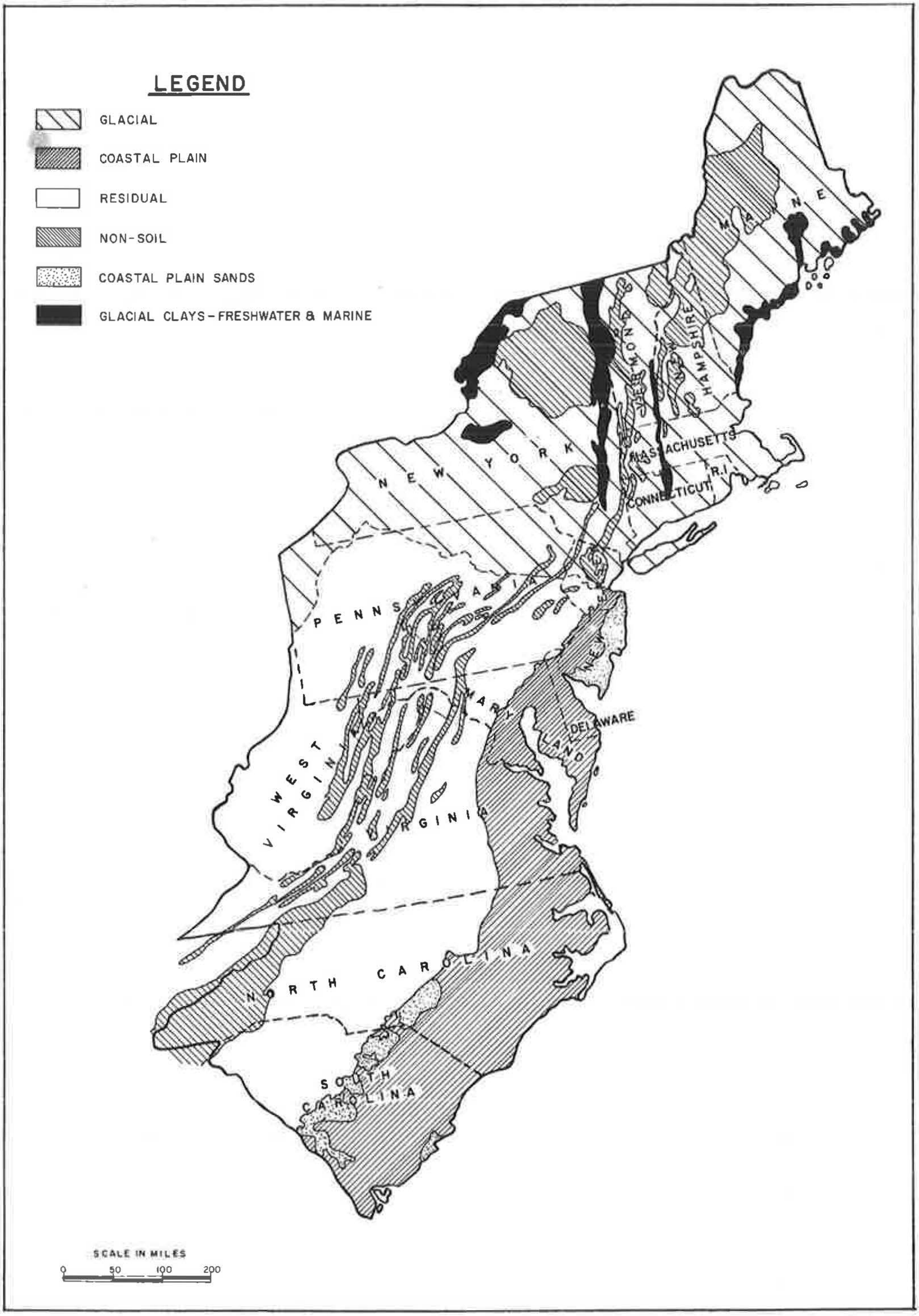


Figure 1. Soils map, Zone 1.

Minor deposits appear throughout the glaciated area, however, in small glacial lake sequences. Fresh water clays are usually varved; that is, they consist of a rhythmic alternation of clay and silt.

Marine glacial clays are not varved and are generally found interbedded with silt and sand. For the most part they are found immediately adjacent to the existing coast. However, in the Champlain Valley Lowland an immediate post-glacial incursion of the sea resulted in the deposition of large deposits of glacial marine clays in an area now quite distant from the sea.

Organic silts, clays, and peat, of both fresh water and marine origin, are found in the glaciated area. Because of their low strength and high compressibility, these soils are usually removed in highway construction, at least for the full depth of frost penetration. Therefore, they have no significance in the frost design problem.

Although glaciated areas in the East are not generally thought of as containing much loess, there are in fact large areas overlain by this material. These windblown, very fine sands and inorganic silts occur in relatively thin surface deposits compared to the great thicknesses found in some western and midwestern states. Eastern loessial deposits average 2 to 3 ft and rarely exceed 6 ft in thickness. As loess is a surface deposit, the topsoil (which may vary from a few inches to as much as 2 ft) is developed in this material and the thin horizon of loess remaining is not generally recognized as such. In addition, because most thin loessial deposits are well within the zone of frost action, gravel, cobbles and boulders have worked their way up into the loess from the underlying soils by frost action.

For the most part, the sand and gravel deposits found in the glaciated area contain materials that are non-frost susceptible or only slightly frost susceptible. These materials offer ideal subgrade conditions and serve as sources for base and subbase courses in highway construction. On the other hand, fresh water and marine glacial clays and inorganic silts are highly frost susceptible. These soils exhibit high frost heaving characteristics and severe loss in strength during frost melting periods.

In the subgrade of highway cuts, the transition zone between non-frost-susceptible and highly frost-susceptible soils, is critical from the standpoint of differential heaving. The occurrence of pockets or zones of inorganic silt within an otherwise non-frost-susceptible sand or gravel subgrade is commonly a source of abrupt changes in the pavement surface profile.

Coastal Plain

The soils in the coastal plain group form a complex assemblage of marine and terrigenous deposits. The complexity arises from their having been deposited during a period of continuing fluctuating sea levels ranging from the Cretaceous to the present. Fluvial sands and gravels were reworked by advances of the sea and intermixed or interbedded with marine clays and silts. These deposits were in turn eroded, reworked and redeposited by rivers during the retreat of the sea. The engineering properties of these soils may be expected to change rapidly both laterally and vertically.

Large areas of the coastal plain are overlain by sand. Large sand deposits occur adjacent to the coast along the fall line in Virginia and the Carolinas. They are the principal surface deposits in the coastal plain of New Jersey.

A number of geologic processes are responsible for the various sand deposits. Those in New Jersey are primarily glacial outwash, whereas coastal sands are essentially marine. Some of the fall-line sands are of marine origin; others are residual sands and have been reworked by the wind, for example, the Congaree sand hills of South Carolina.

Residual

In Zone 1, these soils are confined exclusively to the Piedmont and Appalachian plateau provinces of the Appalachian highland. These products of the chemical and mechanical weathering of rocks accumulate in relatively level or gently rolling bedrock areas. Since the climatic variation in the zone where residual soils are found is not great, the character of the parent bedrock material is the decisive variable that influences the following types of residual soils.

1. Sandy topsoils underlain by sandy clays are the characteristic soils derived from the crystalline rocks (granite, gneiss and gneissoid schist) in the Piedmont province.
2. The soluble carbonates of limestone rocks tend to dissolve and leach away leaving insoluble iron oxide and hydrous silicates. The resulting soils are reddish clays, which contain fragments of more or less unaltered limestone.
3. Micaceous schist generally weathers to friable sand soils, but the properties may vary from sand to clay dependent on the quantity and type of other minerals, particularly feldspar and quartz, in the parent material and on the degree of weathering.
4. Because shales are composed chiefly of relatively stable clay minerals, weathering can produce little change in their mineral composition. However, it can break the strong bonds produced by consolidation so that shales readily revert to soft clay soils.

The residual soils are for the most part frost susceptible.

Non-Soil Areas

In mountainous areas where stream gradients and topography are steep and erosion active, residual soils and the products of mechanical weathering are removed nearly as fast as they are formed and the bedrock is exposed at the surface. It should not be inferred that the area denoted as non-soil in Figure 1 is one vast bedrock outcrop. Within this area are stream valleys and their associate alluvial deposits which often attain a considerable thickness. In glaciated areas there are thin patches of till and high level kame deposits. High in the Appalachians, are small, relatively level meadow lands in which residual soils have accumulated. In general, however, the area is one in which there is little or no soil cover.

CLIMATE

The contours of air freezing index for the coldest year in 10 are shown in Figure 2. For example, the freezing index in Maine is ten times that which occurs in Virginia. Figure 3 shows the contours of freezing index for the coldest year in 30. Figure 4 shows the estimated depth of frost penetration into clean granular soils beneath pavements kept free of snow and ice for the coldest year in 10. Figure 4 was prepared from an empirical relationship between air freezing index and frost penetration by the Corps of Engineers (Fig. 5).

Average annual precipitation is shown in Figure 6. The amount of rainfall within this zone does not vary widely, the mean generally varying from about 35 to 50 in. per year. In the western portion of the Carolinas, however, a mean annual rainfall of up to 80 in. is recorded in a small area.

SURVEY OF DESIGN PRACTICE

A questionnaire (basically the same as prepared by Erickson for the western survey) was sent to each state highway department in Zone 1. Table 1 abstracts and summarizes the replies.

Frost is considered in the design of highway pavements in all states north of New Jersey. In some of these states, the variation in freezing conditions is appreciable. For example, there is approximately 2 ft greater frost penetration in northern New Hampshire and Vermont than in the southern parts of these states. The variation is somewhat greater in Maine. Delaware considers frost in Kent and New Castle counties only. Maryland considers frost only in Garrett and Alleghany counties (approximately 12% of the total state area with approximately 6% of the state routes). In North Carolina, the only areas where frost is considered a factor in the design of pavements are mountainous areas above approximate 2,500-ft elevation. South Carolina reports negligible frost effects. Georgia experiences only a few cycles of freezing; the maximum penetration into base course materials is a few inches. By emphasizing a free-draining base in the northern two-thirds of the state, the minor frost problem has been virtually eliminated.

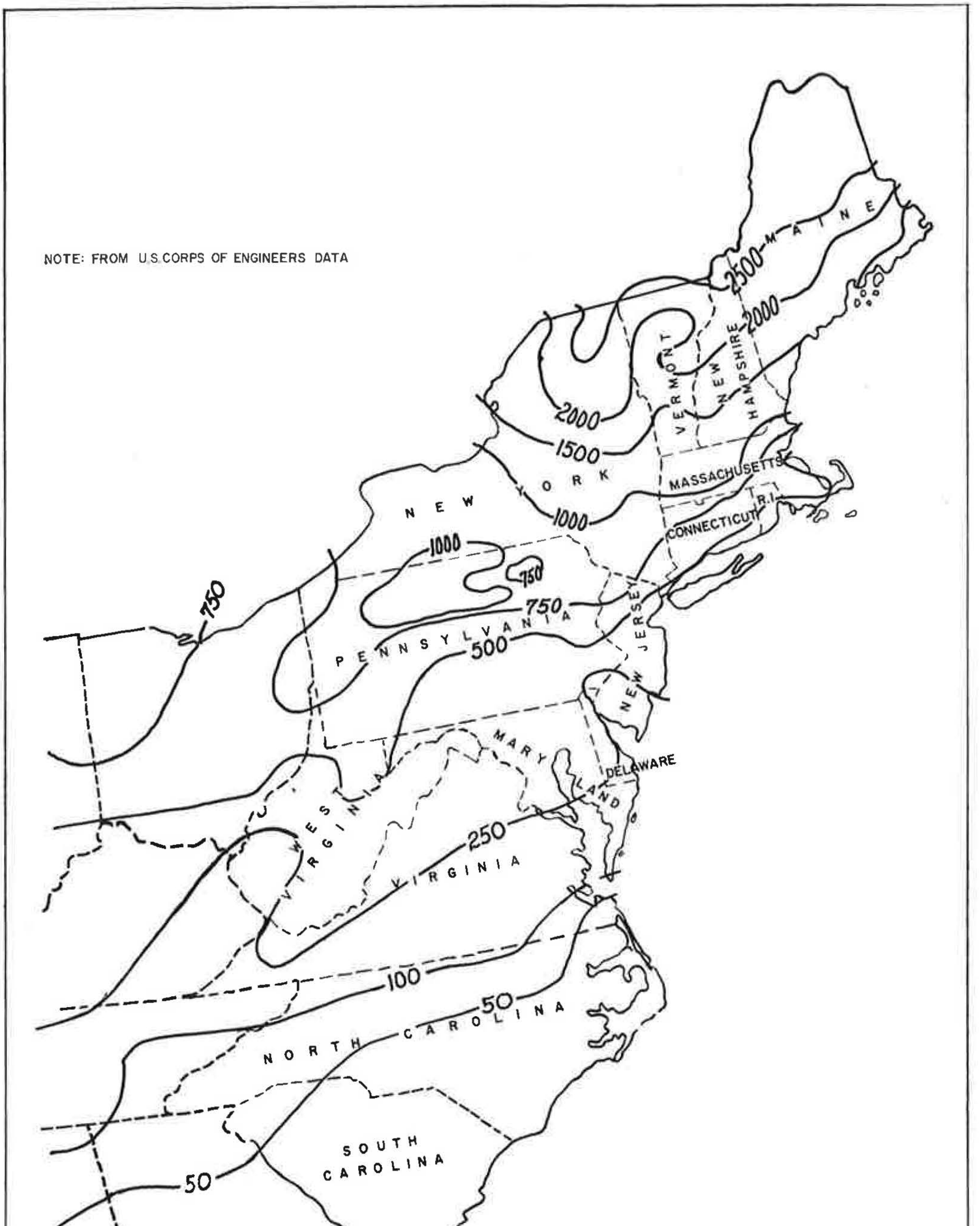


Figure 2. Air freezing index, coldest year in 10.

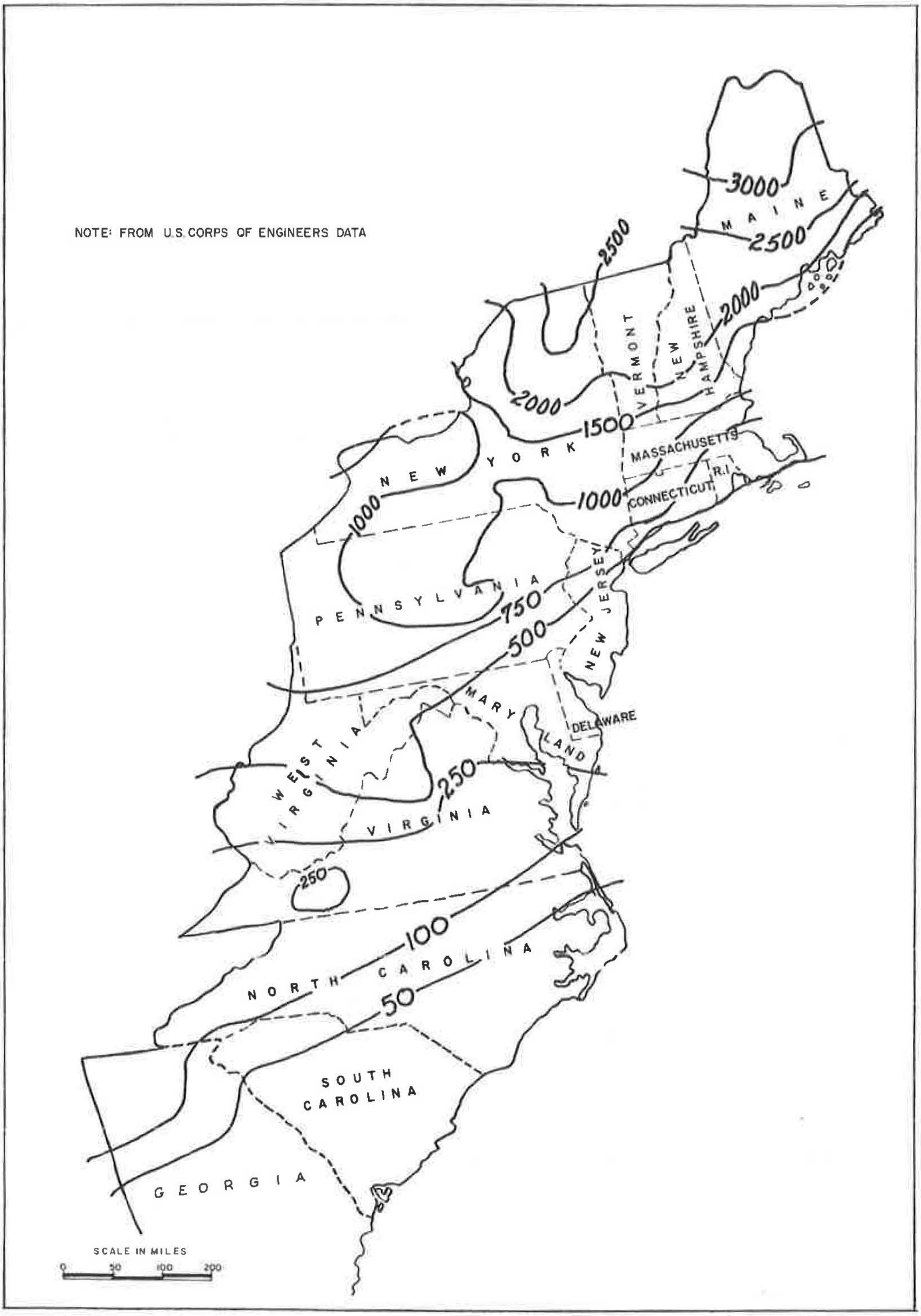


Figure 3. Air freezing index, coldest year in 30.

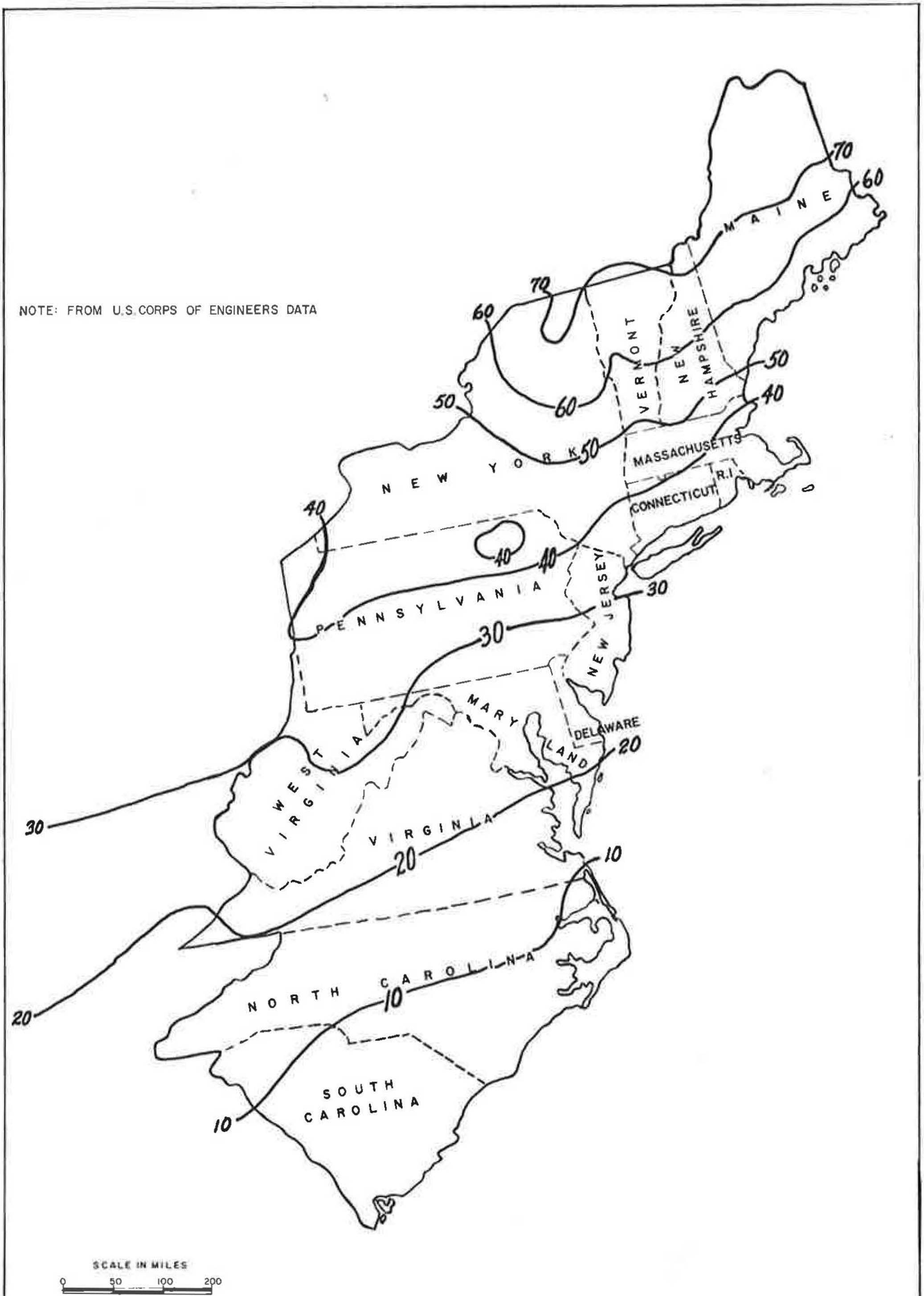


Figure 4. Frost penetration (in.) into clean, granular soils beneath pavements kept free of snow and ice.

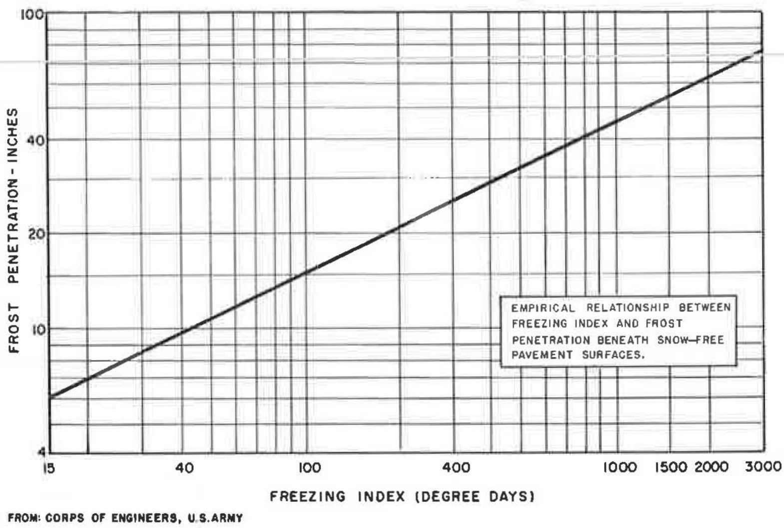


Figure 5

LOAD RESTRICTIONS

All heavy duty highways are being designed for unrestricted use in all seasons. Maine restricts the use of town and state aid roads during the thawing period when conditions warrant. The following is abstracted from a sign which the state posts to restrict the use of such roads:

> The State Highway Commission hereby closes from November 15, 1962 to May 15, 1963 all parts of such state and state aid highways, (including the bridges thereon) as will be in danger of abuse from the traffic hereinafter described:
>
> Travel by all motor trucks, tractors, trailers <u>or other vehicles or objects</u>, when
>
> 1. The gross weight of the vehicle (vehicle and load combined) exceeds the weights set forth in the following schedule:

Type of Vehicle	Allowable Gross Weight
2-axle truck	8 tons
3-axle truck	9 tons
2-axle tractor and 1-axle semitrailer	10 tons
2-axle tractor and 2-axle semitrailer	11 tons
3-axle tractor and 1-axle semitrailer	11 tons
3-axle tractor and 2-axle semitrailer	11 tons

> 2. The load imposed upon the road surface exceeds 400 pounds per inch width of tire (manufacturer's rating).
> 3. The vehicle or object has attached to its wheels (or such part of the vehicle as has contact with the road) any clamp, rib or other object likely to injure the surface of the highway.
>
> Provided, however, that this rule and regulation shall not apply to any particular closed way when the way is solidly frozen.

In New Hampshire, the principal roads are designed for the legal weight limit during the frost melting period, but the issuance of overweight permits is restricted during

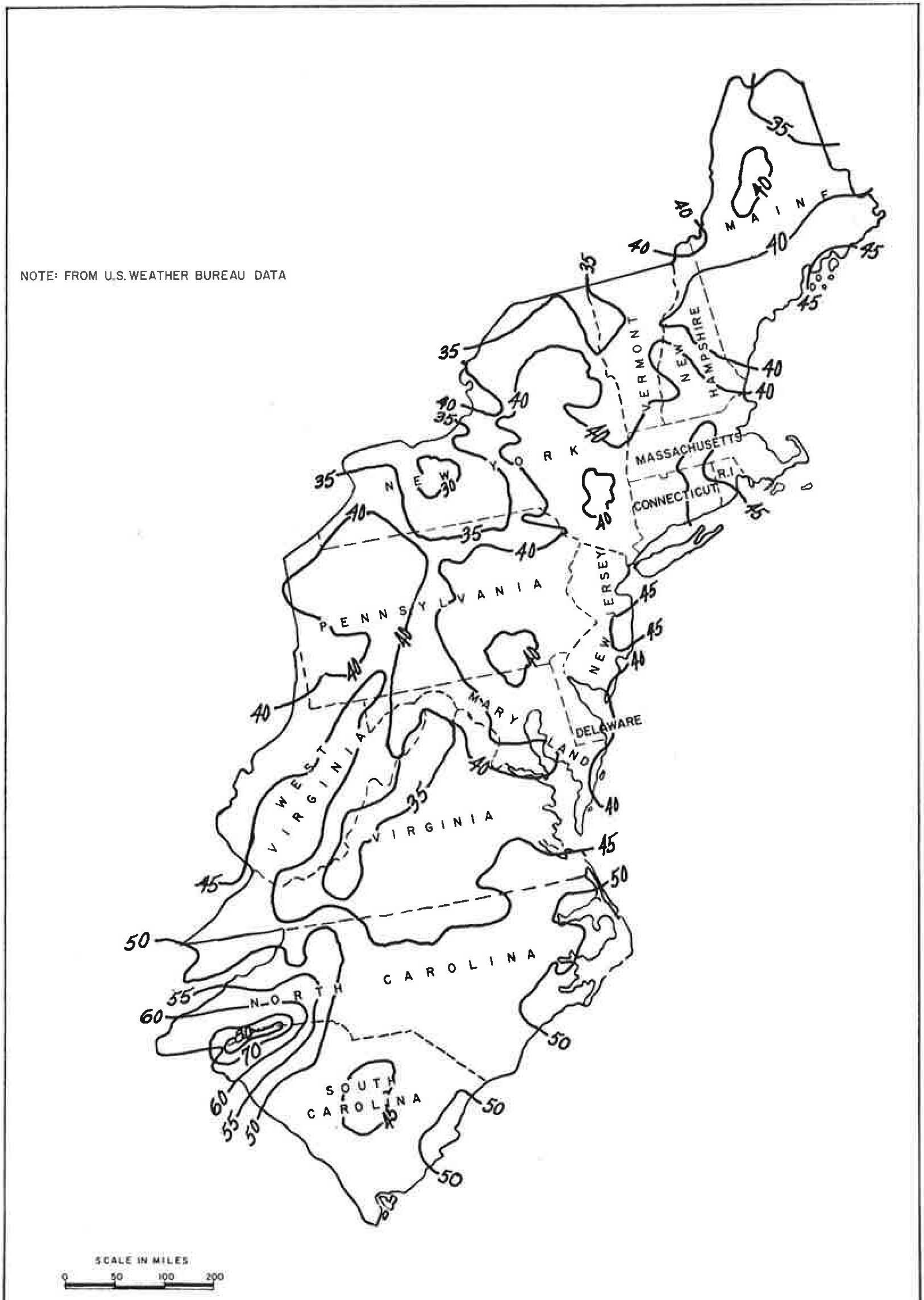


Figure 6. Average annual precipitation.

TABLE 1

FROST CONSIDERATIONS IN PAVEMENT DESIGN—EASTERN UNITED STATES

State	Frost Susceptibility Criteria	Subgrade Preparation	Drainage Criteria	Pavement Section	Remarks
Maine	Frost susceptibility is determined by running a grain size with a hydro meter analysis similar to the Corp. of Engineers classification. The #200 sieve is usually the controlling factor. H. R. B. Bulletin 71.	In general removal of frost susceptible soils is left up to the field engineer. In transition areas between varying types of soil the engineer can require undercutting and selective backfilling.	1. Side ditches are deepened by at least 1 ft. in problem areas. 2. Subgrades are carried for the full width of the roadway and sloped 1/4" to the foot. 3. Base thickness is increased and underdrains installed in shady areas under bridges where greater frost penetration occurs.	1. For both rigid and flexible pavements no ledge is allowed within 5 ft. of the finished grade. 2. Both rigid and flexible pavement sections are increased by 6" in cut sections.	1. Although there are no specifications to this effect, efforts being made to insure uniformity of subgrade soils to a depth of 4 ft. and to eliminate boulders 12" width in the area of frost action
New Hampshire	With respect to frost, soils are grouped from good to poor using the percent passing the #200 sieve as the criterion for grouping. No other tests are used. Soils broadly classed relative to frost heaving as follows - A2, Good less than 10% - #200 slight A2, Fair 10-20% - #200 moderate A2, Poor 25-35% - #200 severe A4, More than 35% - #200 remove for full depth of frost penetration.	Care is taken to see that frost susceptible soils, especially silts, do not lie within the zone of freezing after earthwork has been completed.	1. No special drainage controls are used other than typical use of ditches and free draining gravel. In areas where the subbase does not extend through the fill, gravel weepers are sometimes used to carry off excess water. This is only in areas where gravel fill is scarce. 2. Bottom of V-ditches are carried 4 ft. below the edge of the shoulder.	1. On all primary and interstate roads the thickness of pavement structure is based upon complete removal of all frost susceptible soils within the zone of freezing. 2. On lesser roads the heaving material is removed to a depth of 50% of the depth of frost penetration. 3. Sand subbase or blanket course are not considered to add strength to the pavement structure. 4. No rigid pavements are being built.	
Vermont	Mechanical and hydrometer analyses of soil are used. Soil is considered susceptible if more than 10% passes the #200 sieve or more than 3% finer than 0.02 mm. Other specifications for subbase and base materials include the #4, #100, and #270 sieves.	1. Types of material for backfilling are specified, but final decision lies with the field engineer. 2. Working mats of sand and gravel over wet fine grained soils are specified where needed.	1. Longitudinal underdrains below the subgrade line in conjunction with drainage ditches 9"-21" below roadway subgrade are used. Underdrains are outletted on slopes rather than in drainage structures. 2. Culverts are kept a minimum of 2 ft. below the subgrade to prevent a roadway depression during the winter months.	1. Design is fairly uniform throughout the state. Depths of frost free material used range from 18 to 42 inches. 2. Sand and gravel subgrades require 1 to 2 ft. less thickness of pavement section. 3. Pavement structure thickness is not varied between cuts and fill sections. 4. A one mile test section of rigid pavement is now being studied to evaluate subgrade requirements.	
Massachusetts	Soils with 15% or more passing the #200 sieve are considered frost susceptible.	1. The appearance of a susceptible soil does not influence embankment design, however no fill can be placed on frozen soil. 2. Specifications include what type material is to be removed and what is to be used as backfill. 3. In embankments the poorer material is sandwiched between layers of select granular material.	1. In areas of high ground water such as ledge, cuts and lowlands, both increased subbase thickness and underdrains are utilized.	1. Total flexible pavement structure is determined using an average frost penetration and applying it to the AASHO formula. 2. Rigid pavement structures are a constant 8" thickness underlain by a minimum 12" gravel layer. 3. A proposed revision to specifications advocates the use of a 2 ft. layer of granular fill at the top of embankments. 4. Pavement structure thickness is not varied between cuts and fills.	

TABLE 1 (Continued)

State	Frost Susceptibility Criteria	Subgrade Preparation	Drainage Criteria	Pavement Section	Remarks
Rhode Island	No criteria reported.	No specific details reported.	No specific details reported.	No specific details reported.	Pavement sections being used with substantial bases and subbases have practically eliminated frost problem. Minor maintenance on older secondary roads.
Connecticut	1. 3" or 6" increases in subbase thickness in questionable cases for heavy-duty roads. 2. In the northern half of the state subbases are often 6" thicker than the minimum. In the N.W. corner of the state the subbase is often increased 12" thicker than the specified minimum. 3. For silt and clay subgrades in the frost zone the subbase is increased 6" or 12"±.	1. Susceptible soils are identified through use of Casagrande's criterion. 2. Requirements for subbase material include material having less than 10% passing the #100 sieve and that the soil be non-plastic.	1. Subbase extends laterally to meet underdrain, if any, in cuts or ditch; in fills to meet embankment slope. Ditch invert 6" below subgrade surface. 2. Where G.W.T. is 3 ft. below frost zone in glacial till and 6 ft. in silts and clays, it is not considered to affect frost action. Only surface water would be a problem. 3. Where the G.W.T. is 4 ft. below the pavement, underdrains are placed 5 ft. below the pavement or ditches 4 ft. below the pavement.	1. For heavy duty roads the pavement and base = 13", subbase = 10" minimum for all cuts and fills except rock cuts where the minimum subbase = 18". 2. For other secondary roads the pavement thickness = 8" with a subbase varying from 6" to 18" depending on embankment or cut conditions. 2. For further information see "Subgrade Preparation."	
New York	1. The basic criterion is to obtain uniformity of the subgrade in the zone of freezing. 2. Granular materials are a separate pay item and are used to replace excavated susceptible soil when called for on the plans or as by order of the engineer. 3. Often times use of granular material is dictated by need of a working mat rather than the presence of frost susceptible soils. 4. A granular filter course is used over fine grained soils to prevent intrusion into the subbase or base.	1. Potentially frost susceptible soil is that having more than 3% passing the 0.02 mm. sieve. 2. Subbase materials must have less than 10% passing the #200 sieve and a plasticity index no greater than 3.	1. Every means within economical reason is taken to keep the water table below the zone of freezing under the pavement. 2. All subbase courses extend across the entire roadway section. 3. The invert of side ditches is carried a minimum of 24" below the top of the subgrade or 48" below the top of the pavement.	1. Pavement structures are designed based on an evaluation of performance of similar pavements under similar conditions. 2. Selected backfill where unstable soils are encountered is not considered as part of the structure and is considered to impart strength to the roadway during construction only.	Standardized treatment and criteria in this field are impossible, impractical and/or uneconomical. Success of finished work depends mainly on skill, experience and judgment of Engineer.
New Jersey	1. Frost susceptible materials are excavated, as shown on plans or directed by the engineer and selected backfill is placed. 2. The "Vicksburg" filter criteria is used to check intrusion of fine grained soils into the subbase. 3. Subbase and base courses extend for the full width of the roadway cross section.	1. The Casagrande and Corps of Engineers criteria are used to determine frost susceptibility.	1. In areas where the G.W.T. is a problem, french drains are installed in a herringbone pattern beneath the pavement, along the longitudinal edge of the pavement, at the toe of cut slopes or in the slopes. 2. Low points of subbase materials are drained either to the edge of the embankment or catch basins by modified french drains. 3. Storm drains used are comparable to city street designs.	1. The total pavement structure for flexible and rigid pavements is usually equal to the depth of the zone of frost penetration. 2. No differing criteria for varying soils is used. 3. When rock embankments are constructed the gradations are specified such that the top course is considered as part of the flexible pavement.	

TABLE 1 (Continued)

State	Frost Susceptibility Criteria	Subgrade Preparation	Drainage Criteria	Pavement Section	Remarks
Delaware	1. Frost susceptible soils are those which have more than 35% passing the #200 sieve. 2. Susceptibility is considered to increase with an increasing % passing the #200 sieve and decreasing plasticity.	1. If frost susceptible soil is located within 26" of the proposed pavement it is excavated and backfilled according to plans and specifications. If susceptible soil is discovered, below this depth, during construction it is not removed unless it is unstable. 3. All materials used for base and subbase are specified to be non-frost susceptible.	1. No special design considerations are used when the G. W. T. is within susceptible soil. It is assumed that a water source will always be available to frost susceptible soil. 2. Ditches are made deeper in frost areas to intercept drainage.	1. Frost is considered to be a problem in Kent and New Castle counties only.	1. If the design depth of both flexible and rigid pavements by any method is less than the frost penetration the subbase thickness is increased so that the total structure thickness is equal to the maximum depth of frost penetration. 2. Both full width and box construction are used when placing base courses.
Maryland	1. Frost susceptible soils are determined through use of the Corps of Engineers criteria.	1. In frost areas, susceptible soil is excavated to a depth as determined by the engineer and then backfilled with locally available materials. 2. A 2" compacted blanket is used over fine materials to prevent intrusion.	1. Different drainage plans are used, but generally not for reasons of frost.	1. The total pavement structure is thicker in areas of fine grained soils due to possibility of frost action. 2. Reference is made to Corps of Engineers frost criteria.	1. The main criteria used with respect to frost is a reliance on engineering and experience. 2. Frost is a problem in Garrett, and Allegany counties only.
Virginia	1. Silty micaceous soil is considered frost susceptible, but no design criteria with regard to frost is used in this state.	1. No special consideration is given for frost effects. Design is based on elimination of saturated material only.	See "Subgrade Preparation."	1. See "Subgrade Preparations." 2. On some high-type roads sufficient select subbase material is provided to overcome an 18" frost penetration. (See "Remarks")	1. Frost considerations in northern part of state only.
West Virginia	1. Frost susceptibility is determined by use of the Corps of Engineers classification system, and the HRB criteria. 2. The amount of material finer than 0.02 mm is the controlling factor.	1. Location of frost susceptible soil in a horizon not considered critical. 2. Placement of selective soil is not provided for in design.	In frost areas ditches are deepened when economically feasible. 2. No special drainage criteria in frost areas.	1. The Bergren formula is used to calculate depth of penetration. 2. Material used for capping rock-embankments must have no stones 3" and must be compactable.	1. Choice of materials to be used is the decision of the field engineer acting upon advice from a soils engineer. 2. Univ. of West Virginia conducting frost research at this time.
North Carolina	1. Frost susceptible soil is that which has more than 3% finer than 0.02 mm.	1. The location of frost susceptible material in a horizon is not considered critical.	No special drainage criteria exists for frost susceptible areas.	No special pavement criteria exists for frost susceptible areas.	1. Frost is a problem in areas above 2500 ft. in elevation only.
South Carolina					1. Frost is not a design consideration.
Georgia	1. No frost susceptibility criteria exists.		Shoulder to shoulder granular subbase is provided mainly in the northern two-thirds of the state.		1. Frost is not a design consideration.

NOTES:

1. LOAD RESTRICTIONS - All states design their roads for unrestricted use except Maine, New Hampshire, and New York which may restrict heavy traffic on some of their secondary roads during the spring melt period.

2. GENERAL FROST CONSIDERATIONS - All states consider frost in pavement design except as noted in the "Remarks" column.

this period. New York State may post town and county roads during the critical frost melting period.

FROST SUSCEPTIBILITY CRITERIA

Most Zone 1 states use the so-called Corps of Engineers' or Casagrande criterion for evaluating whether or not a soil is frost susceptible. Such a determination involves sieve and hydrometer tests to determine the percentage by weight finer than the 0.02-mm fraction.

In Massachusetts, the proposed revision to the state specifications limits the percent passing the No. 200 mesh sieve to 15 percent. In the author's opinion, gravels with fines near the 15 percent value could be frost susceptible. Although the amount of heaving in such soils would be relatively minor, the loss of shear strength during the thawing period could be quite significant. In Delaware, the upper limit is specified as 35 percent passing the No. 200 sieve. Based on cold room tests at the Corps of Engineers Frost Effects Laboratory, it is believed that soils in this upper limit would be susceptible to heaving and weakening. In Virginia, the relatively common silty micaceous soil is considered to be frost susceptible but a criterion based on grain size is not used.

ADMIXTURES

The questionnaire requested information on the use of admixtures to control or minimize frost susceptiblity of soils. Most Zone 1 states have not used such admixtures; for those states that have, the results generally have not been promising except for portland cement. In Maine, a test section is undergoing its first cycle of freezing; a dirty base material was treated with calcium chloride, lime, and sulfite liquors. New Hampshire has tried calcium chloride and sulfite liquors on a limited scale. Due to the permeable nature of the soils, however, the effect of these admixtures was lost after the first spring. No admixture used to date except soil cement has survived the spring thaw.

Connecticut highway maintenance crews placed calcium chloride flakes and brine in a limited area after drilling 2-in. diameter holes approximately 3 ft on centers through both flexible and rigid pavements. The benefits from this treatment were inconclusive. Maryland reports that portland cement in silt soils, and lime in clay soils have been successful in improving the supporting value and the frost heave control of frost-susceptible soils. North Carolina has used sodium chloride beneath bituminous surface treatments and thin plant-mix pavements and reports that its use has been satisfactory. However, such admixtures are not used beneath asphaltic concrete pavements 3 in. in thickness and over. Georgia has added portland cement to the native soils to stabilize and waterproof base materials for secondary roads. There is no record of failure of such materials due to freezing and thawing.

In summary, the use of admixtures has received relatively limited acceptance. The most commonly used admixture, and the one apparently most successful, has been portland cement.

DRAINAGE

The questionnaire answers were unanimous as to the importance attached to drainage. The vast majority of the states report that base and subbase courses are extended laterally in cuts to meet the underdrain, if any, or ditch; and in fills, to meet the embankment slope. Some states, such as New Hampshire, report that in areas where gravel is in short supply, gravel weepers may be designed to carry off the accumulated water. This, however, is not the customary practice.

Several states report that when the groundwater table is near the surface of the more frost-susceptible soils, the side ditches are lowered or subsurface drains are installed at the shoulder. The surface of the subgrade is sloped to permit drainage of the base material. Two states report occasional use of herringbone drainage systems under pavement areas.

Several states attempt to lower the water table in the frost-susceptible subgrade on the more important roads; however, this practice is not generally used on secondary roads because of the expense involved.

PAVEMENT SECTION

The answers to the questionnaire indicated that standard criteria for thickness of pavement base and subbase over certain types of soils within a state are not generally used. In New Jersey and other states to the south, the total thickness of pavement and base used for heavy-duty roads is generally equal to or greater than the depth of frost penetration. The use of a free-draining base material has generally become the accepted practice in these states.

Full protection against the penetration of freezing temperatures into frost-susceptible subgrades in the northern states requires very thick sections. In the more frost-susceptible soils, such as the inorganic silts in New Hampshire and the varved clays in the Connecticut River valley, combined thickness of pavement and base used on the more recent projects is equal to the anticipated depth of frost penetration.

In many instances the thick subbase courses are specified for reasons that are not exclusively concerned with frost action. The need of a thick working platform in order to operate construction equipment for construction of base courses and pavements dictates a greater thickness of subbase than would normally be required to prevent frost penetration in the subgrade. On a recently constructed Interstate route in western Massachusetts, construction personnel requested that the design include a 36-in. thick gravel working platform over varved clay subgrades and a 24-in. working platform over inorganic silts. For stability under construction equipment, the Massachusetts personnel prefer the use of gravel for the working platform. In a recent project in Connecticut, a 21-in. sand working platform was used over varved clay of medium consistency.

In New York, plans usually indicate the removal of soils anticipated to be unstable for construction conditions in cuts. The purpose is to obtain a stable working platform for construction operations, not frost considerations. However, most unstable soils are frost susceptible to a considerable degree. The marine and glacial lake clays and silts in northwestern New York would generally require a thick working platform. Although some of the states report that they do not consider the working platform as adding to the structural strength of the pavement, they are unquestionably of great benefit in preventing loss of pavement supporting capacity.

SUMMARY

The survey of frost design practices in the eastern states indicates that all of the states have established criteria for designing pavements to resist the detrimental effects of frost action. Although in design approach and detail the criteria may vary from state to state, the objectives and end results have a marked similarity.

Many design measures that are employed for reasons other than frost action are of benefit to the frost problem. On the other hand, some of the design procedures that are made primarily for the purpose of protecting pavements against frost action have other benefits. Thicker free-draining non-frost-susceptible base courses to attain adequate pavement supporting capacity during the frost melting period also improve the strength and durability of pavements under normal use. The advantages of a free-draining base as compared to a base with a relatively high percentage of clay and silt are not confined to the frost problem. Water entering the free-draining base course at the pavement edges or through joints and cracks in the pavement surface will be less detrimental to pavement supporting capacity during all seasons of the year.

It is believed that drainage trenches and ditches at the edges of wide expressways have little effect on the magnitude of frost heaving, particularly in the more fine-grained frost-susceptible soils. An exception to this would be a drain to intercept flow from a side-hill cut. The primary benefit derived from drainage trenches and ditches at the pavement edge is more rapid drainage of water released into the base course by frost melting. This drainage would tend to reduce the length of the critical period of subgrade weakening and the resultant loss of pavement supporting capacity.

ACKNOWLEDGMENTS

The author wishes to thank the personnel of the various highway departments who contributed their time in preparing informative replies to the questionnaire. Donald E. Reed prepared the section on surface soil conditions.

Frost Considerations in Highway Pavement Design: East-Central United States

K. A. ALLEMEIER and L. J. COOK, respectively, Assistant Engineer of Soils and Assistant to the Engineer of Soils, Soils Division, Office of Testing and Research, Michigan State Highway Department, Lansing

Pavement design considerations for frost conditions in the east central States are summarized, on the basis of facts furnished by the individual States. States included in the east central area are Wisconsin, Michigan, Illinois, Indiana, Ohio, Kentucky, Tennessee, Mississippi, Alabama, and Georgia. Factors influencing frost conditions are presented, such as soils and climate, including frost depth and precipitation. Design considerations for spring thaw support-loss as well as for detrimental frost heaving are discussed. Design loads and spring load restrictions are included. Use of granular subbase and subbase type, depth, and drainage are also discussed. The report compiles the bases for design considerations for frost as reported by the east central States and indicates whether design is based on experience, theoretical concepts, or both, and reports the extent of research performed by the States. Frost considerations with regard to design of culverts and structures are also included. In summary, the paper reports the extent of the frost problem in the east central States, discusses the influencing factors which cause the problem, and presents the methods and design techniques used by the various States in providing satisfactory pavement design.

• THE OBJECT of this report is to present current design considerations for highway pavements in frost areas of the east central States. Theories or details of research studies concerning frost action are not included because there are many excellent HRB and Corps of Engineers publications on the subject.

Questionnaires were sent to the east central States (Fig. 1) to determine current design practices for frost action. The first was an essay type seeking general information on the extent of the problem, research and use of findings, and basic design considerations for frost. A more detailed questionnaire pertaining to specific design practices was then circulated. This information (Tables 1 and 2) allows comparison of current design practices for similar climate and soils.

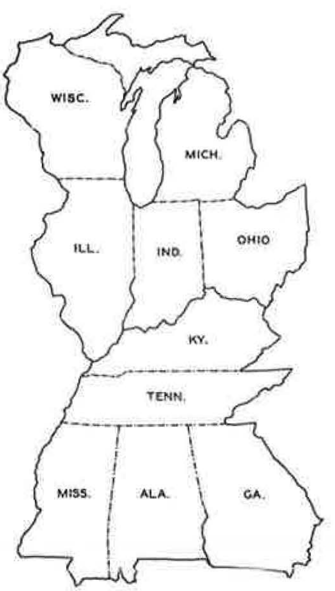


Figure 1. East central States.

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

TABLE 1
QUESTIONS REGARDING FROST INFLUENCE IN DESIGN
Parenthesized numbers refer to additional data as presented on the following pages

STATE	WHAT CRITERIA, METHODS OR TECHNIQUES ARE USED IN PAVEMENT DESIGN TO PROVIDE FOR EFFECTS OF FROST?	TO WHAT EXTENT IS PAVEMENT DESIGN IN FROST AREAS BASED ON THEORETICAL CONCEPTS AND TO WHAT EXTENT IS IT BASED ON EXPERIENCE?	WHAT STUDIES OR RESEARCH HAVE BEEN CONDUCTED IN RELATION TO EVALUATION OF FROST DAMAGE?	TO WHAT EXTENT HAVE RECENT RESEARCH FINDINGS BEEN USED IN PAVEMENT DESIGN?	ARE RIGID AND FLEXIBLE PAVEMENTS TREATED DIFFERENTLY IN REGARD TO DESIGN FOR FROST?	DOMINANT SOIL CONDITIONS ESPECIALLY IN RELATION TO TEXTURE AND ORIGIN.
ALABAMA Edward Eiland, Ass't Mat'ls and Research Engineer	We do not have a frost problem in Alabama except in about ten of the counties in the northern part of the state.	We do not consider frost action in any of our base and pavement designs. (1)	—	—	—	—
GEORGIA John M. Wilkerson, Jr., State Road Design Engineer	Subgrade drainage is the most serious design problem. To control the water, a granular subbase is provided in all major pavement designs which carries through from shoulder slope to shoulder slope on a gradient steeper than the pavement crown. In the northern third of the state, particular emphasis is given to the granular material under the shoulder to assure adequate drainage to the ditch.	Experience has shown that any pavement deterioration due to freezing is due to free moisture in the pavement. If the problem of adequately draining the subgrade is solved, as a by-product, failures caused by freezing are eliminated.	None	We feel that if we eliminate this main cause of damage from frost, i.e., moisture, we will have accomplished all that is necessary to combat frost damage. Cretaceous limestone aggregates which freeze and thaw are not used in the northern two-thirds of the state.	One basic design consideration for secondary roads is that the use of portland cement as an additive to native soils increases the stability and waterproofs the base to the extent that they will not absorb moisture. We have no record of a base course, having been stabilized with portland cement, ever failing due to freezing.	High water table is a serious problem in two-thirds of the state.
ILLINOIS E. L. Sherertz, Engineer of Design	Average frost penetration, HRB soil classification with group index, soil drainage classification and volume of truck traffic are the four indices used in the construction of charts contained in the Illinois manual "Policy on Design Thickness of Subbase, Base and Surface Courses for Highways" (Table 3 and Fig. 6). Frost penetration directly influences required subbase thickness. (2)	Current design practice is based almost entirely upon past experience.	No recent research programs on frost action. Over the years, various District highway laboratories have investigated frost heaves, and have developed data which has proved very helpful in the design of pavement structures.	The Illinois Division of Highways currently has the recently released "AASHO Interim Guide for the Design of Flexible Pavement Structures" under study to determine its adaptability to the Illinois program. It is anticipated that a similar study will be made of the AASHO guide for the design of rigid pavement when it is released. The Department reviews new procedures as they are developed to keep abreast of new methods and to check them against Illinois experience.	Rigid and flexible pavements are treated similarly in Illinois current design practice.	The greater part of Illinois has been glaciated one or more times, and soils are typical of those developed on moraines, till plains, and outwash plains. The northeastern corner of Illinois is possessed of extensive deposits of granular materials. Such materials are not prevalent further to the south and west. Central Illinois soils are more typically developed on till plains. (3)
INDIANA W. T. Spencer, Soils Engineer, Materials & Tests	This is a difficult question to answer. However, frost does affect, directly or indirectly, some of the following factors: a - Thickness of flexible pavement, b - Design of subbases and bases, c - Drainage of subbases, d - Shoulder design of paved or surfaced shoulders, e - Increased structural requirements, f - Higher quality aggregates, etc.	Primarily based on experience.	Numerous spring "break-up" surveys made by the Joint Highway Research Project, Purdue University	Adequate drainage of subbases or bases.	Yes. Flexible pavement design recognizes the reduced bearing values of various subgrades in the spring.	Predominant soils classify A-4, A-6, or A-7-6. Included are beach and dune sands in northwest area, glacial drift to south of central Indiana, and residual soils of silts, silty clays, and clays in the lower central area.

TABLE 1 (cont.)

QUESTIONS REGARDING FROST INFLUENCE IN DESIGN

Parenthesized numbers refer to additional data as presented on the following pages

STATE	WHAT CRITERIA, METHODS OR TECHNIQUES ARE USED IN PAVEMENT DESIGN TO PROVIDE FOR EFFECTS OF FROST?	TO WHAT EXTENT IS PAVEMENT DESIGN IN FROST AREAS BASED ON THEORETICAL CONCEPTS AND TO WHAT EXTENT IS IT BASED ON EXPERIENCE?	WHAT STUDIES OR RESEARCH HAVE BEEN CONDUCTED IN RELATION TO EVALUATION OF FROST DAMAGE?	TO WHAT EXTENT HAVE RECENT RESEARCH FINDINGS BEEN USED IN PAVEMENT DESIGN?	ARE RIGID AND FLEXIBLE PAVEMENTS TREATED DIFFERENTLY IN REGARD TO DESIGN FOR FROST?	DOMINANT SOIL CONDITIONS ESPECIALLY IN RELATION TO TEXTURE AND ORIGIN.
KENTUCKY W. B. Drake, Director of Research	The use of free-draining low plasticity base materials. We do not consider frost action as a primary factor in our pavement designs, undoubtedly it has effected adequate structural thickness in these designs.	Practically 100 percent on experience in the area.	A study of existing flexible pavements "Investigation of Field & Laboratory Methods for Evaluating Subgrade Support in the Design of Highway Flexible Pavements." (4)	Recent research findings have been checked in conjunction with design practices.	Insulation courses of from 3 to 6 in. depths are used under concrete pavements. Flexible pavements are constructed over graded aggregate base courses. Base drainage is provided for in both types of pavements.	Residual soils derived from limestone and sand - stones are most prevalent.
MICHIGAN A. E. Matthews, Engineer of Soils	Grade heights are maintained 5 ft. above water tables, poorly drained soils, and peat deposits. Relatively thick free-draining granular subbases are provided. Based on pedological soil classifications, design charts provide quantities for excavation of frost-susceptible materials and quantities for under drains to control seepage or capillary water. The exact locations and quantities are determined during construction, as needed. On roads which are to be reconstructed, frost heave logs are made and corrections are recommended.	Primarily based on experience. Thickness design is based on soil conditions and anticipated traffic volumes and types. Although pavement design has resulted directly from experience, design charts have been prepared which correlate the adopted thicknesses with soil strength indices such as CBR.	Pavement condition surveys, including evaluation of frost damage and subgrade support are being carried on. An intensive research program concerning frost damage was conducted in the early 1930's. A research project investigating the amount of limestone fines in limestone bases in now in progress. Preliminary reports indicate that limestone fines are more subject to frost action than natural soil binder.	Research findings relative to frost action are compared to present practices and the findings are incorporated if there is an apparent need.	Basically there are no differences. The thicker subbases required for flexible pavements are needed to provide pavement support during the spring breakup period. A rigid pavement provides more "bridging" action over unstable soils.	Michigan is a glaciated state with soils ranging from sands and gravels to loams to clays and silty clays. Many peat deposits are present. Limestone, sandstone, and igneous bedrock are present in some parts of the state.
MISSISSIPPI J. P. Steinwinder, Jr., Roadway Design Engineer H. O. Thompson, Testing Engineer	The upper layers of the pavement system under the pavement subject to freezing temperatures are generally cement-treated on main highways. From a design standpoint the primary objective is wheel load capacity and not frost penetration. Generally all designs are for all season conditions. The design varies with wheel load frequency and traffic volume expected.	—	No data	Based on experience and research, bases and subbases are being cement-treated. The local material being treated with cement consists of sand-clay, semi-gravel, and/or clay gravel. Crushed stone for roadbuilding purposes is not available within the state.	—	Predominant soils range from heavy clays to silty clays. Mississippi is a sedimentary state and the surface contains a great many soil groups. A soil profile before and after grading is required on each project for design purposes.

TABLE 1 (cont.)

QUESTIONS REGARDING FROST INFLUENCE IN DESIGN

Parenthesized numbers refer to additional data as presented on the following pages

STATE	WHAT CRITERIA, METHODS OR TECHNIQUES ARE USED IN PAVEMENT DESIGN TO PROVIDE FOR EFFECTS OF FROST?	TO WHAT EXTENT IS PAVEMENT DESIGN IN FROST AREAS BASED ON THEORETICAL CONCEPTS AND TO WHAT EXTENT IS IT BASED ON EXPERIENCE?	WHAT STUDIES OR RESEARCH HAVE BEEN CONDUCTED IN RELATION TO EVALUATION OF FROST DAMAGE?	TO WHAT EXTENT HAVE RECENT RESEARCH FINDINGS BEEN USED IN PAVEMENT DESIGN?	ARE RIGID AND FLEXIBLE PAVEMENTS TREATED DIFFERENTLY IN REGARD TO DESIGN FOR FROST?	DOMINANT SOIL CONDITIONS ESPECIALLY IN RELATION TO TEXTURE AND ORIGIN.
OHIO W. J. Cremean, Engineer of Location & Design and H. E. Marshall, Engineer-Geologist	The primary technique used in pavement design for prevention of detrimental frost effects is that of providing additional subbase thickness in soils known to be susceptible to frost action. Special consideration is also given to the utilization of subdrainage systems to the best advantage in these locations. (5)	Pavement design practice in frost areas is based primarily upon experience; however, some attention is given to the theoretical concepts and necessary adjustments are made for situations which fall out of the realm of the ordinary.	No formal studies or research have been conducted in relation to evaluation of frost damage in recent years.	As previously state, our pavement design in frost areas is largely a matter of application of facts established from past observations and experience. Research findings are reviewed and incorporated in design in those instances where established procedures need further refinement. The Corps of Engineers manuals and HRB publications of this subject have been of much benefit in our studies of frost conditions.	The conventional rigid pavement designs in use are 9 and 10 in. reinforced concrete pavement on 6 in. granular subbase. Flexible pavement thicknesses are determined on a project to project basis and have varying subbase thicknesses. Experience in Ohio has indicated the desirability of providing a thickness of granular subbase equal to one-half the depth of frost penetration for the prevention of frost-heaving. The depth of insulating material over and above that used in the original design is determined from the anticipated depth of frost penetration in a given area.	The frost susceptible soils are commonly of glacial origin, but may be found outside the glaciated portions of the state.
TENNESSEE R. S. Patton, Engineer of Surveys and Design	Inasmuch as frost penetration in Tennessee will vary from only 4 to 6 in. in depth, we do not take account of frost in our pavement design. (6)	Our soils are classified, using the Bureau of Public Roads numbering system, and based upon our previous experiences with soils of the various types encountered, we use varying thicknesses of mineral aggregate bases under both our concrete and bituminous pavements. (6)	— (6)	— (6)	— (6)	The dominant soils in the eastern and middle sections of the state are clay resulting from the decomposition of limestone. The dominant soils in the western part of the state are clay and sand in their natural state. None of these soils provide a satisfactory subgrade for pavement insofar as their load bearing capacities are concerned.
WISCONSIN J. S. Pilty, Engineer of Design	Soils have been catalogued in relation to frost susceptibility with a range of F0 to F4 where the higher numerical figure indicates the greater susceptibility. Soils engineers provide classification. Where adverse conditions are too general for elimination by cover fill, undercutting, or other economically feasible means, a granular subbase is added to the design as a correction factor.	Major developments have been based on experience with theoretical concepts cautiously taken into consideration for new designs which go beyond the scope dictated by experience. Design, in general, is based on the concept that the strength elements will not alleviate the effects of the frost action so it is necessary to take due consideration of total pavement depth and heavy vehicle traffic volumes.	—	The material compiled through national collaboration of the member states of AASHO is being intensely studied to the extent that designs are being cross-checked with a view toward elimination of as much guesswork as possible.	Not inclined to differentiate between pavement types. Since we consider good granular subbase as a structural element of flexible designs, it is more often than not that the required total depth is attained in the structural design. This would be the major difference from the rigid design since granular material added for depth protection against frost would not normally be required as a strength element in that design.	Difficult to name a dominant soil. The glacial soils cover most of the state and range from silts and clays to gravels. The southwest part of the state consists of non-glaciated soils where the parent rock consists of limestone or sandstone.

DATA SUPPLEMENTING TABLE 1 (Frost Influence in Design)

Alabama

(1) Frost action is not considered in any base and pavement designs. In the northern part of the State where there is some damage, it is concentrated only in the thin surface treatment type pavements and usually occurs about once every ten years. No damage has been reported to the high type pavements; that is, concrete or 4 in. of asphalt. The thin pavements are repaired by the application of a liquid seal and chip course.

Illinois

(2) Frequently, additional precautions are taken by removal and replacement of frost heaving soils, or utilization of subgrade drainage installations.

(3) Subsequent to the glacial age, a mantle of loess covered nearly all of Illinois. The depths of the loess vary from close to 50 ft adjacent to the major river valleys on the western side of the State to depths of such insignificance in some other areas that they may prove difficult if not impossible to detect. Many of the morainic deposits are rather complex in character in that there is a complex interbedding of materials of different grain sizes. These areas frequently necessitate the employment of short cut and fill sections in highway building, and consequent cutting of several different soil types in a relatively short distance. Such conditions are usually associated with the more severe differential frost heaves.

Kentucky

(4) This study did not deal with frost action and frost heave directly, but took into account the effect of these actions in the performance of the pavements.

Ohio

(5) The following is from Ohio's design manual:

> "E-150.00 FROST HEAVING SOILS
>
> .10 Frost heaving may occur under certain conditions of moisture and temperature in any soil which contains more than about 15 percent passing a No. 200 mesh sieve; however, it is common only in some of the very fine dirty sands, sandy silts, and silts (A-3a, A-2, A-4). In the sands and sandy silts, sufficient protection is usually afforded by adequate drainage. For the class A-4b soils, particularly in all new construction, it is advisable to replace a portion of this material with non-frost susceptible granular material. Material meeting I-22 requirements is usually used for this replacement. In the northern part of the state and in local areas where frost conditions appear to be especially severe, 18 in. of subbase should be used beneath the usual 8 or 9 in. pavement. In the central and southern part of the state, a thickness of 12 in. of frost resistant material beneath the pavement should be adequate in most cases."

Note that the effects of frost are given special attention where A-4b high silt-content soils make up the subgrade. For other soils, frost is only considered in a general way as it may affect the supporting strength of the subgrade.

Tennessee

(6) To sum up the whole matter, this department does not feel that frost action is of sufficient importance to be taken into consideration in the design of either pavements or structures. Although no definite studies or research have been conducted to evaluate frost damage, field forces in the maintenance division report such damage. To date, such damage, if any, has been so small that it is not felt necessary to take frost action into consideration in the design and construction of either roadways or bridges.

TABLE 2
DETAILED DESIGN DATA

Parenthesized numbers refer to additional data as presented on the following pages

STATE	Do you use Granular Subbase over non-granular soils? Yes or No	Reason: Spring Thaw Support Loss	Reason: Load Distribution	Reason: Pumping Control	Reason: Reduce Differential Heaving	Thickness Rigid Pri.	Thickness Rigid Sec.	Thickness Flexible Pri.	Thickness Flexible Sec.	Type: Open Graded	Type: Dense Graded	Gradation: Sieve Size	Gradation: % Passing	Subbase Drainage: Through Shoulder To Slope	Subbase Drainage: Under Drains	Does water table influence your grade height? Yes or No	Reason: Frost Damage	Reason: Subgrade Softening	Height Maintained Above Water Table
ALABAMA Edward Elland, Ass't Mat'ls and Research Engineer	Yes	—	Yes	Yes	—	6"	Not used	12"	6"	X		2" 1" 4 10 40 60 200 (1)	100 75-100 30-80 15-55 30-95 20-80 10-40	Yes	Yes	Yes	Yes	Yes	3'
GEORGIA John M. Wilkerson, Jr., State Road Design Engineer	Yes (2)	No	Yes	Yes	No	8"	0	6" & 8"	6"	X	Seldom	2" 1 1/2" 3/4" No. 10	100 95-100 30-90 25-40 (3)	Yes	Yes (where needed)	Yes	No	Yes	Keep W.T. 12" below bottom of sub-base
ILLINOIS E. L. Sherertz, Engineer of Design	Yes	Yes	Yes	Yes	Yes	0"-14" (4)	0"-14" (4)	0"-14" (4)	0"-13" (4)		X	1" 1/2" 4 8 16 200 (Crushed stone)	100 60-90 40-60 25-50 20-40 5-15	No. French drains sometimes used.	Only used where it appears necessary.	Yes	No	Yes	3 1/2' (5)
INDIANA W. T. Spencer, Soils Engineer, Materials & Tests	Yes	Yes	Yes	Yes		5" - 7 1/2"	4" min.	5"-10"	4"-6"	X	X	2 1/2 30 200	100 55 (7)	Yes	Yes	Yes	Yes	Yes	3'
KENTUCKY W. B. Drake, Director of Research	Yes (9)	Yes	Yes	Yes	No	6"	3"-5"	13" Base	8" Base	—	X (10)	1" 3/4" 3/8" No. 4 No. 10 No. 40 No. 200	100 70-100 50-80 35-65 25-50 15-30 5-15	Yes (for interstate)	Yes (for primary & secondary)	Yes	Yes	Yes	3'-5'
MICHIGAN A. E. Matthews, Engineer of Soils	Yes	Yes	Yes	Yes	Yes	14"	14"	25" Subbase 11" Base	18" Subbase 7" Base	X		2 1/2" 1" No. 100 Loss by washing	100 60-100 0-30 0-7	Yes	Yes (12)	Yes	Yes	Yes	4'-5'
MISSISSIPPI J. P. Steinwinder, Jr., Roadway Design Engineer, H. O. Thompson, Testing Engineer	Yes (15) (16) (17)	No	Yes	No	No	0-18"	0-18"	3"-25"	3"-19"	X (18)	X (18)	4 10 40 60 270 Silt Clay	100 25-100 20-100 15-85 4-35 0-20 0-20	Yes	Yes (19)	Yes	No	Yes	3'-4'
OHIO W. J. Cremean, Engr. of Location & Design, H. E. Marshall, Engr.-Geologist	Yes	Yes	Yes	Yes	Yes	6"-24" (23)	Not used	4"-18" (23)	4"-12" (23)	X	X (24)	3" 2" 1" 200 (25)	100 90-100 70-100 0-15 (25)	No	Yes (26)	(27)	No	No	—
TENNESSEE R. S. Patton, Engineer of Surveys and Design	Yes	No	Yes	Yes	No						(31)	1 1/4" 1" 3/8" 4 16 100	100 95-100 50-60 36-65 20-40 8-15	Yes	Yes				
WISCONSIN J. S. Pilty, Engineer of Design	Yes (34)	Yes	Yes	—	Yes	6"-9"	6"-9"	6"-12"	6"-12"	X (35)		Grade 1 No. 4 No. 40 No. 100 No. 200 (36)	100 0-75 0-15 0-8	Yes (37)	Do not use underdrains where it is feasible to drain thru shoulder to slope.	Yes	Yes	Yes	4' ±

STATE	Are spring load restrictions required to project softened subgrades during the frost melting period? Yes or No	Restricted: Single Axle, lbs	Restricted: Tandem Axle, lbs	Normal: Single Axle, lbs	Normal: Tandem Axle, lbs	Do frost bumps (frost heaves) occur in your State? Serious Enough to Require Correction	Type of Treatment: Re-place	Type of Treatment: Mix	Type of Treatment: Raise Grade	Type of Treatment: Other	Does frost in any way influence design of Culverts? Yes or No	In What Way?	Do you require that structure footings be placed below the depth of normal frost penetration? Yes or No	Depth	Average frost penetration for your State: North Part	South Part
ALABAMA	No	—	—	18,000	32,000	No	—	—	—	—	No	—	No		3"	0"
GEORGIA	No	—	—	20,340	40,680	Rarely	Yes	—	—	—	Yes	Selected Backfill	Yes	Below frost line, if any.	4"	0
ILLINOIS	—	Not restricted on primary, variable on secondary.	Not restricted on primary, variable on secondary.	18,000	32,000	Yes (6)	Yes	Yes	Yes	Drainage	No	—	Yes	4'	54" max.	6" min.
INDIANA	Very Seldom	—	—	18,000	32,000	Yes (8)	Yes	Yes	Yes	—	Yes	2' of cover over pipe	Yes	Indefinite	25"	10"
KENTUCKY	Yes (11)	—	—	—	—	No	—	—	—	—	No	—	Yes	18"	12"	9"
MICHIGAN	No (13)	75% of normal for rigid, 65% of normal for flexible	75% of normal for rigid, 65% of normal for flexible	18,000	26,000 (32,000 on main routes)	Yes	Yes	Yes	Yes	—	Yes	(14)	Yes	5' below ground cover.	54"	40"
MISSISSIPPI	No (20)	—	—	18,000 (21)	32,500	Yes (22)	Yes	—	—	Chemical treatment	No	—	Yes	1' 6"	3"	1"
OHIO	Yes (28)	14,300	24,500	19,000	32,500	Yes (29) (30)	Yes	—	—	—	No	—	Yes	4'	24"	10"
TENNESSEE	No (32)			18,000	32,000	No					No	(33)	(34)		6" max.	4" max.
WISCONSIN	Yes	(38)	(38)	18,000 (39)	30,400 (39)	Yes	Yes	—	Yes	—	No	—	Yes	4'	60"-70"	40"-50"

DATA SUPPLEMENTING TABLE 2 (Detailed Design Data)

Alabama

(1) In addition to the sieve requirements, subbase material is further limited as follows: clay, 20 percent maximum; liquid limit, 26 maximum; plasticity index, 6 maximum.

Georgia

(2) Subbase is also used to provide for subgrade drainage.

(3) The gradation for subbase material is varied from job to job to utilize local materials.

Illinois

(4) Subbase thickness for Interstate routes or routes having more than 1,600 trucks per day ranges from 6-in. minimum to 14-in. maximum. Depth of subbase is based on drainage, frost penetration, and soil type. No subbase is required over adequate native granular subgrade soils. If other soils are involved, 4-in. minimum subbase is used under rigid pavement and 3- to 4-in. minimum under flexible depending on class of highway.

(5) The height of grade above the water table may be varied with the anticipated depth of frost penetration.

(6) Differential frost heaves have been experienced over wide areas in Illinois, but in general, it may be stated that such differential heaving increases in frequency and severity in the northern sections. Experience indicates that the worst heaves are associated with cut sections or in zones of transition from cut to fill. Localized heaves have been experienced during periods of severe cold that have heaved differentially several inches and constitute a definite hazard to the motorist. The spring breakup is a real problem in these areas.

Indiana

(7) Indiana specifications provide for two types of subbase:

C1102.1. Gradation Requirements for Type 1 (Open-Graded)

Sieve sizes through which substantially all material passes. Approx. top size.	Total Percent Passing Sieves Having Square Openings									
	2-1/2"	2"	1-1/2"	1"	3/4"	1/2"	No. 4	No. 8	No. 30	No. 200
2"	100	95-100	75-98	60-90	50-85	40-80	25-60	15-45	5-25	0-5*
1-1/2"		100	95-100	75-98	60-90	45-85	25-65	15-50	5-25	0-5*
1"			100	90-100	75-98	60-90	30-70	20-55	5-30	0-5*
1/2"			100			90-100	50-90	30-70	10-40	0-5
No. 4			100				95-100	80-95	20-55	0-5

* In addition to its other requirements, the amount passing the No. 30 sieve shall not be less than two times the amount passing the No. 200 sieve.

C1102.2. Gradation Requirements for Type II (Dense-Graded)

Passing the 2-in. square sieve, percent 95-100
Passing the No. 4 square sieve, percent............... 35-100
Passing the No. 30 not more than 55

The material shall contain sufficient binding material (that portion passing the No. 200 sieve) to compact satisfactorily; however, such binding material shall not be less than 5 percent. If a method of draining the subbase material in place is provided, then the binding material shall be between 5 and 10 percent. If a method of draining the subbase material in place is not provided, then the binding material may exceed 10 percent provided the fraction passing the No. 200 sieve is not greater than one-half the fraction passing the No. 30 sieve, nor greater than one-fifth the fraction retained on the No. 30 sieve.

(8) The most severe differential frost heave problems are generally encountered in localized areas of wet, extremely fine sands and silts and may be found anywhere in Indiana. These materials are generally excavated to depth of 2 to 3 ft below subgrade.

Kentucky

(9) Primarily load distribution and pumping control.

(10) Dense-graded aggregate used for base for flexible and as a subbase for rigid.

(11) Very light initial treatment pavements are not adequate for frost penetration, and it has been the policy to restrict loads for spring thaw conditions on some of these.

Michigan

(12) Drainage of subbase through shoulder to slope is standard. However, in urban sections where curb and gutter is used, underdrains are used for subbase drainage.

(13) On the older trunklines, load limits are required. For the past few years, all roads have been designed for year-round legal loads.

(14) Depth of cover over pipe. Selected backfill.

Mississippi

(15) Subbase also used to prevent the intrusion of fine-grained soils into the base course and maintain moisture content more uniform for all seasons.

(16) Have in the past but discontinuing this practice on expansive fine-grained very plastic soils.

(17) When required by reference on chart (not shown), any subgrade (design soil) with CBR of 5 or less shall be lime-treated; except that when a project contains a few short, isolated sections of subgrade material, the thickness shown on the charts (not shown), in the zero treatment column, may be used.

When the subgrade material (design soil) has a CBR of 6 to 10 and the soil and weather conditions warrant, consideration will be given to the use of lime treatment or of plating material classified as a 4-6 plastic or better. Plating material will not be considered a part of the structure thickness.

The granular subbase shown in the charts may be reduced or eliminated if economically justified, by any of the following: (a) increasing depth of treated subgrade; (b) increasing depth of treated base; and (c) providing soil-cement or cement-treated subbase. (The depth in each case to be equal to 75 percent of the depth of replaced granular subbase.)

(18) This gradation is an example of Class 9, Group C; maximum liquid limit, 30; maximum plasticity index, 10.

(19) Use underdrains when necessary for proper subbase drainage.

(20) Loads are restricted on some secondary roads where the structure thickness is inadequate for legal loads during the spring season.

(21) Legal wheel load limit of 9,000 lb.

(22) Occasionally have frost heaves on D.B.S.T. pavements, but not serious enough to influence our design.

Ohio

(23) The thicker subbases are used over silt soils only (more than 50 percent silt and plasticity index less than 10).

(24) Most Ohio subbase material is natural sand and gravel and is fairly dense graded.

LEGEND FOR SOILS MAP

PODZOL SOILS

- CA — Caribou
- IM — Iron River - Milaca
- OT — Ontonagon - Trenary
- RR — Roselawn - Rubicon

GRAY-BROWN PODZOLIC SOILS

- CBL — Clinton - Boone - Lindley
- FL — Fairmount - Lowell
- HF — Hagerstown - Frederick
- MB — Miami - Crosby - Brookston
- MK — Miami - Kewaunee
- MWZ — Muskingum - Wellston - Zanesville
- PA — Porters - Ashe
- PC — Plainfield - Coloma
- W — Westmoreland
- WM — Wooster - Mahoning
- SP — Spencer

WIESENBODEN, GROUND WATER PODZOL, AND HALF-BOG SOILS

- LB — Leon - Bladen
- NM — Newton - Maumee
- TV — Toledo - Vergennes
- CPB — Coxville - Portsmouth - Bladen

ALLUVIAL SOILS

- A — Alluvial soils

PLANOSOLS

- PV — Putnam - Vigo - Clermont

PRAIRIE SOILS

- CC — Carrington - Clyde
- CW — Clarion - Webster
- TM — Tama - Marshall

RED AND YELLOW PODZOLIC SOILS (Lateritic Materials)

- DB — Dickson - Baxter
- DDC — Decatur - Dewey - Clarksville
- GM — Greenville - Magnolia
- MG — Memphis - Grenada
- MH — Maury - Hagerstown
- NR — Norfolk - Ruston
- SSR — Susquehanna - Savannah - Ruston
- TI — Tifton - Irvington
- NS — Norfolk sands
- GA — Georgeville - Alamance

LITHOSOLS AND SHALLOW SOILS (Humid)

- HM — Hartsells - Muskingum
- TF — Talladega - Fannin
- UM — Upshur - Muskingum
- RI — Undifferentiated rough, stony land and shallow Podzols (forested)

BOG SOILS

- P — Peat and Muck

RENDZINA SOILS

- SV — Sumter - Vaiden

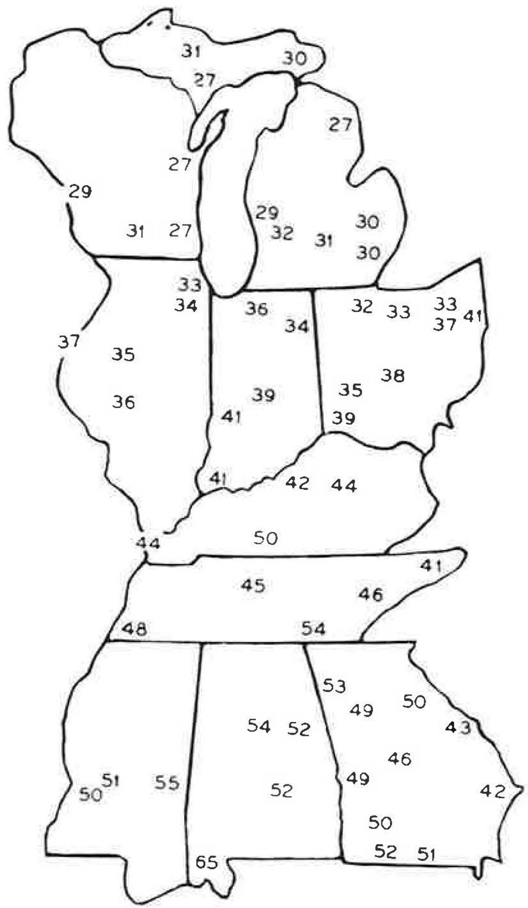


Figure 3. Average annual precipitation in inches: 1921-1950 (U.S. Department of Commerce, Weather Bureau records).

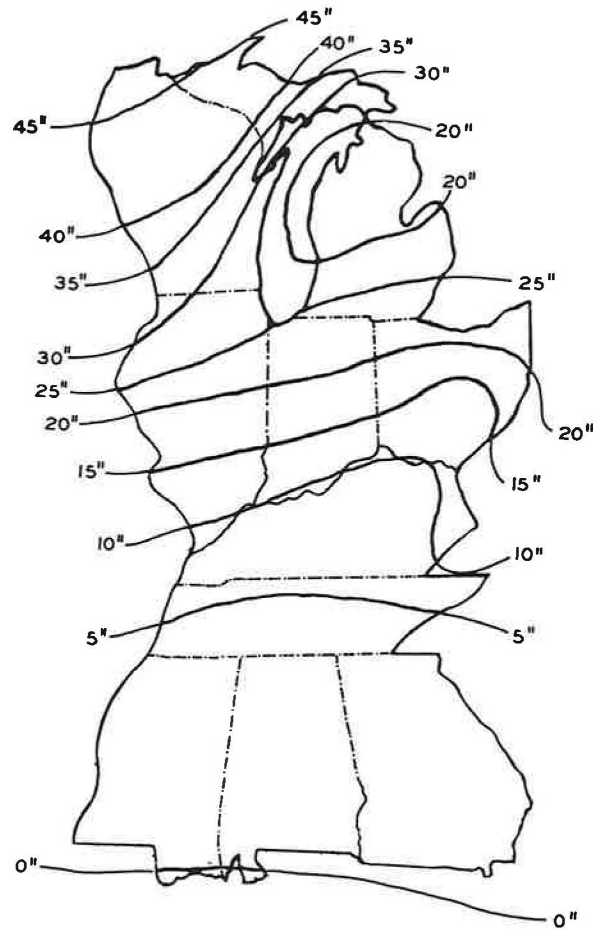


Figure 4. Average annual frost penetration, in inches (based on State averages after U.S. Weather Bureau).

than three-quarters of the thickness of such subbase course or layer being placed. The liquid limit of the material shall not be greater than 25 and the plasticity index shall not be more than 6.

(37) There is evidence that the subbase drainage may be blocked at shoulder by top-soil added for seeding.

(38) Wisconsin law states that gross weight limitations on Class "B" highways are 60 percent of the Class "A" tolerated weights.

(39) Statutory excluding tolerances.

SOILS AND CLIMATE

In the section of the United States considered in this report, the geology and soils have a wide range. The geology varies from the glaciated areas of Michigan, Wisconsin, and the northern parts of Ohio, Indiana, and Illinois to the mountainous regions of Kentucky, Tennessee, and northern Georgia, and the alluvial deposits of Mississippi and the residual deposits of the southern States.

There is also great variation in soil textures throughout the area, ranging from sands and gravels to loams to clay and silty clay, as well as bedrock regions of the mountainous areas. The great soils groups consist of the following: podzols, gray-brown podzols, groundwater podzols, prairie soils, planosols, red and yellow podzols or laterites, lithosols (humid), chernozems, and rendzinas. With this wide range of soils, it is difficult to draw conclusions or to make comparisons. The States in which the frost problem is most severe are those in which the podzol, gray-brown podzol, or groundwater podzol soils predominate. A generalized soil map of the region is shown in Figure 2.

Climatic conditions within the east central States vary extremely from northern Michigan on the Canadian border to southern Mississippi on the Gulf of Mexico. For this reason, no attempt at generalization would be significant in relation to pavement design throughout the area. For climatic conditions within any given State, Figures 3 and 4 show average annual precipitation and average annual frost penetration, respectively.

CONCLUSIONS

In the opinion of the writers, considerations for frost effects in pavement design fall essentially in two categories:

1. Spring thaw support loss, or the loss of bearing capacity of the natural subgrade soil, as the frost leaves the ground in spring.

2. Frost heaving during the freezing period which may cause cracking and destruction of the pavement or in severe cases may even be hazardous to traffic.

By far the more important of the two is the problem of loss of support at the time the frost leaves the ground. In some cases, it appeared that answers to the questionnaires did not discuss this aspect to the extent expected, possibly because it is more an indirect effect and occurs after the frost has left. It is noted, however, that all States use a granular subbase over non-granular soils. And in most cases, the thickness depends on soil classification, group index, CBR, etc. By such means the soils which undergo the greatest strength loss in the presence of water require the strongest pavement design. It appears, therefore, that even in the southern States other sources of moisture such as precipitation and water table notwithstanding, pavement design does provide protection against support loss during the frost melting period. In a northern State such as Michigan, there is no question that the weakest subgrade condition which must be designed for occurs during the frost melting period and is a direct result of the excess moisture accumulation caused by frost action.

Figure 5. Extreme example of pavement damage resulting from frost action.

The second important design consideration for frost effect is protection against heaving. In all but the most uniform of frost-susceptible soils, heaving can cause pavement cracking and shortened pavement life (Fig. 5). Generally coincident with heaving is a rough riding surface. In extreme cases, local frost heaves are dangers to traffic. Subbase thicknesses which provide for load distribution during the spring thaw also automatically provide a cushion which helps to damp differential frost heaving. Michigan, for example, with extremely variable glacial soils and deep frost penetration, designs for pavement smoothness and added pavement life by use of subbase thicknesses adequate to reduce a large percentage of the minor differential heaving caused by variable soil textures. Wisconsin, Michigan, Illinois, Ohio, Indiana, and Mississippi reported that frost bumps or sharp frost heaves are a problem serious enough to require correction. The prime solution to the problem in all States seems to be replacement of the heaving soil with a non-heaving material. Mississippi also reported chemical treatment.

Although it is assumed that the subject of paving aggregates is beyond the scope of this symposium, chert, soft stone, iron concretions, etc., are destructive aggregates which must be considered in pavement design in frost areas. Air entrainment in portland cement concrete is a similar consideration.

Although the southernmost States of Alabama, Georgia, and Mississippi generally report that frost is of very little consequence, their reports do reveal certain design considerations which, although not primarily established for frost reasons, do provide protection against the minor freezing conditions which occur.

Mississippi generally cement-treats the upper layers of the pavement system under the pavement subject to freezing temperatures, although they report that from a design standpoint the primary consideration is wheel-load capacity and not frost penetration.

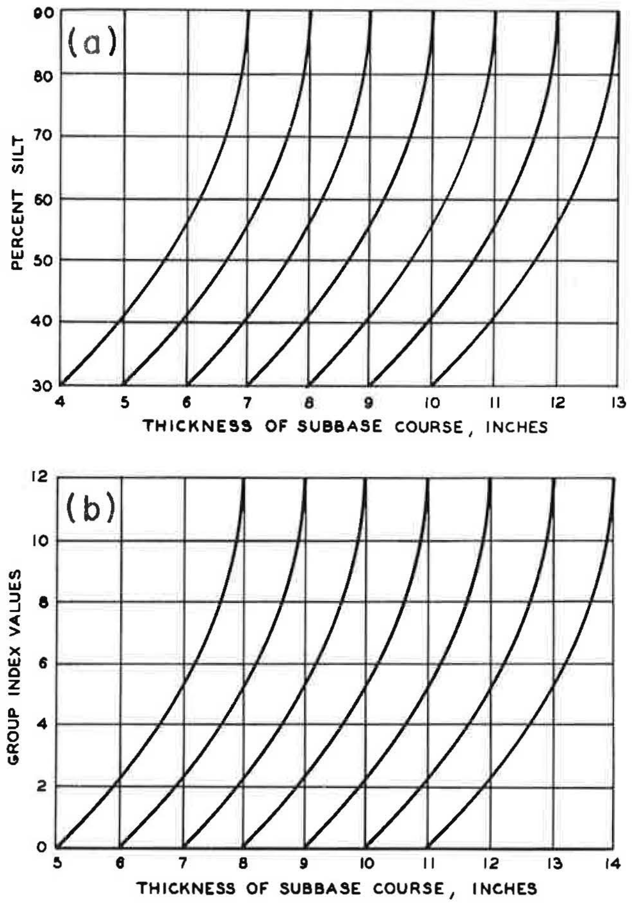
Georgia reports that high water table in two-thirds of the State causes subgrade drainage to be the most serious design problem and that by adequately draining the subgrade, any failures caused by freezing are eliminated as a by-product. Georgia further reported that on less-traveled roads, native soils are stabilized by portland cement thereby waterproofing them to the extent that water is not absorbed, thus eliminating any damage due to freezing. Also interesting is Georgia's experience with cretaceous limestone which cannot be used in the northern two-thirds of the State because the material freezes and even heaves with only a light freeze of short duration.

Of the ten States in the east central area, it appears that Illinois is the only one that employs design criteria using a frost penetration index in establishing individual pavement design. As can be seen from Table 3 and Figure 6, pavement thickness is determined by four factors, namely: soils classification, drainage, average frost penetration, and volume of truck traffic. Table 3 and Figure 6 are included in the Illinois "Policy on Thickness Design of Subbase, Base and Surface Courses for Highways."

TABLE 3
SUBBASE COURSE THICKNESSES IN INCHES
FOR USE WITH PORTLAND CEMENT CONCRETE PAVEMENT
ON HIGHWAYS CARRYING 160 TO 800 TRUCKS DAILY
From Illinois "Policy on Design Thickness of Sub-base, Base and Surface Courses for Highways" as revised September 29, 1951

Foundation Soils Group Classification	Good Drainage			Fair Drainage			Poor Drainage			Very Poor Drainage		
	Average Frost Penetration, in.			Average Frost Penetration, in.			Average Frost Penetration, in.			Average Frost Penetration, in.		
	0-18	18-36	36-54	0-18	18-36	36-54	0-18	18-36	36-54	0-18	18-36	36-54
A-1-a	0	0	0	0	0	0	0	0	0	0	0	0
A-1-b	0	0	0	0	0	0	0	0	0	0	0	0
A-3	0	0	0	0	0	0	0	0	0	0	0	0
A-2-4	0-4*	0-4*	0-4*	0-4*	0-4*	0-4*	0-6**	0-6**	0-6**	0-6**	0-6**	0-6**
A-2-5	0-4*	0-4*	0-4*	0-4*	0-4*	0-4*	0-6**	0-6**	0-6**	0-6**	0-6**	0-6**
A-2-6	4	4	4	4	4	4	6	6	6	6	6	6
A-2-7	4	4	4	4	4	4	6	6	6	6	6	6
A-4	4	4	4	4-7[a]	5-8[a]	6-9[a]	6-9[a]	7-10[a]	8-11[a]	8-11[a]	9-12[a]	10-13[a]
A-5	5	5	5	5-8[b]	6-9[b]	7-10[b]	7-10[b]	8-11[b]	9-12[b]	9-12[b]	10-13[b]	11-14[b]
A-6	4	4	4	4	4	4	6	6	6	6	6	6
A-7-5	4	4	4	4	4	4	6	6	6	6	6	6
A-7-6***	4	4	4	4	4	4	6	6	6	6	6	6

a. see Fig. 6a
b. see Fig. 6b
* Use 4 in. when material is not well graded and plasticity index exceeds 6.
** Use 6 in. when material is not well graded, plasticity index exceeds 6, and drainage is poor or very poor.
*** A-7-6 soils composed of peat and muck should not be used as foundation soil.



Portland Cement Concrete Pavement Thickness, in.	Daily Truck Traffic
8	160 to 400
9	400 to 800
10	over 800

Figure 6. (a) Design thickness of subbase course for A-4 foundation soil; and (b) design of subbase course for A-5 foundation soil (2).

One of the more prominent conclusions which can be made from information supplied by the east central States involves the extent to which design is based on theoretical concepts or on experience. The reply from Wisconsin generally typifies the latter group in that "the major developments for design against frost have been based on experience, with theoretical concepts cautiously taken into consideration...."

From a review of the tabulated answers, it appears also that nearly all the States provide drainage for their subbase sections by means of through-shoulder drainage or underdrains, at least on primary routes. In the opinion of the writers, this is an extremely important consideration in maintaining subgrade stability, especially during the critical frost melting period.

As a final conclusion, it is noted that the replies regarding research performed or in progress indicate only a slight amount of activity in this area. It occurs to the writers, however, that the term research is probably being interpreted as intense, formal programs of field or laboratory investigation. And it could be interpreted that lack of this activity indicates poor engineering—which may not necessarily be the case. In fact, many engineers believe that in many respects the pavements now in existence constitute the only dependable sources of information on which to base future designs, and the writers believe this is the case with most of the States reported here in the east central area. In Michigan, certainly, the dominant feeling is that the performance of in-service roads furnishes the best information for future design.

REFERENCES

1. Jenkins, Belcher, Gregg, and Woods, "The Origin, Distribution and Airphoto Identification of U. S. Soils." Fed. Aeronautics Admin. Tech. Dev. Rpt. No. 52 (May 1946).
2. "Policy on Design Thickness of Sub-base, Base and Surface Courses for Highways." Illinois Div. of Highways (Rev. Sept. 29, 1951).

Discussion

K. B. WOODS, Purdue University.—The authors are to be complimented for putting together good design information for frost conditions in the east central States. The answers to the questionnaires and material from other sources produce reasonably good boundaries for the problem for this area. It will be interesting to see how this material fits in with the remaining portions of the United States and with the material from Canada.

This discusser has studied the frost problem in the midwest for the past 30 years and offers some additional information as a supplement to this paper. Figure 7 (1) is an engineering soils map of the region under discussion and can be used as an addition to the authors' soils map of east central States (Fig. 2). It is to be noted that this soils map is a combination of geologic, pedologic, and textural classifications. It lends itself readily to use in pavement design for frost conditions. The following are a few illustrations:

Young Drift Soils

Many frost problems are encountered in transition between cut and fill sections in the till plains (Crosby-Brookston soils). The textural difference between the silty "A" horizon and the plastic "B" horizon is great. The problem is less severe with modern design because high-level grade lines are used, thus avoiding the problem in transitions.

Also, in this region there are many deposits of shallow sands on till. Here, too, the frost problem can be severe in the transition area. This is noticeably true in northwestern Indiana, many areas of Michigan, and, of course, in large areas of southern Ontario.

Old Drift

The old drift of the region under consideration is confined to southern Illinois, southern Indiana, and a small portion of southwestern Ohio. This soil area is generally flat but where erosion has cut through the "A" and "B" horizons by way of deep gullies or even small streams, highways crossing these areas frequently are in trouble when the grade line cuts through the transition between these horizons.

Windblown Silt and Young Drift

The region under consideration has substantial deposits of loess and the frost problem is of considerable magnitude in western Wisconsin, western Illinois, and in

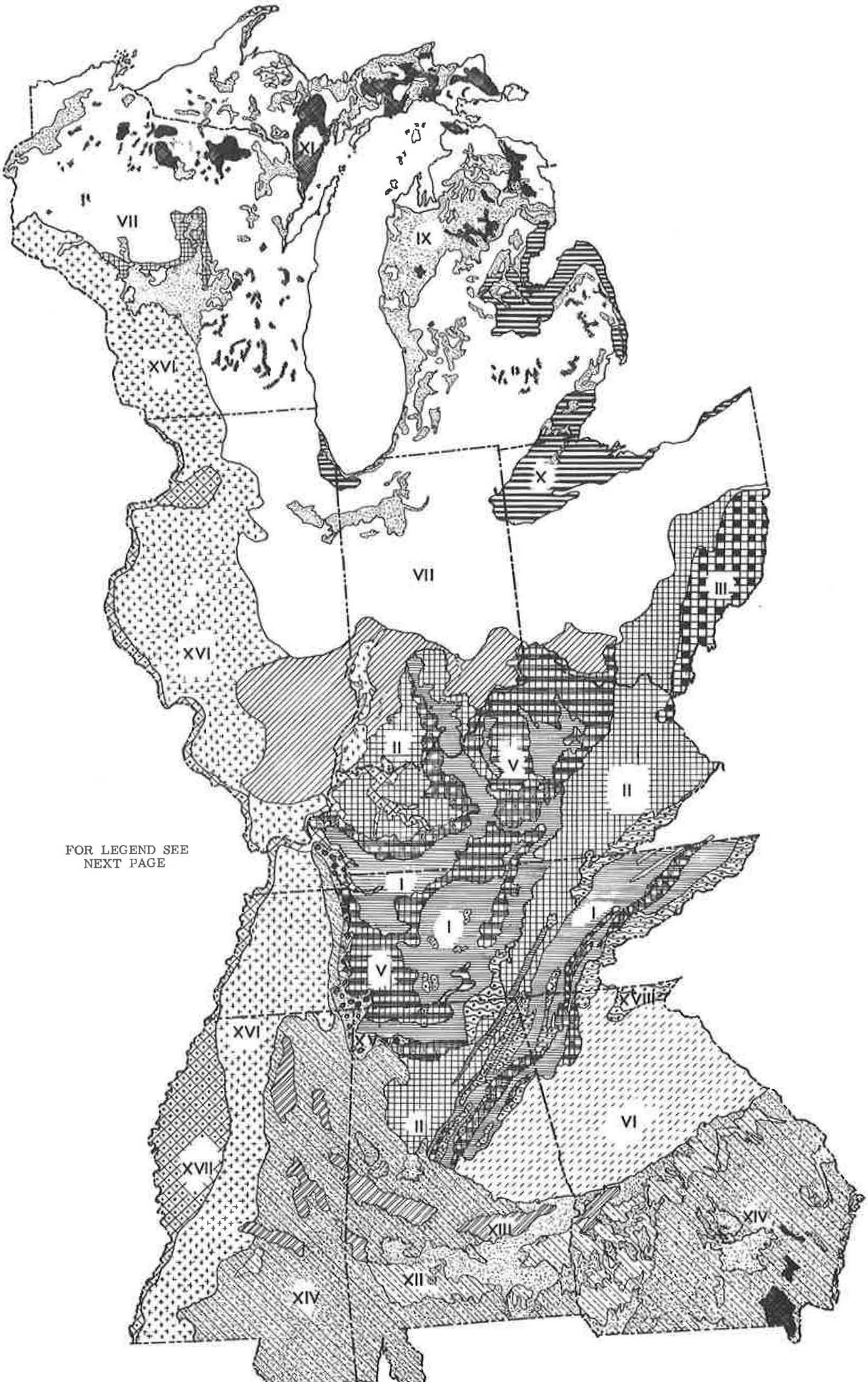
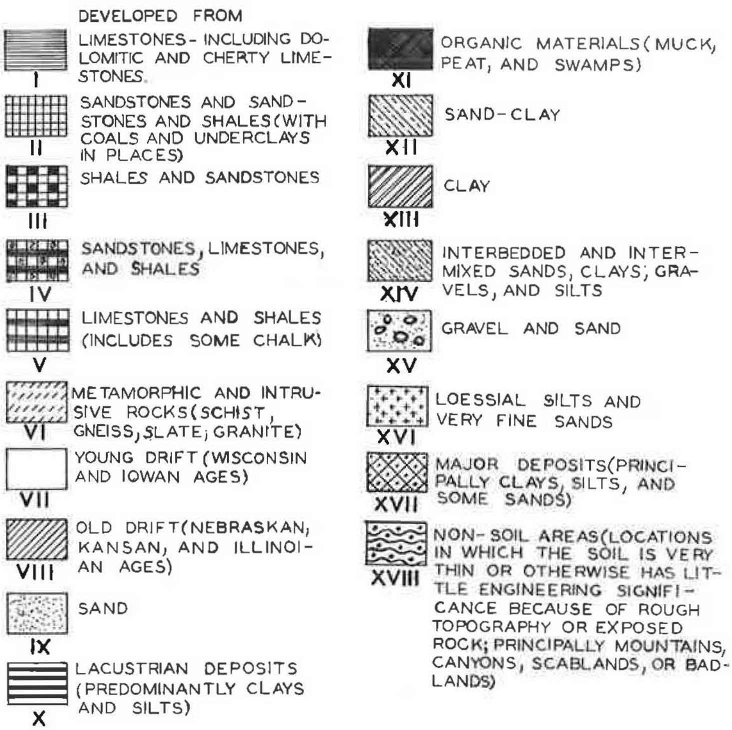


Figure 7. Engineering soils map of east central States (1).

LEGEND FOR SOILS MAP



smaller sections in southwestern Indiana. The silts are quite permeable and when a highway grade line is established close to the transition between the silt and the underlying drift, serious water problems frequently are encountered. Consequently, frost problems are to be expected unless corrective design techniques are employed.

Summary

In those areas where the frost penetration is sufficient to require design considerations, transition zones between soils of unlike textures should receive attention. These layers may be of natural origin such as a natural interbedded-layered system or in cut sections through natural soil profiles.

Frost Considerations in Highway Pavement Design: West-Central United States

F. C. FREDRICKSON, Assistant Materials and Research Engineer, Minnesota Department of Highways

•ASSESSING the harmful effects of frost action on highways and adjusting highway design to eliminate the harmful effects is a major effort in frost areas. The problems are roughness resulting from freezing, weakening of road structures on thawing, and the deterioration of materials and structures resulting from freeze-thaw. The number of problems, their seriousness, and the nature of corrective action depend on the severity of the frost action which is related to geographic location.

The area considered in this report includes Arkansas, Oklahoma, Missouri, Kansas, Nebraska, Iowa, South Dakota, North Dakota and Minnesota.

GENERAL INFORMATION

This area involves regions of diverse climate and topography, ranging from the forest and lake region of northern Minnesota through the vast plains and lowlands to the Ozarks in Missouri and Arkansas. It can generally be subdivided into three physiographic provinces: the Great Plains, Central Lowlands, and the Ozark Plateau region (Fig. 1).

The Great Plains region is part of the high Piedmont area located at the foot of the Rockies. Elevations gradually rise from 1,000 ft in the east to 5,000 ft in the west. Grazing and winter wheat farming reflect the moisture deficiency of the area.

Elevations in the Central Lowlands are fairly uniform ranging from 500 to approximately 1,500 ft. This province, trending north-south through the area, forms the basis for the rich agricultural economy of the Cotton Belt, Corn Belt, and the Spring Wheat regions in the Dakotas.

The Ozark Plateau, located in the southeast portion of the study area, stands at 1,500 to 2,000 ft elevation. The plateau is composed primarily of sedimentary rocks. Early settlement occurred here because of a good supply of timber and spring water leading to a small farm type of agriculture. Today the area produces fruit and truck farm products.

The Rocky Mountains, located west of the study area, have a direct influence on the climate of the area, especially in regard to precipitation. Precipitation and temperature seem to have a more direct bearing on soil formation and frost problems than does topography, and for this reason climatic relations will be discussed more fully than topography. W. Koppen and R. Geiger in "Hanbuch der Klimatologie" (1936) devised a climatic classification based on temperature and precipitation measurements (Fig. 2). Their climatic zones serve as convenient divisions for the discussion of frost conditions in the nine-state area.

Climatic Region BS_w

In western North Dakota, South Dakota, Nebraska, Kansas, and in the Oklahoma Panhandle (at the eastern foot of the Rockies) is a semi-arid "Steppe Climate" classified by Koppen as BS_w (Fig. 2). This type of climate normally occurs in continental interiors where mountain barriers shut off rain-bearing winds. This region suffers from a precipitation deficiency, a high evaporation rate, and in addition, has a severe

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

daily temperature range which causes it to have the highest frequency of freeze-thaw cycles of any of the four regions discussed (Fig. 3). Generally, the Rockies depress the growing season by 40 days and the mean annual temperature by 10 to 15 degrees. Soils formed in this semi-arid climate are lime-accumulating Chestnut and Brown soils of the Pedocal group which develop under a grassland cover. These soils are arranged in north-south belts in Central United States succeeding one another from east to west as aridity increases, until desert soils replace them west of the study area (1). The A horizon is thin; decreasing or increasing with annual precipitation. Zones of lime accumulation usually occur 3 to 5 ft below the surface in the upper part of the B horizon. Due to aridity of the soils and a high evaporation rate, but in spite of a high freeze-thaw frequency, frost damage is not as serious here as in the areas of higher precipitation farther east. Oklahoma and Kansas report frost is not a very serious problem.

Climatic Region Cfa

Farther east, away from the Rockies, the Gulf of Mexico has a moderating effect on temperatures. It also increases precipitation and lengthens the frost-free season by 30 days (3, p. 16). This is the zone of Koppen's "Cfa Humid Sub-tropical Climate" which extends from the coast inland almost to Iowa (Fig. 2). It includes most of Kansas, Missouri, Oklahoma, and all of Arkansas. The climate is warm and temperate with rain occurring in all seasons. Mean annual precipitation ranges from 24 to 56 in. (Fig. 4). Frost occurs during 5 to 7 months of the year, with the soil freezing 1 to 4 in. in the south and 6 to 18 in. in the north (Fig. 3). Soils located in this zone are least affected by frost. Missouri and Arkansas report few or no frost problems. Due to a high annual precipitation, non-lime-accumulating soils of the Pedalfer group developed in this zone. The Prairy soils in the southern Central Lowlands formed under grasslands from parent material of decomposed limestones. The Ozarks, because of higher precipitation (52 in.) developed Red and Yellow soils under a heavy forest cover. The A and B horizons are relatively thick and strongly leached.

Climatic Region Dfa

Iowa, southern Minnesota, southeastern South Dakota, eastern Nebraska, and small parts of Kansas and Missouri are included in the Dfa Climate, differing only from the Cfa Climate in that it has colder winters. Black soils of the Pedocal group are found in South Dakota, but Prairy soils of the Pedalfer group developed in Iowa and southern Minnesota. The west boundary of the Prairy soils shows a definite correlation with annual precipitation and generally follows the north-south 24 in. precipitation isoline, despite divergent parent material and topography. A large part of this area was heavily glaciated, generally north of the Missouri River. In the glaciated area, Prairy soils developed on the older transported glacial tills and wind-blown loess deposits associated with the Nebraskan, Kansan, Iowan, and Wisconsin glacial advances. Directly south of the Missouri River, similar residual soils developed on unglaciated parent material. Mean annual precipitation for the Dfa region ranges from 24 to 32 in. The soil normally freezes 18 to 36 in., with only 3 to 4 months without frost (Fig. 3). Freeze and thaw cycles based on the difference between the annual number of nights with frost and the number of days continuously below freezing, range from 90 to 100 per year (3, p. 135). Frost problems are more complex in this area because glaciation has produced a high diversity of soil types and poor drainage. All states involved report frost problems.

Climatic Region Dfb

The Canadian Climate (Dfb) dominates North Dakota, northern Minnesota, and northeastern South Dakota. This area includes large dairying and spring wheat agricultural regions. It has snowy, cold winters and moderately warm summers. Mean annual precipitation ranges from 16 to 28 in. with most falling during the crop season (Figs. 4 and 5). Freeze-thaw cycles, based on the frequency of a temperature of 28° or lower followed by one of 32° or higher, range from 68-91 per year (Fig. 3). All states in the area report difficult frost problems with extensive studies being made. In 1941, F. C.

Figure 1. Major physiographic provinces (1, Fig. 448, p. 650).

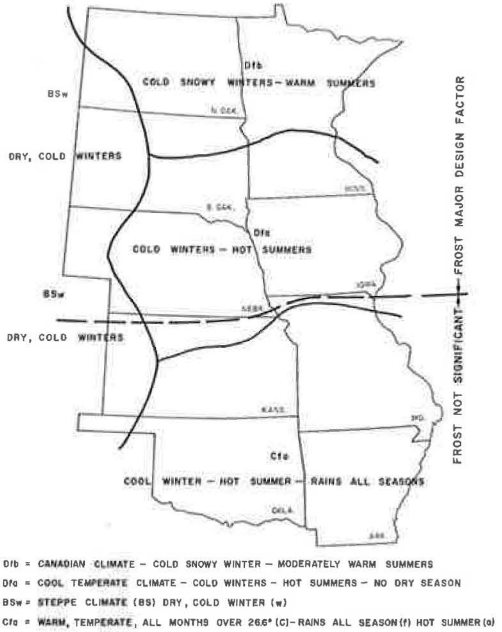


Figure 2. Climatic regions (3, Fig. 981).

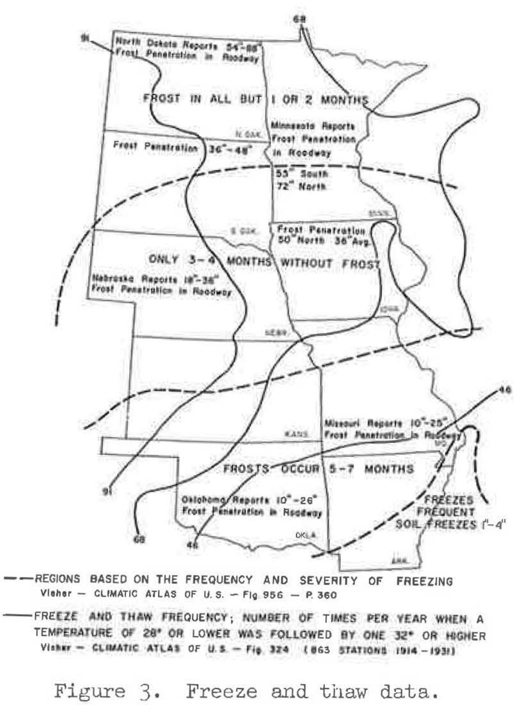


Figure 3. Freeze and thaw data.

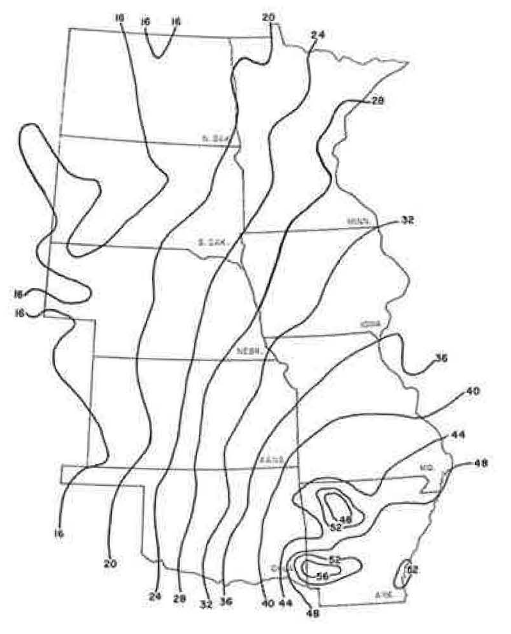


Figure 4. Mean annual total precipitation, in inches (U. S. Weather Bureau).

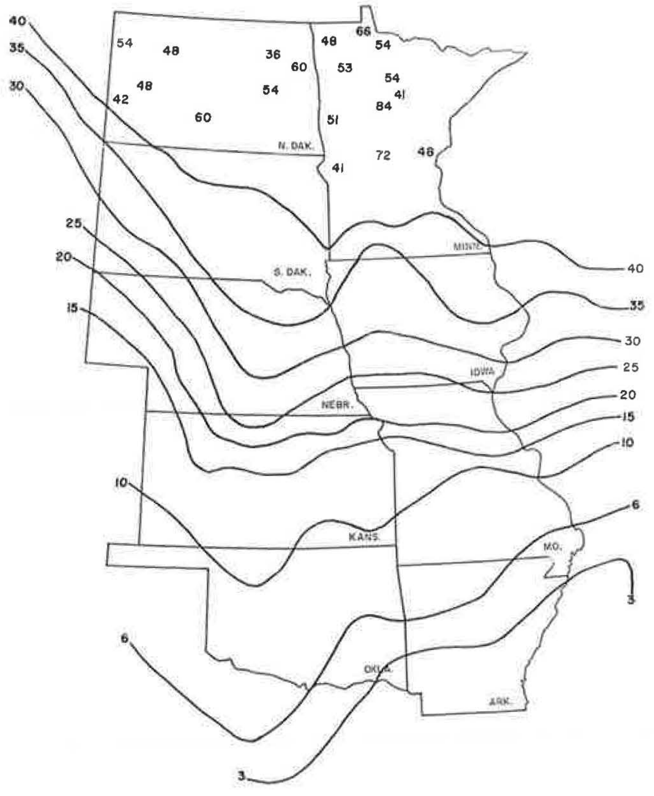


Figure 5. Average depth of frost penetration, in inches, in normal terrain not in roadway, 1899 to 1938 (4).

Lang (2) reported on frost penetration and freeze-thaw cycles occurring in a concrete slab and subbase section exposed to the elements in Minneapolis, Minn. This showed a total frost penetration in excess of 60 in. Freeze-thaw cycles for the winter observed varied from 43 at the surface of the 7-in. slab to 14 at the bottom. Only one cycle was recorded at the 60-in. level (2).

Soil types in the Dfb region range from the infertile Podzols in heavily-wooded northern Minnesota, to the rich Black soils of the Pedocal group developing on the lacustrine deposits of glacial Lake Agassiz. The area was glaciated during the late Wisconsin ice advance and the topography reflects the youthful features of this recent glaciation: massive terminal moraines, young soils, numerous swamps and poor drainage. Minnesota has the most complex glacial history, receiving successive invasions from the Keewatin (grey drift) and Patrician (red drift) ice centers.

PROBLEMS ASSOCIATED WITH FROST ACTION

Because frost problems are related to depth of frost penetration, the problems caused by frost action are most numerous and serious in the northern portion of the West Central States where the penetration is deep, but become less significant southward as frost penetration decreases. Frost damage is also related to the moisture content in the freezing zone. Without moisture there would be little damage, but as moisture increases, the damage increases. Frost enters the roadway from the top and penetrates downward. In the spring, thawing progresses from both top and bottom.

A seasonal fluctuation generally occurs in soil moisture with the moisture content greatest in the spring. Scientific studies attribute much of the increase to the transfer of film and vapor moisture from the warm soil below the zone of seasonal temperature change toward the cooling soil in the temperature change zone where it collects and increases the total moisture content. During the summer, when the temperature of the upper zone increases, the movement is reversed. It is probable that periodic recessions of temperature, caused by the release of latent heat of fusion and the time required to dissipate the heat, results in a fluctuating frost line during the freezing period. Warming periods contribute to temperature recession. During sunny winter days, thawing can occur to depths of more than 6 in. below the top of bituminous surfaces. A fluctuating frost line is conducive to moisture gain.

The percentages of contained water which freeze at normal freezing temperatures vary widely for different soils. In general, these percentages vary inversely with the clay content of the soil. Normally, soil does not freeze until the temperature of the soil reaches about - 4 C.

When freezing, moisture moves from small capillaries and thick films around the soil particles to larger capillaries. When drawn into the larger capillaries this moisture assumes the properties of lubricating moisture. Physically combined and loosely chemically combined water exists in the colloids. On freezing, the colloids coagulate and the combined water is liberated. The capillaries are destroyed by freezing and additional unfree water is liberated. With each cycle of freezing and thawing, more free water is liberated which freezes at normal temperatures. During the summer months the above processes are reversed.

In relation to the bearing value of a soil, moisture may be divided into two classifications: lubricating or free moisture and adhesive moisture. A soil is stable when the absorbed or adhesive moisture is the dominating influence. As lubricating water increases in proportion to adhesive moisture, the bearing value decreases.

Laboratory tests were conducted by the Minnesota Highway Department in 1948. After sealing to prevent moisture change, soils were subjected to freezing and thawing. Bearing tests made before and after freezing and thawing indicated losses in bearing value ranging from 18 to 39 percent.

Plate bearing tests made in the field indicate losses in bearing value of 50 to 62 percent in early spring. It appears the loss of strength in the spring is only partially caused by total moisture increase. Because water expands 9 percent in volume when becoming solid, there is a disrupting effect when materials containing water become frozen.

The gain in moisture on freezing is not necessarily confined to soil materials. This occurrence was observed numerous times in granular (sand gravel) bases. The moisture gain in base material is more evident in aggregates with a high content of soil fines than in aggregates low in fines. When the base thaws, "bleeding" of water through the surface occurs if the moisture gain is substantial.

The moisture gain in a bituminous surface is difficult to detect because it is rare and does not appear to occur in any detectible amount in properly constructed surfaces. When it does happen, stripping of the bituminous material from the aggregate sometimes results. Moisture can be considered a related frost action problem.

Arkansas, Oklahoma, Kansas and Missouri report that frost does not penetrate sufficiently into the subgrade and the frost action is not of sufficient duration or severity to create problems requiring special design considerations for the subgrade, base or surfacing. Occasionally some frost damage does occur, but it is not a major problem. Therefore, few data on frost design practices are available from this area. Missouri reports some subgrade frost problems in the northern counties, but their only frost design consideration relates to the durability of portland cement concrete in pavements and bridge decks. Oklahoma does consider frost penetration in the determination of base thickness, but frost damage is not indicated as a major factor. The deepest frost penetration is in the northwestern part of Oklahoma, but frost damage is unlikely because of low rainfall and the presence of low frost-susceptible soils.

The following are area problems that include Nebraska, South Dakota, Iowa, North Dakota and Minnesota.

Subgrade Problems

Frost heaves and frost boils were am' ıg th first highway defects caused by frost action to be recognized by highway en',.neers. Because these problems have been recognized for a long time and have a l ,ng history f study, corrective action has succeeded in practically eliminating objectionable defec s of this nature from modern highways. They are still a major consideration in design, ; nd in the maintenance of the older roads.

Differential frost heaves (bumps) occur when there are pockets or layers of highly capillary soils in sections predominantly composed of moderately capillary or granular soils, or where cohesive soils have ready access to localized moisture. The heaving can occur as a single bump in a cut, as a series of bumps, or as general irregularity (Fig. 6). The heaving is caused by ice crystal growth in the soil, and its severity varies with the depth and rate of frost penetration, availability of moisture, and the nature of the soil. The height of heaving above surrounding areas varies from one year to the next for the same heave, and, in some cases the height of the same heave will increase with age. The maximum height of heaving in this area varies from 12 in. in northern Minnesota to 4 or 5 in. in Nebraska. Differential frost heaves are a much more serious problem in the northern portion of this area than in the south portion.

In the northern portion, frost penetration is so deep that general surface roughness occurs where soils and moisture are variable. The problem is not always serious, but it is costly to correct and the offending materials or conditions are difficult to detect.

Frost boils are localized areas where there is almost complete loss of soil strength during thawing. The loss of strength is caused by the large accumulation of moisture that occurs as ice crystal growth during freezing. Boils occur when there is insufficient cover of frost-free material to bridge the weakened frost-susceptible material. Frost boil areas do not always develop detrimental heaving, nor do frost heaves necessarily boil in the spring.

A great effort is made by the states in this area to identify materials subject to frost heaves and boils, and to assess conditions that contribute to the problems. Detailed soil surveys are performed prior to design to visually classify the various soils present and evaluate other conditions that influence this performance. Representative samples are physically tested in the laboratory for classification.

Most cohesive soils are frost-susceptible to some degree depending on their physical and chemical composition. If less than 20 percent of the material passes the No. 200 sieve, it is considered a coarse-grained soil and normally is not subject to frost heaves

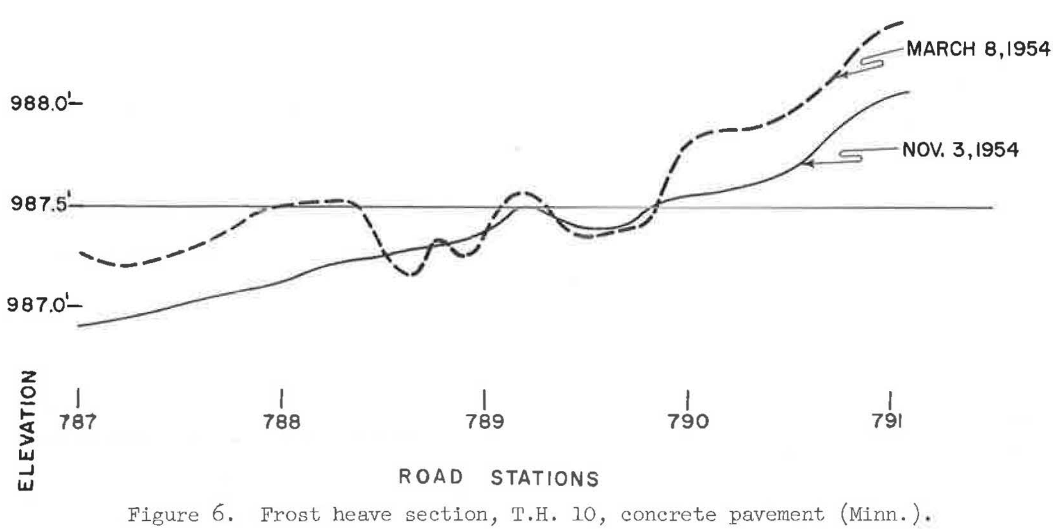


Figure 6. Frost heave section, T.H. 10, concrete pavement (Minn.).

or boils, although very fine sands will heave and boil under adverse moisture conditions. There are no available recognized limiting test values for assessing the degree of susceptibility for various soils, but it is generally recognized that silty soils (A-4) are the most susceptible to boils and heaves. Other cohesive soils may heave or boil when in a non-uniform, pocketed or layered condition, or when adjacent to ledge rock or coarse-grained soil material. Moisture conditions that are recognized to contribute to these problems are springs, seepage areas, high ground water table, perched water, restricted drainage, or underground water supplied by faulty utility installations. Detrimental frost action can be expected where these conditions occur. When encountered in the soils survey, they receive special consideration.

Reduction in soil strength in subgrades during the spring period is a critical problem particularly in connection with flexible pavements. The strength loss in the general subgrade soils for the area varies from 30 to 62 percent of the fall season strength. Because frost penetration is less in the south portion of this area than in the north, there is correspondingly less strength loss. This loss of stability is a result of a reduction in cohesion and internal friction in the soil caused by the loosening effect of freezing, a reorientation of the contained moisture into free moisture, and a possible moisture increase. This strength loss can be expected in all soils including granular material.

The period from spring break-up to substantial recovery in strength of cohesive soils averages approximately three months. There is some variation from year to year. It is believed that coarse-grained materials recover substantially in a much shorter time, and because the strength is still comparatively high during the weakest period, the loss related to granular subgrades is not significant.

Contraction of frozen subgrade soil after a drop in temperature is not considered a serious problem in this area. Contraction cracking occurs in sand subgrades as well as in cohesive soil subgrades during the winter. Both transverse and longitudinal cracks occur, although longitudinal cracking is generally limited to cohesive soil subgrades (Fig. 7). When the cracks carry through the base and surface courses, some damage from spalling or break-down of the crack may occur. The occurrence of this problem appears to be limited in extent.

Base Problems

Granular bases lose strength as the base thaws in the spring, but the duration of the reduction strength is estimated at only one week. Granular base materials become mealy and appear slightly loosened during this period, but regain firmness in a short time under traffic. The reduction in strength is very difficult to measure in the road,

Figure 7. Large crack caused by shrinkage of dense clay subgrade following deep frost penetration (Minn.).

therefore no test data are available. The loss is primarily the result of the disruptive stresses produced by the freezing expansion of the contained moisture, combined with the reduced stability contributed by any increase in moisture. Seasonal moisture content fluctuation in the sand-gravel base on a section of flexible pavement in Minnesota is shown in Figure 8. Moisture migrates upward in vapor form to the cold surface. As freezing occurs, moisture collects, and fluctuations of temperature in the freeze-thaw range promote accumulation. In bases containing excess fines, additional moisture could also be supplied by capillary migration of moisture from the subgrade. Coarse, well-graded bases containing a minimum of fines accumulate less moisture than finer-graded bases containing excess cohesive fines, and consequently are less affected by frost action. Loss of stability in granular base covered by bituminous surfacing is evidenced, if failure occurs, by alligator cracking or breaking of the surface without appreciable displacement. The deflections involved are usually relatively small because there is no reflection of deflection in the frozen subgrade. The base usually regains firmness before substantial thawing developes in the subgrade.

Data on loss of density are limited, but it has been observed that in coarse, well-graded base aggregates, the loss of density is negligible but finer aggregates show some loss. Any losses in density that occur are usually recovered under traffic during thawing.

Substantial loss in strength on thawing can be expected in subbase and base materials that exceed the general limits for gradation and unsound particles indicated under design practices for bases.

In general, materials used for base courses are sampled and their suitability determined by gradation tests, shale tests, Los Angeles rattler tests and Atterberg limits. The locations of representative portions to be sampled are determined by visual examination.

Bituminous stabilized and treated bases are not appreciably affected by frost action. Contraction cracking does occur in bituminous bases, but because of the relatively short period of experience with these bases, it is not recognized as significant. Little is known about detrimental effects, but this could become a future

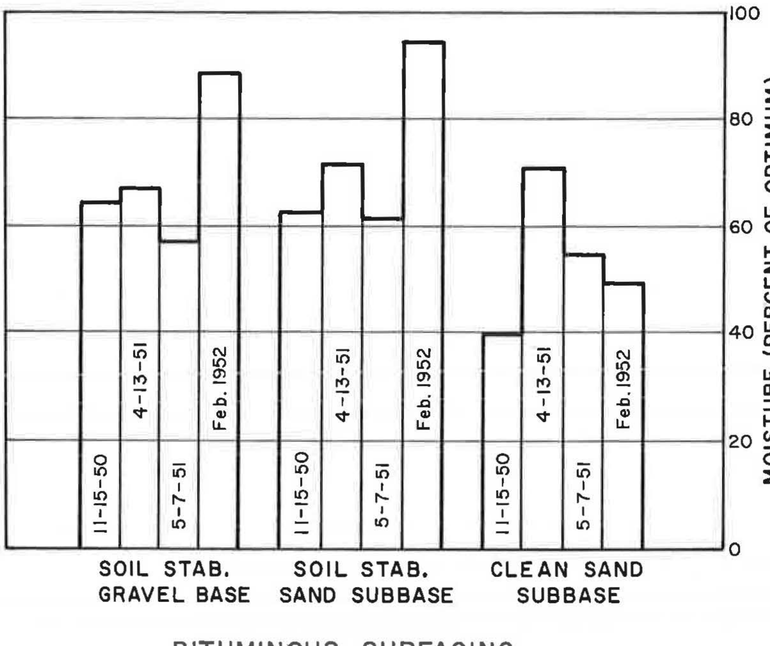


Figure 8. Seasonal moisture fluctuations, avg. values, T.H. 10 Randall-Lincoln, Minn.

problem. Because there is no history of bituminous bases being frost-susceptible, there is little knowledge regarding methods of determining the susceptibility of materials of mixtures; consequently, limiting values are unavailable.

Soil-cement and cement-treated bases are not significantly affected by frost action except for contraction cracking. Contraction cracking is a problem, especially in the northern portion of the area where winter cracking causes considerable maintenance crack filling. Otherwise, these bases appear durable and structurally sound.

Considerable contraction cracking can be expected when fine-grained soil material is used in the soil-cement mixture. Considerable reduction can be obtained if granular materials are used in place of soil. In general, the cracking increases in proportion to the increase in cement used.

Structures and Pavements

Heaving of approach fills adjacent to bridges and culverts is a problem in the northern portion of the area (Fig. 9). The causes and detection procedures are the same as previously described for frost heaves. Pronounced heaving can be expected where frost-susceptible soil is placed in relatively shallow fills adjacent to structures if there is water readily available for ice crystal accumulation. Heaving is rare in the higher fills. In the extreme northern portion of the area where frost penetration is deep, shrinkage can occur in "fat" clay fills causing a slight bump at the culvert.

Culvert heaving is a problem in the northern half of the area where there is less than approximately 5 ft of cover over the culvert top (Fig. 10). The heaving is caused by ice crystal accumulation in the soil under the culvert. The occurrence can be suspected for culvert placement on most non-granular soils where moisture is readily available and there is no protection to prevent freezing below the culvert. Unfrozen water in the culvert or heavy snow cover will prevent freezing under the culvert. The heaving will vary from year to year depending on the amount of freezing protection from snow cover. On rare occasions, fill shrinkage occurs in conjunction with culvert heave causing a more pronounced bump (Figs. 11 and 12).

Permanent uplift (jacking-out) of culverts is a problem associated with culvert heaving. Not all heaved culverts are subject to permanent uplifts. Jacking appears to oc-

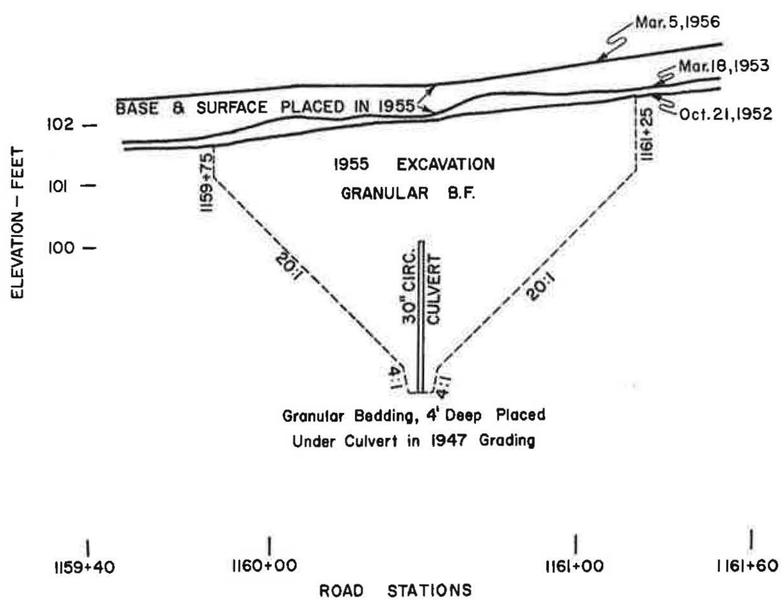


Figure 9. Pipe culvert fill heave and treatment (Minn.).

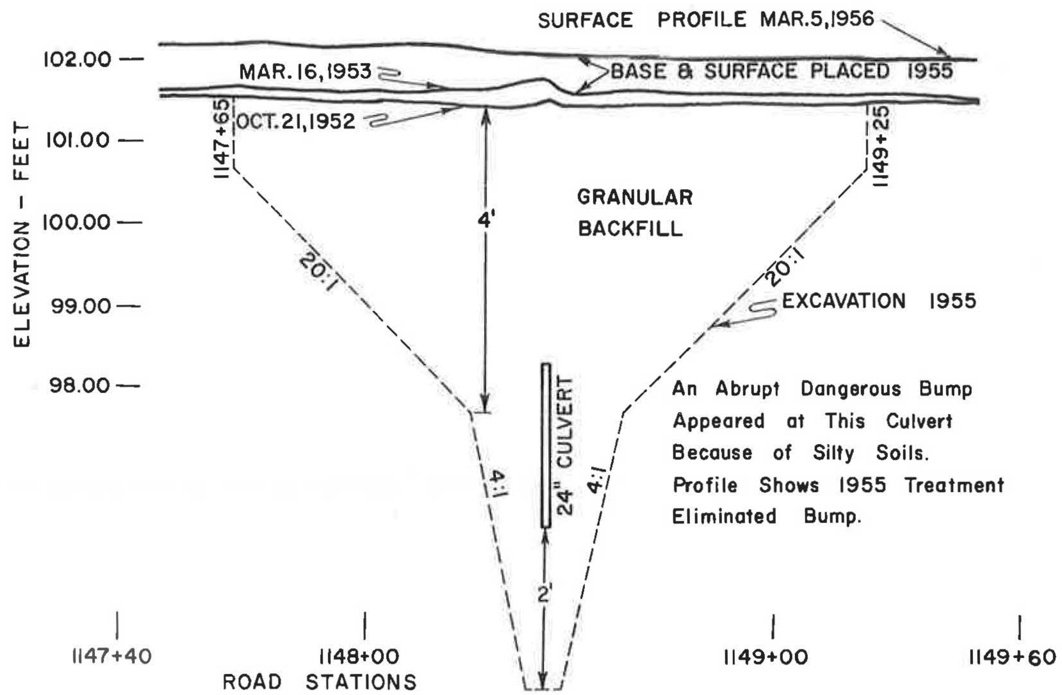


Figure 10. Typical culvert heave treatment--longitudinal section (Minn.).

Figure 11. Combination culvert heave and fill shrinkage causing dangerous bump (northern Minn.).

cur where there is shallow fill over the culvert in areas of "lean" clay loam or cohesive sandy loam soil. The jacking occurs during the spring thaw when soil below the culvert thaws more rapidly than the adjacent fill soil, thus creating a void under the culvert which is prevented from subsiding by the adherence of the frozen fill. Small amounts of loose soil fall under the culvert each year resulting in a progressive uplift.

Portland cement concrete pavement warp (high joints) caused by frost action is a

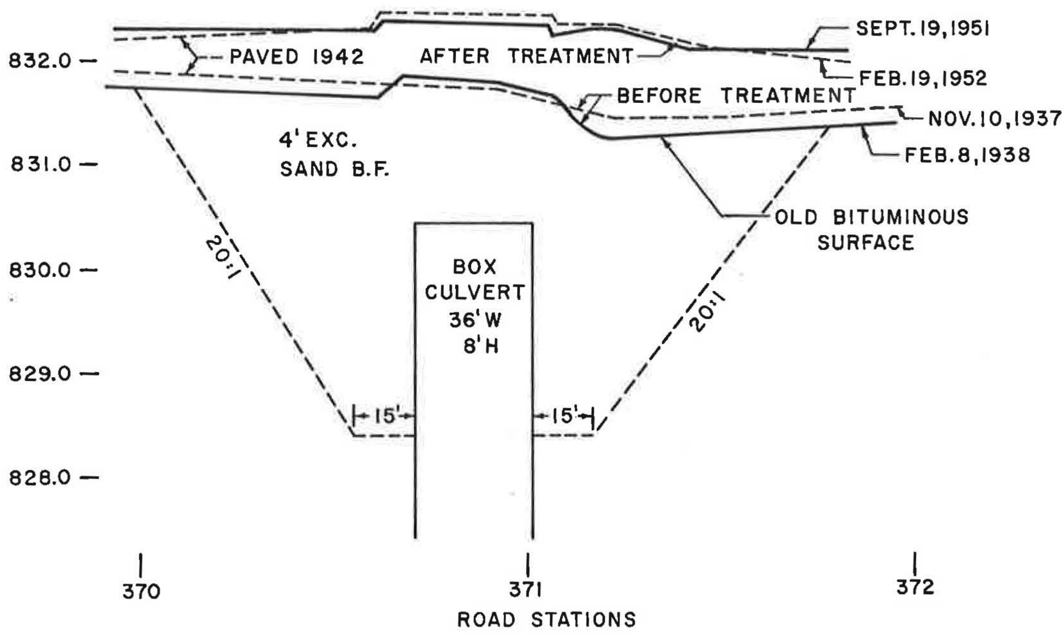


Figure 12. Combination culvert heave and fill shrinkage (low density gumbo clay, northern Minn.).

problem in South Dakota and Minnesota. It is considered an inconvenience to traffic in the form of a slightly rough-riding surface during the frozen period and only occurs occasionally. The roughness is much more noticeable and distressing to trucks than to passenger cars. The high joints, which subside in the early spring period, are primarily caused by the formation of ice lenses in the soil at the joint. The moisture is supplied by water leaking through joints that open during contraction. High joints can be expected in pavements constructed on relatively dry subgrades composed of plastic lacustrine soil of clayey glacial till from the grey drift. Moisture and density control in the subgrade and tightly sealed joints can reduce and sometimes prevent the warp.

Cracking caused by differential subgrade heaving is a problem in the northern portion of the area. This does not occur in newer pavements because of corrective action taken when constructing the subgrade. Consequently the problem is minor in extent. It occurs more in older pavements causing roughness, and in some cases, pavement failure during the spring break-up.

Progressive increase in the permanent roughness of rigid pavements can be partially attributed to subgrade movement caused by freezing where substantial frost penetrates the subgrade. Surface irregularities caused by frost movement are not likely to return entirely to original smoothness after thawing especially if cracks develop. Cracks never close entirely, but become progressively open after repetitive movement. This progressive roughening of the pavement is not serious, but is objectionable and leads to shortening of the pavement life. It is not as evident in bituminous pavements because of the flexible nature of the structure and the dampening effect of substantial subbase and base thicknesses.

ROAD LOAD TESTS—FLEXIBLE PAVEMENTS

Plate Bearing Tests

The loss of load-carrying capacity as related to frost action is considerable. With no increase in moisture, the loosening effect of freezing and the character change of the contained moisture reduce the internal friction and cohesion sufficiently to cause an appreciable loss in stability. In addition, subgrade soils normally increase in moisture content because of moisture migration to the freezing zone. This is dependent on frost penetration, temperature fluctuation, precipitation and chemical properties of the soil.

By plate bearing tests (Fig. 13), Minnesota has derived an average curve showing the loss of load-carrying capacity from fall to spring and the rate of recovery through the summer. Bearing values, measured at various times during spring and summer, are adjusted to maximum fall values by multiplying the value obtained by the factors in Table 1. Spring strengths are generally estimated by applying a 50 percent reduction to the fall value. The 50 percent was selected as an average year-to-year value for all roads.

In the plate bearing test, load-carrying capacity is evaluated by loading a 12-in. diameter steel plate in uniform increments and determining the unit pressure at 0.2-in. deflection. The plate bearing test is used in Minnesota for research, evaluating the strength of existing road structures, and to assist in establishing spring load restrictions. The test is not used directly for design purposes, but the information developed is considered and influences design. Figure 14 shows typical loss and recovery curves developed by Minnesota, Nebraska and Iowa. Nebraska's plate bearing study confirms that flexible pavement strengths are lowest in the first few weeks after spring thaws, and the loss of strength is not as great for subgrades composed of sand as for subgrades of silt-clay materials. Their curve indicates that average strength loss is not as great in Nebraska as in colder Minnesota. Iowa's curve is very similar to that of Minnesota.

Figure 15 shows average strength loss and recovery curves developed by Minnesota for various bituminous-surfaced base structures on various subgrade soils. Test loads were applied to the surfacing. Sand subgrades lose considerably less strength than do

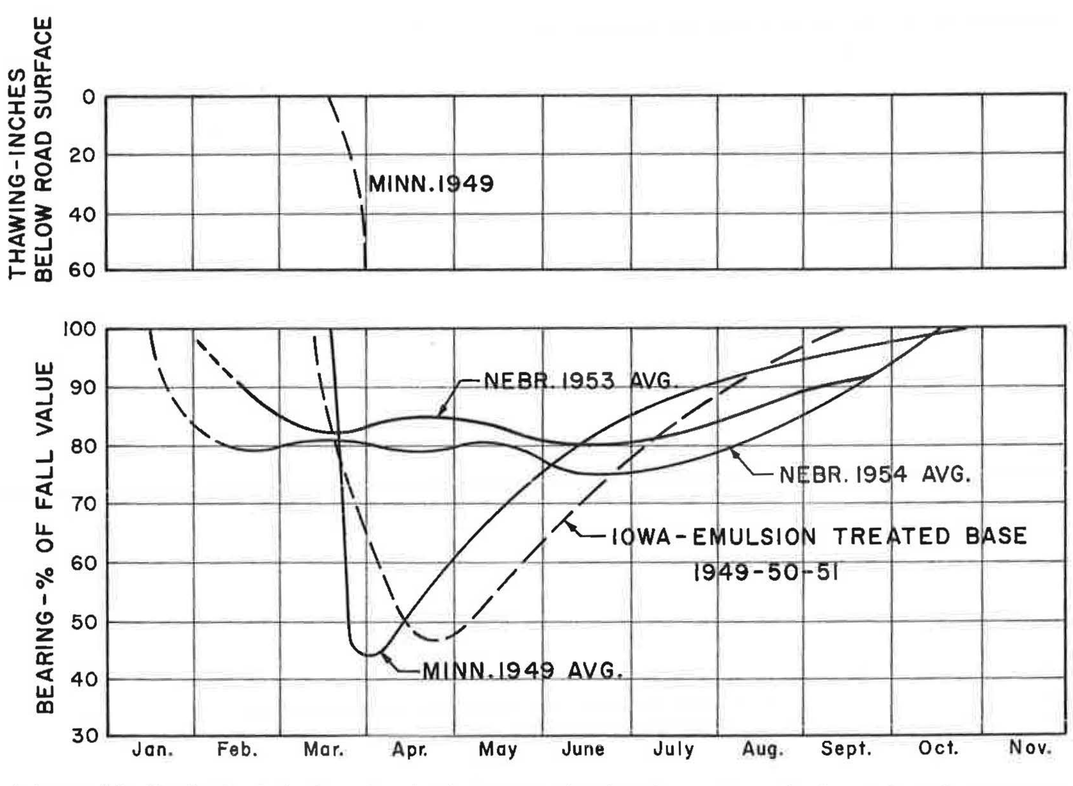


Figure 13. Typical plate bearing test curves showing loss of road strength and recovery.

cohesive soil subgrades. Similarly, road structures consisting of hard semi-rigid bases such as soil-cement on cohesive soil subgrades lose considerably less strength than do structures of more flexible bases such as gravel on cohesive soil subgrades. The strength loss and rate of recovery also varies moderately from year to year.

Table 2 gives the average values of retained strength for several combinations of subgrade soils and bases as a percent of fall load-carrying capacity.

Loss of road structure strength because of frost action varies with the type of subgrade soil, type of base, and depth of frost penetration.

TABLE 1

MULTIPLICATION FACTORS TO OBTAIN FALL BEARING VALUE

Test Period	Cohesive Soil Subgrade	Sand Subgrade
June 1-15	1.370	1.176
16-30	1.284	1.149
July 1-15	1.221	1.125
16-31	1.176	1.099
Aug. 1-15	1.135	1.077
16-31	1.095	1.055
Sept. 1-30	1.042	1.022
Oct. and Nov.	1.000	1.000

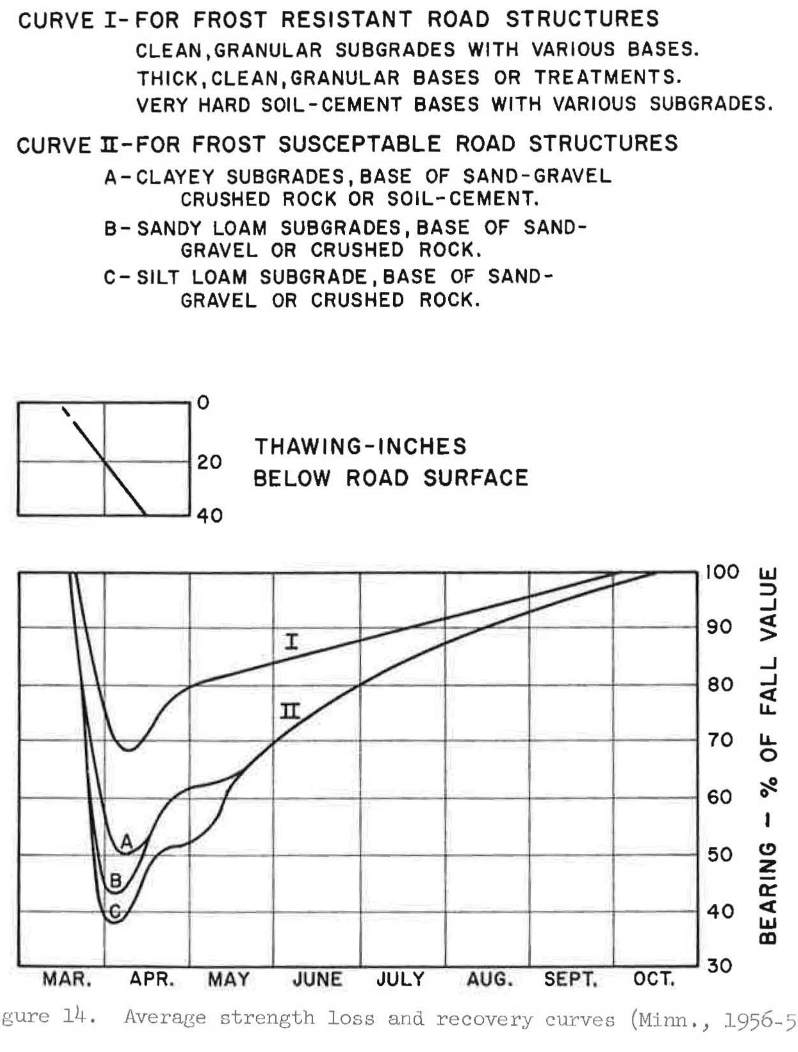


Figure 14. Average strength loss and recovery curves (Minn., 1956-57).

Loaded Vehicle Pavement Deflection Tests

The Benkelman beam test has been used in research work by both Minnesota and Nebraska (Fig. 16).

Minnesota is conducting research using both plate bearing and Benkelman beam procedures. This study is being made in cooperation with the counties. Roads in 50 counties are included. A correlation between the two tests is needed to develop a procedure that will permit use of the Benkelman beam test in place of the more expensive plate bearing test.

Nebraska is using the Benkelman beam test in a study which began in 1960 and is still in progress. The study is based on road condition and the deflection measured using a 9,000-lb wheel load. The arithmetic means and standard deviations were calculated and a limiting value of deflection was selected for several flexible pavement designs. Benkelman beam tests have also been used to check deflections on highways with load restrictions.

Figure 15. Plate bearing truck—trailer.

TABLE 2

AVERAGE VALUES OF RETAINED STRENGTH AS PERCENT OF FALL CAPACITY

Base	Subgrade	Strength Retained in Spring (%)
Sand and gravel	Silt loam	38
Sand and gravel	Sandy loam	43
Sand and gravel	Clayey	48
Sand and gravel	Sand	66
Crushed rock	Clayey	49
Soil-cement (soft)	Clayey	53
Soil-cement (hard)	Clayey	71

DESIGN PRACTICES ASSOCIATED WITH FROST ACTION

The following design practices pertain only to the area covered by Nebraska, Iowa, South Dakota, North Dakota, and Minnesota.

General

The design of good roadway drainage, which is common practice in planning most highway construction, deserves special attention in frost problem areas when attempting to prevent high subgrade moistures and reduce frost damage potential. Springs, perched water, seepage areas and low areas are drained as much as possible. Underground drainage systems are provided in areas where free water may be a problem. Minimum ditch depths vary from 3 to 5 ft below the finished shoulder for rural sections.

Figure 16. Benkelman beam and truck.

General practice is to provide a grade line over low wet areas of sufficient height to insure against penetration of frost into unstable foundation soils that are likely to create problems. The minimum elevation of the top of the subgrade above natural ground varies from 2 to 4 or 6 ft depending on moisture conditions and the nature of the foundation soil. Base and surface thicknesses are additional.

Maximum legal loads for the area are uniformly 9 tons per axle. Primary highways are designed to withstand this loading during the low strength period. Highways not constructed to this standard are usually restricted to lower axle weights in the spring period. These restrictions are usually placed in late February or early March and remain in effect until the latter part of May. Minnesota designs some secondary routes to support maximum axle loads of 7 tons during the weakest period. These routes are restricted to the design loadings during the spring, but are expected to support 9-ton axle loads the remainder of the year.

Restrictions on construction work during the freezing season prevail throughout the area, resulting in the suspension of most construction from freeze-up time until there is sufficient stability recovery in the spring to support construction equipment.

Grading with frozen soil or placing embankment on frozen ground is generally not permitted. Swamp excavation is permitted in the winter in Minnesota, but backfill with frozen material is not permitted. Base construction with frozen material or on frozen subgrade is generally not permitted. Surfacing construction is generally restricted to temperatures well above freezing.

Subgrades

The design of treatment for frost boil and frost heave sections consists of excavating the offending soil from the subgrade to depths of 1 to 3 ft in the southern portion of the area and 2 to 4 ft in the northern part. The exact depth of excavation is governed by conditions determined by soil borings and prevailing frost penetration. Base and surface thicknesses are additional, and in the extreme north, result in a maximum total depth of 6 ft from pavement surface to bottom of treatment. The width of excavation generally includes the shoulder width of the finished surface. Seepage trench outlets or perforated drain pipes are provided as needed in sections where there is a danger of water accumulations. Sand or gravel backfill is usually provided, although suitable mineral soil material may be provided in sections where water is not a problem. In Minnesota, suitable soil is used when feasible for backfill in the interest of economy and because it offers more resistance to frost penetration than do granular materials. Granular materials are used where there are adverse moisture conditions.

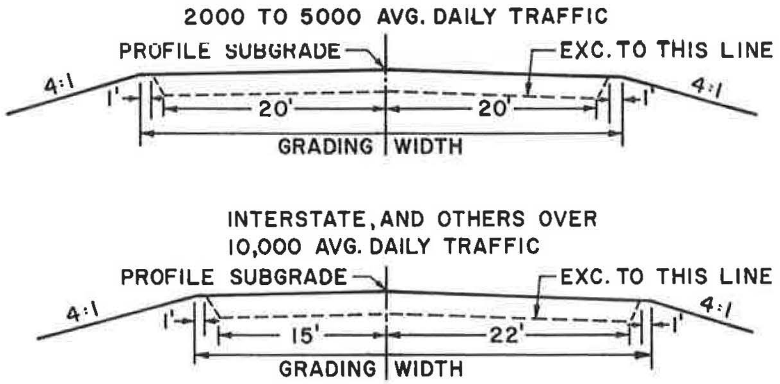
Adequate tapers, ranging from 1 ft to 10 to 1 ft in 20, are provided at the ends of all treatments and subcuts to avoid abrupt changes in soil. Figure 17 shows typical standard designs for frost heave treatment as practiced in Minnesota.

Soil selection is practiced in grading throughout the area. Topsoils, silty soils and other unsuitable soils are placed below the upper 3 ft of the subgrade. The better soils are reserved for the upper 3 ft of the subgrade for uniformity and to minimize frost susceptibility. Where sufficient granular materials are available, they are selected for the upper portion of the subgrade. Unless deeper treatments are provided, cuts are generally subcut 1 to 2 ft for soil selection and compaction to obtain uniformity. Figure 18 shows typical standard subcut designs used in Minnesota. When previously unknown areas of unsuitable soil are discovered during grading construction, they are removed and replaced with selected suitable material. Any necessary underground drainage systems are installed.

It is the practice to provide underground drainage systems of perforated drain pipe to intercept the infiltration of seepage or spring water into the subgrade or to depress the elevation of free water. Trenches filled with open-graded granular material or special ditches are sometimes used. When free water exists in the roadway, an underground drainage system is installed in the subgrade excavation. Gravel-filled trench outlets are usually provided for gravel-filled excavations if ditches are sufficiently deep to provide run-offs.

In rock cuts, the rock is removed 12 in. below the bottom of finished surface and backfilled with suitable soil material or sand-gravel. South Dakota uses bituminous-treated material for this purpose. Soil is removed from pockets to a depth of at least 3 ft at the ends of the rock. Adequate tapers are provided and sand-gravel materials are used for backfill.

Although compaction and moisture control are primarily to develop stability and are not considered a positive treatment for frost effects, they do promote uniformity in subgrade construction and contribute some resistance to strength loss. Nebraska requires that the upper 6 in. of the subgrade soil be compacted to not less than 90 percent of maximum density. For flexible pavements the moisture content of the upper 6 in. of subgrade must not be more than 4 percent above or below a value which is 90 percent of optimum, and for rigid pavements, not more than 3 percent above or below optimum



NOTE: TAPER EACH END OF SUBGRADE EXCAVATION 20:1 SLOPE UNLESS OTHERWISE RECOMMENDED. BACKFILL MATERIAL TO BE SUITABLE SOIL OR GRANULAR MATERIAL AS RECOMMENDED. PROVIDE DRAINAGE FOR ALL SUBGRADE CORRECTIONS DURING CONSTRUCTION. PROVIDE SEEPAGE TRENCHES WHEN BACKFILLED WITH GRANULAR MATERIAL. PLACE PERFORATED PIPE DRAINAGE AND OUTLET PIPE AS REQUIRED. WHEN THE PROPOSED SUBGRADE CORRECTION IS LOCATED WHERE THERE IS BASE AND BITUMINOUS SURFACING OR CONCRETE PAVEMENT INPLACE, THE ENDS OF THE TAPERS SHALL BE VERTICAL FOR THE DEPTH OF THE BASE AND BITUMINOUS OR CONCRETE PAVEMENT. DEPTH OF SUBCUT, AS RECOMMENDED BY SOILS ENGINEER, VARIES FROM 3' TO 4.5' BELOW TOP OF SUBGRADE.

Figure 17. Subgrade correction or treatment (Minn.).

moisture. Optimum moisture and maximum density are determined by AASHO Designation T 99. North Dakota specifies that embankments are to be constructed in layers not to exceed 12 in. and must contain not less than 75 percent of optimum moisture when being compacted. Compaction of embankment material to a density of not less than 95 percent of maximum density is required for the upper foot and to not less than 90 percent below the upper foot. Optimum moisture and maximum density is determined by AASHO Designation T 180. South Dakota requires compaction of fill material to 95 percent of maximum density with moisture controlled to within 2 percentage points below and above optimum moisture as determined by AASHO Designation T 99. Iowa requires compaction of the lowest lift of fill to 90 percent of maximum density (AASHO T 99) and to 95 percent for subsequent lifts. The control of moisture is also specified. Minnesota requires embankment materials to be compacted in layers not to exceed 6 in. at moisture contents between 65 and 102 percent of optimum moisture for the upper 3 ft and at not more than 115 percent for material below the upper 3 ft. The lower limit of 65 percent does not apply to granular materials, and the upper limit of 102 percent may be increased slightly for "fat" clay. In areas where pavement warp is prevalent, the moisture in the upper 12 in. of clayey subgrades is limited for 80 to 100 percent of optimum. Compaction of embankment materials to a density of not less than 100 percent of maximum density is specified for the upper 3 ft and to not less than 95 percent below that. Optimum moisture and maximum density are determined by AASHO Designation T 99.

In recent years Nebraska used hydrated lime to stabilize the upper 6 in. of the sub-

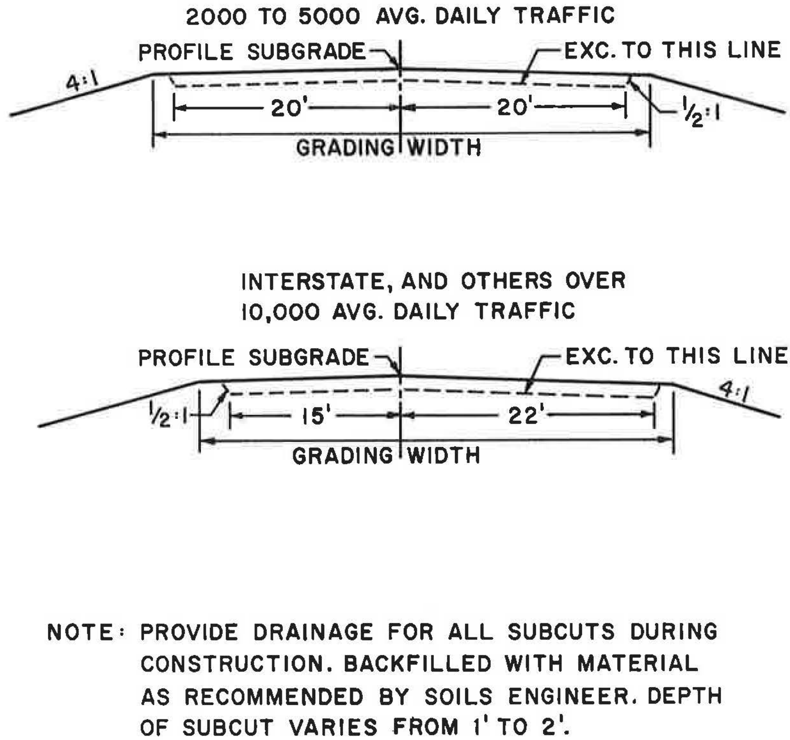


Figure 18. Typical subcut for compaction (Minn.).

grade in weak areas. The addition of 3 to 6 percent has been effective in adding bearing strength to the subgrade, but because of the high cost, it is only used in areas where there is a shortage of granular materials. Hydrated lime, cement, and a combination of hydrated lime and cement have been used in Nebraska to stabilize the upper subgrade on experimental construction projects. The results have been satisfactory, but it is too early for definite conclusions on the permanency of this type of treatment. South Dakota has recently treated some subgrades with lime, but results have not been evaluated.

Bases

Because of strength loss in the subgrade during the spring period, it is the practice to provide adequate thickness of subbase and base to withstand traffic loads during the weak period. No data are available on the added thickness used to compensate for frost effects, but it is believed this is in the order of 30 to 40 percent of the total thickness required on cohesive soil.

Granular materials used for base are generally limited to a maximum of 10 percent of material passing the No. 200 sieve, 35 percent passing the No. 40 sieve, and 65 percent passing the No. 10 sieve. A limiting value of 7 percent rather than 10 percent is considered more appropriate in the portion of the area where freeze-thaw cycles are numerous. Subbase materials are generally limited to a maximum of 10 percent passing the No. 200 sieve. These limiting gradation values are subject to variation in relation to other characteristics of the base aggregate. An excess of soft, deleterious or unsound particles in the aggregate, such as shale, soft limestone and soft limey sandstone, contributes to strength loss. The limiting values in the area vary from a maximum of 7 percent shale in high quality base materials to a maximum of 10 or 15 percent in subbase aggregates and lower quality base aggregates. For both subbase and base, plastic limits are restricted to a maximum of six and liquid limits to a maximum of 25. Iowa specifications permit soil-aggregate subbase and base materials to contain soil fines and soft particles in considerable excess of these limits, but these materials are seldom used. Iowa base courses on primary roads are usually asphalt or bituminous treated to add resistance to frost action. It is becoming common practice in flexible pavement design to provide a granular subbase plus a base composed of granular material stabilized with bituminous material or portland cement to develop added strength and resistance to frost action. They are placed full width of the subgrade. The aggregates used in the base are sand or gravel with gradation deficiencies compensated for by the increase in the amount of stabilizing agent. Bituminous-stabilized bases are used extensively on the high-traffic roads.

Soil-cement bases are used primarily on secondary roads in areas where satisfactory base aggregates are not economically available. Sand materials are generally preferred in place of cohesive soil for cement treatment. Cement factors are determined by the freeze-thaw durability tests and wet-dry durability tests. Density to 98 or 100 percent of maximum density (AASHO T 134) is required.

The following are limiting values for freeze-thaw durability tests on soil-cement mixtures performed in accordance with AASHO Designation T 136-57 (values established by Portland Cement Association):

AASHO Soil Classification	Maximum Allowable Loss (%)
A-1, A-2-4, A-2-5, A-3	14
A-2-6, A-2-7, A-4, A-5	10
A-6, A-7	7

These maximum allowable loss values are the same for the wet-dry durability test, AASHO Designation T 135-57.

Stabilization of aggregate bases by the use of additives other than bituminous material and cement, such as lime, calcium chloride, or sodium chloride, is not standard practice although they have been tried experimentally. Subbase and base for portland cement concrete pavement is not generally predicated on the basis of frost effect.

Minnesota relates flexible pavement quality and thickness to the traffic loads and subgrade soil. The thicknesses have been established on the basis of experience and performance studies. The following are three typical design standards for A-6 soil subgrade and 9-ton axle loads related to heavy commercial average daily traffic count:

<u>150-300 H.C.A.D.T.—Single-Roadway Type</u>

10-in.	sand-gravel subbase	Full width
5-in.	crushed gravel base	Full width
1-in.	road-mixed bituminous base using crushed gravel	26 ft wide
3-in.	hot-mixed bituminous surfacing	24 ft wide

<u>600-1,100 H.C.A.D.T.—Two-Roadway Type</u>

6-in.	sand-gravel subbase	Full width
6-in.	sand-gravel subbase, high type	Full width
5-in.	high-type crushed gravel base full with or 4 in. of bituminous-treated lower type crushed gravel	28 ft wide
3-in.	hot-mixed bituminous base	26 ft wide
4-in.	asphaltic concrete surface	24 ft wide

Interstate or Over 1,100 H.C.A.D.T.—Two-Roadway Type

8-in. sand-gravel subbase	Full width
6-in. sand-gravel subbase	Full width
4-in. bituminous-treated crushed gravel	28 ft wide
4-in. hot-mixed bituminous base	26 ft wide
4-in. asphaltic concrete surface	24 ft wide

For granular subgrade, the base structure is reduced by 50 percent. For A-4, A-5, A-7-6 and A-7-5 soils, the base structure is increased up to 20 or 30 percent. Compaction to 100 percent of maximum density (AASHO T 99) is required for granular subbase, granular base and bituminous-treated gravel base. The lower limit for the moisture content of granular subbases and bases at the time of compaction is 90 percent of optimum (AASHO T 99), except when vibrating compactors are used and this may be reduced to 75 percent. Hot-mixed bases must be compacted to 90 percent of Marshall density determined when the mixtures are placed on the road.

Nebraska requires soil-aggregate base courses and granular subbases to be compacted to 100 percent and 95 percent, respectively, of maximum density (AASHO T 99). No moisture limits are required. Iowa and South Dakota provide for density control in the compaction of base courses.

Flexible Pavement Surfaces

There is no general practice for the determination of the quality and thickness of bituminous surfaces in relation to frost effects. North Dakota has adopted the criteria and formulas developed by the AASHO Test Road Staff, and Minnesota is studying this approach for application to design. The primary consideration is traffic loading.

Frost effects do have an influence on the design of bituminous surfaces in Minnesota, except that selection of the kind of bituminous mixture used is predicated on traffic volume. Thicknesses were increased an estimated 30 percent because of frost effects on the aggregate base courses.

Rigid Pavements

Frost action is not presently recognized as a design factor in rigid pavement thickness determination. The design of steel reinforcement in pavements is not related to frost action except where reinforced panels are used over culverts where heaving is a possibility.

To control pavement warp in certain areas, Minnesota provides for the compaction of the upper 12 in. of clayey subgrades to 100 percent of maximum density (AASHO Designation T 99) at moisture content between 80 and 100 percent of optimum. Nebraska generally provides granular subbase under rigid pavements to promote even moisture distribution in the subgrade for the control of joint heaving. South Dakota reports the use of thicker subbase to improve riding qualities.

The effectiveness of subbase under rigid pavement and the thickness needed are controversial. It was observed in Minnesota that pavements placed on aggregate base do not become as rough as pavements placed without base. Iowa reports an increased use of subbase under rigid pavements. South Dakota spokesmen indicate that the best riding pavements are those with thicker subbases. They believe the thicker subbases provide better protection against frost uplift. This writer believes that in cohesive soil sections and where frost penetration is deep, the use of good quality and properly consolidated bases of substantial thickness will result in pavements that remain smooth longer, perform better, and last longer than those placed on thin bases.

Bridges and Culverts

The treatment of bridge approaches and culverts is confined to the northern portion of this area. Iowa places wedge-shaped granular backfill adjacent to abutments in bridge approaches, but does not provide placement treatment for culverts. South Dakota intends to study the effects of frost on structures and methods of placement.

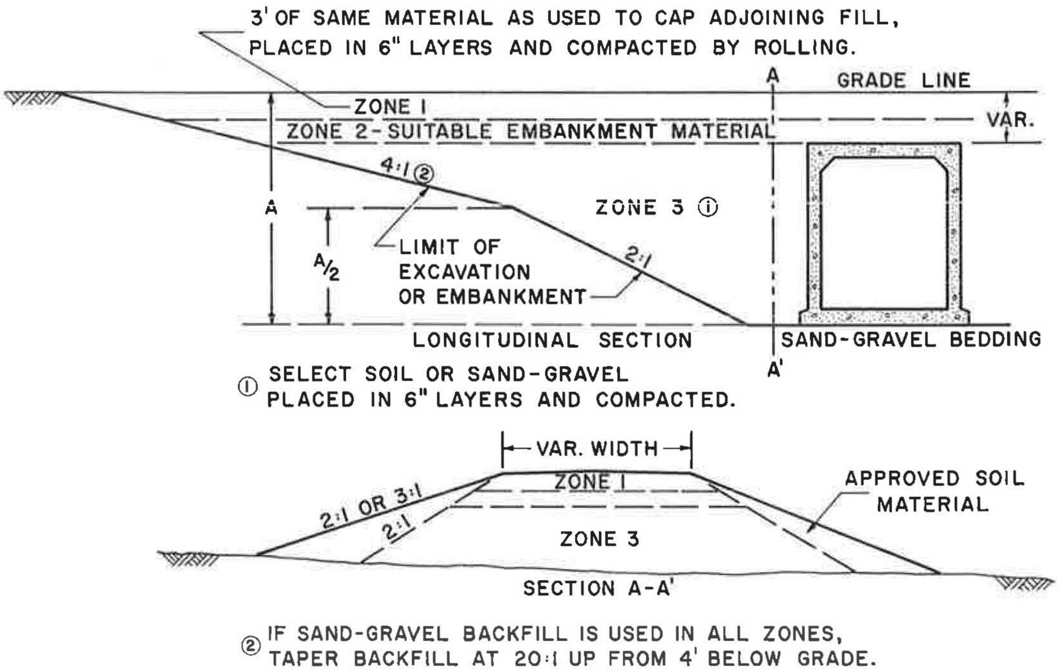


Figure 19. Box culvert approach fill treatment (Minn.).

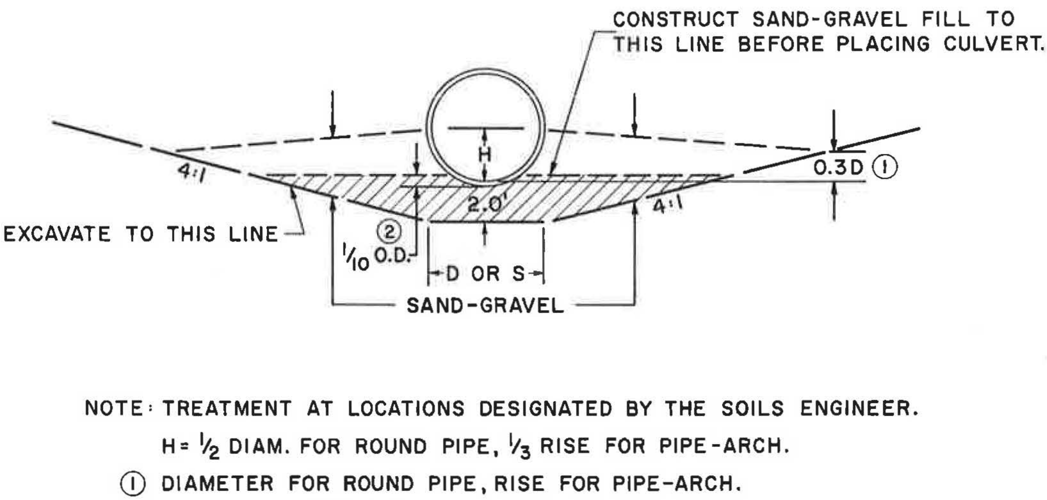


Figure 20. Treatment for centerline culvert in cohesive soils (Minn.).

North Dakota uses pit run gravel, well drained, behind the abutments of overhead structures. The bottoms of abutments are generally placed 4 ft or more below ground, and pier footings 5 to 7 ft below ground. Drainage culverts are bedded in one to several feet of gravel where foundation soils are questionable, but this has not completely solved the heave problem where fills are shallow over the culverts.

In Minnesota, wedge-shaped granular fill is often used adjacent to bridge ends, and the bottoms of pier and abutment footings are placed at sufficient depth below ground to avoid frost complications.

Foundation soils below box culverts are subcut sufficiently to provide space for 1 to 2 ft of granular material below the floor. The approach fills are generally designed with suitable local soil. If only silty soils are available, granular materials are selected for a tapered fill adjacent to the culvert. Figure 19 shows a typical design for box culvert treatment. Minnesota's bedding design for pipe culverts with less than 10 ft of cover provides for excavation to 2 ft below the culvert with 4 to 1 tapers (Fig. 20). Granular backfill is placed up to the center of the culvert. The remainder of the fill consists of suitable grading soil. This is a relatively new practice and is being observed to evaluate its effectiveness. This type of installation is not required where natural soils are sand and gravel.

SUMMARY

There are no significant frost problems in the southern portion of the West Central States other than the deterioration of portland cement concrete which is not discussed herein. Progressing northward, the base courses and subgrade soils become successively involved in frost action resulting in more numerous deep-seated problems. Design and construction practices used in severe frost areas cannot be applied in less severe areas without adjusting to local conditions.

General frost design practices used in states where problems exist have been quite effective in overcoming or minimizing most of the problems. A few are not fully solved and improvements are possible in the area of base design for rigid and flexible pavements, culvert placement, and a number of others. The greatest advances have been made in solving subgrade and base problems through improved design, grading, and base construction procedures.

Future research is suggested as follows:

1. Cheaper and more effective stabilization of subgrade soil.
2. More effective treatment of low-quality aggregate for base courses.
3. Development of chemicals for combination with soils to neutralize frost effects, and more effective methods of injecting them into foundation soils to correst existing problems.
4. Evaluation of the loss of density in base courses and subgrade soil as a result of frost action.

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Discussion

E.B. MCDONALD, Materials Engineer, South Dakota Department of Highways--The following, presented in the order that the various points are given in this report, is written to add to the information presented by Mr. Fredrickson rather than as a critique. Others should have a more complete study made before definite conclusions are made or criteria set up in design.

Topography is quite important, especially in glaciated areas where kettle-holes are found in combination with the A-4 and A-5 soil groups. Serious frost boils have developed in such areas where pockets of these types of soils were not detected on soil surveys and were also passed up on construction.

Recent studies made on several roads built in the past five years bear out the moisture transfer theory associated with freezing and also the loss of subgrade support. A difference as great as 10 percentage points has been found between the soil 18 in. below the subbase as compared to the soil 6 in. under the subbase.

Subgrade problems, as described, are typical of what has happened on many South Dakota roads; namely, differential heaves—some permanent and others reverting to a nearly normal condition in spring and summer. Some permanent roughness remains in all of the affected areas. It is believed that in the expansive soils the alternate freezing and thawing causes the moisture vapors to gradually accumulate in the subgrade soil and over a period of years a permanent uplift develops due to the expansive nature of the soil, even though no free-water ice lenses were formed and the system does not have access to any moisture other than that presented by rainfall. It has been found that many South Dakota roads located in the highly expansive soil areas react like a solid and many transverse and longitudinal cracks develop during the winter. Also, there have been transverse cracks only that were so evenly spaced it appeared they were built into the road.

The problems listed in the report as associated with base courses are essentially the same as have been found in South Dakota. It is planned to treat the upper 6 in. of expansive soil with lime and lime-asphalt, and phosphoric acid to see if reflected cracking due to contraction can be prevented and also to reduce the expansion due to moisture accumulation associated with freezing.

The structure and pavement approach fill problems, as submitted, are certainly typical of the South Dakota area. A study of this problem is being made to determine, if possible, the cause or causes of the uplift. It is suspected in some instances, that lack of proper control has resulted in some of the roughness connected with approaches to structures.

High joints have been the object of a rather intensive study made in the past two years. Figures 21 and 22 were made in connection with this study, and also a summary of the conclusions. Study of the figures tends to conclude that where moisture contents were not held at or slightly above optimum, a higher roughometer count was noted.

An extremely rough portion of the pavement project studied does not completely recover in the spring. Some portions are extremely rough. A highly saturated condition exists in the area of subgrade directly below the subbase. The soil has a rather high organic content and is quite plastic resulting in some expansion which has contributed to the permanent rough condition.

Plate bearing tests on 34 projects throughout South Dakota showed a reduction in the bearing capacity of the soil in the spring as compared to the bearing capacity in the fall. These tests were made during the spring and fall of 1961 and 1962. The per-

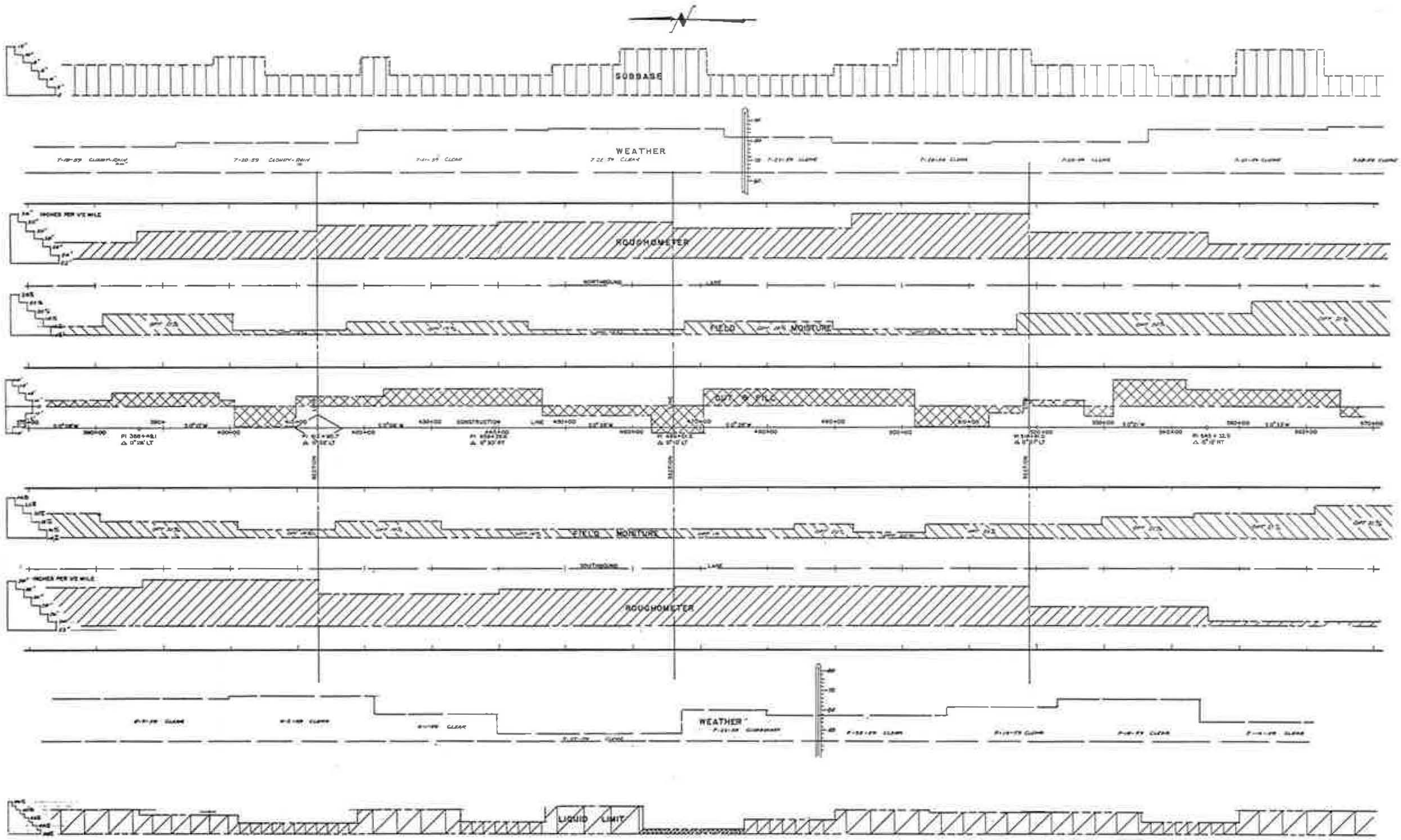

SUBBASE
WEATHER
ROUGHOMETER
FIELD MOISTURE
CUT & FILL
FIELD MOISTURE
ROUGHOMETER
WEATHER
LIQUID LIMIT

Figure 21.

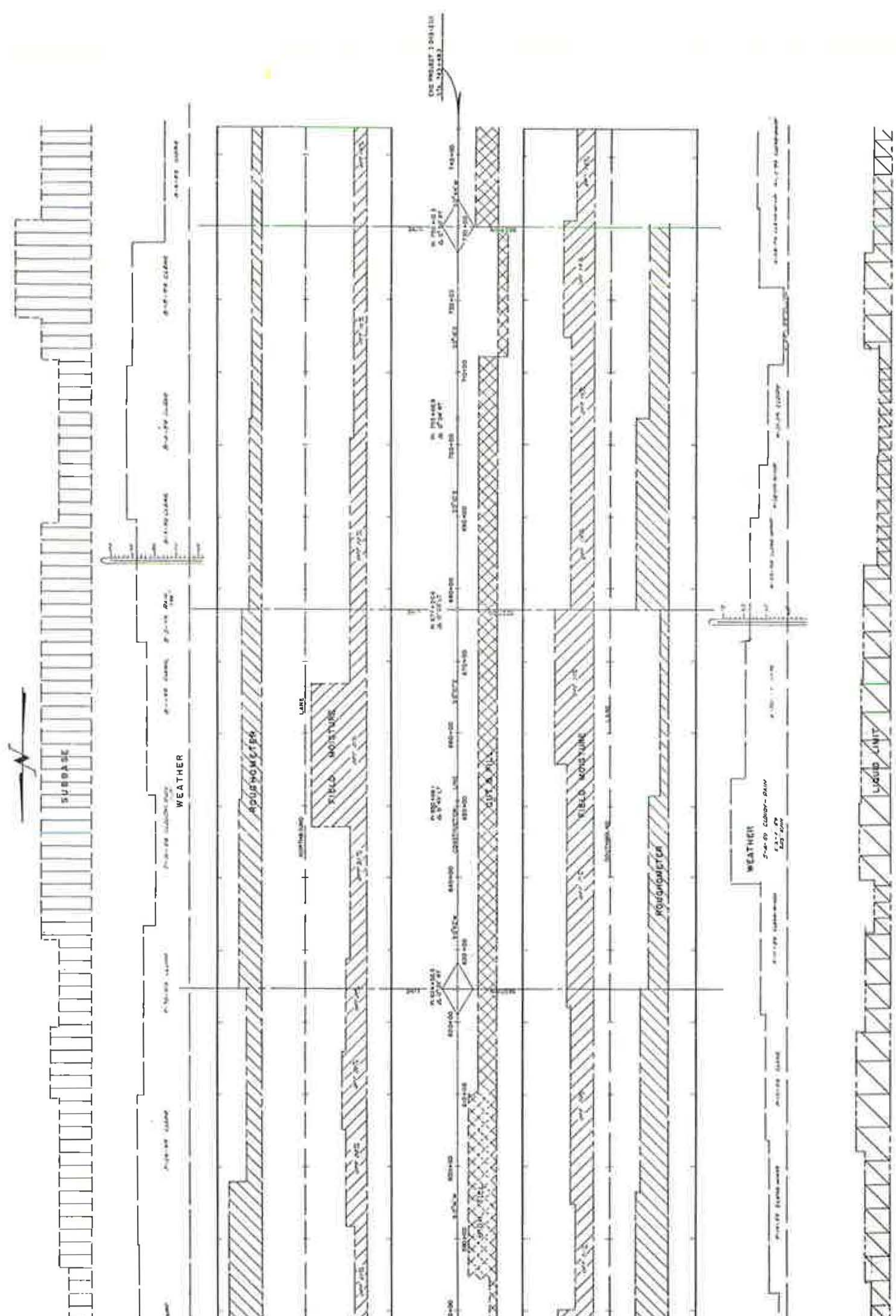

SUBBASE
WEATHER
ROUGHOMETER
FIELD MOISTURE
CUT & FILL
FIELD MOISTURE
ROUGHOMETER
WEATHER
LIQUID LIMIT

Figure 22.

centage of loss was not as large as noted in Minnesota. The average loss for all projects was only 13 percent. However, some projects did show as much as 43 percent loss. A few showed a higher bearing in the spring than in the fall. This latter finding points up the extreme difficulty in trying to predict exactly what will happen in any given year on the same project.

Benkelman beam tests were run on all of the projects where plate bearing tests were made. Results of the two tests did not correlate very well. A comparison of deflections between the spring and fall readings shows that, on the average, the spring reading is only 6 percent less than in the fall.

Recent studies of subgrade conditions on some of the projects built in South Dakota in the past five years indicate that if moisture is held closer to the optimum less roughness will develop due to frost. This appears to be especially true in subgrades that contain higher liquid limit soils (Figs. 21 and 22). More study is needed to determine just how closely the moisture content should be controlled on construction.

South Dakota has one Interstate project in which the upper 6 in. of subgrade has been treated with 3 percent lime to reduce the P.I. This project is the next project south of the one that has shown much warping and roughness. To date, the lime-treated project has not shown any roughness or warping, even though it is located in the same area of subgrade soil, terrain and moisture conditions that exist on the rough project. This is the second winter for the lime-treated project which is being closely watched to see if any roughness develops.

One Interstate project in the highly expansive soil area has a 4-in. bituminous-treated base course along with a standard 4-in. base and 4-in. mat. It is now in its third year and seems to be standing up quite well. It does not appear that the quality or thickness of the wearing surface or base in flexible pavements would be a factor in design criteria, insofar as frost action is concerned. The overall total thickness of frost-free material is believed to be the most important factor. Total thickness in South Dakota is usually two-thirds of the expected frost penetration.

Although there is presently no design factor in rigid pavement thickness determination, South Dakota experience indicates that a thicker subbase generally provides a better riding surface. It appears that this is an area where more research should be made to determine to what degree roughness could be reduced in pavement.

The design of beddings under culverts and approaches to bridges, as described in the report, appears to be the proper approach to the problem of uplift and heaving at the structures. As mentioned previously, it appears that possible lack of control in placement of these items on construction may be a factor. Indications are that this may be true because on some projects several of the culverts and approaches will be very smooth while others placed in the same soil will be rough.

There is no doubt that all the roughness is not due to frost. It is true that compaction on berms is rather difficult to achieve on the slope and possibly some special equipment could be developed for this purpose. Also, it is rather difficult to eliminate all of the internal differential settlement, especially in the higher fills.

Figures 21 and 22 were developed in an analysis of the causes of rough joints. Several factors, along with the roughness index have been placed on an abbreviated profile, in graph form, to see if there was any definite correlation that might determine the causes of rough joints.

It should be pointed out that the roughness index is general in that there may be areas within the sections marked rough that were fairly good. This is indicated by some of the tests that seem to contradict the average or general trend in these sections. To have made a more accurate study of what actually happened it would have been necessary to set up special instrumented test sections so that a before and after record could have been made for comparison.

Test data indicate that several factors were involved in causing the joint roughness.

1. Natural inherent curl in the concrete due to temperature and moisture differential;
2. Low grade heights, especially in the areas of high water table;
3. High-plasticity soil combined with moisture differential; and
4. Frost action.

The fact that water moved back into the test hole after being channeled out to the edge of the pavement indicates that the top of the subbase material was not in contact with the bottom of the slab.

Figures 21 and 22 as plotted on the abbreviated plan, seem to indicate that generally where the grade line is in shallow cut or on a shallow fill, the roughness index increases. This appears to be especially true in the low, poorly drained areas.

The graph depicting the field moistures, as measured at the time of construction, tends to indicate that when the moistures were from 2 to 4 percent below optimum the roughness index increases. The fact that moistures are presently over 2 percent above optimum at these same depths would indicate that the additional moisture which has collected would cause expansion in the high-plasticity soils.

There is no doubt that frost action caused some of the roughness. Heaves were visible in a few short areas and were responsible to a certain degree in other areas. The recovery of the joints to a relatively smooth condition, early this spring, would indicate that the frost had contributed to the rough condition.

In conclusion, it would appear that the following changes in design and construction methods would tend to reduce roughness.

1. Construct shorter slab lengths. Several sections of concrete should be laid down with the joints constructed in the manner adopted by New York State. This type of joint provides for a channel of specified width and depth, usually $\frac{3}{4}$ in. wide and $\frac{1}{2}$ in. to $\frac{3}{4}$ in. deep, to be cut through just above the normal sawed joint. This channel, when filled with sealing compound, allows the compound to move with the slab and prevents separation between the compound and the slab.

2. It does not appear that the earth subgrade should be less than 2.0 to 2.5 ft above the original ground line on any portion of a project. All grades should be built to this minimum even if it is necessary to bring in borrow material from outside the right-of-way.

3. Indications are that the moisture contents of the soil should be held to a closer tolerance, than is now allowed, for optimum moisture control. It appears that it would be better to hold the moisture slightly above optimum.

4. Sufficient time has not elapsed since stabilizing additives have been added to the subgrade soils on some South Dakota projects. Preliminary tests indicate that the waterproofing will retard the action of frost in the material directly below the slab. On the basis of these tests it may be desirable to include some type of stabilization in the soils prior to placing the subbase. It may also be desirable to treat the subbase material to reduce the plasticity index.

Observations and studies will be made of all of the projects built between Sioux City and Sioux Falls in order to determine if the changes, that have been made, have corrected the rough joints to any appreciable degree.

In conclusion, Mr. Fredrickson has covered the subject of frost problems as related to design very well. Many of the problems associated with frost action have been eliminated or at least reduced to a point where the ridability of present roads is quite good.

Economy seems to be a big factor in eliminating completely the effects of frost. Most of the personnel connected with surfacing design are familiar with methods of design which could almost completely eliminate all frost problems throughout an entire project. However, on some projects where frost-susceptible soil is practically the only soil available and the water table and terrain are such that sufficient grade height is difficult to maintain, the cost of design to eliminate all effects of frost is prohibitive.

Frost Considerations in Highway Pavement Design: Western United States

L. F. ERICKSON, Research Engineer, Idaho Department of Highways

The western United States highway departments all seem to have a similar approach in their consideration of frost effects in soils for the design of pavements. Geographically, all the western states' area with the exception of parts of California, Arizona, and Texas is considered subject to frost effects. Generally all roads are designed for all-season unrestricted loading and operations with the exception of very low traffic roads of secondary classifications in three states.

Soil characteristics considered as setting a criteria to frost susceptibility are generally silt by classification and any material having in excess of 10 percent finer than a No. 200 sieve. No special tests are reported as being used to measure susceptibility.

The location of soils considered frost susceptible within the horizon and with respect to the grade line varies. Some states are endeavoring to waste or otherwise dispose of frost-susceptible soils within an arbitrary zone with respect to the finish grade line of the highway, whereas others report no consideration given this condition. Similar considerations appear to be given the elevation of the water table with respect to the grade line.

No special geometric section or drainage feature or controls are attributed to frost effects in soils. It appears that generally accepted sections and treatments were developed with this feature considered as all present designs are believed adequate. Admixtures have been used only in very few instances as a means of controlling frost. Flexible pavement design criteria vary considerably. Some states appear to make no differentiation for frost susceptibility or their standards of design have "built in" a factor that is not identifiable directly. Others do give special consideration to frost by requiring added thickness, or set a minimum thickness of pavement structure depending on the depth of frost penetration, or the frost susceptibility of the soil. Designs for structure thickness are not varied for embankments or cut sections.

Rigid pavement designs seem to apply very similar criteria for the total thickness design. Subbase materials must meet general requirements similar to those for flexible pavements. Present designs are for all-season loading and operations of the pavement except for the very lowest traffic volumes on secondary highways in three states for either rigid or flexible pavements.

No special treatment of soils in foundations for structures is reported. Generally footings are carried well below frost line. Backfill material is required to meet general criteria for cleanness and free-draining properties. It appears that the frost considerations are so much a part of each department's routine oper-

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

ation that it is hard to separate specific considerations in design due to frost. All recognize frost problems and their design methods appear to provide a pavement structure considered adequate.

•THE western United States geographically encompasses an area of climatic extremes. Elevations for regularly used highways range from below sea level to over 11,000 ft. The latitude varies from semitropical areas to the 49th parallel, or an area where winters can be very severe except as modified by the Japanese current along the Pacific Coast. Rainfall varies from less than 1 in. per year to more than 150 in. per year. With these variations in precipitation and temperature, it is apparent that frost effects would likewise vary through very great extremes.

Figures 1 and 2 show the mean minimum and maximum temperatures for January. Figure 3 shows the range of the mean annual precipitation. Figures 1 and 2 indicate the extensive areas subject to daily freeze and thaw conditions.

The questionnaire submitted to all highway departments in the WASHO, with the exception of Alaska and Hawaii, requested clarification geographically of the areas within their state requiring special consideration for frost effects.

A different approach to the frost effect problem appears immediately. All of the western states acknowledge that frost must be given consideration throughout a part of their state, however, only Colorado, Idaho, and Washington report that 100 percent of their state systems require special consideration due to frost.

The remaining states report limiting design considerations because of frost:

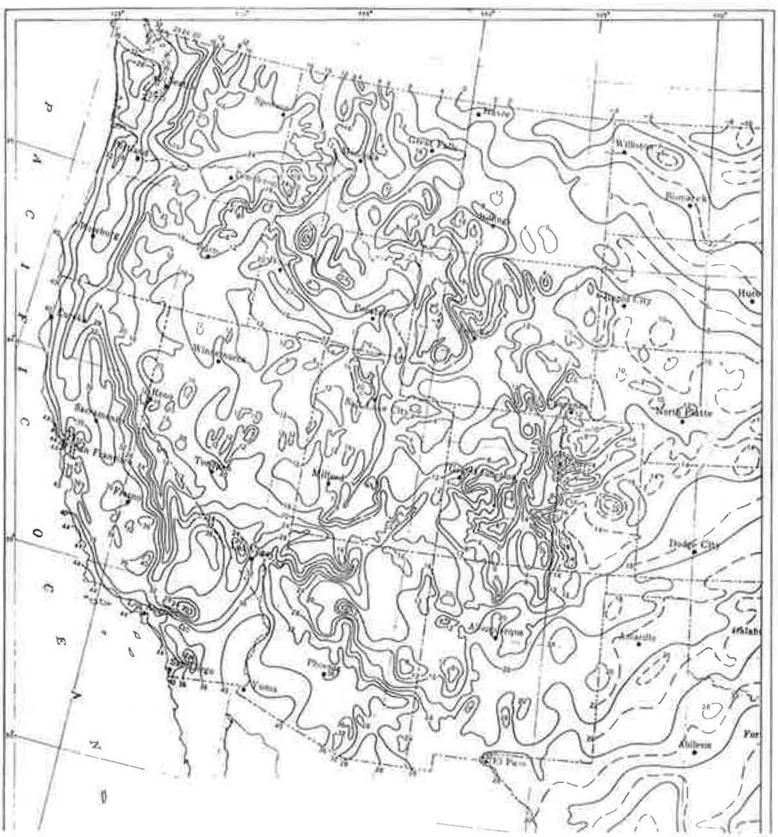


Figure 1. Mean daily minimum temperatures (°F) January.

Arizona	Northern half.
California	Mountain regions.
Montana	Area west of the Continental Divide, north central area and any area of silty soil having a high water table.
Nevada	Northern half.
New Mexico	Elevations above 6,500 ft.
Oregon	East of the western foothills to the Cascade Mountains.
Texas	Northwestern part.
Utah	Areas where moisture and frost are conducive, about 25 percent of state.
Wyoming	Only irrigated areas.

Thus, the approach to the frost problem varies greatly, and the degree of frost susceptibility considered to require attention varies.

Land use of land, that is, forested, cultivated, irrigated, etc., is not recognized by Arizona, California, Idaho, Nevada, New Mexico, Oregon, and Texas. However, Colorado, Montana, Utah, Washington, and Wyoming recognize land use as it affects the elevation of the water table.

As expected, all states are designing their heavy-duty concrete highways for all-season unrestricted legal axle loadings.

All states except Wyoming consider their asphaltic concrete pavements adequate for

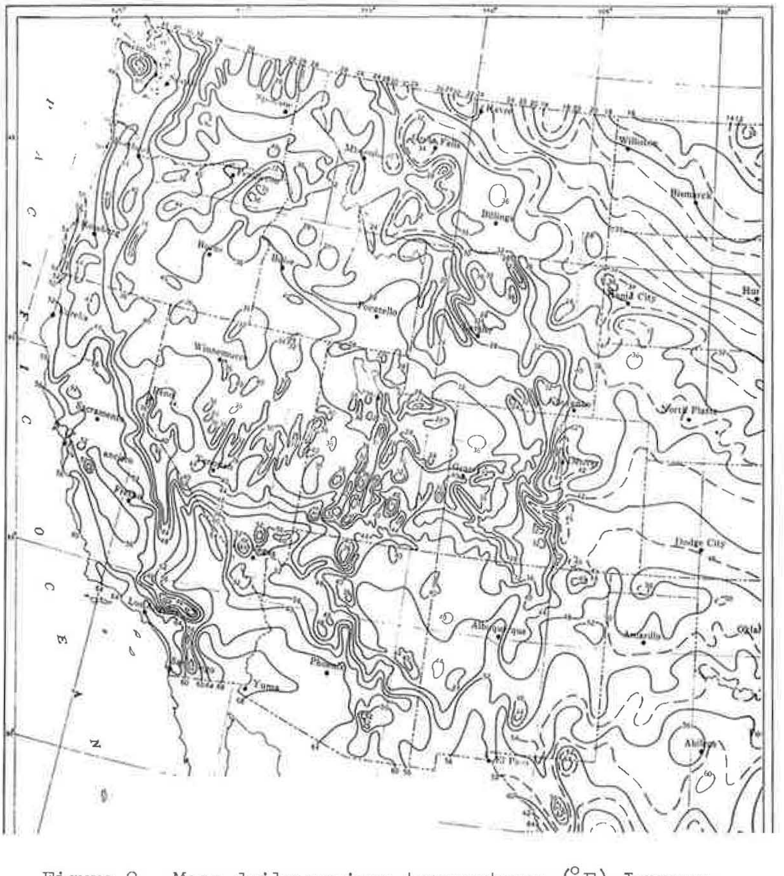


Figure 2. Mean daily maximum temperatures (°F) January.

Figure 3. Mean annual total precipitation (in.).

unrestricted legal axle loadings. Wyoming uses the 5,000-lb equivalent wheel-load method of evaluating axle loading and does not believe this method adequately provides for frost. Other states using the 5,000-lb equivalent wheel load for axle loading either believe it adequate or provide other means of adjusting thicknesses because of frost.

All states except Wyoming and Texas provide for unrestricted legal axle loading and operation for their intermediate roads. Again, Wyoming feels the 5,000-lb equivalent wheel loading is inadequate. Texas, without further clarification, reports that restrictions are applied to some roads.

Several states have designed parts of the secondary road system for springtime load restrictions. Montana applies load limits if the ADT is more than 100 vehicles per day. New Mexico and Texas apply restrictions to some roads and not to others.

Oregon and Idaho report studies for strength loss of soils or softening of the road bed during the spring. Oregon has reported work in previous HRB Proceedings and bulletins. This work was conducted for the Committee on Load Carrying Capacity of Roads and Airfields as Affected by Frost Action. Idaho conducted Benkelman beam deflection measurements during summer 1954 and spring 1955 with a few isolated tests since then. Their original work was reported at the WASHO Conference at Phoenix in 1956.

SOIL CONSIDERATIONS

The western states were asked if they had established any criteria for a "frost-susceptible soil" and any test or combination of tests to measure the degree of susceptibility. Conversely, they were also asked if they had any criteria or tests to assure that soils were not susceptible to frost. Answers vary from "No" to all questions to some interesting and apparently practical considerations.

The percentage passing the No. 200 sieve, together with liquid limit and plasticity index tests, appears to be the most accepted approach for determining if a material is frost susceptible. The classification "silt" was also noted as being a criterion for frost susceptibility.

The percentages passing the No. 200 sieve varied from a maximum limit of 8 to a maximum limit of 25 for a non-frost-susceptible soil. Generally, values less than 12 percent were reported as the maximum percentage passing the No. 200 sieve for base courses. Limiting values for liquid limit or plasticity index were not reported.

Colorado considers all its soil as frost susceptible. Arizona has established maximum percentages passing the No. 200 sieve for base materials depending on elevations with 12 percent permitted to an elevation of 2, 500 ft, 10 percent to 3, 500 ft, and 8 percent for elevations above 3, 500.

Montana considers A-1a(o), A-1-b(o), and A-2-4(o) soils least susceptible to frost. Utah reports that any sand or silty soil is susceptible if more than 25 percent passes the No. 200 sieve. Washington limits passage of the No. 200 to 10 percent for base courses and considers lesser percentages as non-frost susceptible.

SOIL PROFILE AND HORIZON CONSIDERATIONS

Consideration of the location of the frost-susceptible soil within the soil profile and the subgrade is given by all but four states. It appears possible that other considerations such as the depth to the water table are the governing factors in these four states.

States that consider the frost-susceptible soils to require special consideration base their action on the positon of the soil with regard to the subgrade. Idaho takes precautions to remove all top soil at the grade point and to further reinforce this area with granular materials. Montana uses a minimum of 2 ft of selected granular materials in the top of the embankments together with a thicker surfacing section. Other states report raising the grade line and wasting the frost-susceptible soils or burying them in the lower portions of embankments.

The depth to the water table is given special consideration by seven states and four report the water table presents no problem. Most states reported raising the roadway grade line if the water table was high. It appears that the dividing line between a high and low water table is considered to be about 4 or 5 ft.

The states giving consideration to the water table elevation remarked that their consideration was based on the influence of the water table on the moisture content of the soil.

Selective placement of soil is given consideration by nearly all states. Reasons given by several are to reinforce or strengthen the subgrade and to reduce the quantities of high-type base. Several report that the poor soils are buried low in the embankment. One state reports that it is too costly to consider selective placement. Another reports that uniformity of the subgrade is stressed, and still another that the poor soils are merely given added reinforcement with base.

The general specifications appear to be about evenly divided in requiring or not requiring selective placement of soil. Several states reported that the special provisions or plans provided for selective placement when desired. Only two states made a special note that payment for cross-haul was made, although the question was not asked. Several other states may also do this.

Five states show on the plans the soils to be excavated and replaced due to frost susceptibility. Others remarked this was done to increase the structural strength of the subgrade. It is important to note that several states report the soil areas are too extensive for this type treatment.

The quality of backfill material is mentioned in only two specifications, but several states provide for central laboratory, material engineer's, or another engineer's approval of the material to be used. It appears that this is not a specification-described material.

The use of a material to prevent intrusion of fine-grained soils into the coarser base or subbase materials is reported by six states; three of which use the Corps of Engineers D15/D85 ratio of less than five as their control. One state uses a A-3 sand if available, otherwise a bituminous membrane in the bottom surfacing course. Another specifies a material at least 15 percent finer than the No. 40 sieve and 25 percent finer than the No. 10 sieve. This is required only when the soil has more than 65 percent passing the No. 200 and a PI or linear shrinkage greater than 5.

GEOMETRIC DESIGN CONSIDERATIONS

None of the states appear to have special geometric designs due to frost considerations. Several note widened or deepened ditches to provide for snow storage. A few made occasional slight changes for short sections.

None of the states have any special drainage design features specifically for frost areas, although occasionally special drainage using perforated pipe underdrains is used for lowering the water table.

USE OF ADMIXTURES

None of the states appear to have used admixtures in any general way to control frost-susceptible soils. One state reports occasional use of sodium chloride in an attempt to prevent frost heaves. Another reports using portland cement and lime to control PI and upgrade aggregates. Several western states have used portland cement and bituminous materials to upgrade or stabilize base courses, although not specifically to reduce frost affects.

DESIGN OF FLEXIBLE PAVEMENTS

Seven states have provisions for varying their design thickness requirements because of frost. The others use a standard design throughout, but make variations in design due to type of soil, water table, and other considerations.

The criterion used for design is geographic in five states, i.e., regions wherein frost is no problem are noted and not given any consideration for frost. Three states use the maximum measured frost penetration. Two of these set a minimum thickness of the pavement structure, pavement, base, and subbase equal to one-half the frost penetration unless the soil strength calls for a greater thickness. Colorado has a table of factors which gives added thickness requirements depending on the penetration of frost and moisture conditions (see Appendix B).

Two states report an arbitrary thickness increase where frost considerations dictate—one state providing 2 in. of base, the other 4 in.

Apparently design considerations are applied to all soils as only two states made reference to this factor. One reported designs applied to all soils having more than 10 percent passing the No. 200, and the other remarked they wasted soils of high PI or "bentonite" type soils.

Limitation of axle loads apparently is not considered in design for frost except by Wyoming where it is believed that the equivalent wheel load method is insufficient to provide for all-season legal loads.

The use of material to prevent intrusion of fines into the subbase or base courses as a part of the total design thickness is common to all states except two. Further comments indicate that the material was considered in the design only if it was better structurally than the subgrade material.

None of the states report making any change in the design thickness for cuts or embankments. One state increases the thickness in cuts if the tendency toward a wet situation exists, then backfills with selected granular materials.

Embankments constructed from rock are capped with granular materials by 8 states. One reports using selected material only for construction purposes, and another reports inferior materials are avoided. Only two states report that no special materials were provided or attention given.

DESIGN OF RIGID PAVEMENTS

Only two states have any different considerations in the design of concrete pavement over frost- versus non-frost-susceptible soils. Both states add additional subbase over frost-susceptible soils even though both have used base material beneath the slab.

Two states use the same frost penetration criteria for rigid pavement designs as for flexible pavements using base or subbase to obtain the necessary thickness.

Four states consider the base and subbase as a part of the thickness design but furnish no details as to the manner applied.

Two states treat the base or subbase with portland cement beneath the concrete pavement to prevent pumping. Another state does this, but not because of frost. Pumping is caused by water and not necessarily frost, but spring thaws seemingly provide the greatest water supply at any time and in this way can be associated with spring breakup.

States using portland cement concrete pavements do not make use of any special material to cap rock embankments other than suitable material for a leveling course.

SUBBASE AND BASE COURSES

Four states reported on their criteria for measuring frost susceptibility of subbase and base courses. Three used the same criteria as for crushed-base materials, i.e., gradation, LL and PI. Only one state reports any specific test, that developed by McDonald (1). The state reports that although the test is not very precise, it has provided considerable information.

All states apparently pretest and designate sources of materials for subbase and base during the preliminary engineering phases. Approval is given to sources, but one state reports frost considerations are not included.

Seven states report that the gradation taken with the Atterburg limits or the sand equivalent determines the quality of the materials to be used. Some report that their standard specifications require the same limitations for percent passing the No. 200 and for LL and PI for subbase material as they require for crushed-base materials.

Ten states report that subbase and base courses are carried full width, and one other reports this is done when necessary for drainage. The ditch is also carried below the subgrade in 9 states with depths reported from 0.5 to 3 ft. Two states did not answer.

Only one state reports using any admixture to control frost susceptibility of base or subbase courses. In this instance, they permit 20 percent passing the No. 200 sieve and use cement or lime to stabilize the material if it is above 12 percent or the plasticity index is more than 6.

STRUCTURES

Limited information was obtained regarding frost-susceptible soils or backfill materials. Those reporting placed footings below the frost line and considered drainage of backfill materials of sufficient importance to mention.

GENERAL COMMENTS

The results of the questionnaire show that all the western states have some criteria that they use in design for frost-susceptible soils. Most of the factors are incorporated into their overall design criteria, but it is not always evident that certain requirements are essentially because of frost. This is particularly true in those states having a definite winter season throughout the entire state. Only those states having areas with limited or no winters apparently have recognized any major difference in designs.

Even though criteria differ throughout the states, it appears to be mostly the means to the end that differs. Essentially all states strive to keep the better soils in the subgrade, elevate the grade line to reduce effects of the water table and keep a free-draining subbase and base material over the subgrade. Criteria for the gradation limits and other properties do vary. However, as was pointed out previously, when the extremes of precipitation and climate are considered, this certainly must be no surprise. States having moderate to heavy precipitation with definite winters tend to have the most restrictive requirements for their subbase and base materials. Others with equally cold winters but limited precipitation apparently have found they can be less restrictive.

One factor that appears to be limiting special treatments of frost-susceptible soils is the extensive areas of materials that can be classified as definitely susceptible. In these instances, the design must be such that the roadway structure can carry all-season traffic even though these materials are used. The use of a pavement structure, i.e., subbase, base, and surfacing equal to half the frost penetration, is one approach used by two states. Others apparently find this uneconomical or not necessary.

Realizing that the availability of materials for use in subbase and bases is limited in many areas, it is understandable that available local material, which experience has shown to give acceptable service, is used extensively. Attention to the quality of bases and subbases and the upgrading of these materials by cement, lime, and bituminous materials is gaining in importance. All of the states apparently want to build roads capable of carrying legal axle loads all seasons of the year.

The factors involved in frost susceptibility are numerous. No one has developed a specific test for frost susceptibility as such, but reliance is placed on soil-identification tests, depth to water table, position of the soil within the roadway grade, etc., in determining the design. This approach appears to be giving good results.

REFERENCE

1. McDonald, C.H., "Investigation of a Simple Method of Identifying Base Course Material Subject to Frost Damage." HRB Proc., 29: p. 392-400 (1949).

Appendix A

QUESTIONNAIRE
HIGHWAY PAVEMENT DESIGN IN FROST AREAS—DESIGN CONSIDERATIONS

Questions asked the various State Highway Departments are answered in the Tables below. The numbers at the head of the columns of the tables refer to the question numbers below.

1. GENERAL INFORMATION

(a) What is the geographical extent of areas within your state wherein special consideration is given to frost effects in the design of pavements?

(b) Does land use (irrigated tracts - forest lands, etc.) provide a guide or limit to the geographic areas given special consideration? If so, please explain.

(c) Are pavements designed for all season unrestricted loading and operations on:

Heavy Duty Roads
Intermediate Roads
Secondary Roads

If other classification is used, please explain

(d) Have you made any special studies regarding the loss of strength of soils or of the softening of the roadbed during the spring. Please give reference to any reports.

2. SOIL CONSIDERATIONS

(a) Have you established any criteria for a "Frost susceptible soil"? If so, please furnish criteria.

(b) Have you any criteria or specific tests or combination of tests to measure the degree of frost susceptibility? Please furnish details.

(c) Have you any criteria or specific tests or combination of tests to assure that soils are not susceptible to frost? Please furnish details.

State	1(a)	1(b)	1(c)				1(d)	2(a)	2(b)	2(c)
			H	I	S	Other				
Arizona	Practically ½ state high enough elevation to require frost consideration	No - Altitude and soil analysis	Yes	Yes	Yes		No	Cover frost susceptible soils with sufficient material that frost no longer is considered. Specify use of non-frost susceptible base	Base materials to have maximum percent passing No. 200 sieve Elevation Feet / Maximum Percent Under 2500 — 12 2500- 3500 — 10 Over 3500 — 8	Grading and plasticity index
California	Primary Routes in Mountain Regions	No	Yes	Yes	Yes		No	No	No	No
Colorado	Entire State	Yes - irrigated lands or other land where ground saturated when frost present. Ref. Colorado Dept. of Hwys. Design Manual, Table 5-606.4 (See Appendix B)	Yes	Yes	Yes		None	All soils considered susceptible	None	None
Idaho	Entire State. Doubt special consideration given but do have few "built in" controls like percent 200 in base and an empirical soil number.	No	Yes	Yes	No		Periodic Benkleman Beam Deflection measurements	A-4 - A-5	No	No
Montana	Northwest area west of Continental Divide. North central area in Milk River drainage & areas of high water table.	Yes - irrigated and flood irrigated areas Combination excess water, high water tables, and heavy silty clayey soils can cause heaves.	Yes	Yes		Yes on secondary roads if over 100 ADT.	No - observation of past performance	No	Past experience with individual soil types	Consider A-1a(o),A-1-b(o), and A-2-4(o) soils least frost susceptible. Percent passing No. 200 sieve, liquid limit plasticity index on both No. 40 and No. 200 fractions usual guide.
Nevada	Northern ½ State	No - generally severity climate is guide - irrigated tracts considered where encountered.	Yes	Yes	Yes		No	No	No	No
New Mexico	Northern State, elevations over 6,500.	No - See 1(a)	Yes	Yes		Secondary roads - varies	No	Yes - Silt tested to verify	Permeability and freeze-thaw	No
Oregon	East of Cascade Mtns. western foothills	No	Yes	Yes	Yes		Yes - See HRB Proceedings Vol. 28 & 34, Research Report 10 D - Bulletin 40, 54, & 96.	Soils having more than 10% passing No. 200 sieve	No	No
Texas	Northwestern area only	No	Yes			Some intermediate & secondary are restricted	None available	No	No	No
Utah	Limited to areas where frost and moisture are conducive - 20-25% of State. Aridity of State not conducive to detrimental frost action.	Provided use furnishes moisture to frost susceptible soils.	Yes	No Ans	No Ans		No	Non - or slightly permeable fine sands and silts with more than 25% passing No. 200 sieve	No	Field experience - believe tests alone will not show areas susceptible.
Washington	Entire State	Yes - irrigation water effect on ground water has increased areas where frost must be considered.	Yes	Yes	Yes		No	Soil or aggregate with more than 10% Passing No. 200 sieve	No	No
Wyoming	Areas with light to heavy irrigation	Yes - irrigated areas only ones with high water table.	Concrete Yes Asphalt No	No	No	Asphalt pavement std. Section based on traffic count.	No	No	No	No

3. SOIL PROFILE OR HORIZON CONSIDERATIONS

(a) Does the location of a frost susceptible soil in a horizon influence your design? Please explain.

(b) Does the water table elevation with relation to the frost susceptible soil influence your design? Please explain.

(c) Are requirements for selective placement of soil considered and provided for in design? Please explain and if possible illustrate.

(d) Does your standard specifications provide for soil types (granular material) to be used selectively?

(e) Do you show on your plans areas of frost susceptible soils which are to be excavated from below subgrade and replaced with suitable backfill?

(f) Is the quality of backfill material used to replace frost susceptible soils specified in your general specifications or is choice of material left to your field engineers?

(g) Do you have any criteria for the use of a choker or blanket course immediately over fine grained soils to prevent intrusion into a subbase or base material? If so, please give details.

State	3(a)	3(b)	3(c)	3(d)	3(e)	3(f)	3(g)
Arizona	No	Water table is usually too low to influence frost action	No	In base and subbase only	Not necessary	No answer	Varies with available material controlled by PI & No.200 specification.
California	Only in special cases such as I-80 in the mountains.	No	No - for structural requirements only.	No	No	Where applicable included in specifications for project.	No
Colorado	No	Refer Colorado Design Manual Table 5-606.4 (See Appendix B)	Yes - when practical poor soils placed in lower portions of embankments.	No	No	Neither	No
Idaho	Yes - Topsoil or frost susceptible soils at transition cut and embankments excavated, backfilled with granular material - drainage provided. Depth below finished grade is to bottom topsoil or twice depth of "ballast" section whichever is least.	Keep ditch bottom 0.5' below base or any select granular material.	Specifications require saving granular material for selective placement. Project design may call for use in capping embankments.	Yes, see 3(c)	Yes, See 3(a)	Only that it be granular sources investigated during project development. Field engineers choice.	Yes - If percent passing No. 22 exceeds 65 & PI or linear shrink exceeds 5. Blanket material must have at least 15% passing No. 40 & 25% passing No. 10. Place 0.25' to 0.40' thickness.
Montana	Yes - Use selected granular soils in top of embankment (Min. 2') also thicker surfacing section.	Yes - Construct higher embankment or protect surfacing course with sand choke or bituminous membrane.	No - Poor soils in lower horizons. Best soils on top. Pay crosshaul - placed in 8" layers.	No - covered by special provisions	Yes	Left to field engineers judgment	Yes - Use 5" of A-3 sand when economically feasible based upon piping ratio of 5. When sand not available use a bituminous membrane full width in bottom of surfacing courses consisting of 3" - 4" depth roadmixed with SC4 or MC3 plus top and shoulders of surfacing courses given 2 applications Bituminous treatment down through membrane course.
Nevada	Silt pockets or layers are removed or covered by free draining material.	Yes - Roadway elevated above watertable elevation and placing free-draining base material above subgrade in low surfacing.	No - Control is in base Thickness - poor soils given greater thickness.	Yes - Thicker base courses	Yes - Generally in high water table areas.	Quality of backfill specified - obtain from roadway cuts or borrow sources.	No
New Mexico	Yes - Grade line kept high to avoid moisture in embankment - special pitrun material may be used.	Yes - Higher water tables cause greater susceptibility. See Question 2.	Yes - uniformity cannot otherwise be obtained.	No - Plans have notes indicating where conditions warrant selective placement.	No - See 3(c) for method used.	No - selection by project engineer based upon criteria from preliminary soil testing.	No standard - on high type roads a layer of cement treated base is often used over fine silty clays.
Oregon	Yes - if soil in subgrade zone.	This is not a problem	Yes - if free-draining granular material is used in subgrade to reduce base rock requirements.	Yes	Yes - in particular instances	General Specifications	Corps of Engineers criteria for filter material.
Texas	Yes - where experience indicates.	No	Yes - where economically feasible.	Yes	Yes - under special conditions.	Not in General Specifications.	No specific criteria
Utah	If location of soil and moisture conditions are such as to require. If soil 4' below final grade not considered.	Yes - frost penetration without available moisture does not produce serious subgrade failures.	Yes - Well graded granular materials as subgrade reinforcement. Thickness these materials plus base and surfacing roughly equivalent frost penetration	Yes	No - Generally areas are too extensive - Treatment is prescribed in design recommendations.	No - When specified central laboratory determines type and quality.	Not in Specifications. Central laboratory determines if warranted and based on type and quality of materials economically available.
Washington	Yes - Avoided, wasted or buried or covered with adequate depth frost free material where they must be used.	Yes - If water table is expected within 5 feet of subgrade elevation frost design called for if frost susceptible soils involved.	Yes - See 3 (a)	Not specifically for frost - Usually covered in special provisions for project.	See 3 (d)	Not in General Specifications. Approval of Materials Engineer required. Quality see 2(a).	15% size of Surfacing less than 4 to 5 times 85% size foundation material.
Wyoming	No	Yes - Water table elevation used determine where soils could be susceptible. Rainfall, frost action, water table and general conditions used to determine final design.	No - Would make costs prohibitive	Yes - Special granular backfill used around culverts and specified drainage areas.	Generally No	No	No - However, used in certain areas.

4. DRAINAGE

(a) Are different geometric sections used in areas subject to frost problems than in non-frost areas (ditch depths, shoulder slopes, etc.)? If any, please describe.

(b) Are any special drainage features or controls, if any, used in conjunction with your subgrade in frost areas versus non-frost areas? If so, please explain.

(c) Are any special drainage features or controls, if any, used in conjunction with your subbase or base materials? If so, please explain.

5. USE OF ADMIXTURES

(a) Have any admixtures been specified to control frost susceptible soils?

1. Calcium Chloride
2. Sodium Chloride
3. Bituminous materials
4. Portland cement
5. Lime
6. Sulphite Liquors
7. Other

(b) Please describe your success with the use of admixtures, i.e., degree of increased support attained, duration of effectiveness, control frost heave, etc.

State	4(a)	4(b)	4(c)	5(a) 1	2	3	4	5	6	7	5(b)
Arizona	No	No	No	No	No	No	No	No	No	No	None used
California	No	No	No	No	No	No	No	No	No	No	No answer
Colorado	No	No	Under drains where moisture conditions require	No	No	No	No	No	No	None	None
Idaho	No	No	See 3(a) for gradepoint treatment	No	Yes	No	No	No	No	No	NaCl placed in frost heave areas by drilling through pavement.
Montana	No - consider snowfall and snow storage in geometric design. Can expect 5 feet of frost penetration.	All sections designed for good drainage.	No	No	No	Yes See 3(g)	Yes	Yes	No	No	Have used soil cement where aggregates are scarce and hydrated lime to reduce PI in poor gravels - Reduces susceptibility to frost with satisfactory results.
Nevada	No - Ditches constructed below bottom of base course	In extreme cases perforated under drains carry water from base and subbase - bedding and backfill aggregate sand or sand-gravel with less 2% pass No. 200.	See 4(b)	No	No	No	Yes	No	No	No	One project stabilized with cement to prevent decomposed granite from heaving. Completed in 1961 - Satisfactory to date.
New Mexico	No - In general adopted wider roadway cut ditch to eliminate excessive water.	No - Use perforated pipe for sub-drains.	Cement treated bases tends keep moisture from working into underlying surfacing and subgrade.	No	No	No	No	No	No	No	None
Oregon	Emphasis on good drainage	Not in particular	None other than free draining specification material.	No	No	No	No	No	No	No	None
Texas	No answer	No answer	No answer	No	No	No	No	No	No	No	No experience
Utah	To date only some section changes in short road sections. Primarily of side ditch interceptions of water by cut widening, drain ditches, or drain pipes.	4(a)	4(a)	No	No	No	No	No	No	No	None
Washington	Deeper ditches used where snow may remain in ditch and plug them.	Not for frost	4(b)	No	No	No	No	No	No	No	None
Wyoming	No	No	No	No	No	No	No	No	No	No	None

6. DESIGN OF FLEXIBLE PAVEMENT STRUCTURE THICKNESS (Pavement, base, and subbase)

(a) Do you have any differing criteria for total pavement structure thickness in frost areas versus non-frost areas? Please explain.

(b) If any differing criteria is used, is it applied to soils types generally or to any specific soil type? Please explain.

(c) If limited axle loadings are provided for, how are these adjustments made in your design?

(d) Do you consider any choker or blanket course used as a part of your total pavement structure thickness?

(e) Do you vary pavement structure thicknesses for embankments versus cuts for the same soil types? Please give criteria if any.

(f) Are any special materials specified to be used in capping rock embankments? Please give details.

State	6(a)	6(b)	6(c)	6(d)	6(e)	6(f)
Arizona	Standard design thickness	No answer	Not limited	Considered part of total thickness.	No	Attempt to avoid inferior materials - Material used depends upon material available.
California	No	No answer	No adjustments	Yes	For drainage purposes primarily; frost effects considered only in special cases.	Only for structural and construction purposes
Colorado	Total thickness of subbase, base course and surfacing is partly determined by depth of frost penetration (Ref. Colorado Design Manual - 5-606.4) (See Appendix B)	None	See Table 5-606.4 Appendix B	No	No	Yes - we use subbase material for leveling course.
Idaho	No			Yes	Extra thickness of granular materials provided in cuts if wet conditions are anticipated.	No - Specifications provide for using "approved granular materials".
Montana	No - Use HRB Group Index for thickness design with thicker bases over poorer soils.	Applied to all soil types & group indexes modified by local soil and moisture conditions.	Design for 20 years Projected traffic type and volume.	Yes - for choke course but not for blanket course of granular material.	No - Use same thickness	Variable - Depending on local conditions and type of rock.
Nevada	On Interstate projects total base and surface thickness is increased 4 inches in northern ½ of State or in frost areas.	Total base thickness is increased over minimum where poor soils are encountered. Determined during design from soil samples.	No	No	No	Yes - granular material specified and used as a cushioning material to cap rock fills and cuts.
New Mexico	Regional factor for thickness of Pavement Structure.	Applied to soils generally	Reflected in Traffic Index	Yes 3(g)	No	Yes - subbase or base course is used to level out rock cuts and fills, to profile grade.
Oregon	In frost areas total pavement structure equal to ½ frost penetration if exceed "R" Value for Soil.	All soil, if 10% or more material passes No. 200 sieve.		Yes	No	No
Texas	No - due to limited depth of frost problem.				No	
Utah	No - Extra thickness is applied in subgrade. Reinforcement with granular materials.			No - See 6(a)	No	Only a base leveling course.
Washington	Frost design thickness equal to ½ frost penetration in area for frost susceptible soils.	Only to frost susceptible soils.		If it is of better quality or higher "R" value than subgrade on which placed.	No	No
Wyoming	Yes - frost areas noted on soils profile by field engineers - total thickness designed will be increased 2" over that required by Stabilometer.	Only on specific type such as "Bentonite". Soils having extremely high PI are noted on soils profile to be wasted.	Axle loading are limited and design done accordingly	When recommended included as part of total pavement structure thickness	No	Yes - Rock kept at least 2 ft. below profile grade on embankments. Special material used limited in maximum size and clay type soils are not to be used in the top 6" of the grade.

7. DESIGN OF RIGID PAVEMENT STRUCTURE THICKNESS
(Pavement and Sub-base)

(a) Do you have differing criteria for total pavement structure thickness in frost areas versus non-frost areas? Please explain.

(b) If differing criteria is used, is it applied to soil types generally or to any specific type? Please explain.

(c) If limited axle loadings are provided for, how are adjustments made in your design?

(d) Is the sub-base course, blanket or choker courses considered as imparting any structural strength?

(e) Are you providing for treating of the sub-base material with admixtures to prevent pumping? Please explain.

(f) Do you vary the pavement structure thickness for embankments versus cuts for the same soil types? Please explain if used.

(g) Are any special materials specified to be used in capping rock embankments? Please give details.

State	7(a)	7(b)	7(c)	7(d)	7(e)	7(f)	7(g)
Arizona	No		Not limited	Problematical	No	No	Attempt to avoid inferior materials - Material used depends upon types available.
California	No			No	Yes - All projects have 4" - 6" cement treated material directly under the PCC.	No	No
Colorado	None			Yes	No	No	Yes - Sub-base material used for leveling course.
Idaho	Have no design Standards						
Montana	Usually Minimum 1 ft. of base course over free-draining embankment soils - Over frost susceptible soils 2 ft. blanket or sub-base or good granular material placed.	Soil types generally	Slab designed to carry expected traffic volumes with a 6" cement treated base - Additional base and sub-base courses are used to protect from frost heave with thicknesses based upon soil types or local conditions.	Yes - Slab is designed to carry traffic volumes and loads without additional base.	No	No	Use available material
Nevada	No answer						
New Mexico	No		Not limited legal only	Yes	Yes - 4" of cement treated base under PC pavement	No	Yes - See 6(f)
Oregon	Frost areas - Total pavement structure equal to at least ½ frost penetration.	See 6(b)		Yes	No	No	No
Texas	No				Yes - Not in connection with frost.	No	No
Utah	No			No	No	No	Gravel base materials as a leveling course.
Washington	See 6(b)	See 6(b)	No	Yes - See 6(b)	No	No	No
Wyoming	Yes - Areas of frost action thoroughly investigated. Standard rigid pavement designed and 4" of crushed base provided where frost is detrimental - additional sub-base to above used.	Yes - Applied to specific soil types in relation to water table.	Standard pavement design based upon overloadings as outlined in PCA manual "Concrete Pavement Design".	No	No - Sub-base is a specification material.	No	See 6(f)

8. SUBBASE AND BASE COURSES

(a) Do you have any criteria for frost susceptibility of subbase or base materials?

(b) Are sources of subbase materials pretested and designated for use?

(c) Are sources of base materials pretested and designated for use?

(d) What test methods and test limitations are specified to control quality of subbase and base materials for frost susceptibility?

(e) Are subbase and base courses constructed full width from subgrade shoulder to subgrade shoulder?

(f) Are ditch bottoms carried at a level lower than the subgrade on which any subbase is placed in frost areas? How far?

(g) Are admixtures ever specified for use to control frost susceptibility of subbase or base materials? If used, what is your criteria, testing procedures and test value limitations?

State	8(a)	8(b)	8(c)	8(d)	8(e)	8(f)	8(g)
Arizona	Use test reported in Highway Research Board Proceedings, Vol. 29, 1949, Page 392. Investigation of a simple method of identifying base course material subject to frost damage	Yes	Yes	Maximum % Pass No. 200 and plasticity index.	Yes	Usually	No
California	No	Not for frost action	Not for frost action	None	When necessary for drainage.		No
Colorado	No	Yes	Yes	None	No	Not in all cases	No
Idaho	No	Yes	Yes	No quality criteria for frost. Have gradation and sand equivalent controls otherwise.	Yes	Yes 0.5 feet	No
Montana	Yes - limit % Pass No. 200 to 12% and LL to 35, PI to 6 by special provisions.	Yes	Yes	% Pass No. 200, LL & PI	Yes	Usually 1 foot	No
Nevada	No answer						
New Mexico	No	Yes	Yes	None	Yes	No	No
Oregon	Yes	Tested and recommended for use	Tested and recommended	Standard Specifications for crushed materials	Yes	Yes - in special instances to 3 feet	No
Texas	No	Yes	Yes	None	Yes - generally	Yes	No
Utah	No	Yes	Yes	Use Standard AASHO tests for gradation. Limit percentages fine sand and silt fractions in soils designated to use.	Yes	Yes	No
Washington	See 2(a)	Yes	Yes	Grading only See 2(a)	Nearly always except when special free-draining shoulder section is used	Ditch bottom carried 6 inches below subgrade elevation frost or no frost	No
Wyoming	Use Specification materials - these considered not susceptible.	Yes	Yes	Subbase crushed to pass 1½" square sieve with less 20% pass No. 200 LL & PI meet base specifications. LL less 25 PI less 6.	Yes	Yes - ditch bottoms always below subbase.	Yes - when subbase or base has PI greater than 6, cement or lime used to improve quality.

9. STRUCTURES

(a) Are any special criteria employed for structures in areas subject to frost effects? If so, please explain.

(b) Do you use a special backfill or embankment material that is non-frost susceptible adjacent to structures or culverts? If so, please give details.

(c) Are any other treatments of soil made due to frost with regard to structures and pipe?

10. OTHER

Please add any comments you wish regarding Design Considerations relative to frost action in soils.

State	9(a)	9(b)	9(c)	10
Arizona	No	All backfill for structures both in or out of frost areas shall have the sum of the % passing No. 200 and plasticity index not to exceed 23.	No	
California		Granular material is required for all structure backfill.	No	
Colorado	Bridge footings placed below frost line.	No	No	
Idaho	No	Yes - granular material with less 5% pass No. 200 and Sand Equivalent of at least 70.	No	
Montana	No	Permeable granular material, less 15% pass No. 200	No	Good embankment construction by layer placement with compaction control and selective placement of poor soils in lower horizons producing embankment uniformity together with adequate surfacing courses can eliminate detrimental frost action. We have found no cheap method of frost heave control, but have generally eliminated this condition by good embankment construction and adequate depths of surfacing materials.
Nevada	No Answers			
New Mexico	Footings below frost and use granular backfill.	Yes - Specifications require a specified granular backfill.	No	
Oregon	No	Yes - Backfill with a free-draining material.	No	
Texas	No	No	No	
Utah	Foundations below frost elevation.	Backfill conforms to Specifications for imported borrow.	No	
Washington	No	Yes - Standard backfill materials are non-frost susceptible.	No	
Wyoming	Footings are a minimum of 4 feet below ground line.	Yes - Replace poor material due to consolidation rather than frost.	No	Considerations are given in design to frost action in soils even though specific tests to determine this are not run. By having specifications on subbase and base and having embankment placed at 95% compaction we believe detrimental frost action is at a minimum.

Appendix B

The Colorado Department of Highways, Design Manual, Section 5-606, Design Procedure for Flexible Pavements provides for varying the total thickness as follows:

"When (CBR) values are used on basement soils, the following procedure shall be used to determine the required thickness of subbase material:"

"The design curve to be used is determined by summing up the values assigned to the FROST conditions, moisture conditions and traffic conditions on Table 5-606.4."

"The gravel equivalent of the total thickness of subbase, base course surfacing and pavement is determined from Figure 5-606.4 by drawing a vertical line from the indicated (CRB) to an intersection with the designated design curve. From this intersection point, a horizontal line drawn to the left side of the chart will indicate the gravel equivalent of the combined thickness."

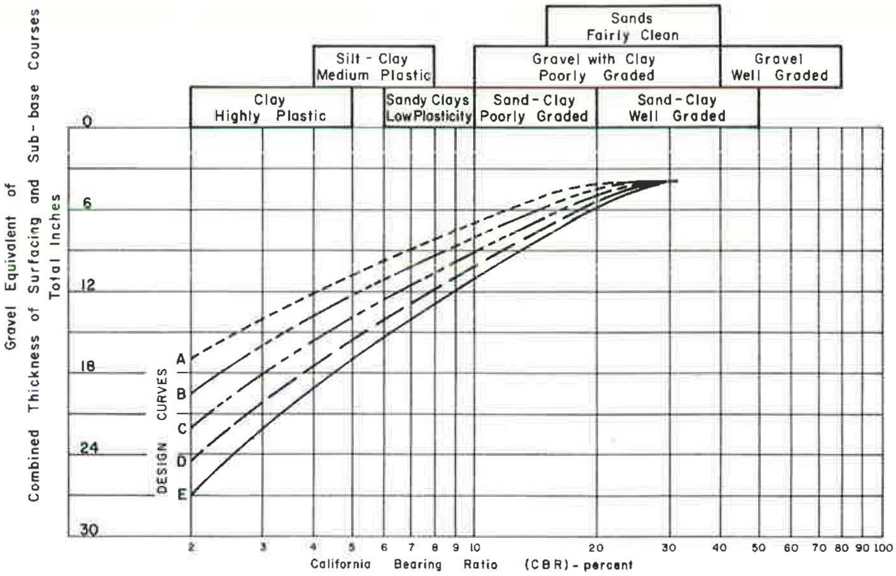
"The required subbase thickness is determined by subtracting from this gravel equivalent, the gravel equivalent of the base course surfacing and pavement."

Table 5-606.4

Design Curve Selection

FROST CONDITIONS	ASSIGNED VALUE
Penetration of 0" to 12"	3
Penetration of 13" to 24"	5
Penetration of 25" to 36"	7
Penetration over 36"	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 400,000 EWL	0
Traffic of 400,001 to 800,000 EWL	1
Traffic of 800,001 to 1,600,000 EWL	2
Traffic of 1,600,001 to 2,400,000 EWL	3
Traffic of 2,400,001 to 3,200,000 EWL	4
Traffic of 3,200,001 to 5,600,000 EWL	5
Traffic of 5,600,001 to 8,000,000 EWL	6
Traffic of 8,000,001 to 12,000,000 EWL	8
Traffic from 12,000,000 EWL	10
SUM OF ASSIGNED VALUES	
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

DESIGN CHART FOR THICKNESS OF SURFACING & SUB-BASE COURSES



Discussion

R.V. Le CLERC, Washington Department of Highways—Mr. Erickson has summarized a rather voluminous amount of information into a form which can be easily read and assimilated. As such, he has made it possible for the utmost benefit to be derived from such a questionnaire. The summary text will enable the reader to see what others are doing and perhaps obtain a few new ideas—further details can be found in the questionnaire or from the state source of the information.

Most of the frost design methods appear to be based on recognition of the three basic conditions nearly always associated with frost problems (frost-susceptible soil, available water, freezing temperatures) and removal of one or more from consideration by various means.

Opposite ends of the scale of frost design methods might be the design which seeks to prevent any freezing or frost heave at all, and the design which ignores the frost prevention and provides a structural section to accommodate the weakened, thawed subgrade. Most of the methods, by plan or happenstance, fall somewhere between these--at least it is believed that the Washington approach does. It falls into the group which calls for a depth of frost-free cover equal to one-half of the maximum depth of frost penetration in the location concerned. This is used only where the three elements of frost potential are evident, and the depth of cover to provide this frost protection is considered together with that called for by the stabilometer "R" value and that indicated by swell pressure in arriving at the design cover depth for the subgrade soil. A depth necessary to meet all these requirements is used.

The depths of maximum frost penetration were obtained by field measurements during an exceptionally cold winter of 1949-50. The measured depths and their locations were spotted on a state map and rough contour lines of equal frost penetration drawn through these points. The maximum frost penetration depth used in the design is taken from this map.

Frost susceptibility is judged by the amount passing the U.S. No. 200 sieve—any soil having more than 10 percent passing this size sieve is considered frost susceptible.

If water table at the onset of freezing temperatures is within 5 ft of roadway elevation, water is considered available for frost damage through formation of lenses and frost heave.

Although the above criteria are somewhat classic and deal with frost heave and its consequent damage, it is believed that frost contributes to roadway deterioration in another manner which is less obvious and which does not require the classic conditions listed heretofore.

The sequence of events occurs in this manner:

The rains in the fall season are somewhat continuous and contribute to an increase in the water content of the underlying base, subbase, and possibly subgrade. A prolonged cold spell moves in with freezing temperatures prevailing for approximately a week or ten days. During this period, particularly if there is snowfall, the thawing which might occur during the day does not penetrate to a depth sufficient to permit vertical drainage, and lateral drainage is hardly ever present, even in unfrozen shoulders. The alternate freezing and thawing tends to accumulate water under the roadway, and promotes an abnormally high water content in the surfacing (base and subbase) courses. Also, the freezing contributes to some expansion and consequent loss of density in these materials. The freezing temperatures need not be exceedingly severe, just 15 or so degrees below freezing, to bring about this condition.

With the warming trend that follows, the accumulation of water plus the loss in density, however small, is manifested in increased amplitude of roadway surface deflection under load. Before the water can be dissipated and the density of the affected base and subbase courses regained, the fatigue life of the pavement surface has been seriously shortened or surpassed.

The sequence of events has been noted often in Washington and no doubt occurs in other states. The rapid appearance of surface cracking is usually noted, and close observance will show that the extent of cracking increases with each cycle of similar freezing conditions.

The work of Oregon and Idaho in measuring loss of strength in roadbeds during spring is directed to the solution of this problem—particularly the studies which involve measurement of deflection. It is believed that the continued studies of roadway deflection and behavior during this critical period will lead eventually to a much better understanding of this problem and give design engineers more tools with which to work, or at least another factor to add to the list of those which must be considered in designing a roadway for reasonable life expectancy.

Corps of Engineers' Pavement Design In Areas of Seasonal Frost

K. A. LINELL, Chief, Experimental Engineering Division, U.S.A. Cold Regions Research and Engineering Laboratory, Hanover, N.H.;
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Definitions pertaining to design for frost conditions are presented. Conditions necessary for ice segregation and the need for considering the effects of frost action in pavement design are discussed. In addition, discussions are presented on frost-susceptible soils, the detrimental effects of frost action and investigational procedures for determining frost susceptibility and its magnitude. Base course composition requirements are discussed and frost design procedures are presented with examples. Also, requirements for field control of construction for frost conditions and standard laboratory frost susceptibility test procedures are given.

• SUBSTANTIAL design, construction, operation and maintenance difficulties were experienced by the Department of the Army in regions of seasonal frost and permafrost during World War II. The special problems of constructing pavements in these regions were especially apparent in the northern part of the 48 States, Canada and Alaska. As a result, the Frost Effects Laboratory was organized in the New England Division of the Corps of Engineers in 1944 and the Permafrost Division was established in the St. Paul district in 1945. These two organizations carried out extensive separate investigations in the period 1944 through 1953, and their successor organizations, the Arctic Construction and Frost Effects Laboratory, and now the U.S. Army Cold Regions Research and Engineering Laboratory, have continued these studies. Thus, the Corps of Engineers has carried out special investigations to improve the design of pavements in frost regions for nearly 20 years. In this time a great deal has been learned about the performance of pavements subject to frost action. Although much of the Corps of Engineers' effort has been aimed at development of designs to accommodate the great increases in weight and speed of aircraft and the requirements for longer and smoother runways, the design principles which have evolved are also applicable to roads and highways, even though the latter involve a much smaller range of wheel loadings. The first design criteria developed in these investigations were issued in the mid-1940's and successive revisions have been made in intervals since then.

This report summarizes the current practices (1). It includes three appendices which discuss the Corps of Engineers Standard Laboratory frost susceptibility test, field control of pavement construction for frost conditions, and design of base course drainage. Design examples are also given.

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

DEFINITIONS

The following specialized frost terms are used by the Corps of Engineers:

Frost and Soil Terms

Frost action. —A general term for freezing and thawing of moisture in materials and the resultant effects on these materials and structures of which they are a part or with which they are in contact.

Frost boil. —The breaking of a localized section of a highway or airfield pavement under traffic, and ejection of subgrade soil in a soft and soupy condition caused by the melting of the segregated ice formed by frost action.

Frost heave. —The raising of a surface due to formation of ice in the underlying soil.

Frost-melting period. —An interval of the year during which the ice in the foundation materials is returning to a liquid state. It ends when all the ice in the ground has melted or when freezing is resumed. Although in the generalized case only one frost-melting period is visualized, beginning during the general rise of air temperatures in the spring, one or more significant frost-melting intervals may occur during a winter season.

Frost-susceptible soil. —Soil in which significant detrimental ice segregation will occur when the requisite moisture and freezing conditions are present.

Non-frost-susceptible materials. —Cohesionless materials such as crushed rock, gravel, sand, slag and cinders in which significant detrimental ice segregation does not occur under normal freezing conditions.

Ice segregation. —The growth of ice as distinct lenses, layers, veins and masses in soils, commonly, but not always oriented normal to the direction of heat loss.

Pavement pumping. —The ejection of water and soil through joints, cracks and along edges of pavements caused by downward slab movements actuated by the passage of heavy axle loads over the pavement after the accumulation of free water beneath the pavement.

Period of weakening. —An interval of the year which starts at the beginning of the frost-melting period and ends when the subgrade strength has returned to normal summer values.

Base or base course. —As used herein, all non-frost-susceptible material between the pavement surfacing layer and the subgrade. For frost design purposes, any frost-susceptible materials underlying the base, whether subbase, embankment, or natural in-place soils, are considered as subgrade.

Temperature Terms

Average daily temperature. —The average of the maximum and minimum temperatures for one day or the average of several temperature readings taken at equal time intervals during one day, generally hourly.

Mean daily temperature. —The average of the average daily temperatures for a given day for several years.

Degree-days. —The degree-days for any one day equals the difference between the average daily air temperature and 32 F. The degree-days are minus when the average daily temperature is below 32 F (freezing degree-days) and plus when above (thawing degree-days). Figure 1 shows curves obtained by plotting cumulative degree-days against time.

Freezing index. —The number of degree-days between the highest and lowest points on a curve of cumulative degree-days versus time for one freezing season. It is used as a measure of the combined duration and magnitude of below freezing temperatures occurring during any given freezing season. The index determined for air temperatures at 4.5 ft above the ground is commonly designated as the air freezing index, and that determined for temperatures immediately below a surface is known as the surface freezing index.

Design freezing index. —The average air freezing index of the three coldest winters in the latest 30 years of record. If 30 years are not available, the air freezing index

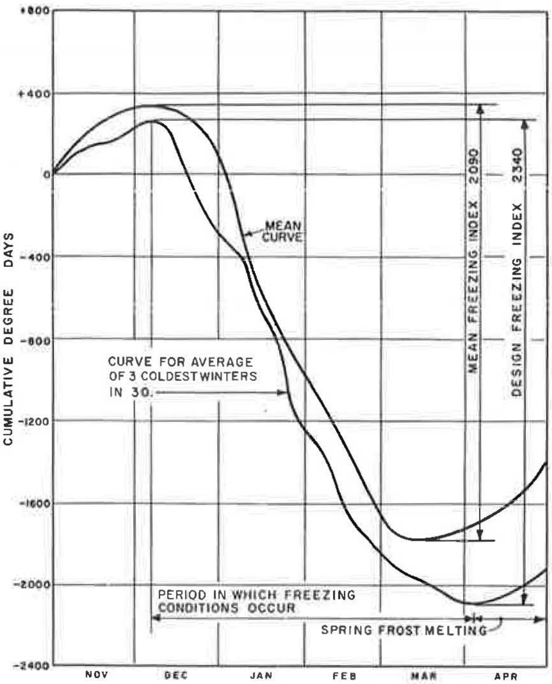


Figure 1. Determination of freezing index.

for the coldest winter in the latest 10-year period may be used. To avoid the necessity for adopting a new and only slightly different freezing index each year, the design freezing index at a site with continuing construction need not be changed more often than once in 5 years unless the more recent temperature records indicate a significant change in thickness design requirements for frost (Fig. 1).

Mean freezing index. —The freezing index determined on the basis of mean temperatures. The period of record over which temperatures are averaged is usually a minimum of 10 years (preferably 30) and should be the latest available (Fig. 1).

NEED FOR CONSIDERING EFFECTS OF FROST IN PAVEMENT DESIGN

The detrimental effects of frost action in subsurface materials are manifested by non-uniform heave of pavements or other structures during the winter as a result of ice segregation, and by loss of strength of affected soils with a corresponding reduction in load-supporting capacity during the period of weakening which ensues. Other related detrimental effects are possible loss of compaction, development of permanent roughness, restriction of drainage by the frozen strata, and cracking and deterioration of the pavement surface. In pavements, these effects may result in hazardous operational conditions, excessive maintenance, or pavement destruction.

Except in cases such as airfield pavement overrun areas where other criteria are specifically established, Corps of Engineers' design policy for permanent-type pavements requires that they be designed so that there will be no interruption of traffic at any time due to differential heave, reduction in load-supporting capacity, or deterioration of the pavement resulting from frost action.

CONDITIONS NECESSARY FOR ICE SEGREGATION

Three conditions of soil, temperature, and water must be present simultaneously in order for ice segregation to occur in the subsurface materials:

(a) The soil must be frost susceptible.

(b) Freezing temperatures must penetrate the soil. In general, the thickness of ice layers is inversely proportional to the rate of penetration of freezing temperature into the soil.

(c) A source of water must be available, such as an underlying ground water table, infiltration, an aquifer, or the water held within the voids of fine-grained soils.

The degree of ice segregation which will occur in any given case is markedly influenced by environmental factors such as transitions between cut and fill, lateral flow of water from side of cuts, and localized pockets of perched ground water.

DESCRIPTION OF ICE SEGREGATION IN SOILS

A strong attraction exists between unfrozen water immediately below the plane of freezing and ice crystals forming at the freezing plane. The water flowing to the crystals solidifies on the crystals as new ice. Continuing crystal growth leads to formation of an ice lens. A lens continues to grow in thickness in the direction of heat transfer, and at the same time laterally, until ice formation at a lower elevation cuts off the source of water, or until the temperature of the soil just below the surface of ice formation rises above the normal freezing point.

EXTENT OF FREEZING CONDITIONS IN THE NORTHERN HEMISPHERE

The extent and distribution of freezing conditions in the Continental United States, based on U. S. Weather Bureau data, are shown in Figures 2, 3 and 4.

The relationship between mean air freezing index and values computed on various other statistical bases is shown in Figure 5.

Distribution of freezing conditions in Canada, Alaska and Greenland is shown in Figures 6 and 7.

FROST-SUSCEPTIBLE SOILS

The potential intensity of ice segregation in a soil is dependent to a large degree on its void sizes, and for pavement design purposes may be expressed as an empirical function of grain size as follows:

> Most inorganic soils containing 3 percent or more of grains finer than 0.02 mm in diameter by weight are frost susceptible for pavement design purposes. Gravels, well-graded sands and silty sands, especially those approaching the theoretical maximum density curve, which contain 1 1/2 to 3 percent finer by weight than 0.02 mm size should be considered as possibly frost susceptible and should be subjected to a standard laboratory frost-susceptibility test (Appendix B) to evaluate actual behavior during freezing. Uniform sandy soils may have as high as 10 percent of grains finer than 0.02 mm by weight without being frost susceptible. However, their tendency to occur interbedded with other soils usually makes it impractical to consider them separately.

Soils classed as frost susceptible under the above criteria or determined as such by standard laboratory freezing tests, may be expected to develop significant ice segregation if frozen at normal rates with free water readily available.

Figure 8 shows results of laboratory frost susceptibility tests performed at the former Arctic Construction and Frost Effects Laboratory on natural soil gradations ranging from well-graded gravels to fat clays, using the standardized freezing procedure. Average daily rate of heave is plotted against percentage finer by weight than the 0.02 mm size. Test specimens were 6 in. high and 6 in. in diameter and were frozen with water made available at the base. The soils are representative of materials found in frost areas. The grain size distribution, dry unit weight, void ratio, uniformity and curvature coefficients, Atterberg limits, average rate of heave and frost susceptibility classification for each test specimen are given in Table 1.

The four diagrams at the left side of Figure 8 show individual test results for each of the four major soil groups: gravel, sand, silt and clay. A family of overlapping envelopes is shown at the right of Figure 8, which depicts the laboratory test results by various individual soil groupings as defined by the Unified Soil Classification System. A frost susceptibility adjective classification scale relating the degree of frost susceptibility to the exhibited laboratory rate of heave is shown at the left side of the latter diagram. Because of the severity of the laboratory test, the rates of heave shown in Figure 8 are not rates which may be expected under normal field conditions. Soils which heave in the standard laboratory tests at average rates up to 1 mm per day are considered satisfactory for use under pavements in frost areas unless unusually severe conditions of moisture availability and temperature are anticipated. In Figure 8, soils classed as non-frost susceptible under the criteria given at the start of this section are not necessarily free from susceptibility to frost heaving. Also, soils which are

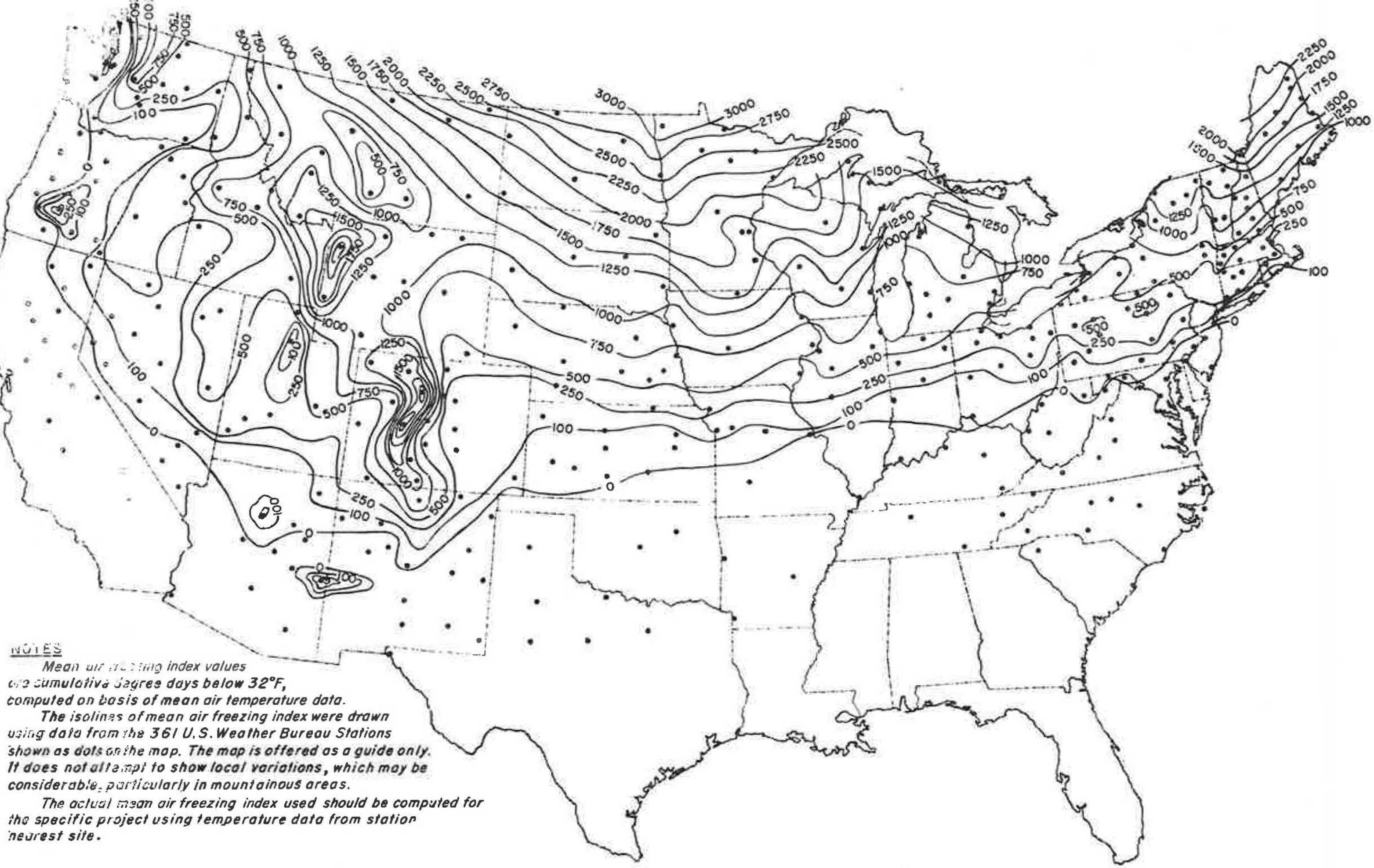


Figure 2. Distribution of mean air freezing index values in Continental United States.

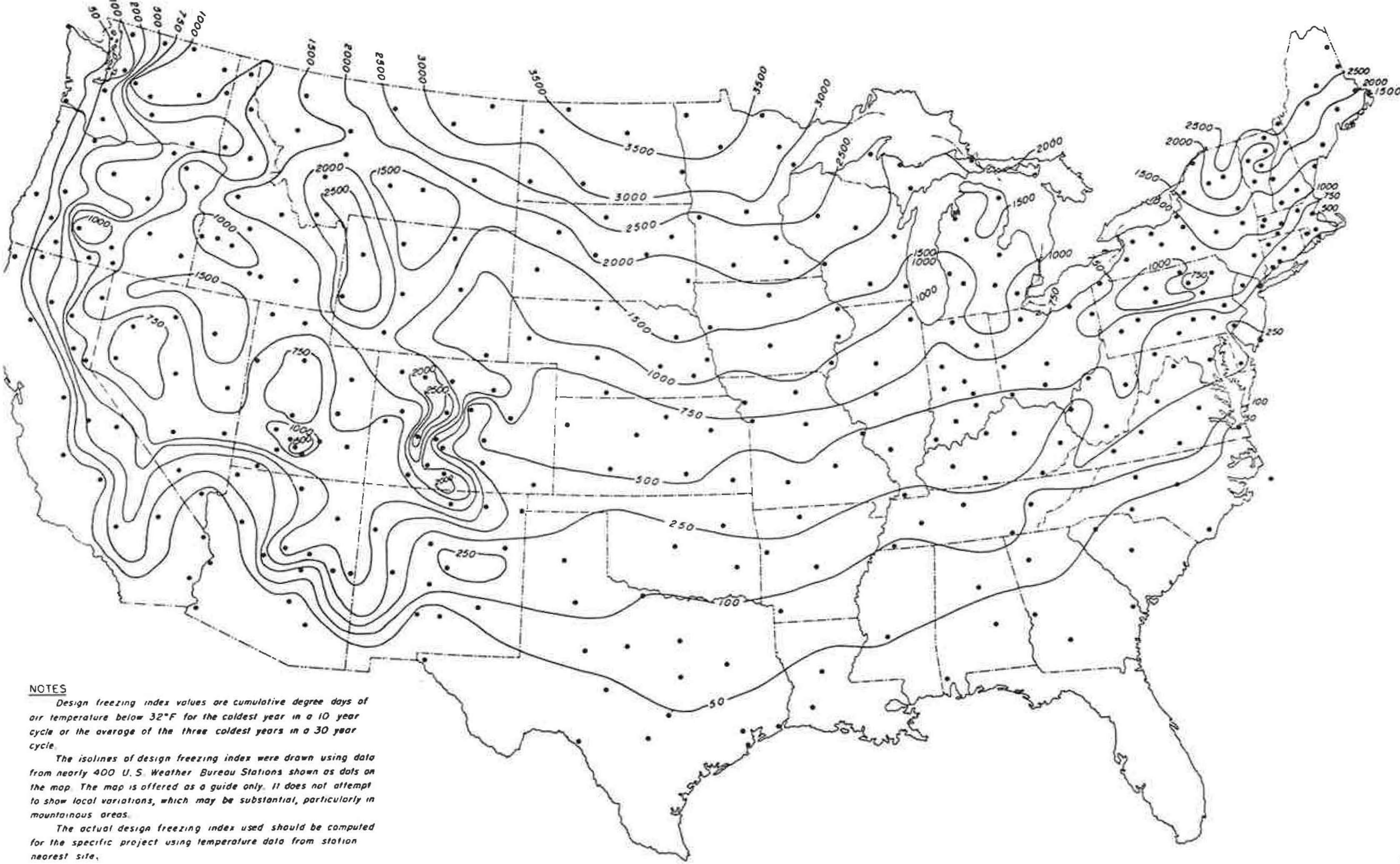
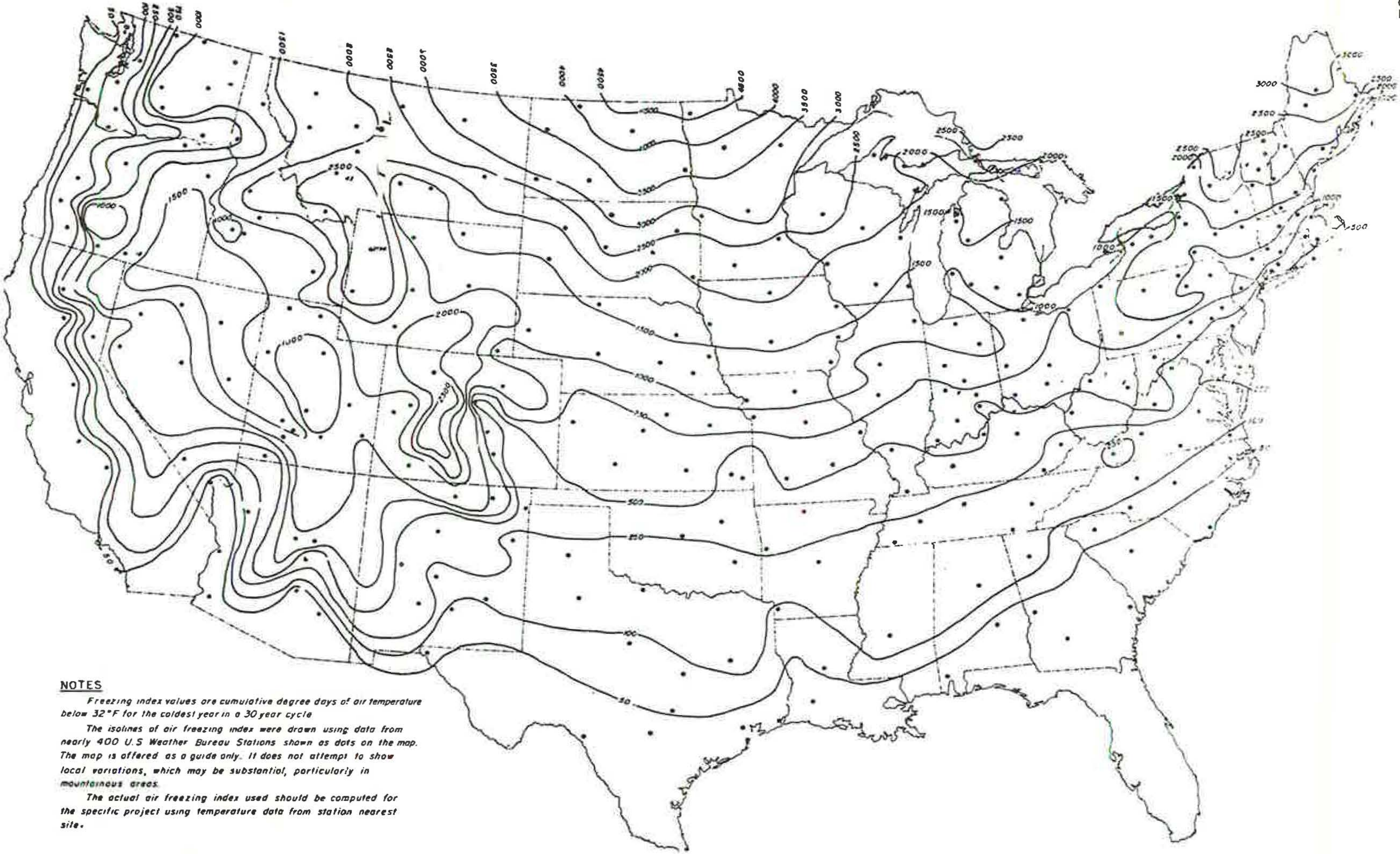


Figure 3. Distribution of design freezing index values in Continental United States.



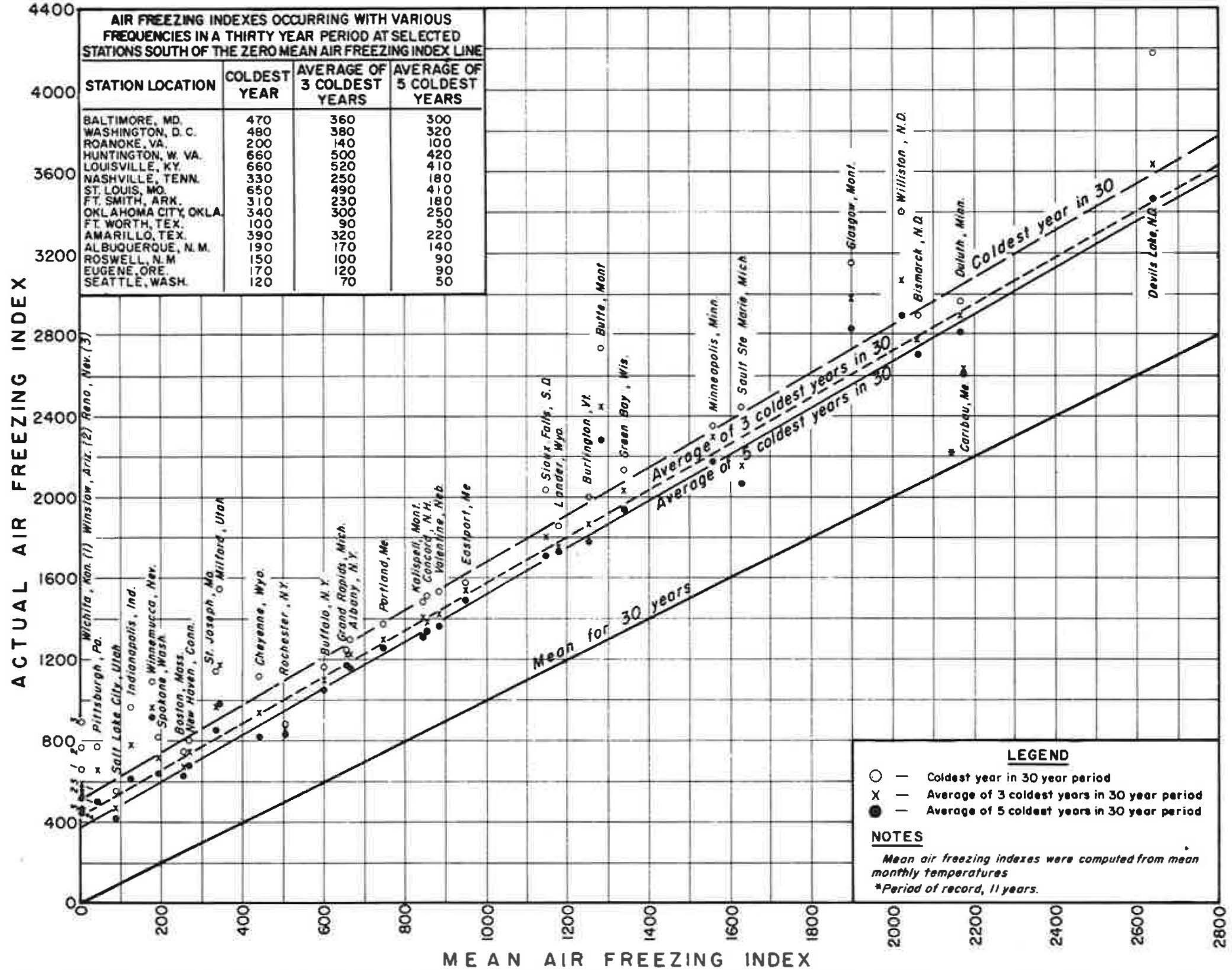
NOTES

Freezing index values are cumulative degree days of air temperature below 32°F for the coldest year in a 30 year cycle.

The isolines of air freezing index were drawn using data from nearly 400 U.S. Weather Bureau Stations shown as dots on the map. The map is offered as a guide only. It does not attempt to show local variations, which may be substantial, particularly in mountainous areas.

The actual air freezing index used should be computed for the specific project using temperature data from station nearest site.

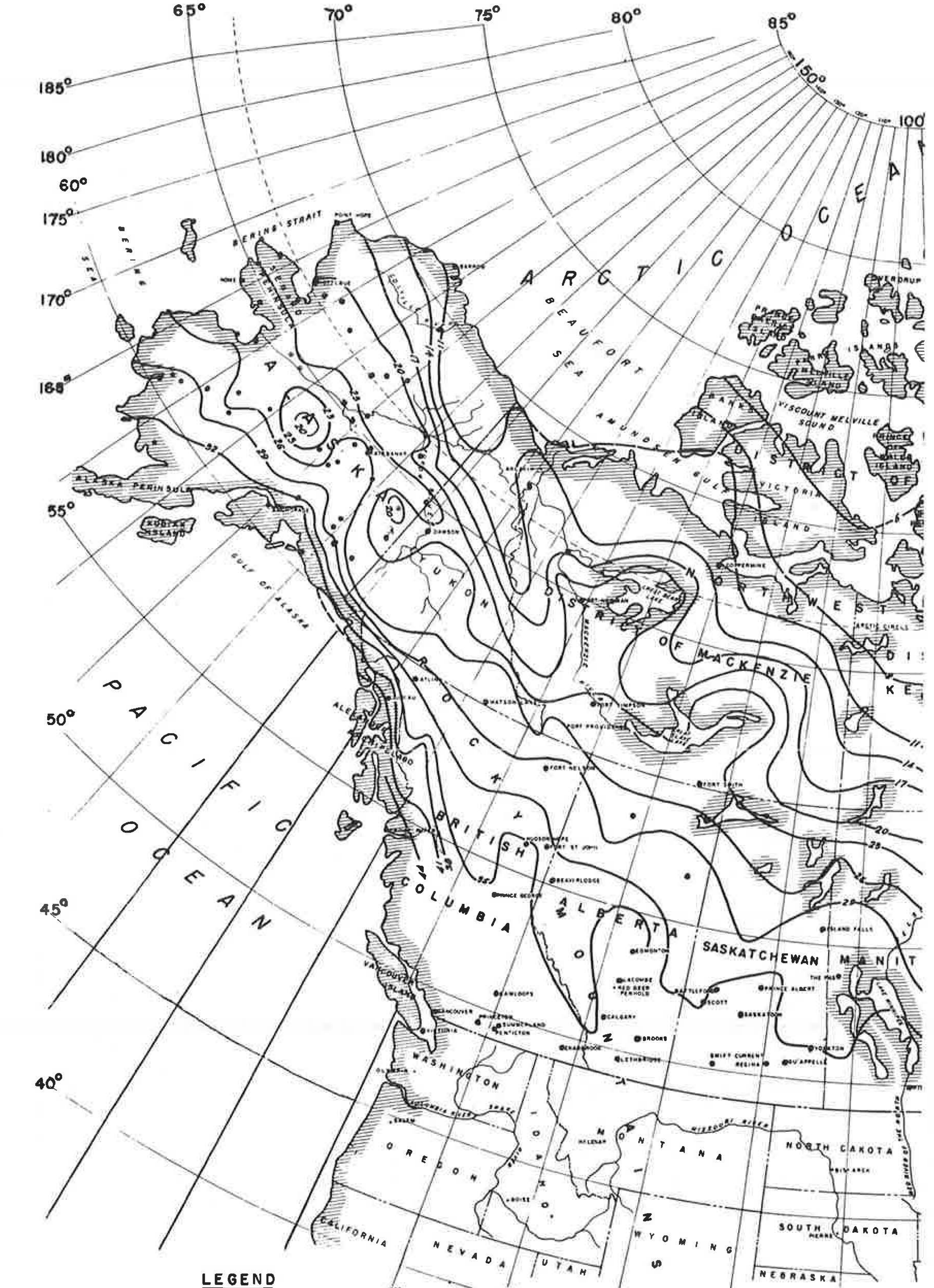
Figure 4. Distribution of air freezing index values in Continental United States for the coldest year in 30.



AIR FREEZING INDEXES OCCURRING WITH VARIOUS FREQUENCIES IN A THIRTY YEAR PERIOD AT SELECTED STATIONS SOUTH OF THE ZERO MEAN AIR FREEZING INDEX LINE

STATION LOCATION	COLDEST YEAR	AVERAGE OF 3 COLDEST YEARS	AVERAGE OF 5 COLDEST YEARS
BALTIMORE, MD.	470	360	300
WASHINGTON, D. C.	480	380	320
ROANOKE, VA.	200	140	100
HUNTINGTON, W. VA.	660	500	420
LOUISVILLE, KY.	660	520	410
NASHVILLE, TENN.	330	250	180
ST. LOUIS, MO.	650	490	410
FT. SMITH, ARK.	310	230	180
OKLAHOMA CITY, OKLA.	340	300	250
FT. WORTH, TEX.	100	90	50
AMARILLO, TEX.	390	320	220
ALBUQUERQUE, N. M.	190	170	140
ROSWELL, N. M.	150	100	90
EUGENE, ORE.	170	120	90
SEATTLE, WASH.	120	70	50

Figure 5. Relationship between mean air freezing index and freezing indexes during colder years for 30 consecutive years.

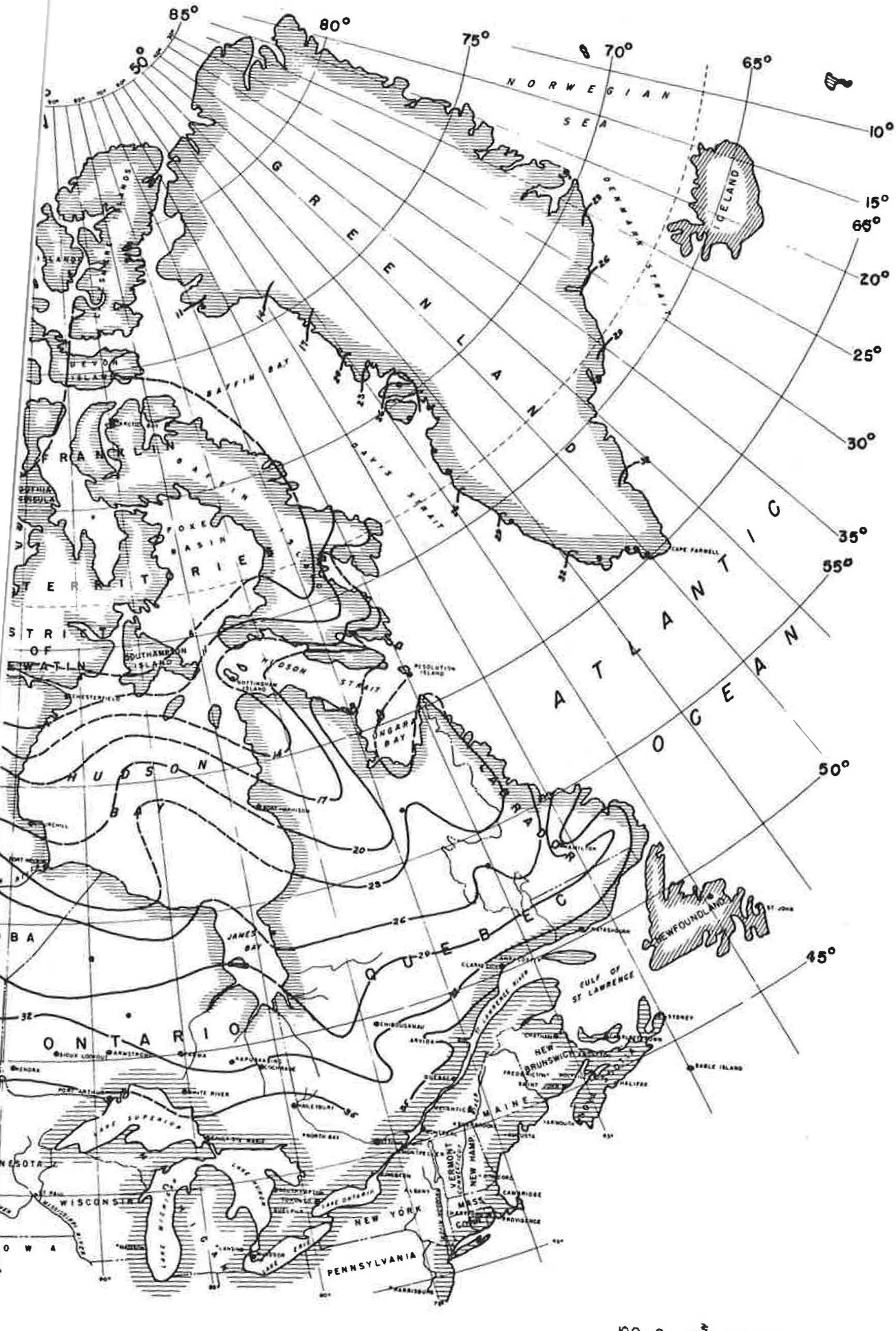


LEGEND

MEAN ANNUAL TEMPERATURE (INTERVAL 3°F)

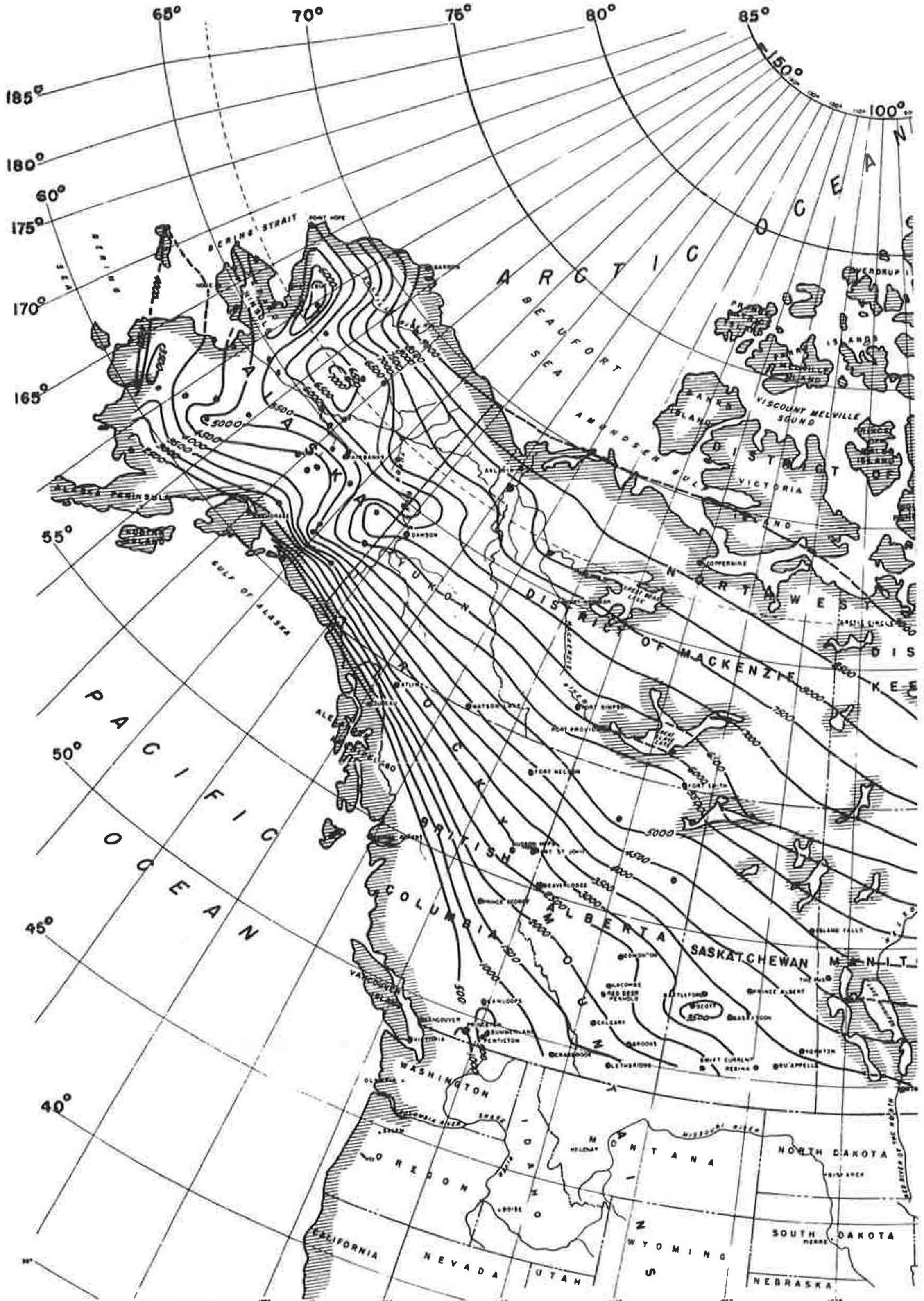
WEATHER STATION

Figure 6. Isolines of mean annual



perature in Canada, Alaska and Greenland.

100 0 100 200 300 400 500
SCALE IN MILES



LEGEND

——— DEGREE-DAYS OF FREEZE (INTERVAL, 500 DEGREE-DAYS F)

● WEATHER STATION

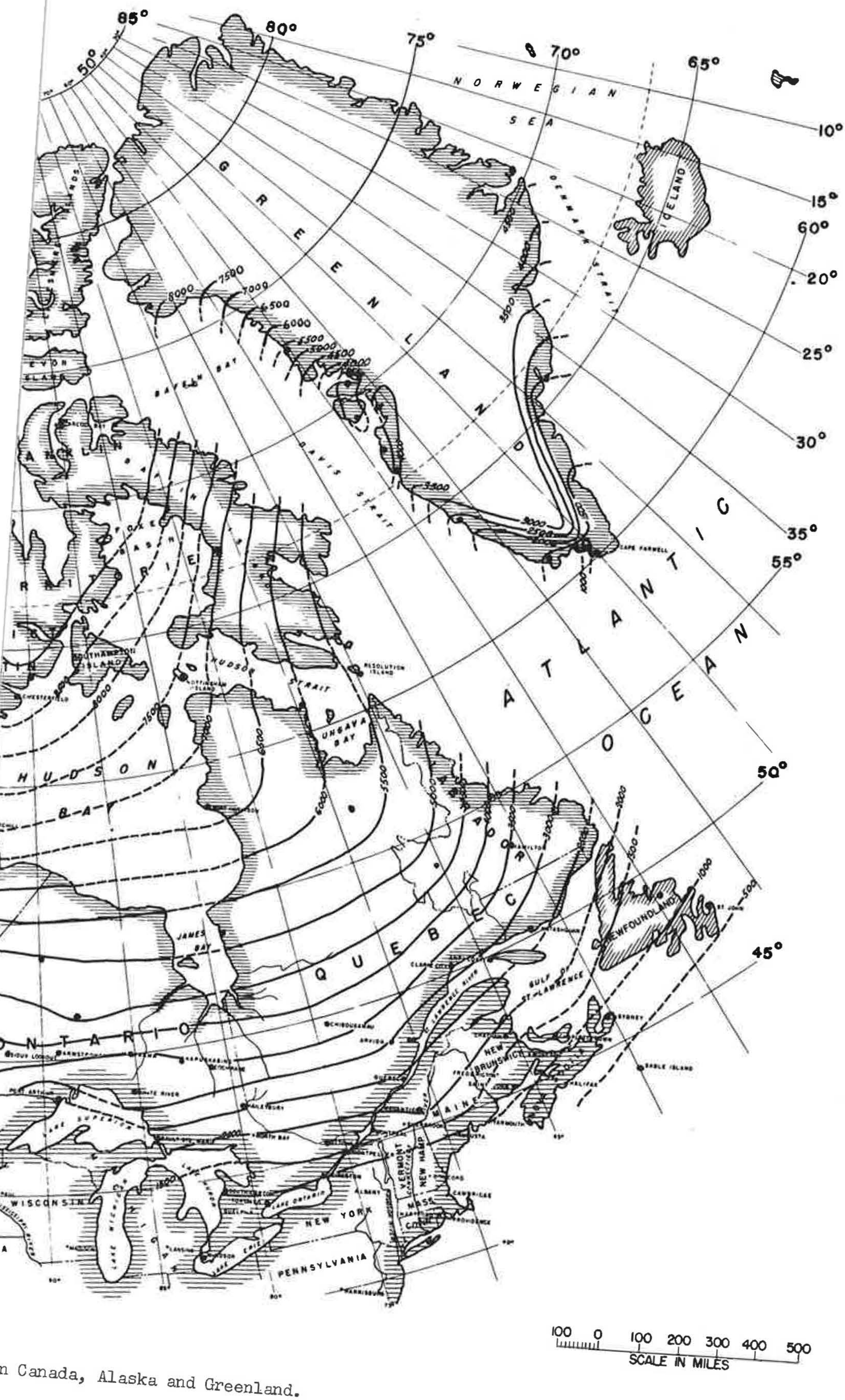
Figure 7. Isolines of mean freezing

Silty GRAVELS	GM	2-1/2	55	28	28	15	6.3	4.1	2.0	139	0.218	167	0.9	Non-plastic		2.9	Medium
		1	58	51	38	27	10	5.0	2.2	127	0.338	270	0.1	"	"	4.4	High
Clayey Sandy GRAVELS	GP-GC	3/4	37	25	14	11	6.6	5.0	3.2	134	0.265	145	16	24.9	8.7	2.0	Medium
		3/4	33	23	15	12	8.7	6.9	(5.0)	131	0.250	315	32	22.3	8.1	1.2	Low
Clayey Silty GRAVELS	GM-GC	1-1/2	54	45	30	20	15	9.0	5.0	129	0.320	485	1.9	24.8	6.8	3.7	Medium
Clayey GRAVELS	GC	1-1/2	48	42	36	22	17	15	12	133	0.252	4000	1.2	42.6	24.6	3.4	Medium
						SANDS											
SANDS and Gravelly SANDS	SW	2	53	36	13	3.8	1.8	1.4	0.9	129	0.277	20	1.1	Non-plastic		0.8	Very low
		2	58	40	15	4.9	2.3	1.5	1.1	136	0.254	23	1.3	"	"	2.4	Medium
		2	58	40	15	4.9	2.3	1.5	1.1	136	0.250	23	1.3	"	"	2.2	Medium
	SP	1-1/2	59	50	20	2.1	1.0	0.8	0.5	130	0.281	24	0.3	Non-plastic		0.4	Negligible
		1-1/2	59	50	20	2.1	1.0	0.8	0.5	130	0.283	24	0.3	"	"	0.3	Negligible
		1	72	24	7.0	3.0	1.3	0.9	0.5	139	0.440	5.3	2.0	"	"	0.2	Negligible
		2	85	70	8.6	3.6	1.3	1.2	(1.0)	116	0.469	3.4	0.2	"	"	0.6	Very low
		2	70	47	6.9	3.4	1.4	1.3	(0.9)	125	0.368	4.7	1.3	"	"	0.5	Negligible
		1-1/2	72	68	36	4.5	1.8	1.4	1.0	124	0.338	5.1	0.7	"	"	0.3	Negligible
		1-1/2	72	68	36	4.5	1.8	1.4	1.0	125	0.329	5.1	0.7	"	"	0.6	Very low
Silty Gravelly SANDS	SW-SM	3/4	57	42	16	5.0	1.4	(1.0)	(0.5)	111	0.532	27	1.1	Non-plastic		0.1	Negligible
		1	68	48	12	5.6	2.9	2.3	1.8	117	0.467	10	1.0	"	"	1.5	Low
		1-1/2	68	60	11	7.0	3.5	2.3	1.2	128	0.365	6.7	1.4	"	"	0.9	Very low
		2	69	52	20	9.6	3.8	(2.1)	(1.8)	135	0.268	28	1.8	"	"	2.8	Medium
		2	68	55	26	9.1	4.0	2.9	1.8	139	0.214	31	1.1	19.3	4.3	2.7	Medium
		2	68	55	26	9.1	4.0	2.9	1.8	138	0.224	31	1.1	19.3	4.3	2.5	Medium
		1-1/2	70	60	29	9.7	4.4	3.2	2.5	131	0.285	24	1.2	Non-plastic		1.2	Low
		1	57	31	20	8.7	5.0	3.5	2.0	144	0.179	43	1.1	"	"	6.3	High
		1-1/2	57	48	30	12	8.7	7.1	5.8	137	0.253	183	1.1	19.0	2.0	1.3	Low

TABLE 1

STANDARD LABORATORY FROST-SUSCEPTIBILITY TESTS ON NATURAL SOILS[a] (Continued)

UNIFIED SOIL CLASSIFICATION		MAX. SIZE	GRAIN SIZE[b] mm-% Finer							INITIAL DRY UNIT WEIGHT[c]	INITIAL VOID RATIO	COEFFICIENTS		ATTERBERG LIMITS		AVERAGE RATE OF HEAVE	FROST SUSCEPTIBILITY CLASSIFICATION[e]
SOIL TYPE	SYMBOL	in.	4.76	2.00	0.42	.074	0.02	0.01	.005	pcf	e	C_u[d]	C_c[d]	LL	PI	mm/day	
					SANDS (Cont'd)												
Silty Gravelly, SANDS (Cont'd)	SP-SM	3/4	60	55	39	9.7	1.8	0.8	-	137	0.246	62	0.2	Non-plastic		1.4	Low
		2	84	67	11	5.3	1.9	1.7	(1.0)	121	0.421	4.0	1.6	"	"	0.6	Very low
		-	-	100	71	8.8	2.2	1.3	-	114	0.473	4.3	1.5	"	"	0.2	Negligible
		-	-	100	86	5.0	2.6	2.4	1.8	115	0.450	2.0	0.9	"	"	0.1	Negligible
		-	-	-	100	6.3	2.6	2.2	1.7	109	0.516	1.9	1.0	"	"	0.1	Negligible
		-	-	-	100	6.3	2.6	2.2	1.7	109	0.514	1.9	1.0	"	"	0.3	Negligible
		1-1/2	73	48	11	5.2	2.7	2.2	1.8	129	0.316	8.1	0.9	"	"	0.5	Negligible
		3/4	66	50	18	6.0	2.8	1.7	1.0	133	0.278	15	0.9	"	"	1.3	Low
		1-1/2	74	51	25	6.9	3.2	2.7	1.8	127	0.329	15	0.6	"	"	1.1	Low
		1-1/2	77	57	27	7.1	3.3	3.0	2.6	127	0.329	13	0.7	"	"	1.3	Low
		3/4	99	97	84	10	3.3	3.0	2.0	113	0.484	3.4	1.8	"	"	0.8	Very low
		3/4	99	97	84	10	3.3	3.0	2.0	112	0.487	3.4	1.8	"	"	0.8	Very low
		1-1/2	98	98	80	8.8	3.3	2.0	0.9	108	0.518	2.8	1.4	"	"	0.8	Very low
		-	-	100	82	9.0	3.4	2.0	0.9	106	0.542	2.8	1.4	"	"	0.4	Negligible
		-	-	100	82	9.0	3.4	2.0	0.9	105	0.552	2.8	1.4	"	"	0.4	Negligible
		2	56	44	17	6.0	3.5	2.4	-	140	0.222	28	0.7	"	"	3.6	Medium
		1	66	49	22	6.5	3.9	3.8	3.0	134	0.238	17	0.9	"	"	0.8	Very low
		3/4	79	69	13	8.1	4.1	3.7	1.5	128	0.361	6.4	3.2	"	"	1.0	Low
		3/4	92	90	67	9.0	4.5	2.9	1.8	115	0.438	4.2	1.2	"	"	0.9	Very low
		3/4	92	90	67	9.0	4.5	2.9	1.8	119	0.396	4.2	1.2	"	"	2.7	Medium
		1-1/2	92	90	67	9.0	4.5	2.9	1.8	120	0.367	4.2	1.2	"	"	5.2	High
		3/4	71	63	46	10	4.5	4.0	1.8	138	0.215	20	0.3	"	"	1.9	Low
		1-1/2	59	55	39	8.5	4.5	2.5	1.6	134	0.280	6.0	0.2	"	"	1.4	Low
		1	80	54	24	6.5	4.9	3.8	3.0	135	0.228	15	0.8	"	"	1.0	Low
		1	63	51	30	7.0	5.0	3.0	2.0	137	0.212	28	0.4	"	"	1.1	Low
		1	71	55	27	7.8	5.0	4.0	3.2	132	0.250	16	0.6	"	"	1.2	Low
		2	61	50	29	9.7	5.1	4.2	3.1	130	0.289	52	0.7	"	"	2.1	Medium
		2	94	93	83	10	5.6	5.0	3.6	120	0.364	3.0	1.5	"	"	2.3	Medium
Silty SANDS	SM	-	100	99	95	28	1.5	1.2	0.9	107	0.567	2.5	0.9	"	"	0.1	Negligible
		-	100	99	95	28	1.5	1.2	0.9	109	0.540	2.5	0.9	"	"	0.1	Negligible
		-	-	-	100	33	2.5	(2.0)	-	112	0.551	1.6	1.0	"	"	1.7	Low
		-	-	-	100	33	2.5	(2.0)	-	111	0.565	1.6	1.0	"	"	0.9	Very low
		-	-	100	86	20	2.5	(1.8)	-	115	0.458	4.1	1.2	"	"	0.2	Negligible
		-	100	99	95	20	3.8	2.2	-	114	0.434	3.7	1.3	"	"	2.2	Medium



n Canada, Alaska and Greenland.

TABLE 1

STANDARD LABORATORY FROST-SUSCEPTIBILITY TESTS ON NATURAL SOILS[a]

UNIFIED SOIL CLASSIFICATION		MAX. SIZE	GRAIN SIZE[b] mm-% Finer							INITIAL DRY UNIT WEIGHT[c]	INITIAL VOID RATIO	COEFFICIENTS		ATTERBERG LIMITS		AVERAGE RATE OF HEAVE	FROST SUSCEPTIBILITY CLASSIFICATION[e]
SOIL TYPE	SYMBOL	in.	4.76	2.00	0.42	.074	0.02	0.01	.005	pcf	e	C_u[d]	C_c[d]	LL	PI	mm/day	
							GRAVELS										
GRAVELS and Sandy GRAVELS	GW	1	40	25	5.0	1.5	0.7	0.4	0.2	124	0.395	14	1.0	Non-plastic		0.5	Very low[f]
		3/4	49	30	10	3.0	0.8	0.8	0.5	109	0.589	17	1.4	"	"	0.1	Negligible[f]
		1-1/2	30	13	6.0	2.9	1.1	0.7	0.4	126	0.462	8.2	1.7	"	"	0.1	Negligible
		2	40	26	10	3.7	1.9	1.5	0.9	132	0.249	22	1.6	"	"	0.8	Very low
		3/4	49	36	12	4.7	2.4	1.7	0.9	138	0.231	20	1.1	"	"	2.2	Medium
		3/4	42	29	13	4.9	2.4	(1.7)	(0.9)	131	0.296	33	2.4	"	"	1.0	Low
		3/4	42	29	13	4.9	2.4	(1.7)	(0.9)	131	0.300	33	2.4	"	"	1.0	Low
		3/4	35	17	7.0	4.8	2.6	1.5	1.0	130	0.322	8.2	1.8	18.0	3.0	0.7	Very low
		3/4	35	17	7.0	4.8	2.6	1.5	1.0	132	0.309	8.2	1.8	18.0	3.0	0.3	Negligible
		3/4	49	32	11	4.9	3.2	2.6	2.0	137	0.237	24	1.4	Non-plastic		2.0	Medium
		2	40	27	8.0	4.6	3.7	3.3	2.7	135	0.255	17	1.0	"	"	1.6	Low
	GP	3/4	46	36	17	1.4	0.4	0.3	0.2	144	0.188	57	0.4	Non-plastic		1.7	Low[f]
		3/4	46	36	17	1.4	0.4	0.3	0.2	140	0.218	57	0.4	"	"	2.2	Medium[f]
Silty Sandy GRAVELS	GW-GM	2	42	33	19	5.7	2.0	1.3	1.0	139	0.200	87	1.1	Non-plastic		0.4	Negligible
		3/4	42	29	14	5.3	2.1	1.2	0.7	120	0.446	38	2.2	"	"	0.1	Negligible
		3/4	42	29	14	5.3	2.1	1.2	0.7	121	0.435	38	2.2	"	"	0.1	Negligible
		3/4	42	33	18	7.0	2.5	1.9	1.3	140	0.228	59	1.7	17.8	2.4	0.6	Negligible
		3/4	44	32	18	7.0	2.9	2.1	1.5	140	0.230	57	2.0	17.8	2.4	1.2	Low
		3/4	49	36	17	8.0	3.2	(2.2)	(1.5)	134	0.274	57	2.1	Non-plastic		1.1	Low
		3/4	49	36	17	8.0	3.2	(2.2)	(1.5)	132	0.288	57	2.1	"	"	1.2	Low
		2	53	40	20	7.4	3.5	2.5	1.3	139	0.231	48	1.0	"	"	2.6	Medium
		2	53	40	20	7.4	3.5	2.5	1.3	141	0.222	48	1.0	"	"	2.1	Medium
		3/4	51	34	12	5.5	4.0	3.3	2.3	137	0.237	22	1.3	"	"	1.9	Low
		3	47	30	13	7.5	4.3	3.2	1.8	132	0.267	47	2.2	"	"	2.5	Medium
		3/4	44	33	14	7.0	4.5	3.1	2.5	140	0.220	32	1.3	16.8	4.7	1.3	Low
		1	48	32	9.0	5.6	4.6	4.1	3.1	134	0.259	16	1.0	Non-plastic		2.0	Medium
		2	44	32	16	7.2	5.4	3.8	2.4	121	0.401	67	2.1	38.6	2.7	2.4	Medium
	GP-GM	2	27	19	10	5.2	3.1	2.0	1.2	121	0.401	40	4.7	38.6	2.7	1.1	Low
		2	47	40	23	9.1	3.2	2.1	1.5	136	0.233	120	0.6	Non-plastic		1.4	Low
		2	51	36	12	5.8	3.3	2.5	1.8	141	0.218	23	0.8	"	"	2.6	Medium
		2	51	36	12	5.8	3.3	2.5	1.8	141	0.221	23	0.8	"	"	2.2	Medium
		2	56	47	32	11	3.7	3.0	2.0	142	0.199	101	0.3	"	"	1.3	Low
		3/4	54	47	32	10	4.0	2.2	1.5	143	0.194	81	0.4	"	"	1.5	Low
		2	45	38	25	11	6.8	6.0	4.0	135	0.262	258	0.7	"	"	1.4	Low
		2	45	38	25	11	6.8	6.0	4.0	135	0.260	258	0.7	"	"	1.2	Low
		2	37	30	20	12	8.5	6.5	5.1	128	0.315	310	3.1	25.7	3.6	1.9	Low

		3/4	79	57	27	14	4.2	2.6	-	133	0.300	47	1.9	"	"	1.2	Low
		3/4	67	52	31	14	4.4	2.6	-	143	0.202	62	0.9	"	"	2.4	Medium
		-	-	-	100	21	4.5	2.5	1.0	106	0.578	3.0	1.1	"	"	0.3	Negligible
		-	-	-	100	21	4.5	2.5	1.0	105	0.593	3.0	1.1	"	"	0.6	Very low
		-	-	100	86	26	5.1	(2.4)	-	114	0.467	27	1.3	"	"	0.7	Very low
		3/4	66	61	45	17	5.2	3.7	2.4	135	0.258	47	0.4	"	"	2.3	Medium
		3/4	66	61	45	17	5.2	3.7	2.4	137	0.244	47	0.4	"	"	4.2	High
		3/4	66	61	45	17	5.2	3.7	2.4	136	0.252	47	0.4	"	"	2.1	Medium
		-	100	99	85	27	7.0	(3.0)	-	117	0.450	6.9	1.2	"	"	1.1	Low
		-	100	99	85	27	7.0	(3.0)	-	111	0.521	6.9	1.2	"	"	0.6	Very low
		2	84	80	47	13	7.5	5.3	3.6	123	0.374	17	1.9	"	"	1.7	Low
		2	76	72	49	17	7.8	4.5	3.0	122	0.384	28	1.4	"	"	1.8	Low
		3/4	98	98	94	29	8.2	5.4	3.7	109	0.560	4.0	1.8	"	"	1.0	Low
		-	-	100	97	48	8.8	4.0	-	120	0.419	4.0	0.8	"	"	1.3	Low
		2	58	46	27	14	8.9	7.5	6.0	128	0.312	250	2.2	21.9	3.0	0.8	Very low
		-	-	100	88	13	11	9.5	7.7	114	0.375	20	7.5	Non-plastic		2.3	Medium
		3/4	78	70	53	23	11	7.5	4.5	131	0.290	38	1.3	"	"	3.3	Medium
		3/4	71	65	34	23	11	6.3	4.0	136	0.280	95	2.2	21.6	2.9	2.5	Medium
		3/4	73	69	47	20	12	9.0	6.9	145	0.243	71	1.8	14.1	2.2	2.5	Medium
		3/4	73	69	47	20	12	9.0	6.9	144	0.248	71	1.8	14.1	2.1	2.9	Medium
		3/4	68	62	45	23	14	9.1	1.2	127	0.333	14	1.2	Non-plastic		4.8	High
		3/4	97	97	75	38	14	(7.0)	-	112	0.483	17	0.8	"	"	4.3	High
		3/4	90	88	79	28	15	12	9.0	130	0.300	36	4.2	"	"	1.3	Low
		3/4	97	91	73	31	17	(14)	(13)	124	0.374	280	18	18.3	2.8	6.3	High
		1-1/2	81	75	58	33	19	12	6.5	119	0.404	56	0.9	20.7	0.9	1.9	Low
		3/4	92	88	79	35	22	15	1.9	139	0.216	55	1.9	14.4	1.6	1.0	Low
Clayey Silty SANDS	SM-SC	1	71	55	28	16	9.0	6.0	4.3	131	0.292	108	3.7	24.1	5.9	1.6	Low
		3/4	65	55	39	22	14	10	7.0	148	0.215	310	0.9	16.1	4.3	2.5	Medium
		3/4	65	55	39	22	14	10	7.0	146	0.223	310	0.9	16.1	4.3	3.3	Medium
		1-1/2	91	79	48	23	15	13	11	120	0.378	225	13	22.0	4.6	1.3	Low
		1-1/2	62	50	33	22	15	10	5.5	135	0.267	400	2.7	22.0	6.1	2.5	Medium
		1-1/2	98	97	62	21	16	14	12	118	0.403	137	14	21.8	6.0	1.1	Low
		3/4	98	95	68	29	18	16	14	119	0.393	195	11	22.0	6.1	1.9	Low
		1-1/2	94	89	75	44	21	15	10	134	0.282	33	1.3	16.8	5.1	1.7	Low
		1-1/2	94	89	75	44	21	15	10	135	0.290	33	1.3	16.8	5.1	1.7	Low
		1-1/2	94	89	75	44	21	15	10	136	0.267	33	1.3	16.8	5.1	1.5	Low
		1-1/2	83	76	63	46	30	25	18	127	0.334	188	0.8	21.1	6.0	5.0	High
		1-1/2	87	76	62	48	32	24	15	127	0.334	100	0.2	21.1	6.0	3.1	Medium
		3/4	84	77	65	50	36	30	21	133	0.279	225	1.0	21.1	6.0	1.5	Low
Clayey SANDS	SC	1/2	98	90	33	18	9.5	7.5	5.5	123	0.374	72	3.2	30.7	10.5	1.1	Low
		3/4	73	68	55	35	23	20	15	134	0.272	500	1.7	24.7	8.1	1.3	Low
		3/4	76	72	60	41	24	18	13	139	0.237	191	1.1	24.0	11.0	0.5	Negligible
		3	82	77	66	48	30	23	17	130	0.293	115	0.9	20.7	7.2	2.2	Medium
		3/4	98	94	78	48	31	(25)	(22)	114	0.478	-	-	28.7	10.7	1.7	Low
		3/4	80	72	58	44	35	31	22	139	0.234	310	0.1	18.6	9.2	1.3	Low

TABLE 1

STANDARD LABORATORY FROST-SUSCEPTIBILITY TESTS ON NATURAL SOILS[a] (Continued)

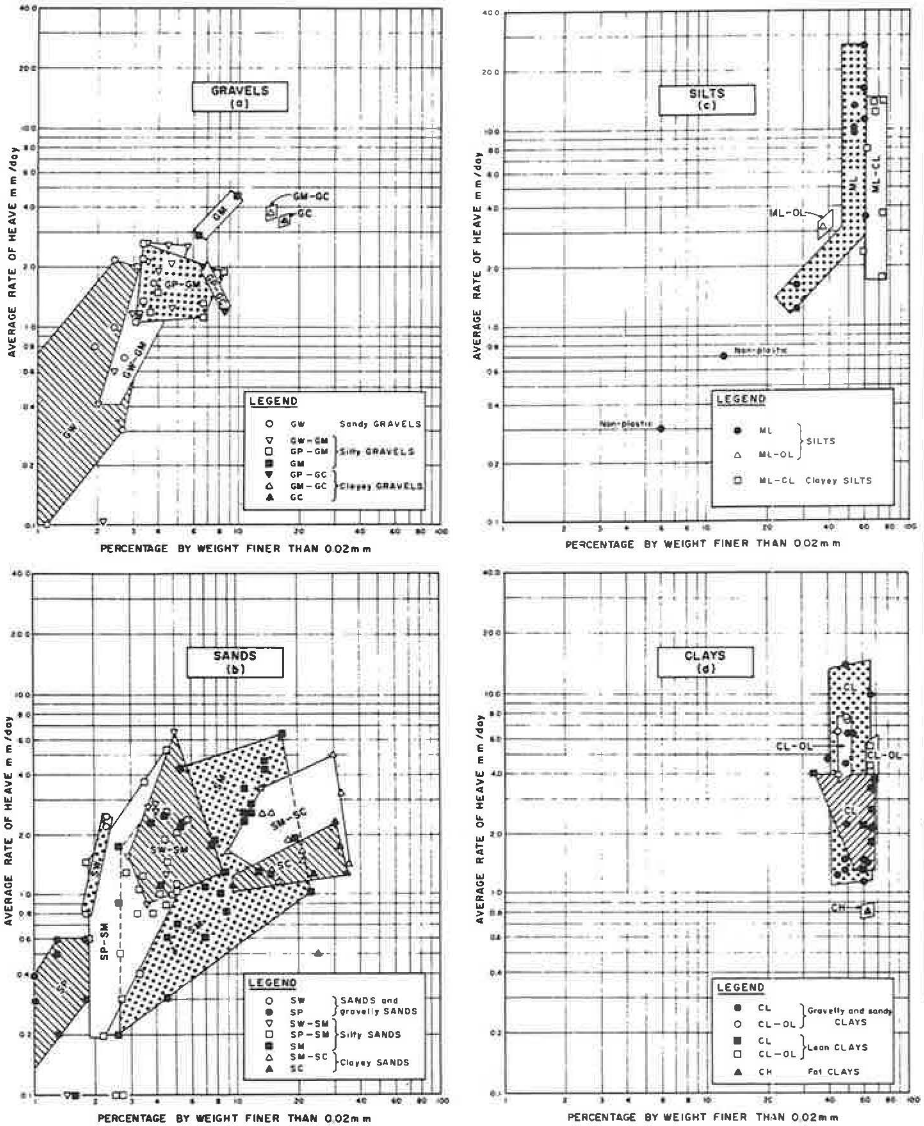
UNIFIED SOIL CLASSIFICATION		MAX. SIZE	GRAIN SIZE[b] mm-% Finer							INITIAL DRY UNIT WEIGHT[c]	INITIAL VOID RATIO	ATTERBERG LIMITS		AVERAGE RATE OF HEAVE	FROST SUSCEPTIBILITY CLASSIFICATION[e]
SOIL TYPE	SYMBOL	in.	4.76	2.00	0.42	.074	.02	.01	.005	pcf	e	LL	PI	mm/day	
						SILTS									
SILT	ML	-	-	100	99	54	6.0	(4.0)	(2.5)	102	0.688	Non-plastic		0.3	Negligible
		-	100	99	91	53	13	(6.0)	(3.5)	112	0.484	"	"	0.7	Very low
		-	-	-	100	95	27	10	(4.0)	106	0.626	26	3.0	1.2	Low
		-	-	-	100	95	27	10	(4.0)	103	0.668	26	3.0	1.5	Low
		-	-	-	100	99	53	25	15	113	0.501	23.7	4.0	9.8	Very high
		-	-	-	100	99	53	25	15	113	0.501	23.7	4.0	10.0	Very high
		3/4	95	94	91	87	54	40	28	104	0.590	32.8	8.1	13.9	Very high
		-	-	100	99	97	60	22	10	105	0.611	26.6	0.1	11.0	Very high
		-	-	100	99	97	60	22	10	106	0.589	26.6	0.1	15.9	Very high
		-	-	100	99	97	60	22	10	108	0.567	26.6	0.1	26.0	Very high
		3/4	97	96	92	83	60	44	28	101	0.611	36.0	5.1	3.5	Medium
Clayey SILT	ML-CL	-	-	-	100	98	60	37	22	123	0.389	25.3	5.8	2.2	Medium
		-	-	-	100	86	61	34	14	105	0.643	24.1	5.9	7.9	High
		-	-	100	96	90	67	36	16	101	0.685	25.0	6.0	12.3	Very high
		-	-	100	96	90	67	36	16	101	0.662	25.0	6.0	14.0	Very high
		-	-	-	100	99	73	37	13	107	0.577	23.7	6.0	1.7	Low
		-	-	-	100	99	73	37	13	106	0.596	23.7	6.0	3.7	Medium
		-	100	99	93	85	73	47	23	101	0.674	26.0	5.0	14.0	Very high
SILT w/organic	ML-OL	-	-	-	100	91	38	12	6	98	0.737	Non-plastic		3.1	Medium
						CLAYS									
Gravelly and Sandy CLAYS	CL	3/4	82	77	70	62	40	31	23	133	0.352	25.6	7.9	4.8	High
		3/4	95	-	87	64	43	-	-	115	0.468	41.0	18.0	1.3	Low
		-	-	-	100	96	49	38	30	109	0.569	30.0	11.7	4.5	High
		1/4	98	97	90	61	49	41	34	110	0.536	43.8	20.3	14.0	Very high
		1/4	98	97	90	61	49	41	34	113	0.504	43.8	20.3	1.3	Low
		1/4	98	97	90	61	49	41	34	117	0.456	43.8	20.3	1.5	Low
		1/4	98	97	90	61	49	41	34	118	0.441	43.8	20.3	2.2	Medium
		3/4	96	96	93	86	51	38	27	118	0.424	26.4	8.4	6.2	High
		3/4	85	84	82	78	53	40	30	119	0.429	27.6	9.5	6.5	High
		3/4	97	95	90	80	60	48	36	125	0.403	28.6	12.6	1.2	Low
		3/4	97	95	90	80	60	48	36	125	0.395	28.6	12.6	1.5	Low
		3/4	97	95	91	81	61	50	35	126	0.389	29.6	13.6	1.4	Low
		1-1/2	94	92	88	80	64	52	37	117	0.448	30.0	12.0	10.0	Very high
		1-1/2	94	92	88	80	64	52	37	118	0.431	30.0	12.0	3.3	Medium

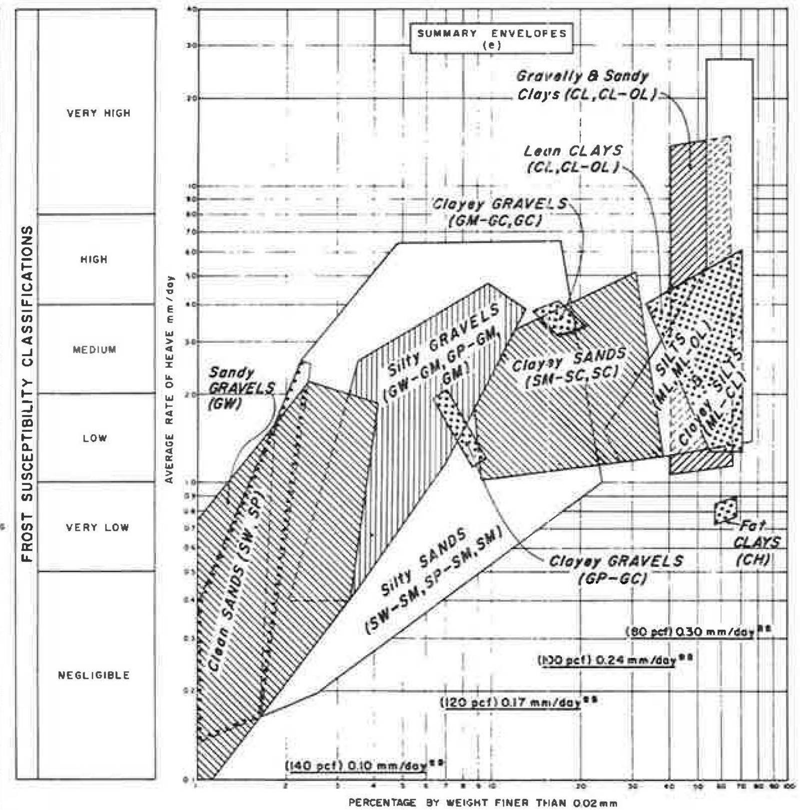
Gravelly and Sandy CLAYS w/organic	CL-OL	3/4	84	80	72	56	44	35	25	130	0.328	23.0	7.0	6.5	High
		3/4	84	80	72	56	44	35	25	130	0.324	23.0	7.0	4.0	High
		3/4	86	81	73	57	50	42	30	129	0.336	21.0	7.0	7.8	High
		3/4	86	81	73	57	50	42	30	130	0.328	21.0	7.0	7.3	High
Lean CLAYS	CL	-	100	99	98	91	33	(24)	(19)	113	0.474	28.0	12.0	4.0	High
		-	-	100	98	91	58	41	31	117	0.485	36.5	16.8	1.4	Low
		-	-	-	100	97	60	43	34	116	0.518	31.3	15.2	2.2	Medium
		-	-	-	-	100	67	37	29	115	0.476	28.0	8.6	2.5	Medium
		-	-	-	-	100	67	37	29	118	0.448	28.0	8.6	3.8	Medium
		-	-	-	-	100	67	37	29	120	0.424	28.0	8.6	1.8	Low
		-	-	-	-	100	67	37	29	123	0.385	28.0	8.6	2.1	Medium
Lean CLAYS w/organic	CL-OL	-	-	100	99	96	65	48	35	98	0.644	37.0	13.0	4.1	High
		-	-	100	99	96	65	48	35	99	0.630	37.0	13.0	5.3	High
		-	-	100	99	96	65	48	35	99	0.627	37.0	13.0	4.2	High
Fat CLAYS	CH	-	-	100	99	74	61	52	42	105	0.715	55.0	37.0	0.8	Very low

NOTES: (a) See Notes on figure 8, Summary of Average Rate of Heave vs. Percentage Finer than 0.02 mm Size for Natural Soil Gradations.
(b) Numbers in parentheses indicate estimated values.
(c) To nearest full pound.
(d) $C_u = \frac{D_{60}}{D_{10}}$

$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

(e) With respect to rate of heave.
(f) Not shown on applicable plot on figure 8





Gravelly Soils	F1	F1	F2	F3
SANDS (Except very fine silty SANDS)		F2		F3
Very fine silty SANDS				F4
All SILTS				← F4
CLAYS (PI > 12)				← F3
CLAYS (PI < 12), varved CLAYS and other fine-grained banded sediments				← F4

NOTES: Standard tests performed by Arctic Construction and Frost Effects Laboratory; specimens 6 in. dia. by 6 in. high, frozen at penetration rate of approximately 0.25 in. per day, with free water at 38°F continuously available at base of specimen. Specimens compacted to 95% or better of applicable standard, except undisturbed clays. Saturations before freezing generally 85% or greater.

* Undisturbed specimen

** Indicated heave rate due to expansion in volume, if all original water in 100% saturated specimen were frozen, with rate of frost penetration 0.25 inch per day.

Figure 8. Summary of average rate of heave vs percentage finer than 0.02 mm size for natural soil gradations.

classed as acceptable, but which approach 1.0 mm per day rate of heave in laboratory tests should be expected to show some measurable frost heave under average field conditions. These facts must be kept in mind when applying the criteria to other than normal pavement practice, and when considering subsurface drainage measures.

The data presented in Table 1 may be used for general guidance to estimate the relative frost susceptibility of similar soils. However, a standard laboratory frost susceptibility test on a sample of the specific soil will give a more accurate evaluation.

Soils are classified into four groups for frost design purposes (Table 2). Soils are listed in approximate order of increasing susceptibility to frost heaving and/or weakening as a result of frost melting. However, the order of listing subgroups under Groups F3 and F4 does not necessarily indicate the order of susceptibility to frost heaving of these subgroups. There is some overlapping of frost susceptibility between groups. The soils in Group F4 are of especially high frost susceptibility.

The F1 group is intended to include frost-susceptible gravelly soils which in the normal unfrozen condition have traffic performance characteristics of GW and GP type materials with the noted percentages of fines. The F2 group is intended to include frost-susceptible soils which in the normal unfrozen condition have traffic characteristics of GM, SW, SP or SM type materials with fines within the stated limits. Occasionally GS or SC materials may occur within the F2 group, although they will normally fall in the F3 category. The basis for division between the F1 and F2 groups is that F1 materials may be expected to show higher bearing capacity than F2 materials during thaw, even though both may have experienced equal ice segregation.

Varved clays consisting of alternate layers of silts and clays are likely to combine the undesirable properties of both silts and clays. These and other stratified fine-grained sediments may present a problem in selection of overall frost classification for design purposes. Because such soils are likely to heave and soften more readily than homogeneous soils with equal average water contents, the classification of the material of highest frost susceptibility should be adopted for design purposes. Usually this will place the overall deposit in the F4 category.

Under special conditions the frost group classification adopted for design may be permitted to differ from that obtained by application of the previous frost group defini-

TABLE 2

FROST DESIGN SOIL CLASSIFICATION

Frost Group	Soil Type	Percentage Finer Than 0.02 mm by Weight	Typical Soil Types Under Unified Soil Classification System
F1	Gravelly	3 to 10	GW, GP, GW-GM, GP-GM
F2	(a) Gravelly	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	3 to 15	SW, SP, SM, SW-SM SP-SM
F3	(a) Gravelly	>20	GM, GC
	(b) Sands, except very fine silty sands	>15	SM, SC
	(c) Clays, PI >12	—	CL, CH
F4	(a) All silts	—	ML, MH
	(b) Very fine silty sands	>15	SM
	(c) Clays, PI < 12	—	CL, CL-ML
	(d) Varved clays and other fine-grained, banded sediments	—	CL and ML; CL and ML and SM; CL, CH and ML; CL, CH, ML and SM

tions, if the difference is not greater than one frost group number and if complete justification for the variation is presented. Such justification may take into account special conditions of subgrade moisture or soil uniformity, in addition to soil gradation and plasticity, and should include data on performance of local pavements. For example, some pavements constructed on varved clay subgrades in which the soil deposit and the depth to ground water table are uniform show comparatively good performance under frost conditions. In such case, adoption of F3 classification in lieu of F4 for design purposes may be justified. However, care must be used in attempting to translate highway experience into airfield applications, and vice versa, and in evaluating experience based on seasons which are warmer and/or drier than normal, or on drainage conditions which will not be applicable to the case in point.

DETRIMENTAL EFFECTS OF FROST ACTION

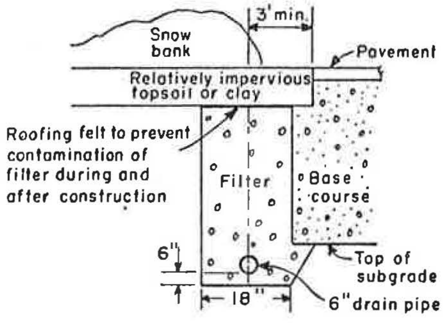
Heaving

Frost heave, indicated by the raising of the pavement, is directly associated with ice segregation and is visible evidence on the surface that ice lenses have formed in the subgrade, base materials, or both. Heave may be uniform or non-uniform depending on variation in the character of the soils and the ground water conditions underlying the pavement.

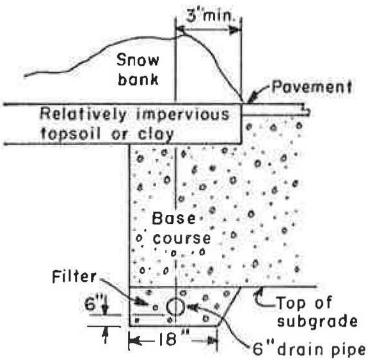
Uniform heave is the raising of adjacent areas of a pavement surface by approximately equal amounts so that the initial shape and smoothness of the surface remains substantially unchanged. Typical conditions conducive to uniform heave may exist in a section of pavement constructed with a fairly uniform stripping or fill depth, uniform ground water depth and horizontally uniform soil characteristics.

When non-uniform heave occurs, there are appreciable differences in the heave of adjacent areas resulting in objectionable unevenness or abrupt changes in grade at the pavement surface. Conditions conducive to irregular heave occur, for example, at locations where subgrades vary between clean non-frost-susceptible sands and silty frost-susceptible materials, at abrupt transitions from cut to fill sections with the ground water close to the surface, or where excavation cuts into water-bearing strata. Drains, culverts or utility ducts placed under pavements on frost-susceptible subgrades frequently result in abrupt differential heaving. Placing such facilities beneath pavements should be avoided wherever possible. Where this cannot be avoided, construction should be in accordance with methods such as indicated in Figure 9d. All drains or similar features should be placed first and the base course materials carried across them without break to obtain maximum uniformity of pavement support. The practice of constructing the base course and then excavating back through it to lay drains, pipes, etc., is unsatisfactory because a marked discontinuity in support will result. It is almost impossible to compact material in a trench to the same degree of compaction as the surrounding base course material. Also, the amount of fines in the excavated and backfilled material may be increased by incorporation of subgrade soil during the trench excavation or by manufacture of fines by the added handling. The poor experience record of combination drains (those intercepting both surface and subsurface water) indicates that the filter material should never be carried to the surface as shown in Figure 9c. Recommended practices are shown in Figures 9a and 9b. Inserted items such as drain inlets in pavements, and fueling hydrants and pavement lighting systems in airfields, are likely to be locations of abrupt differential heave with resultant pavement distress and loss of smoothness. Differences in pavement thickness and/or composition inevitably produce differences in commencement of heave, rate of heave and total heave of the frozen materials.

When interruptions in pavement uniformity cannot be avoided, the best design solutions are use of a sufficiently thick non-frost-susceptible base or use of long transitions. No specific dimensional standards for transition sections have been established. However, transition lengths should vary directly with the speed of traffic and the amount of heave differential. For rigid pavements, transition sections should begin and end directly under pavement joints and should never be shorter than one slab length. For example, at a heavy-load airfield where 1-in. heave differentials may be expected



(a) RIGHT



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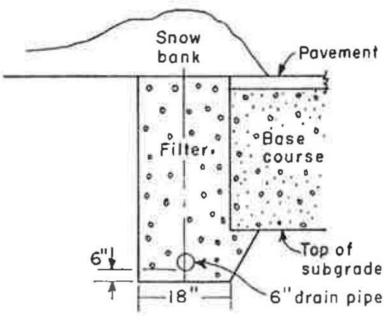
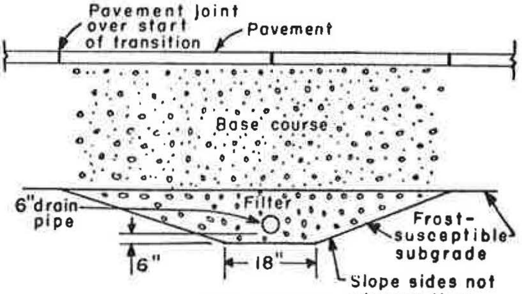


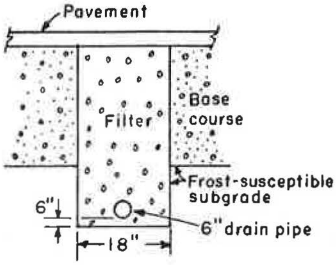
(c) WRONG

Under winter conditions, thaw water accumulating at edge of pavement may feed into base course in detail (c). This detail is poor because filter provides poor surface and is subject to clogging; drain is also located too close to pavement to permit easy repair.

SUBDRAINS ALONG PAVEMENT EDGES



(d) RIGHT



(e) WRONG

Under frost conditions, non-uniformity produced by detail (e) is likely to result in differential heave and pavement cracking.

SUBDRAINS UNDER PAVED SURFACES

NOTE:

For additional details on design of subdrains and filter courses see EM 1110-345-282.

Figure 9. Subdrain details for cold regions.

at changes from one subgrade soil condition to another, gradual changes in base thicknesses should be effected over distances of 200 ft for the runway area, 100 ft for taxiways, and 50 ft for aprons. Pavements designed to lower standards of frost heave control, such as airfield overruns, have less stringent requirements, but nevertheless may need transition sections.

Other possible measures to modify the effects of heave are use of insulation to control depth of frost penetration in limited areas, and use of dowel and slab reinforcement to insure pavement continuity where any doubt remains concerning the design. Reinforcement will not reduce heave or prevent cracking. However, reinforcement will help to hold pavement tightly closed and to assure satisfactory structural performance.

Transitions between cut and fill and changes in character or stratification of subgrade soils should also receive special attention in field control (Appendix A).

Thawing and Reduction in Pavement Supporting Capacity

When ice segregation occurs, reduction of the strength of the soil with a corresponding reduction in load-supporting capacity of the pavement develops during frost-melting periods, particularly early in the spring when thawing is occurring at the top of the subgrade and the rate of melting is rapid. As shown in Figure 10, ice melting from the surface downward releases water which cannot drain through the still frozen soil below or redistribute itself readily. Excess moisture from the wet and softened subgrade soil moves upward into the base course and laterally to the nearest drain. If drainage provisions are inadequate, the base course may become completely saturated. If this occurs, the bearing capacity of the base is substantially reduced, the effects of possible subsequent frost action are increased, water and fines may be pumped through joints and cracks, and accelerated deterioration of the surfacing may occur. Therefore, it is essential that base courses

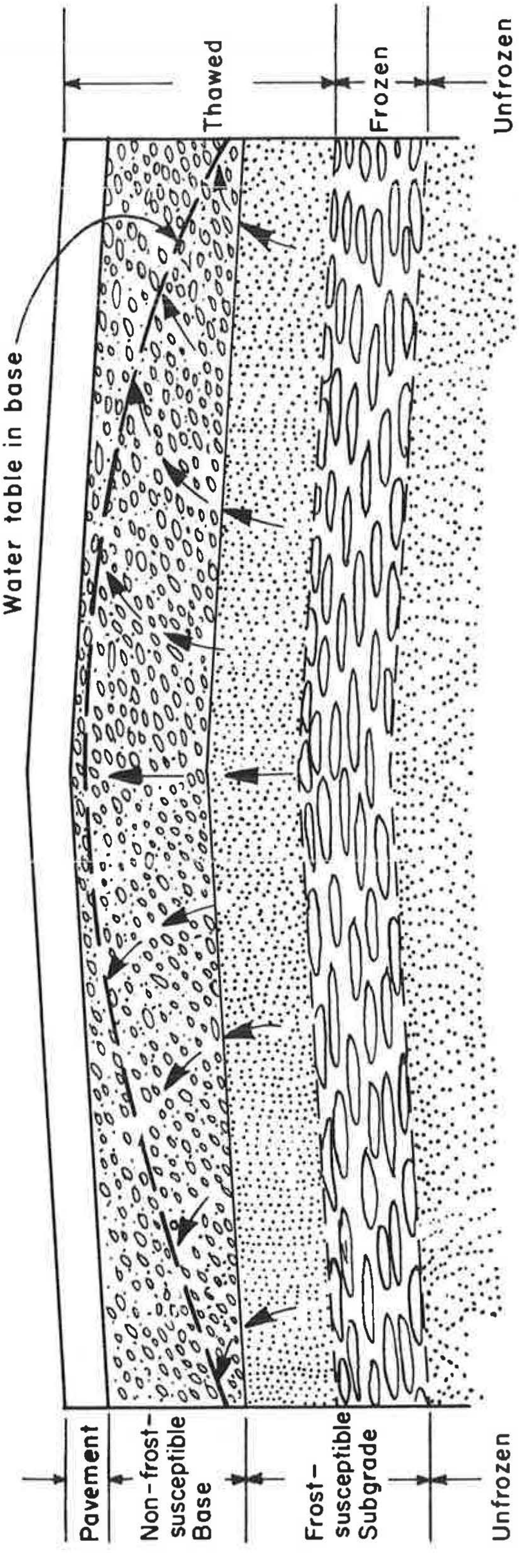


Figure 10. Moisture movement upward into base course during thaw.

in frost regions be designed in strict accordance with the drainage criteria of Ref. (2), pertinent parts of which are abstracted in Appendix C. The possible effects of restriction of subsurface drainage by frozen soils should be considered at all points in drainage design.

Supporting capacity may be reduced in clay subgrades even though significant heave has not occurred, because water for ice segregation is extracted from the voids of the unfrozen clay below and the resulting shrinkage of the latter largely balances the volume of the formed ice lenses. Also, traffic may cause remolding or hydrostatic pressures within the pores of the soil during the period of weakening, thus resulting in further reduced subgrade strength.

The degree to which a soil loses strength during a frost-melting period and the length of the period depend on the type of soil, temperature conditions, amount and type of traffic, moisture supply during fall, winter and spring, and drainage conditions.

Effect of Frost Action and Low Temperatures on Pavement Surface

The most obvious structural effect of frost action on the pavement surface is the formation of random cracking and roughness as the result of differential frost heave. Studies of rigid pavements have shown that cracks may develop more rapidly during and immediately following the spring frost-melting period, as a result of differential thaw, than during the period of active heave itself. For airfield pavements it is especially important that uncontrolled cracking be reduced to an absolute minimum, because deterioration and spalling of the edges of working cracks are a source of debris which may seriously damage jet aircraft. This may be accomplished by control of such elements as base composition and thickness, slab dimensions, horizontal uniformity of base and subgrade materials, uniformity of subsurface moisture conditions, and in special situations, by use of reinforcement and limitation of pavement type. The importance of uniformity cannot be overemphasized, and although true for all pavements, it is particularly important for airfield pavements.

Cracking may also result, particularly in flexible pavements, from shrinkage of the pavement and base under extreme low temperatures. In very cold regions, cracks from this source may penetrate not only the pavement but the underlying materials. As stated, this is essentially a flexible pavement problem because there is no jointing system for control of such stresses. Unfortunately, when the most severe tensile stresses develop, flexible pavements are least ductile. Shrinkage cracking in flexible pavements is not regarded as a structural problem. The only remedial measures considered necessary in seasonal frost areas are periodic sealing of cracks when entrance of surface moisture may be detrimental or when raveling of crack edges may produce surface debris, and resurfacing at required intervals.

INVESTIGATION PROCEDURE

The field and laboratory investigations conducted in accordance with Ref. (3) will usually provide sufficient information to determine whether a given combination of soil and water conditions beneath the pavement will be conducive to frost action. Particular attention should be given to the degree of horizontal variation of subgrade conditions. This involves both soil and moisture conditions and is difficult to express simply and quantitatively. Subgrades may range from uniform conditions of soil and moisture in which variations from point to point are so slight as to result in negligible differential frost heave and thaw settlement, to extremely variable conditions in which frequent and abrupt changes occur between low or negligible and high or very high frost-heave potential. The procedures for determining whether or not the conditions necessary for ice segregation are present at a proposed site follow.

Soil

As stated, the frost susceptibility of soils may be estimated from the percentage of grains finer than 0.02 mm by weight or by laboratory freezing tests. The Corps of Engineers presently requires that such freezing tests in connection with its projects be be carried out by or under the supervision of the U. S. Army Cold Regions Research and Engineering Laboratory, Hanover, N. H.

Temperature

Air freezing index values should, so far as possible, be based on actual air temperatures obtained from a station located in close proximity to the construction site. This is desirable because differences in elevations, topographical position, nearness to cities, bodies of water or other sources of heat may cause considerable variations in air freezing indexes over short distances. These variations are of greater relative importance to design in areas with a design freezing index of less than 1,000 (i.e., mean air freezing index of less than about 500) than they are farther north.

Daily and mean monthly air temperature records for all stations which report to the U. S. Weather Bureau are available at the various Weather Bureau section centers. In general, one of these centers is located in each State. The mean air freezing index may be based on mean monthly air temperatures, but average daily air temperatures are used to compute the design freezing index. Computation of values for determination of the design freezing index may be limited to consideration of only the coldest years in the desired cycles. These years may be selected by inspection of the tabulation of average monthly temperatures for the nearest first order weather station. A "Local Climatological Data" summary containing this tabulation for the period of record is published annually by the Weather Bureau for each of the approximately 150 U. S. first order stations. If the temperature record of the station in closest proximity to the construction site is not of sufficient duration to permit the determination of mean or design index values, the available data are related, for the same period, to that of the nearest station or stations of adequate record. Site index working values may then be computed based on this established relationship and the indexes for the more distant station or stations.

Depth of Frost Penetration

The depth to which freezing temperatures will penetrate the surface of a pavement kept clear of snow and ice depends principally on the magnitude and duration of below freezing air temperatures, properties of the underlying materials, and the amount of water which becomes frozen. The curves in Figures 11 and 12 may be used to estimate values of frost penetration beneath paved areas. They have been computed for an assumed 12-in. thick PCC pavement using the modified Berggren formula (4) and correction factors derived by comparison of theoretical results with field measurements under different conditions. The curves yield maximum depths to which the 32 F temperature will penetrate from the top of the pavement under total winter freezing index values in indefinitely deep homogeneous materials for the indicated density and moisture content properties. Variations due to use of other pavement types and of PCC pavements of lesser thicknesses may be neglected. Where individual analysis is desired or unusual conditions make special computation desirable, the modified Berggren formula may be applied (see Notes, Fig. 11). Neither this formula nor the curves in Figures 11 and 12 are applicable for determining transient penetration depths under partial freezing index values. Values obtained by use of Figures 11 and 12 should be verified whenever possible by observations in the locality under consideration. Methods of estimating frost penetration depths beneath surfaces other than pavements kept free of snow and ice are discussed in Ref. (5).

Water

A potentially troublesome water supply for ice segregation is present if the highest ground water table at any time of the year is within 5 ft of the proposed subgrade surface or the top of any frost-susceptible base materials used. A water table within this depth or less may be considered indicative of relatively adverse ground moisture conditions. When the depth to the uppermost water table is in excess of 10 ft throughout the year, ice segregation and frost heave may be expected to be reduced. Although the reduced frost heave may be tolerable for flexible pavements, it may not be so for rigid

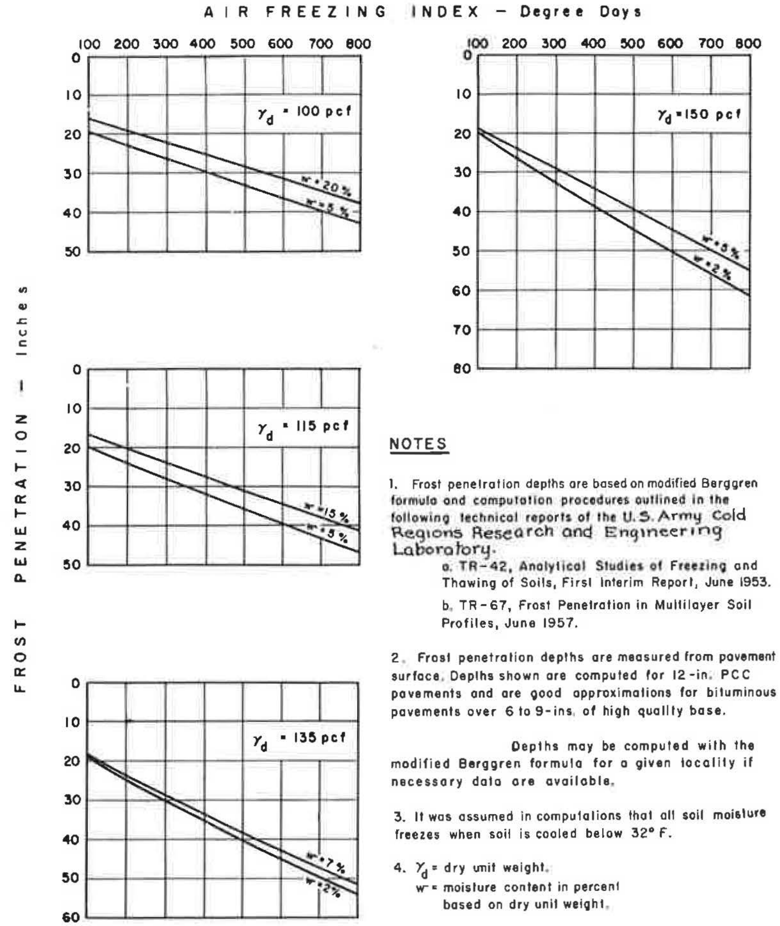


Figure 11. Relationships between air freezing index and frost penetration into granular, non-frost-susceptible soil beneath pavements kept free of snow and ice for freezing indexes below 800.

pavements because of the cracking which may result in the latter, even under reduced heave. In homogeneous clay soils, the water content which the clay subgrade will attain under a pavement is usually sufficient to provide water for some ice segregation, even with a remote water table. Closed system laboratory tests on silt, clays and tills, corresponding to a field condition of a very deep water table, indicate that detrimental ice segregation is unlikely if the moisture content of these soils is below 70 percent of the saturation value. Full advantage can rarely be taken of this, however, because moisture contents near full saturation may occur in the top of the frost-susceptible subgrade from surface infiltration through pavement and shoulder areas or from other sources.

In addition to the conditions stated, it is necessary to consider all reliable information concerning past frost heaving and performance during frost-melting periods of airfield and highway pavements constructed in the area being investigated, with a view toward modifying the frost-design requirements.

BASE COURSE COMPOSITION REQUIREMENTS

All base course materials lying within the determined design depth of frost penetration must be non-frost-susceptible. The dimensions and permeability of the base course

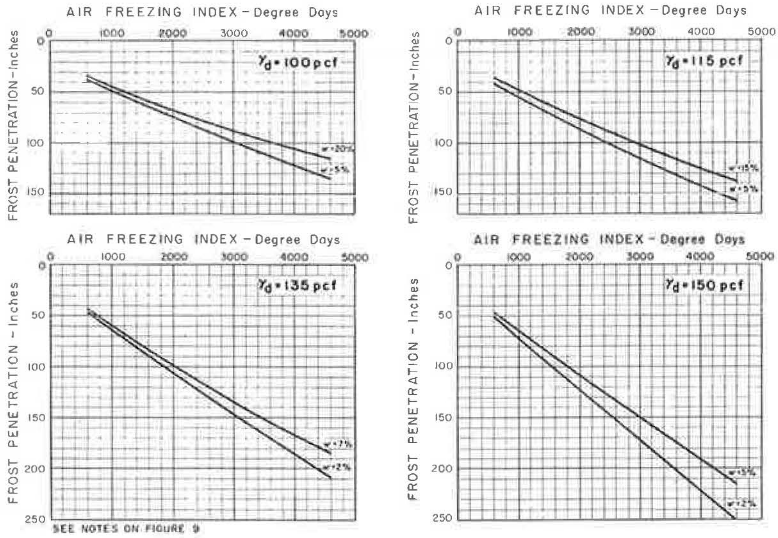


Figure 12. Relationships between air freezing index and frost penetration into granular, non-frost-susceptible soil beneath pavements kept free of snow and ice.

should satisfy the base course drainage criteria given in Appendix C, as well as the thickness requirements for frost design. Thicknesses indicated by frost criteria should be increased if necessary to meet subsurface drainage criteria. Base course materials of borderline frost-susceptible quality should be tested frequently after compaction to insure that the materials meet these design criteria. Where the combined thickness of pavement and base over a frost-susceptible subgrade is less than that required under the limited subgrade frost penetration design method, the following additional design requirements apply:

Filter Over Subgrade

For both flexible and rigid pavements, at least the bottom 4 in. of base should consist of non-frost-susceptible sand, gravelly sand, screenings, or similar material. It should be designed as a filter between the subgrade soil and overlying base course

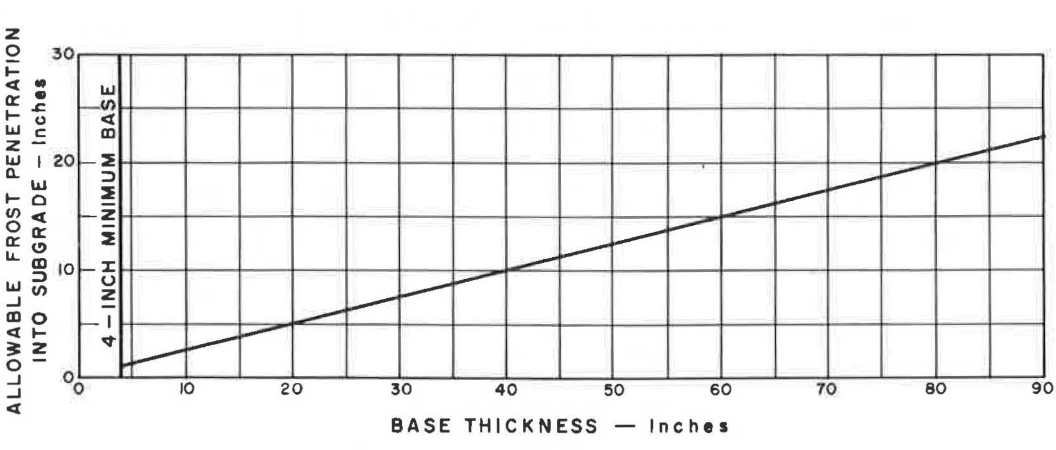


Figure 13. Allowable subgrade frost penetration in design freezing index year for limited subgrade frost penetration design method.

material to prevent mixing of the frost-susceptible subgrade with the base during and immediately following the frost-melting period. This filter is not intended to serve as a drainage course. The gradation of this filter material is determined in accordance with criteria presented in Appendix C, 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. Experience shows that a fine-grained subgrade soil will work up into an improperly graded overlying gravel or crushed stone base course under the kneading action of traffic during the frost-melting period, if a filter course is not provided between the subgrade and the overlying material. Experience and tests indicate that non-frost-susceptible sand is especially suitable for this filter course. The 4-in. minimum filter thickness is dictated primarily by construction requirements and limitations. Greater thicknesses are specified when required to suit field conditions. Over weak subgrades, a 6-in. or greater thickness may be necessary to support construction equipment and provide a working platform for placement and compaction of the base course.

Filter Under Pavement Slab

For rigid pavements, the 85 percent size (the size particle for which 85% of the material by weight is finer) of filter or regular base course material placed directly beneath pavements is required to be equal to or greater than 2.00 mm in diameter (No. 10 U. S. Standard Sieve Size) for a minimum thickness of 4 in. The purpose of this requirement is to prevent loss of support by pumping soil through the joints.

DESIGN OF PAVEMENTS FOR FROST ACTION

The design of pavements in frost areas may be based on either of two basic concepts: (a) control of surface deformation resulting from frost action, or (b) provision of adequate bearing capacity during the most critical climatic period. Under the first concept, sufficient combined thickness of pavement and non-frost-susceptible base must be provided to eliminate or limit to an acceptable amount, subgrade frost penetration and effects thereof. Under the second concept, the amount of heave which will result is neglected and design is based solely on the anticipated reduced strength of the subgrade during the frost-melting period. The following three design methods have been derived from these concepts and are described in detail: complete protection method; limited subgrade frost penetration method; and reduced subgrade strength method.

The reduced subgrade strength method is the most commonly used design procedure for roads, with added thickness of non-frost-susceptible pavement and base used as needed to control heave or insure adequate subsurface drainage. The two procedures are also helpful in road design by establishing limits for frost protection effectiveness. The limited subgrade frost penetration method may sometimes be directly employable in highways.

The first step in determination of design thickness is to select the appropriate design method or methods from Table 3, which summarizes the conditions for which each of the above methods is applicable. The degree of horizontal variability of subgrade soil and moisture conditions may be classified into one of four categories: uniform; slightly variable; variable; or extremely variable. Definitions of these adjective categories are given under the respective adjective headings in Table 3. The distinctions are purely qualitative. Selection of the adjective category involves judgment; it must be based on careful analysis of past performance of pavements in the area and thorough study of site exploration data. An airfield may fall entirely into one adjective category, or it may have to be divided into a number of areas for separate design consideration. Once an adjective category has been chosen, the design approaches which are applicable may be determined from Table 3.

It should be noted that the requirement for sufficient bearing capacity during the normal period (summer and fall) as determined by non-frost design, takes precedence over the frost-design criteria if the former requires greater combined thickness than that obtained by the frost-design methods.

TABLE 3

SUMMARY OF METHODS FOR DESIGN OF AIRFIELD PAVEMENTS FOR FROST CONDITIONS

Design Method	Horizontal Variability of Subgrade Soil and Moisture Conditions			
	Uniform Variations affecting heave potential virtually undetectable by ordinary methods of investigation. Negligible differential frost heave and thaw settlement may be anticipated under reduced subgrade strength design.	Slightly Variable Small variations of subgrade conditions apparent by ordinary methods of investigation.	Variable Subgrade conditions moderately variable. Widespread cracking of rigid pavements and appreciable surface deformation would be expected if reduced subgrade strength design method were used.	Extremely Variable Very large, frequent and abrupt changes in subgrade frost heave potential not permitting use of transition sections.
Complete protection				Applicable only under exceptionally adverse conditions for F3 and F4 subgrades.
Limited subgrade frost penetration[a, b, c]	Required for flexible and rigid pavements: (1) Over F4 subgrade soils (except as noted in Col. (4) below). (2) Over other frost-susceptible subgrade soils when: (a) Cracking of rigid pavements or unacceptable pavement roughness caused by non-uniform frost heave may be expected with lesser design thickness, or (b) Limited subgrade frost penetration design requires less combined thickness or is otherwise more economical than reduced subgrade strength design.			
Reduced subgrade strength[a, b, c]	Applicable for flexible and rigid pavements over F1 thru F3 subgrades when objectionable differential heave or cracking will not occur.[a]	Applicable for flexible pavements over F1 thru F3 subgrades when objectionable differential heave or cracking will not occur.	Applicable for flexible pavements over F1 thru F4 subgrades when pavements are minor, slow speed, and non-critical and heave can be tolerated, except not to be used for F4 subgrade under adverse moisture conditions.	

[a] Transition sections required at any substantial and abrupt changes in subgrade frost heave potential which would produce unacceptable pavement roughness and cracking.

[b] When indicated combined thickness exceeds 72 inches, consider alternatives: (1) limiting total thickness to 72 inches, and, in rigid-type pavements, using steel reinforcement, (2) reduced slab dimensions or (3) base of higher moisture retention. OCE approval required for use of alternatives or thickness over 72 inches.

[c] Thickness intermediate between reduced subgrade strength and limited subgrade frost penetration design values may be adopted when justification based on field experience or special conditions of the design is provided.

[d] Special provision for rigid pavements over uniform subgrades: Instead of base equal to slab thickness, 4-in. minimum base is allowed over F1, F2, F3 subgrades when: (1) Design freezing index 1,000 or, (2) Subgrade is susceptible to pumping and water table is below 10 feet; however, base drainage criteria must be met.

NOTE: Design of highway pavements should be based generally on the Reduced Subgrade Strength Design Method, with additional thickness (based on local field data and experience) used where necessary to keep pavement heave and cracking within tolerable amounts. Where such added thicknesses are used for highways they should not exceed values obtained by the Limited Subgrade Frost Penetration Design Method. Thickness reduction up to 10% may also be allowed on substantial highways fills when justified by field data and experience.

Complete Protection

Under this method of design, surface deformation resulting from frost action is eliminated by providing sufficient thickness of non-frost-susceptible base to completely protect underlying frost-susceptible soils from freezing. This method is used only in exceptional situations, when the subgrade soil is F3 or F4, soil and moisture conditions are horizontally extremely variable, and the limited subgrade frost penetration method will not provide adequate control of heave and cracking.

The combined thickness of pavement and non-frost-susceptible base required for complete protection is the value a (Fig. 14).

Limited Subgrade Frost Penetration

This is the normal method of design for control of surface deformation. It attempts to hold deformations to small, acceptable values, instead of eliminating them completely. It is applicable primarily for slightly variable and variable subgrade conditions which would produce unacceptable cracking of rigid pavements and pavement roughness if the reduced subgrade design method were used. However, it may sometimes be applicable for more uniform subgrade conditions. The combined thickness of rigid or flexible pavement and non-frost-susceptible base course determined by this method should always be used in the following cases:

(1) Over group F4 subgrade soils.

(2) Over other frost-susceptible subgrade soils.

(a) When cracking of rigid pavements or unacceptable pavement roughness caused by non-uniform frost heave may be expected with lesser design thickness.

(b) When limited subgrade frost penetration design requires less combined thickness or is otherwise more economical than reduced subgrade strength design.

Exceptions are those cases where the subgrade conditions are so extremely variable that the complete protection method must be used, or when flexible paved areas in which the effects of appreciable non-uniform heave and cracking are not considered detrimental. At some sites it may be possible to correct the causes of non-uniform heave by the removal of isolated pockets of frost-susceptible soils for the full depth of frost penetration, or by providing gradual transitions at abrupt changes in subgrade conditions. In these cases a lesser combined thickness of pavement and base than required for limited subgrade frost penetration may be used, and design should then be based on reduced subgrade strength. Exception from the full thickness requirements of the limited subgrade frost penetration design method is not permitted where subgrade soils are group F4 under adverse moisture conditions.

The design freezing index should be used in determining the combined thickness of pavement and base required to limit subgrade frost penetration. As with any natural climatic phenomenon, winters which are colder than average occur with a frequency which decreases as the degree of departure from average becomes greater. A mean freezing index cannot be computed where temperatures in some of the winters do not fall below freezing. A design method has been adopted, therefore, which utilizes the average air freezing index for the three coldest years in a 30-year period (or for the coldest winter in 10 years of record) as the design freezing index to determine the thickness of protection that will be provided.

Except in special situations, it is not necessary to construct airfield pavements entirely to prevent frost penetration into the subgrade. Therefore, the following design method permits a small amount of frost penetration into frost-susceptible subgrades for the design freezing index year.

(1) Estimate average moisture contents in base course and subgrade at start of freezing period and dry unit weight of base.

(2) From Figures 11 or 12, as applicable, determine frost penetration a, which will occur in a base material of unlimited depth beneath a 12-in. thick PCC pavement or average bituminous pavement kept free of snow and ice in the design freezing index year. Use straight-line interpolation where necessary. For PCC pavements greater than 12

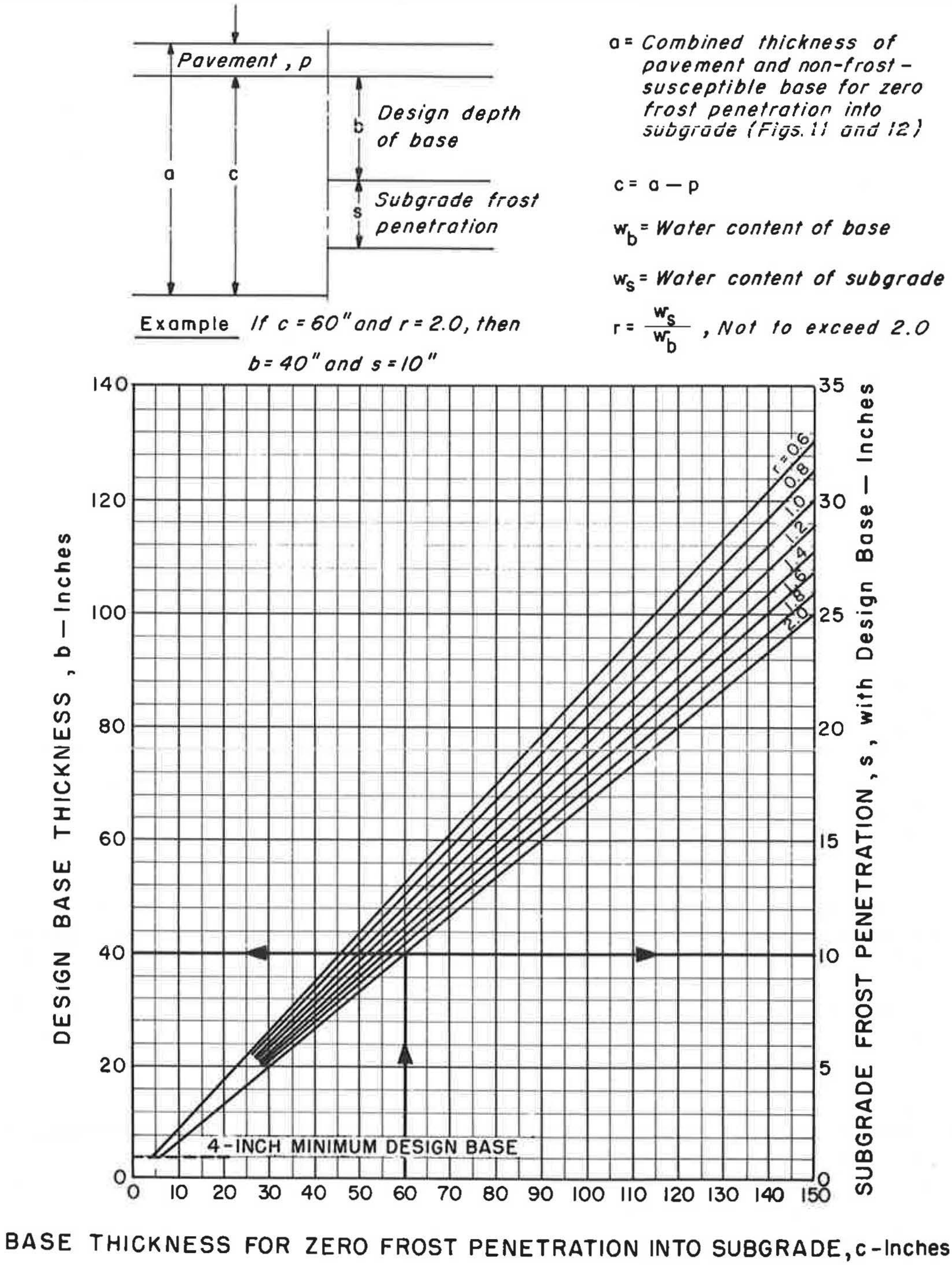


Figure 14. Design depth of non-frost-susceptible base for limited subgrade frost penetration.

in. in thickness, deduct 10 degree-days for each inch of pavement exceeding 12 in. from the design freezing index before entering Figures 11 or 12 to determine frost penetration a. The extra concrete pavement thickness is then added to the determined frost penetration.

(3) Compute base thickness c (Fig. 14) required for zero frost penetration into the subgrade (complete protection) as follows:

$c = a - p$, where p = thickness of portland cement concrete or bituminous concrete.

(4) Compute ratio $r = \frac{\text{water content of subgrade}}{\text{water content of base}}$

(5) Enter Figure 14 with c as abscissa and at applicable value of r, find on left scale design base thickness b which will result in allowable value of subgrade frost penetration s shown on right scale. If r (computed in (4) above) is equal to or exceeds 2.0, use 2.0 in Figure 14.

(6) Values of b and s should show reasonable agreement with plot in Figure 13, which illustrates the basic subgrade frost penetration assumption on which this design procedure is based.

This procedure will result in sufficient thickness of material between the frost-susceptible subgrade and the pavement, so that for average field conditions, subgrade frost penetration of the amount s should not cause excessive differential heave and cracking of the pavement surface during the design freezing index year. The reason for limiting r to a maximum of 2.0 is because not all of the moisture in fine-grained soils will actually freeze at freezing temperatures.

The bottom 4 in. of the design base of thickness b must be designed as a filter, unless the selected base course material already fulfills the filter criteria.

When the maximum combined thickness of pavement and base required by this design procedure exceeds 72 in., special study should be made of alternatives such as the following:

(1) limiting total combined thickness to 72 inches and using steel reinforcement to prevent large cracks in rigid pavements; (2) limiting the maximum slab dimensions (as to 15 ft) without use of reinforcement; (3) reduction of the required combined thickness by use of a base of non-frost-susceptible uniform fine sand with high moisture retention in the drained condition in lieu of more free-draining material.

The first two alternatives would entail a greater surface roughness than obtained under the basic design method because of greater subgrade frost penetration. With respect to the third alternative, it should be noted that base course drainage requirements (Appendix C) must still be met.

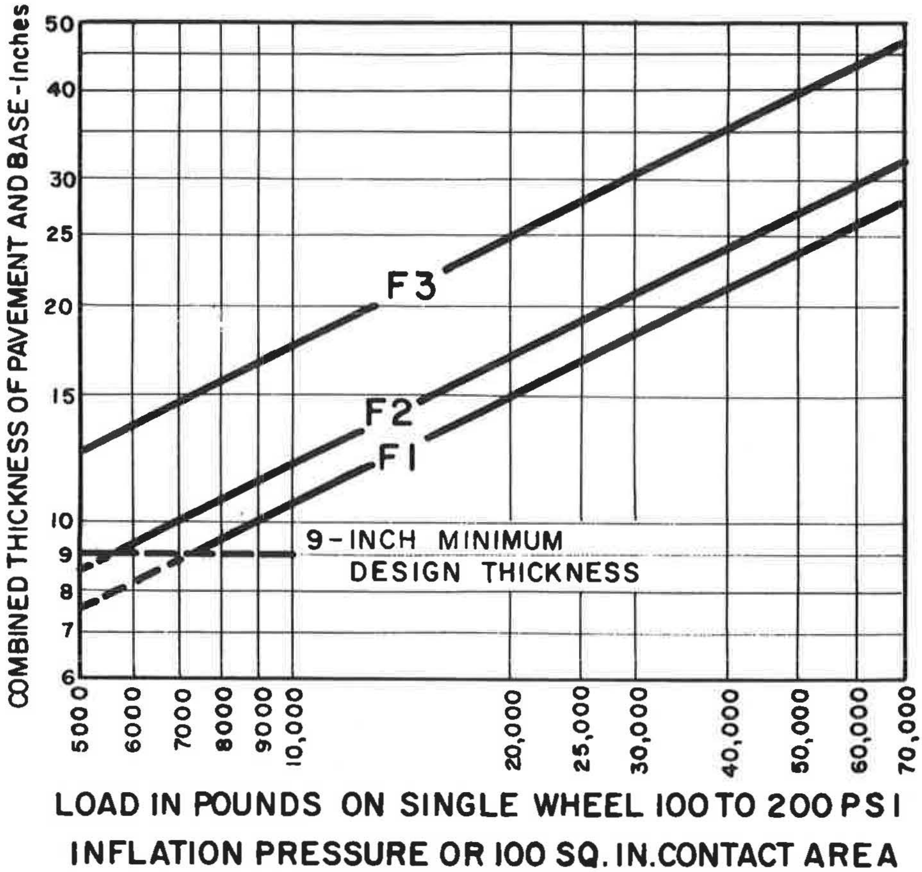
Less total thickness of pavement and base than indicated by the basic design method may also be used if definite justification, based on local experience or special conditions of the design, is provided.

Reduced Subgrade Strength

Thickness design may also be based on the reduction in subgrade strength which occurs during thawing of soils affected by frost action. This design method usually permits less thickness of pavement and base than that needed for limited subgrade frost penetration. The method may be used for both flexible and rigid pavements on F1, F2, and F3 soils when the subgrade is horizontally uniform (or slightly variable for flexible pavements) and significant or objectionable differential heaving and resultant cracking will not occur. The method may also be used over F1 through F4 horizontally variable subgrades for flexible-type pavements of a minor, slow-speed and non-critical character in which heave and its effects can be tolerated. When the reduced subgrade strength method is used for F4 subgrade soils, the combined pavement and base thicknesses should be determined by using the design curves for F3 soils in Figures 15 through 21. When a thickness determined by the reduced subgrade strength method exceeds that

GROUP	DESCRIPTION
F 1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 20 PER CENT FINER THAN 0.02 mm. BY WEIGHT.
F 2	SANDS CONTAINING BETWEEN 3 AND 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT.
F 3	(a) GRAVELLY SOILS CONTAINING MORE THAN 20 PER CENT FINER THAN 0.02 mm. BY WEIGHT. (b) SANDS, EXCEPT VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT. (c) CLAYS WITH PLASTICITY INDEXES OF MORE THAN 12. (d) VARVED CLAYS EXISTING WITH UNIFORM SUBGRADE CONDITIONS.
F 4	(a) ALL SILTS INCLUDING SANDY SILTS. (b) VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PER CENT FINER THAN 0.02 mm. BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF LESS THAN 12. (d) VARVED CLAYS EXISTING WITH NON-UNIFORM SUBGRADE CONDITIONS

NOTE FOR DESIGN OVER F4 SUBGRADE SOILS SEE TEXT

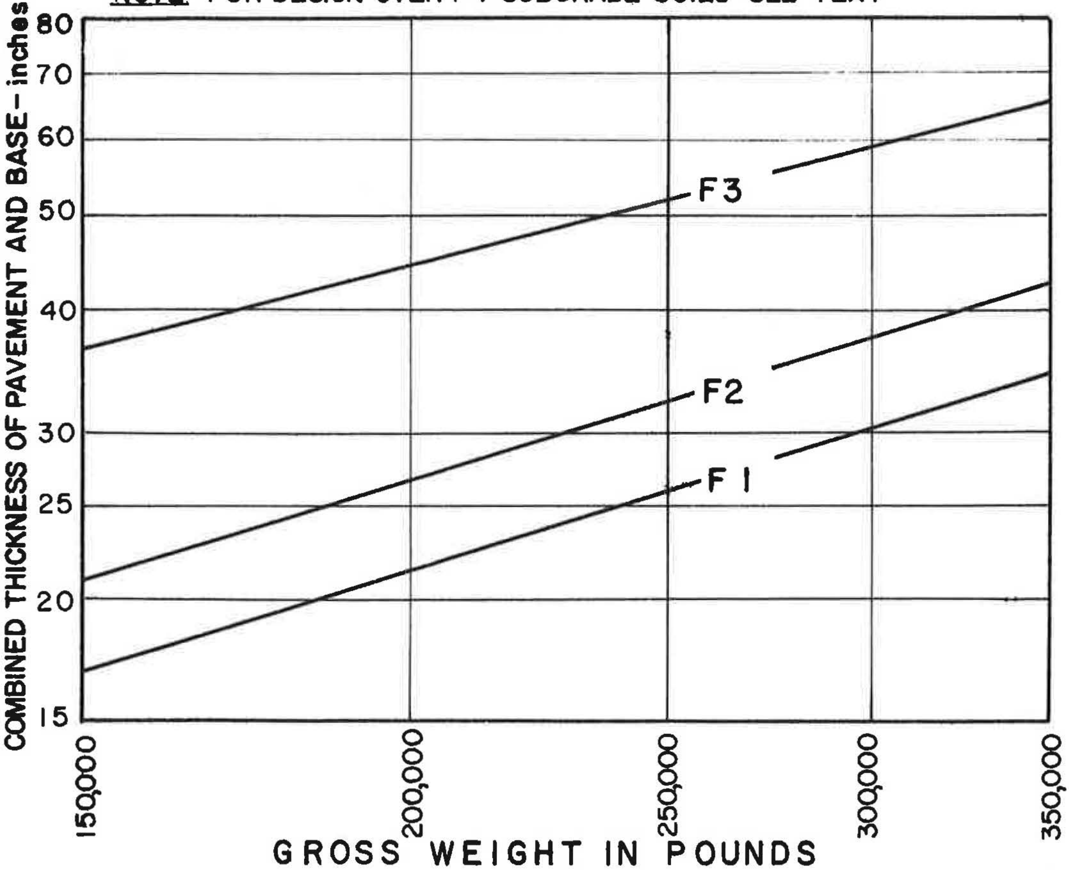


THE THICKNESS WILL BE REDUCED 10 PER CENT FOR RUNWAY INTERIOR (AREA BETWEEN 1000 FOOT SECTION AT EACH END)

Figure 15. Frost condition reduced subgrade strength design curves for flexible pavements.

GROUP	DESCRIPTION
F1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 10 PERCENT FINER THAN 0.02mm BY WEIGHT
F2	(a) GRAVELLY SOILS CONTAINING BETWEEN 10 AND 20 PERCENT FINER THAN 0.02mm BY WEIGHT (b) SANDS CONTAINING BETWEEN 3 AND 15 PERCENT FINER THAN 0.02mm BY WEIGHT
F3	(a) GRAVELLY SOILS CONTAINING MORE THAN 20 PERCENT FINER THAN 0.02mm BY WEIGHT (b) SANDS, EXCEPT VERY FINE SILTY SANDS, CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF MORE THAN 12
F4	(a) ALL SILTS (b) VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02mm BY WEGHT (c) CLAYS WITH PLASTICITY INDEXES OF LESS THAN 12 (d) VARVED CLAYS AND OTHER FINE-GRAINED BANDED SEDIMENTS.

NOTE: FOR DESIGN OVER F4 SUBGRADE SOILS SEE TEXT



THE THICKNESS WILL BE REDUCED 10 PERCENT FOR RUNWAY INTERIOR (AREA BETWEEN 1000 FOOT SECTION AT EACH END)

BOEING 707

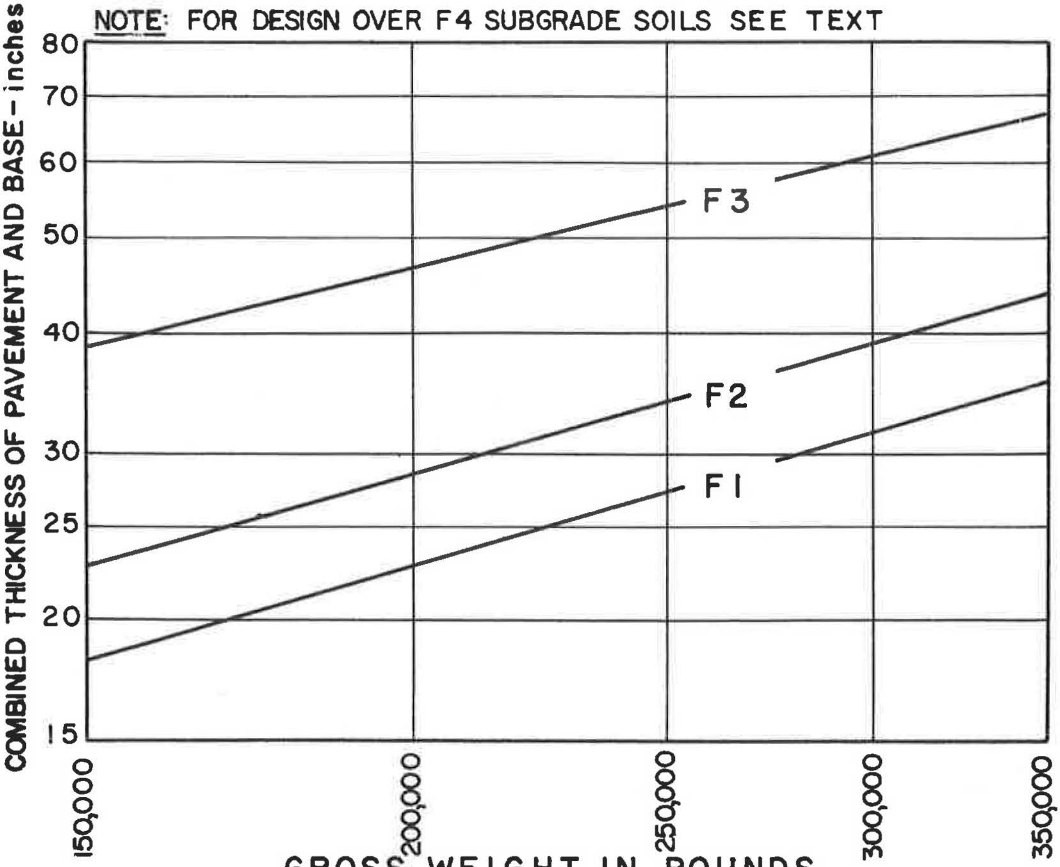
TWIN TANDEM ASSEMBLY-TRICYCLE GEAR

SPACING 34 in., CONTACT AREA 236 sq. in. EACH WHEEL

Figure 16. Frost condition reduced subgrade strength design curves for flexible pavements.

GROUP	DESCRIPTION
F 1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 10 PERCENT FINER THAN 0.02mm BY WEIGHT
F 2	(a) GRAVELLY SOILS CONTAINING BETWEEN 10 AND 20 PERCENT FINER THAN 0.02 mm BY WEIGHT (b) SANDS CONTAINING BETWEEN 3 AND 15 PERCENT FINER THAN 0.02 mm BY WEIGHT
F 3	(a) GRAVELLY SOILS CONTAINING MORE THAN 20 PERCENT FINER THAN 0.02 mm BY WEIGHT (b) SANDS, EXCEPT VERY FINE SILTY SANDS, CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF MORE THAN 12
F 4	(a) ALL SILTS (b) VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF LESS THAN 12 (d) VARVED CLAYS AND OTHER FINE-GRAINED BANDED SEDIMENTS.

NOTE: FOR DESIGN OVER F4 SUBGRADE SOILS SEE TEXT



THE THICKNESS WILL BE REDUCED 10 PERCENT FOR RUNWAY INTERIOR (AREA BETWEEN 1000 FOOT SECTION AT EACH END)

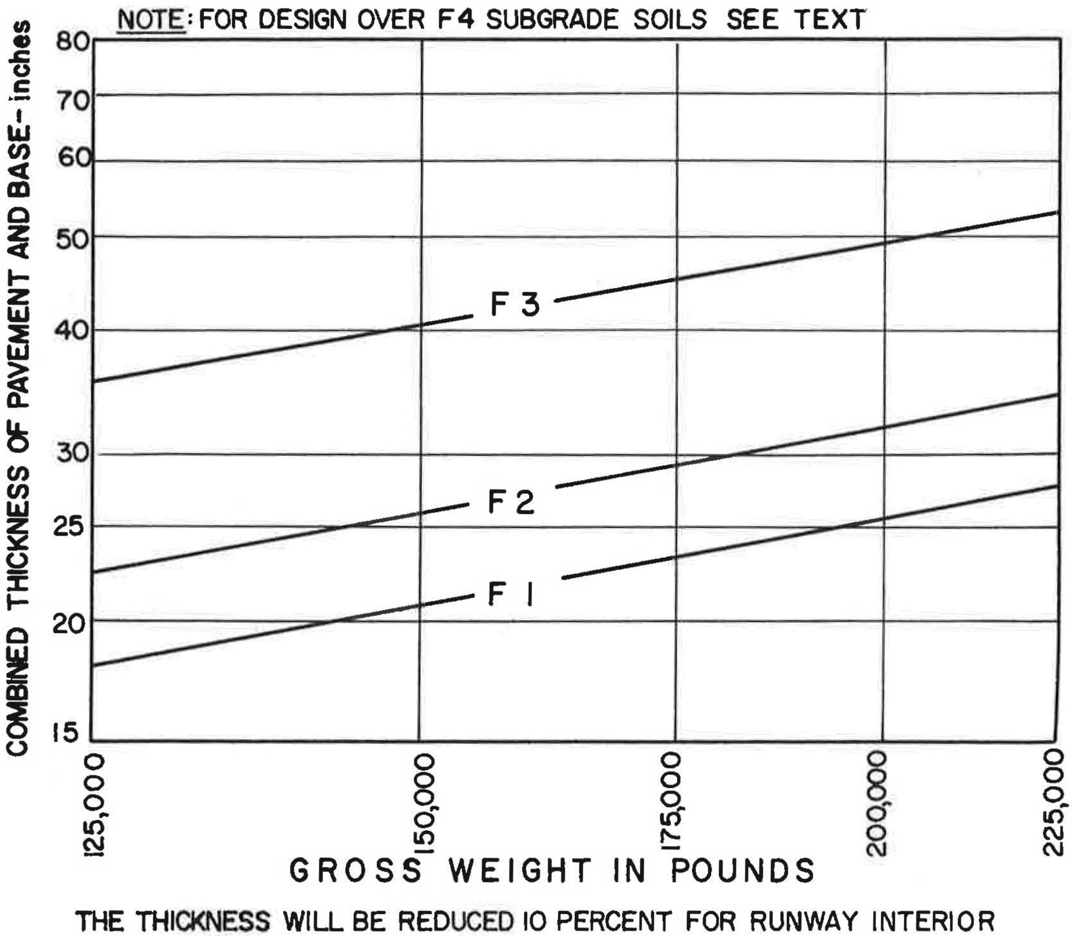
DOUGLAS DC-8

TWIN TANDEM ASSEMBLY-TRICYCLE GEAR

SPACING 30 in., CONTACT AREA 228 sq. in. EACH WHEEL

Figure 17. Frost condition reduced subgrade strength design curves for flexible pavements.

GROUP	DESCRIPTION
F 1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 10 PERCENT FINER THAN 0.02 mm BY WEIGHT
F 2	(a) GRAVELLY SOILS CONTAINING BETWEEN 10 AND 20 PERCENT FINER THAN 0.02 mm BY WEIGHT (b) SANDS CONTAINING BETWEEN 3 AND 15 PERCENT FINER THAN 0.02 mm BY WEIGHT
F 3	(a) GRAVELLY SOILS CONTAINING MORE THAN 20 PERCENT FINER THAN 0.02 mm BY WEIGHT (b) SANDS, EXCEPT VERY FINE SILTY SANDS, CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02 mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF MORE THAN 12
F 4	(a) ALL SILTS (b) VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02 mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF LESS THAN 12 (d) VARVED CLAYS AND OTHER FINE-GRAINED BANDED SEDIMENTS.



THE THICKNESS WILL BE REDUCED 10 PERCENT FOR RUNWAY INTERIOR (AREA BETWEEN 1000 FOOT SECTION AT EACH END)

CONVAIR 880

TWIN TANDEM ASSEMBLY-TRICYCLE GEAR

SPACING 22.5 in., CONTACT AREA 152 sq. in. EACH WHEEL

Figure 18. Frost condition reduced subgrade strength design curves for flexible pavements.

GROUP	DESCRIPTION
F1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 10 PERCENT FINER THAN 0.02 mm BY WEIGHT
F2	(a) GRAVELLY SOILS CONTAINING BETWEEN 10 AND 20 PERCENT FINER THAN 0.02 mm BY WEIGHT (b) 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 0.02mm BY WEIGHT (b) SANDS, EXCEPT VERY FINE SILTY SANDS, CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF MORE THAN 12
F4	(a) ALL SILTS (b) VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF LESS THAN 12 (d) VARVED CLAYS AND OTHER FINE-GRAINED BANDED SEDIMENTS.

NOTE FOR DESIGN OVER F4 SUBGRADE SOILS SEE TEXT.

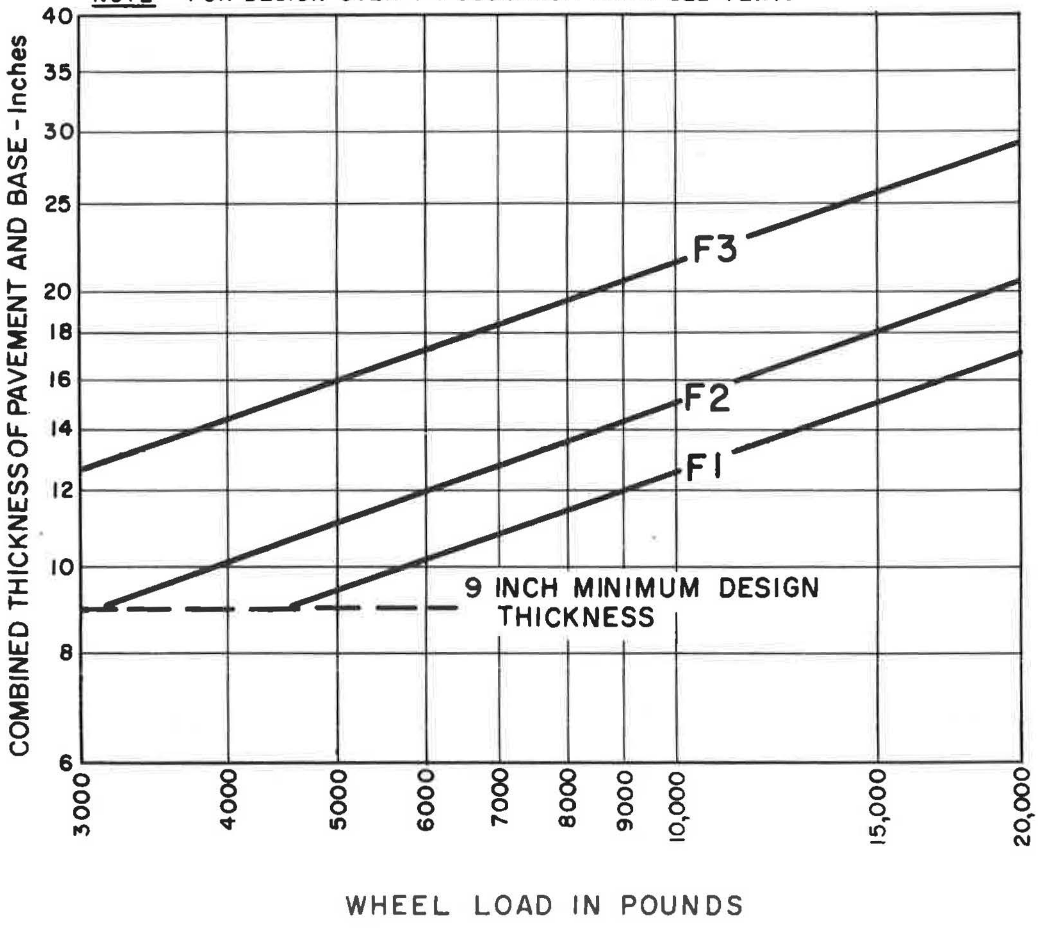


Figure 19. Frost condition reduced subgrade strength design curves for flexible highway pavements.

GROUP	DESCRIPTION
F 1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 10 PERCENT FINER THAN 0.02 mm BY WEIGHT.
F 2	(a) GRAVELLY SOILS CONTAINING BETWEEN 10 AND 20 PERCENT FINER THAN 0.02 mm BY WEIGHT. (b) SANDS CONTAINING BETWEEN 3 AND 15 PERCENT FINER THAN 0.02 mm BY WEIGHT.
F 3	(a) GRAVELLY SOILS CONTAINING MORE THAN 20 PERCENT FINER THAN 0.02 mm BY WEIGHT (b) SANDS, EXCEPT VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02 mm BY WEIGHT. (c) CLAYS WITH PLASTIC INDEXES OF MORE THAN 12.
F 4	(a) ALL SILTS (b) VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02 mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF LESS THAN 12 (d) VARVED CLAYS AND OTHER FINE-GRAINED BANDED SEDIMENTS.

NOTE: FOR DESIGN OVER F4 SUBGRADE SOILS SEE TEXT.

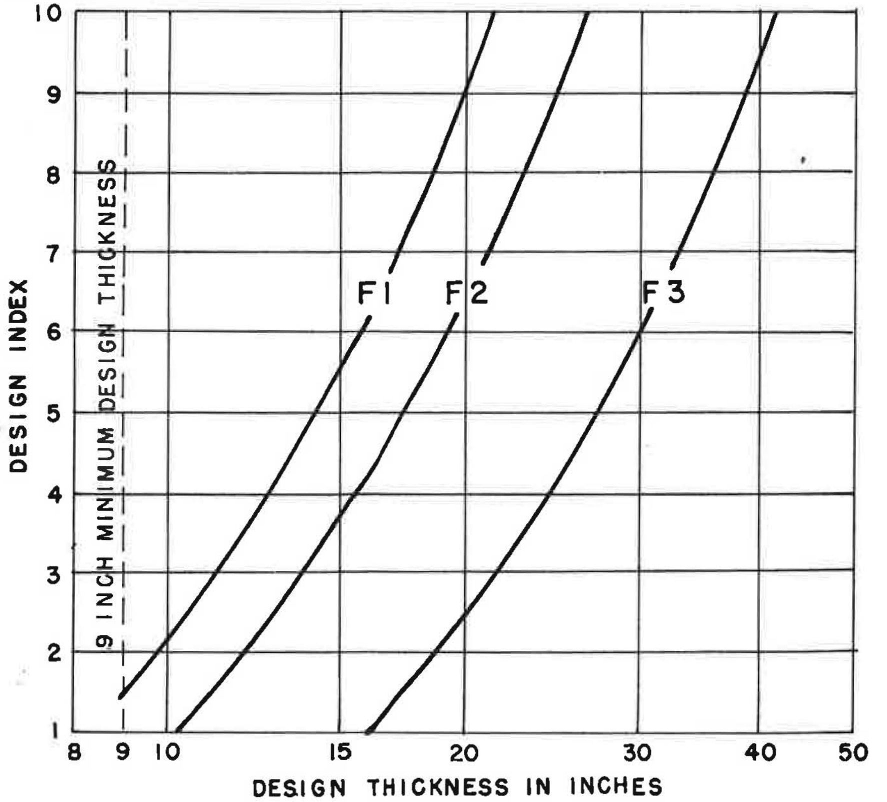


Figure 20. Frost condition reduced subgrade strength design curves for flexible highway pavements.

GROUP	DESCRIPTION
F1	GRAVELLY SOILS CONTAINING BETWEEN 3 AND 10 PERCENT FINER THAN 0.02 mm BY WEIGHT
F2	(a) GRAVELLY SOILS CONTAINING BETWEEN 10 AND 20 PERCENT FINER THAN 0.02 mm BY WEIGHT (b) 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 (b) SANDS, EXCEPT VERY FINE SILTY SANDS, CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02 mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF MORE THAN 12
F4	(a) ALL SILTS (b) VERY FINE SILTY SANDS CONTAINING MORE THAN 15 PERCENT FINER THAN 0.02 mm BY WEIGHT (c) CLAYS WITH PLASTICITY INDEXES OF LESS THAN 12 (d) VARVED CLAYS AND OTHER FINE-GRAINED BANDED SEDIMENTS.

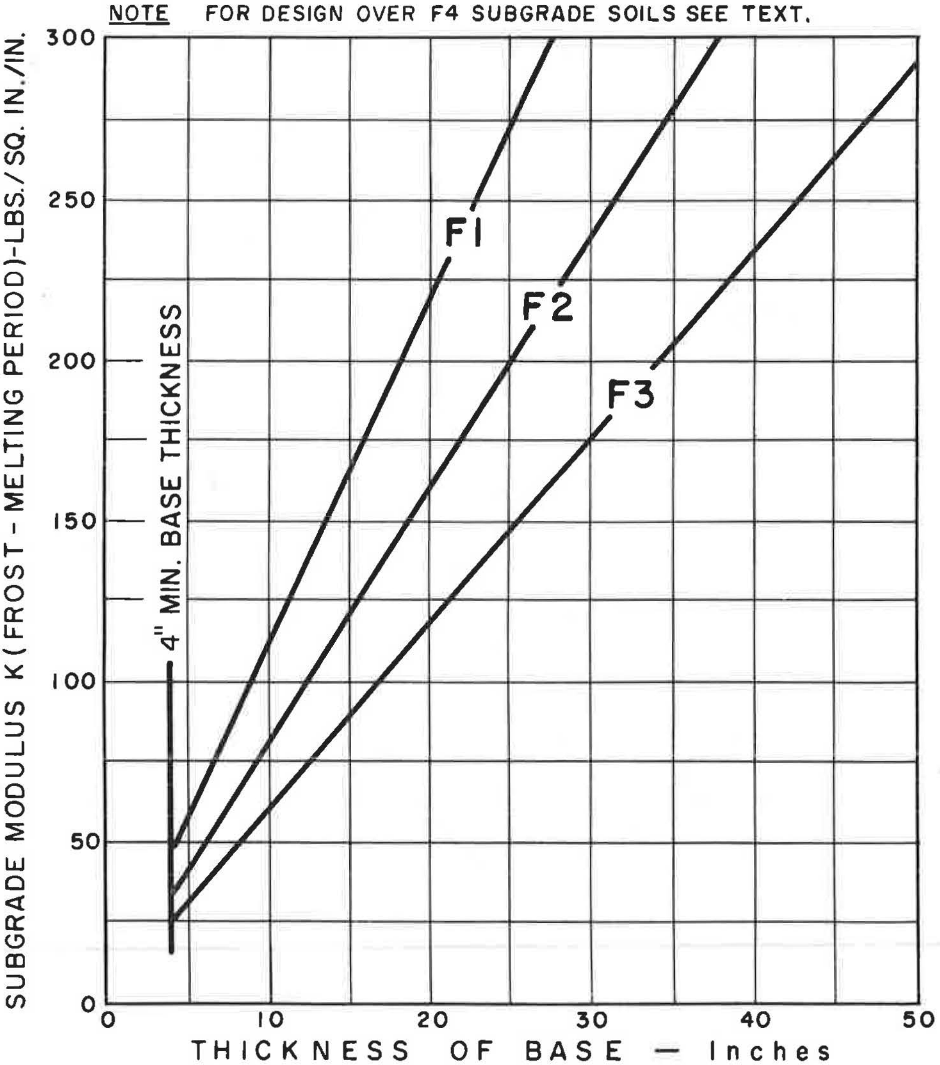


Figure 21. Frost condition reduced subgrade strength design subgrade modulus curves for rigid airfield and highway pavements.

determined for limited subgrade frost penetration or for complete protection, the applicable smaller value should be used, provided it is at least equal to the thickness required for non-frost conditions.

In situations where use of the reduced subgrade strength method might result in objectionable surface roughness or pavement cracking caused by frost heave, but use of the limited subgrade frost penetration design is not considered necessary, intermediate design thicknesses may be used as necessary to prevent objectionable heaving, provided justification is offered on the basis of frost heaving experience developed from existing airfield and highway pavements where climatic and soil conditions are comparable.

(1) Flexible Pavements. In the reduced subgrade strength method of design, the curves in Figures 15 through 18 are used to determine the combined thickness of flexible pavement and non-frost-susceptible base required for aircraft wheel loads and wheel assemblies. Figures 19 and 20 are used for highway design in combination with Ref. (6).

Figure 19 shows no consideration to repetition of loading or to methods for combining the effects of widely varying load. It is used to design pavements for a specified single wheel load selected on the basis of engineering judgment and experience, to represent the anticipated traffic.

Normal Corps of Engineers' practice is to design flexible pavements for roads, streets and similar areas based on a design index. This index represents all traffic expected to use the pavement during its life. It is based on typical magnitudes and compositions of traffic reduced to equivalents in terms of repetition of an 18,000-lb single-axle dual tire load. Development of this method for flexible pavements is given in Ref. (7). Figure 20 shows the required thickness of flexible pavement for the soils of groups F1, F2, F3 and F4 and various design indexes, selection of which is discussed in Ref. (6).

The curves for highways require greater combined thicknesses than the curves for equivalent single-wheel aircraft loadings because of the higher frequency of load applications. General field data and experience indicate that on the relatively narrow embankments of highways, reduction in strength of subgrades during frost melting may be less in substantial fills than in cuts because of better drainage conditions and less intense ice segregation. If local field data and experience show this to be the case, then a reduction in combined thickness of pavement and base of up to 10 percent may be permitted for highways on substantial fills. In no case should the combined thickness of pavement and non-frost-susceptible base be less than 9 in. where frost action is a consideration.

(2) Rigid Pavements. Where frost penetration is permitted in a horizontally uniform frost-susceptible subgrade beneath a rigid pavement, a non-frost-susceptible base course at least equal in thickness to the slab should be used, except for the following conditions:

(a) Where subgrade soils of groups F1, F2, and F3 occur under horizontally uniform conditions (Table 3) and the design freezing index is less than 1,000, the minimum thickness of the non-frost-susceptible base should be 4 in., designed in accordance with the combined filter requirements discussed earlier.

(b) Where soils of groups F1, F2, and F3 subject to pumping occur under horizontally uniform conditions and the depth to the water table is greater than 10 ft, the minimum thickness of the non-frost-susceptible base should be 4 in., designed in accordance with the combined filter requirements.

The base course drainage criteria of Appendix C may require use of base course thicknesses greater than those outlined.

The thickness of concrete pavement should be determined in accordance with Ref. (8) for airfields and Ref. (9) for highways, using the modulus of subgrade reaction for the frost-melting period, $\bar{k}_f$ (Fig. 21), which shows values of equivalent subgrade reduced strength in relation to the thickness of base. If the tested non-frost subgrade modulus value, k, is smaller than the subgrade modulus k_f (Fig. 21), the test value should govern the design. Plate bearing tests performed during the frost-melting period are difficult to evaluate and should not be attempted. Development of rigid pavement thickness requirements for military roads and streets, based on the design index, is given in Ref. (10).

DESIGN FOR STABLIZED RUNWAY OVERRUNS

Frost Condition Requirements

A runway overrun pavement must be designed to withstand occasional emergency aircraft traffic in the form of short or long landings, aborted takeoffs, and possible barrier engagements. The pavement must also serve various maintenance vehicles, such as crash trucks and snowplow equipment. The design of an overrun must provide: adequate stability for infrequent aircraft loading during the frost-melting period; adequate stability for "normal" traffic of snow removal equipment and other maintenance vehicles during frost-melting periods; and sufficient thickness of frost-free base or subbase materials to prevent objectionable heave during freezing periods.

Overrun Design for Reduced Subgrade Strength

In order to provide adequate strength during frost-melting periods, a combined thickness of flexible pavement and non-frost-susceptible base and subbase course should be used, which will be 75 percent of the thickness required for frost capacity operations, based on reduced subgrade strength (Figs. 15-18). The thickness established by this procedure should have the following limitations:

(1) It should not be less than that required for non-frost condition design in overrun areas as determined from Ref. (3).

(2) It should not exceed the thickness required under the limited subgrade frost penetration design method, unless greater thickness is required by the first limitation. For the current principal assembly loadings, use of the tabulation of overrun design thicknesses which follow will avoid the necessity of entering the curves referenced previously.

Overrun Design for Control of Surface Roughness

In addition to establishing the necessary thickness for strength, it may become necessary in some instances to provide additional thickness to restrict maximum differential frost heave to an amount which is reasonable for these emergency areas (generally not more than 3 in. in 50 ft). In selecting a design for restricting frost heave, consideration must be given to type of subgrade material, availability of water, depth of frost penetration, and local experience. In the absence of reliable information on frost heave based on local experience, the following criteria derived from limited tests at Dow and Presque Isle Air Force Bases provide a guide to frost heave limitations for runway overruns:

TABLE 4

COMBINED THICKNESS OF FLEXIBLE PAVEMENT AND BASE (IN.)*
(Equal to 75% of Frost Capacity Operation Thickness)

	F1 Subgrade	F2 Subgrade	F3, F4 Subgrade
188,000 lb, twin-tandem assembly, tricycle gear, spacing 22.5 in., 152 sq in. contact area each wheel (Convair 880).	18	24	36
296,000 lb, twin-tandem assembly, tricycle gear, spacing 34 in., 236 sq in. contact area each wheel (Boeing 707).	23	29	45
310,000 lb, twin-tandem assembly, tricycle gear, spacing 30 in., 228 sq in. contact area each wheel (Douglas DC-8).	24	30	47

*These thicknesses exceed those required for normal operation of snowplow and crash-truck equipment.

(1) For a type F3 subgrade, differential heave can generally be controlled to 3 in. in 50 ft by providing a thickness of non-frost-susceptible base and subbase course equal to 60 percent of the thickness required by the limited subgrade frost penetration design method.

(2) For well-drained subgrades of the F1 and F2 frost types, smaller thicknesses are satisfactory for control of heave. However, unless the subgrade is non-frost-susceptible, the minimum thickness of pavement and base course in overruns should not be less than 40 percent of the thickness required for limited subgrade frost penetration design.

These criteria apply only if they require a combined pavement and base thickness in excess of that described previously for adequate load-supporting capacity.

EXAMPLES OF PAVEMENT DESIGN

Example 1

Design both flexible and rigid class A highway pavements to carry vehicles consisting of 75 percent passenger cars and panel and pick-up trucks, 15 percent two-axle trucks, and 10 percent three-, four- and five-axle trucks, under frost conditions, using the following information:

Design freezing index—800.
Pavement (from normal period design): 3-in. bituminous concrete or 8-in. portland cement concrete.
Base material:
- non-frost-susceptible;
- dry unit weight, 135 pcf;
- moisture content in fall, 5 percent.

Subgrade:
- lean clay;
- plasticity index, 15;
- moisture content, 30 percent;
- uniform conditions;
- normal period CBR, 8 percent.

Highest ground water—3 ft below top of subgrade.
Concrete flexural strength, 700 psi.

The subgrade soil falls into frost group F3.

(1) Flexible Pavement.

Limited Subgrade Frost Penetration.—From Figure 11 the estimated depth of frost penetration below the pavement surface, for base material of 135 pcf dry unit weight, 5 percent moisture content, and unlimited depth, is 52 in. Subtracting the 3 in. of wearing surface, the penetration in base-type material would be 49 in. From Figure 14, required actual base thickness under this design method is 32 in., using a ratio of subgrade to base moisture content of 2.0, the maximum permitted. About 8 in. penetration into subgrade may be expected 1 year in 10. Required combined thickness of pavement and base under the limited subgrade penetration method is 32 + 3 = 35 in.

Reduction in Subgrade Strength.—From Ref. (6), flexible pavement design index is 6. From Figure 20, 30 in. combined thickness of pavement and non-frost-susceptible base are required by the reduction in subgrade strength method for this group F3 subgrade soil. This is 5 in. less than required by the limited subgrade frost penetration method.

Since subgrade conditions are expected to produce uniform heave, the 30-in. thickness is the proper choice. At least the bottom 4 in. of the base should be graded to provide filter action against the subgrade.

(2) Rigid Pavement.

Limited Subgrade Frost Penetration.—From Figure 11, the estimated depth of frost penetration with base of unlimited depth is 52 in. Subtracting the 8-in. slab thickness applicable for normal period design, the penetration in base materials only would be 44 in. From Figure 14, the required actual base thickness is 29 in., which will allow about 7 in. of frost penetration into the subgrade 1 year in 10. Required combined thickness of pavement and non-frost-susceptible base = 29 + 8 = 37 in.

Reduction in Subgrade Strength.—Because the design freezing index is less than 1,000 and subgrade is of a type which produces uniform heave, exception permitting a minimum 4-in. base course to protect against loss of support by pumping is applicable. From Figure 21, the reduced-strength subgrade modulus is 25 psi per in. From Ref. (9), rigid pavement design index is 5, and corresponding required slab thickness is 10 in. after rounding to the next full inch of thickness.

The combined thickness of 10 + 4 = 14 in. is more economical than that obtained by the limited subgrade frost penetration method. However, the design must also be analyzed for conformance with the base drainage criteria of Ref. (2) and Appendix C; these may prove governing. Also, the reduced subgrade strength design can be used only if local experience, records, and study of the specific subgrade conditions indicate that objectionable differential heave and cracking of pavements will not occur. Note that consideration of local experience and records must take into account the severity of freezing conditions actually experienced during the period of record. Frequently these conditions may be well below the design freezing index level.

Example 2

Design flexible and rigid pavements for the following conditions:

Aircraft—Boeing 707, gross weight 296,000 lb, twin-tandem assembly, tricycle gear, spacing 34 in., contact area 236 sq in. each wheel.
Design freezing index—3,000 degree-days.
Subgrade material:
- clay (CL);
- plasticity index, 18;
- water content, 25 percent (avg.);
- normal period CBR, 8;
- normal period subgrade modulus, K - 400 psi/in. (corresponds to test value on top of base of final design thickness).
- Subgrade shows moderate differential heave character in existing pavements and is, therefore, classed as horizontally variable.

Base course material:
- high quality base material (flexible pavement only), graded crushed aggregate, normal period CBR = 100;
- remainder of base non-frost-susceptible sandy gravel (GW), normal period CBR, 50;
- avg. dry unit weight, 135 pcf;
- avg. water content after drainage, 5 percent.

Highest ground water—3 ft below surface of subgrade.
Concrete flexural strength, 650 psi.

(1) Flexible Pavement.

Limited Subgrade Frost Penetration Method.—The subgrade is frost group F3. Table 3 indicates that this design method is applicable for the horizontally variable subgrade condition. From Figure 12, to prevent any freezing of subgrade in the design freezing index year (complete protection), the combined thickness of pavement and base a is 140 in. From Ref. (3), the required flexible pavement thickness p is 4 in. Therefore, thickness of base c for zero penetration of subgrade is 136 in. The ratio of subgrade to base water content r is over 2.0. A ratio of 2.0 is used in Figure 14, which yields a required base thickness b of 91 in. The required combined thickness of pavement and base to limit subgrade frost penetration is 91 + 4 = 95 in. As shown in Figure 14, this will allow about 23 in. of frost penetration into the moderately variable F3 subgrade on an average of 1 year in 10. (Because this is limited subgrade frost penetration design, the same total thickness would apply for all traffic areas.)

This design will limit pavement heaving, cracking, and loss of subgrade strength to tolerable amounts, provided all other requirements are met, such as use of non-frost-susceptible base material, uniformity of the base course as placed, subsurface drainage meeting the criteria of Ref. (2), and use of appropriate transitions at any substantial and abrupt changes in the foundation characteristics.

Because the indicated combined thickness exceeds 72 in., further investigation should be made to attempt to locate a non-frost-susceptible base course material of lower unit weight and/or higher moisture retention. It could be used in lieu of the sandy gravel for at least a substantial part of the base thickness to reduce the amount of frost penetration and hence the design thickness requirements. If this is not successful, a special analysis should be made for each traffic area using all available data, including performance records of other pavements under similar conditions, to determine whether surface roughness of the flexible pavement for each specific case under design freezing index conditions would be excessive if only 72-in. combined thickness is used.

Reduced Subgrade Strength Method.—Referring to Table 3, this design method should not be used when horizontally variable subgrade conditions exist.

About 34-in. combined thickness would be required during the normal period. Thus, the 95-in. thickness determined by the limited frost penetration method is applicable, unless some reduction can be achieved by further analysis.

(2) Rigid Pavement.

Limited Subgrade Frost Penetration Method.—Table 3 indicates this method is applicable. The required pavement thickness p, based on the normal period k = 400 psi/in., is 18 in. Every inch of concrete pavement in excess of 12 in. reduces the design freezing index by 10 degree-days. In this example, the reduction = 10 (18-12) = 60 degree-days. Therefore, the modified freezing index = 3,000 - 60 = 2,940. From Figure 12, the combined thickness of 12-in. pavement and base a required to prevent any freezing of the subgrade is 138 in. Addition of the originally deducted 6-in. thickness of pavement results in a combined thickness of pavement and base of 144 in. Therefore, the thickness of base c required for zero frost penetration into the subgrade is 126 in. From Figure 14, the required design base thickness b is 84 in., which permits a corresponding subgrade frost penetration s of 21 in. in the design year. Because the indicated combined thickness of 84 + 18 = 102 in. exceeds 72 in., special analysis is required for possible reduction of base thickness. The possible use of steel reinforcement, reduced slab dimensions, or base material with smaller unit weight and/or higher moisture retention are considered appropriate.

In the exceptional case of an extremely variable subgrade or of design requirements so stringent that complete protection is required, a combined thickness of 144 in. would be needed using this particular base material. In such case, an attempt should be made again to provide a non-frost-susceptible base material of smaller unit weight and/or higher moisture retention in order to reduce this thickness.

Reduced Subgrade Strength Method.—As indicated in Table 3, this method is not applicable for rigid pavements under horizontally variable subgrade and moisture conditions.

Example 3

Design an overrun pavement for the following conditions:

Aircraft—Boeing 707, gross weight 296,000 lb, twin-tandem assembly, tricycle gear, spacing 34 in., contact area 236 sq in. each wheel.
Design freezing index - 600 degree-days.
Subgrade material:
uniform sandy clay (CL);

plasticity index, 18;
water content, 20 percent (avg.);
normal period CBR, 15.
Base course material:
non-frost-susceptible sandy gravel (GW);
avg. dry unit weight, 135 pcf;
avg. water content after drainage, 5 percent.
Highest ground water—4 ft below surface of subgrade.

For reduced subgrade strength during the frost-melting period, the required combined thickness for F3 subgrade is 45 in.

Under limited subgrade frost penetration design method, using the same computation procedures outlined above and neglecting effect of any surface treatment, the required thickness is 29 in. which would allow about 7 in. of frost penetration into the subgrade 1 year in 10.

ACKNOWLEDGMENT

The design of pavement for frost conditions reported herein is based on studies carried out under the overall direction of the Civil Engineering Branch, Engineering Division, Military Construction, Office, Chief of Engineers. Mr. Thomas B. Pringle is Chief.

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Appendix A

FIELD CONTROL OF PAVEMENT CONSTRUCTION FOR FROST CONDITIONS IN AREAS OF SEASONAL FREEZING

Field control of airfield and highway pavement construction in areas of seasonal freezing should give specific consideration to conditions and materials that will result in detrimental frost action. Ideally, contract plans and specifications should provide for special treatments, such as removal of unsuitable materials encountered, with sufficient information included to identify those materials and specify necessary corrective measures. However, construction operations will quite frequently expose frost-susceptible conditions at isolated locations of a degree and character not revealed by even the most thorough subsurface exploration program conducted during the design phase. It is essential, therefore, that personnel assigned to field construction control be made aware of their responsibility to recognize situations that require special treatment whether or not anticipated by the designing agency.

Subgrade Preparation

Where laboratory and field investigations indicate that the soil and ground water conditions will not result in ice segregation in the subgrade soils, the pavement design is based on the assumption that the soils will not heave during the winter or weaken during the frost-melting period. The construction inspection personnel should check the validity of the design assumptions, and if pockets of frost-susceptible material or wet subgrade conditions are revealed of which the design agency was not cognizant, remedial measures should be initiated. Gradation tests should be performed on any questionable materials encountered during grading operations, and all pockets of frost-susceptible soils in an otherwise non-frost-susceptible subgrade should be removed to the full depth of frost penetration and replaced with materials of the same type as the surrounding soil. Clean granular soils are little affected by frost action. These materials should be employed in situations where seasonal freezing will affect the construction.

At any site where the subgrade conditions are recognized as favorable for frost action, personnel should be alert to observe whether the field conditions as found are in accordance with the design assumptions regarding drainage, gradation and character of materials. Where the design permits freezing of the subgrade materials, the inspector has the responsibility of insuring that the special frost protection measures are adequate and that design provisions are adhered to. One condition that is often left in the hands of the field inspection forces is the case of a subgrade which consists of soils of variable degrees of frost susceptibility. Areas in such a subgrade requiring supplementary design measures can only be defined as to location during grading operations. It may be necessary either to remove a pocket of highly frost-susceptible material for the full depth of frost penetration, or if this is impractical, to provide transition zones between the areas of high and low frost susceptibility so as to minimize non-uniform pavement heave. In general, abrupt changes in subgrade conditions should always be avoided by providing transitions, particularly in high-speed pavements such as runways. Frequent trouble sources in addition to abrupt variations in soil characteristics, are sudden changes in ground water conditions, changes from cut to fill, and locations of under-pavement pipes, drains, or culverts. At the transition between cut and fill sections the topsoil and humus materials should be completely removed for the ultimate depth of frost penetration in otherwise non-frost-susceptible materials, even though specifications may not require general stripping in fill areas.

Special attention should be given to wet areas in the subgrade, and special drainage measures should be installed as required. The need for such measures arises most frequently in road construction where it may be necessary to provide intercepting drains to prevent infiltration into the subgrade from higher ground adjacent to the road.

In areas where rock excavation is required, the character of the rock and seepage conditions should be considered. In any case, the excavations should be made so that positive transverse drainage is provided and no pockets are left on the rock surface which will permit ponding of water within the maximum depth of freezing. The irregular ground water availability created by such conditions may result in markedly irregular heaving under freezing conditions. It may be necessary to fill drainage pockets with lean concrete. Rock subgrades where large quantities of seepage are involved should be blanketed with a highly pervious material to permit the escape of water. Frequently the fractures and joints in the rock contain frost-susceptible soils. These materials should be cleaned out of the joints to the depth of frost penetration and replaced with non-frost susceptible material. If this is impractical, it may be necessary to remove the rock to the full depth of frost penetration.

Base Course Construction

Where the available base course materials are positively non-frost-susceptible, the base course construction control should be in accordance with normal practice. In instances where the base course material selected for use is of borderline frost susceptibility (usually materials having $1\frac{1}{2}$ to 3 percent of grains finer than 0.02 mm by weight), frequent gradation checks should be made to insure that the materials meet the design criteria. If it is necessary for the contractor to exercise selection in the pit in order to obtain suitable materials, his operations should be inspected at the pit. It is more feasible to reject unsuitable material at the source when large volumes of base course are being placed. It may be desirable to stipulate thorough mixing at the pit, and if necessary, stockpiling, mixing in windrows and spreading the material in compacted thin lifts in order to insure uniformity. Complete surface stripping of pits should be enforced to prevent mixing of detrimental fine soil particles or lumps in the base material. The gradation of materials taken from the base after compaction, such as density test specimens, should be determined particularly at the start of the job and checked frequently to see if fines are being manufactured in the base under the passage of the base course compaction equipment. Base course materials exhibiting possible serious degradation characteristics may warrant construction of a test embankment to study the manufacture of fines under the proposed or other compactive efforts. Mixing base course materials with frost-susceptible subgrades should be avoided by making certain that the subgrade is properly graded and compacted prior to placement of base course, by insuring that the first layer of base course provides filter action against penetration of subgrade fines under traffic, and by the elimination of kneading action caused by overcompaction or insufficient thickness of the first layer of base course. Experience has shown that excessive rutting by hauling equipment tends to cause mixing of subgrade and base materials. This can be greatly minimized by the frequent rerouting of material-hauling equipment. After completion of each lift of base, a careful visual inspection should be made before placing additional material to insure that areas with high percentages of fines are not present. These areas may be frequently recognized both by visual examination of the materials, and by observations of their action under compaction equipment, particularly when the materials are wet. The materials of any areas which do not meet specification requirements for frost conditions should be removed and replaced with suitable material. Use of a leveling course of fine-grained material should not be used as a construction expedient to choke open-graded base courses, to establish fine grade or prevent overrun of concrete. Because the base course receives high stresses from traffic, this prohibition is essential so that there will be no weakening during the frost-melting period.

Action should be taken to vary the base course thickness to provide transitions, when necessary, and to avoid abrupt changes in pavement supporting conditions.

Appendix B

STANDARD LABORATORY FROST SUSCEPTIBILITY TEST PROCEDURE

Molding of Specimens

Soil specimens for standard laboratory frost susceptibility tests are generally prepared in a slightly tapered (5.50 to 5.75 in. inside diameter) 6-in. high steel molding cylinder with removable base. The steel cylinder is lubricated with silicone grease and a light coat of paraffin prior to molding to facilitate ejection of the soil specimen. The soil is compacted to an approximate height of 6 in. and to a predetermined dry unit weight by means of a static load and/or vibration. Undisturbed specimens of cohesive soils are prepared by trimming to a uniform diameter and height of about 6 in., respectively.

Two methods are used in molding specimens to the desired dry unit weight. Relatively cohesionless, coarse-grained soils, such as sands and sandy gravels, are generally prepared by an adaptation of the Providence Vibrated Density Test Method (11). In this method, a predetermined weight of soil is placed in the steel cylinder and a load of approximately 1,000 lb is applied by a piston at each end and a heavy spring at the top. The soil within the steel cylinder is compacted by vibrating the cylinder with hammer blows on the sides. Fine-grained soils, such as uniform fine sands, silts and glacial tills are compacted by tamping in layers using the modified AASHO (12) or the Corps of Engineers Airfield Density Test (3) procedures, Appendix C.

Cohesionless soils are either molded dry and then wetted, or are molded at a low moisture content which improves the apparent cohesion and aids specimen handling after molding. For field construction design purposes, cohesive soils are molded at the optimum moisture content and to the dry unit weight determined by the Modified AASHO Test or Corps of Engineers Airfield Density Test, depending on the anticipated field conditions or requirements. For evaluation of the frost potential of materials under existing pavements, subgrade soils obtained from beneath the pavements are tested either in an undisturbed condition or are recompacted in the laboratory to approximately field dry unit weight and moisture conditions.

The remolded specimens are removed from the steel molding cylinder by piston pressure at the bottom of the specimen and are fitted snugly into open-ended tapered lucite cylinders (wider at the upper end) lined with cellulose acetate strips, 1.5 in. wide and 0.007 in. thick. The acetate strips are coated on each side with silicone grease and lapped horizontally in a telescopic manner. This is done to minimize friction between the specimen and cylinder when heave takes place during freezing. Specimens prepared by cutting from undisturbed samples are not tapered because of the difficulty of obtaining a uniform taper manually. Such specimens are fitted snugly into parallel-walled cylinders of lucite or of waxed, laminated heavy cardboard lined with lubricated acetate strips.

Saturation of Specimens

All specimens tested in the open system are saturated prior to freezing. Saturation is carried out in the cold room at a temperature of 38 F. Both ends of the lucite cylinder containing the soil specimen are covered with filter papers, porous discs ($^3/_8$ in. thick) and capped with snug-fitting shallow brass pans which have nipples extending out from the center for connection of tubing. A rubber sleeve-like membrane, 0.02 in. thick, is slipped around the cylinder and a rubber band wound firmly around the membrane over the entire height of the cylinder to seal the specimen against leakage during the air evacuation and the subsequent saturation period. The specimen is first evacuated of air simultaneously from the top and bottom. It is then saturated from the bottom with de-aired water.

Thermocouples in Specimens

Thermocouples are inserted at 1-in. intervals along the longitudinal axis, including top and bottom, in one of the specimen groups in a test cabinet, and at the top and bottom only in one additional specimen. The former installation provides an accurate record of the temperature gradient and the day-by-day advance of freezing temperature into the specimen. The latter installation provides a double check of the start and completion of the freezing test period. The thermocouples are inserted through the side of the specimen container. The entrance points are sealed with a mastic or other suitable waterproofing material. The specimens are placed in freezing cabinets containing cooling plates around three sides at the top. Each cabinet can accommodate up to four 6-in. diameter specimens. A water supply is connected to the bottom of each specimen through the nipple provided on the brass receptacle. The nipple protrudes through a bottom sheet metal pan and grillwork into the open space beneath the freezing cabinet which is about 38 F, the cold room temperature. The free water level in the bottom cap is adjusted and maintained at a height of $\frac{1}{4}$ to $\frac{3}{8}$ in. above the bottom of the specimen. The top brass caps, porous stones and filter papers are removed and the space around the specimens is filled loosely with granulated cork leaving the top surface of the specimens exposed to the cabinet air temperature.

Pressure

All specimens are frozen under a pressure load (lead weights) of 0.5 psi to simulate field conditions consisting of a 6-in. combined thickness of base and pavement. A thin steel base plate ($\frac{1}{8}$ in. thick) is placed on top of the specimen and firmly seated to provide a uniform contact. Four lugs are attached to the base plate to raise the lead weights $1\frac{1}{2}$ in. so that the air may circulate over the top of the specimen.

Freezing Test Procedure

Prior to freezing, the specimens are tempered for 18 to 24 hours at 38 F. Initial freezing is obtained by rapidly lowering the air temperature in the freezing cabinet to about 20 F until crystallization of the soil is visible on the surface. To insure crystallization, the surfaces are seeded with pulverized ice. At this time, the thin 6-in. diameter steel base plates and weight (both tempered at 28 F) are placed on each specimen to provide the necessary pressure intensity. The specimens are then gradually frozen from the top to bottom by sufficiently decreasing the cabinet air temperature to obtain a rate of the 32 F isotherm of about $\frac{1}{4}$ to $\frac{1}{2}$ in. per day. Heave measurements are taken daily with a meter stick or an extensometer placed on a designated point on the surcharge weights over the specimens.

Examination of Specimens

On completion of the freezing tests, usually 24 days, the specimens are removed from the cabinet and containers and are weighed, measured and split longitudinally in two sections. Measurements for amount of heave, and observations for the location, distribution and magnitude of ice lens formations are made on one section. The other section is photographed and retained for supplemental laboratory tests. The water content distribution is obtained for every inch of specimen depth.

Supplementary Laboratory Tests

The following standard laboratory tests are performed on all materials tested, for correlation with the average rate of heave: gradation, permeability, specific gravity, Atterberg limits (if applicable), and compaction characteristics.

Evaluation of Frost Susceptibility

The standard laboratory frost susceptibility test was designed to subject the soil to a severe combination of conditions conducive to frost action and results in virtually the maximum rate of ice segregation and heave which the soil can exhibit under natural field conditions. The results are not usually quantitatively representative of actual heave to be expected in the field. The test procedures are considered satisfactory, however, for determining the relative degree of frost susceptibility of various soils, with the possible exception of unweathered clays which may show unduly low heave for at least the first cycle of freezing. In clays which are unfissured and have not previously been frozen, the rate of heaving may be low initially, but as the clay is repeatedly thawed and frozen and becomes fissured, the rate of heaving may become much greater.

Rate of heave has been found to be relatively independent of rate of freezing over the range of employed freezing rates. Therefore, average rate of heave has been utilized as the basis for expression, comparison, and evaluation of test results. The following tentative scales of average rate of heave have been adopted for rates of freezing between $\frac{1}{4}$ in. and $\frac{3}{4}$ in. per day:

Average Rate of Heave mm/Day	Frost Susceptibility Classification
0-0.5	Negative
0.5-1.0	Very low
1.0-2.0	Low
2.0-4.0	Medium
4.0-8.0	High
Greater than 8.0	Very high

The evaluation given by the standard freezing test should be considered empirical in nature. Average rate of heave does not represent a simple and fundamental physical value because such factors as pressure and moisture availability vary continuously during the test.

Appendix C

DESIGN OF BASE COURSE DRAINAGE

Basis for Design

Where frost action occurs in the subgrade beneath the pavement, base drainage is required. To simplify the analysis of drainage of base courses, it is assumed that the base course is fully saturated and no inflow occurs during drainage, the subgrade constitutes an impervious boundary, and the base course has a free outflow into the drain trench.

Maximum Rate of Discharge

The following equation may be used to determine the maximum rate of discharge for a saturated base course of dimensions shown in Figure 22:

$$q = kH\frac{H_0}{D60}$$

where:

k is the coefficient of horizontal permeability in feet per minute;
H, H_0, and D are dimensions (Fig. 22) in feet; and
q is the peak discharge quantity in cfs per lineal foot of drain.

Degree of Drainage

Degree of drainage is defined as the ratio, expressed as a percent, of the amount of water drained in a given time to the total amount of water that is possible to drain from the given material. Base course design should be based on the criterion that a degree of drainage of 50 percent in the base course should be obtained in not more than 10 days. The following formula may be used to determine the time required for a saturated base course to reach a degree of drainage of 50 percent:

$$t = \frac{n_e D^2}{2880kH_0}$$

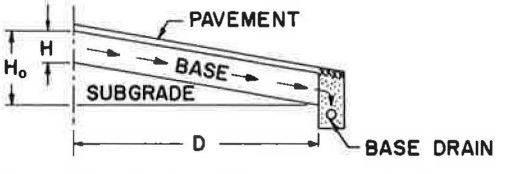

Figure 22. Design of base course drainage.

where:

t is time in days for 50 percent drainage;
n_e is the effective porosity of the soil;
D and H_0 are dimensions (Fig. 22) in feet; and

k is coefficient of permeability of the soil parallel to direction of seepage flow in feet per minute.

The application of the preceding formula may be illustrated by the following example. Assuming a section as shown in Figure 22, let:

$$n_e = 0.1$$
$$D = 75 \text{ ft}$$
$$H_o = 3.6 \text{ ft}$$
$$k = 1 \times 10^{-2} \text{ ft per min}$$

Then:

$$t = \frac{0.1 \times 75 \times 75}{2880 \times 0.01 \times 3.6} = 5.4 \text{ days}$$

Coefficient of Permeability of Base Materials

The base materials generally used immediately beneath airfield pavements consist of sand and gravel, sand, crushed rock, partially crushed gravel and sand, slag, cinders, etc. In many cases the base will consist of several layers, each of different base material. The coefficient of permeability of sand and gravel courses graded between limits usually specified for stabilized material depends principally on the percentage by weight of sizes passing the 200-mesh sieve. The following tabulation may be used for preliminary estimates of average coefficients of permeability for remolded samples of sand and gravel bases:

Percent by weight passing 200-mesh sieve	Coefficient of permeability for remolded samples (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 without many fines, is generally greater than one foot per minute. The coefficient of permeability of sand, and sand and gravel mixtures may be approximated from Figure 23.

The coefficient of permeability of a base in a horizontal direction (parallel to compaction planes) may be 10 times greater than the average value tabulated previously, the average value based on determinations on remolded samples. For uniformly graded sand bases, the coefficient of permeability in a horizontal direction may be about four times greater than the value determined by tests on remolded samples. Very pervious base materials such as crushed rock and slag with few fines, have substantially the same coefficient of permeability in a vertical and horizontal direction.

In all cases for final design, the coefficient of permeability of the material used for base should be determined by laboratory tests. The preceding values are presented as a general guide for preliminary design computations.

When more than one material is used in a given base, the weighted coefficient of horizontal permeability determined in accordance with the following formula results in a reasonable design value.

$$k = \frac{k_1 d_1 + k_2 d_2 + k_3 d_3, \text{ etc.}}{d_1 + d_2 + d_3, \text{ etc.}}$$

where:

k is the weighted coefficient of horizontal permeability;

k_1, k_2, k_3, etc., are the coefficients of horizontal permeability of individual base materials in feet per minute; and

d_1, d_2, d_3, etc., are the thicknesses of the individual layers in feet.

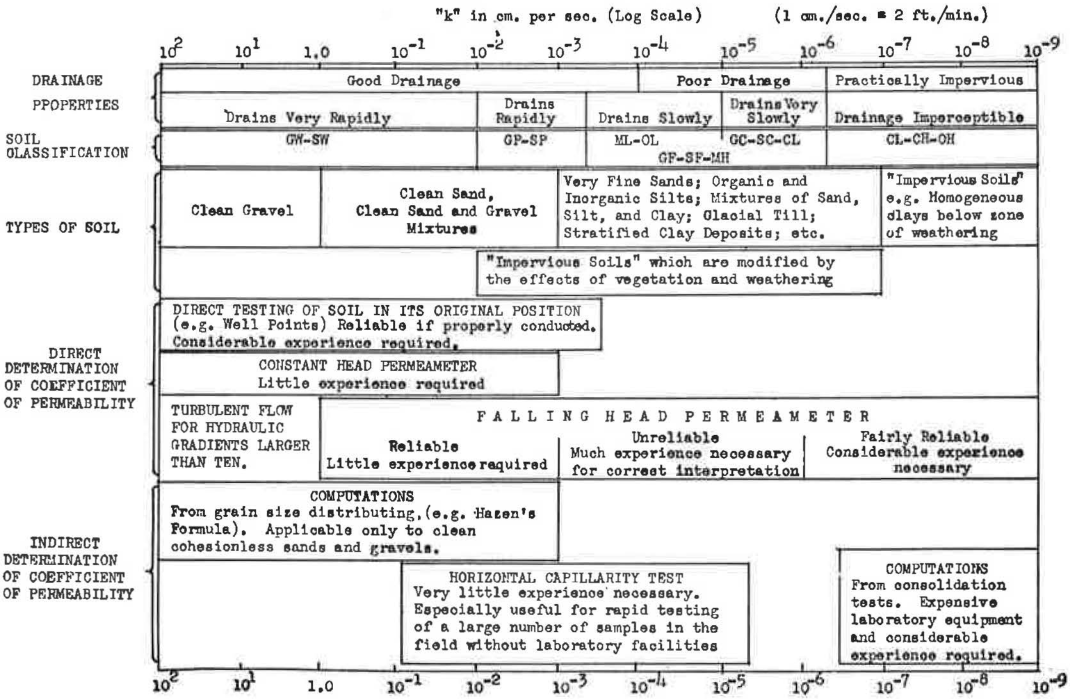

Figure 23. Permeability chart.

Spacing of Drains

Where the time determined for degree of base course drainage of 50 percent is greater than 10 days, the spacing between drains should be decreased until the time for drainage is 10 days or less, a more pervious base material should be selected, or a greater thickness of base should be used in the design.

In general, for most runway and taxiway bases of a width from crown to edge of not more than 75 ft, a single line of base drains along the edges should meet the design criteria. It may be necessary on wider base widths, or where reasonably pervious base course material is not locally available, to install intermediate lines of drains to provide satisfactory base drainage.

Base Course Filter Design

To prevent the movement of particles from the protected soil into or through the filter or filters, the following condition must be satisfied:

$$\frac{15\%\ \text{size of filter material}}{85\%\ \text{size of protected soil}} \leq 5$$

and

$$\frac{50\%\ \text{size of filter material}}{50\%\ \text{size of protected soil}} \leq 25$$

The preceding criteria are used when protecting all soils except medium to highly plastic clays without sand or silt partings, which by the preceding criteria may require multiple-stage filters. For these clay soils, the d_{15} size of the filter may be as great as 0.4 mm and the preceding d_{50} criteria disregarded. This relaxation in criteria for protecting medium to highly plastic clays allows the use of a one-stage filter material.

However, the filter must be well graded, and to insure nonsegregation of the filter material, the coefficient of uniformity should be not greater than 20 (Fig. 24).

Depth of Cover Over Drains

The depth of cover over drains is dependent on loading and frost requirements. (EM 1110-345-283 lists the cover requirements for different design wheel loads.) With respect to frost in areas of seasonal freezing, the depth of cover to the centerline of the pipe should be not less than the depth of frost penetration determined from Figures 11 or 12, based on the design freezing index for the particular location. The trench for subdrains should be backfilled with free-draining, non-frost-susceptible material. Within the depth of frost penetration, gradual transitions should be provided between non-frost-susceptible trench backfill and frost-susceptible materials of drains placed under traffic areas to prevent detrimental differential heave, particularly for the case of frost condition pavement design based on reduced subgrade strength.

Discussion

G. Y. SEBASTYAN, Head, Engineering Design Section, Air Services, Construction Branch, Canadian Department of Transport.—The paper submitted by Messrs. Linell, Hennion and Lobacz was studied with great interest by the engineers of the Engineering Design Section of the Construction Branch, Canadian Department of Transport. The authors did an exceptional job in compiling and making available to the engineering profession, the U. S. Corps of Engineers' design procedures relating to the design of flexible and rigid pavements in areas of seasonal frost. Because almost all Canadian airport pavements are in areas affected by seasonal frost, it was thought interesting and worthwhile to compare the experience and procedures of the Canadian Department of Transport with those given in the subject paper.

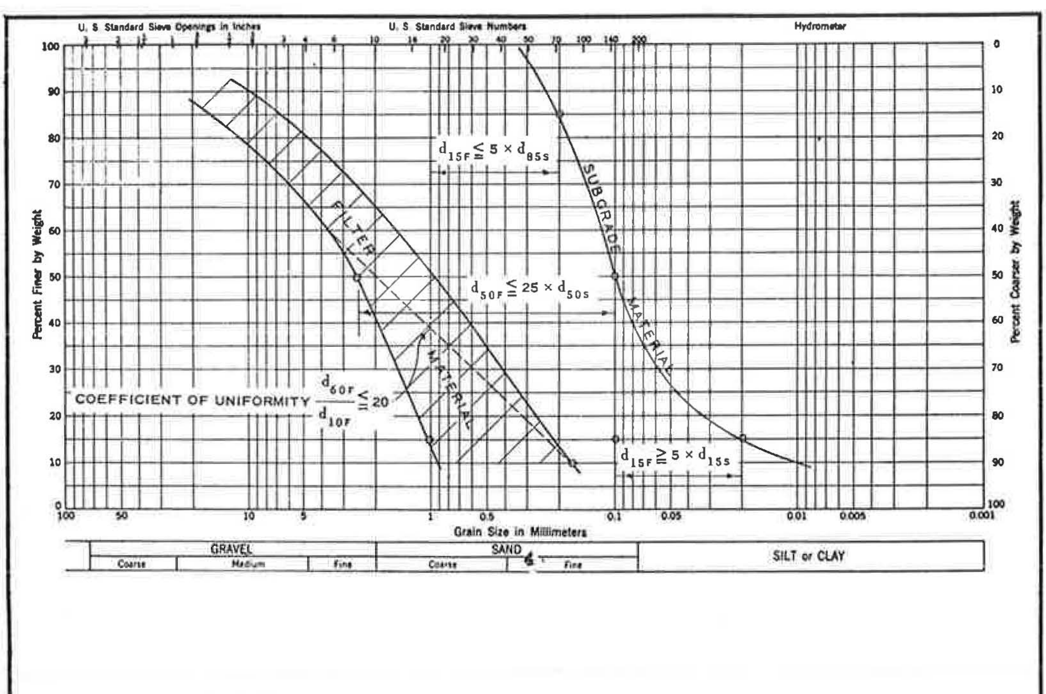

Figure 24. Design example for filter materials.

It is emphasized that the Canadian Department of Transport's design and evaluation procedures are related to Canadian environments and aircraft traffic conditions.

There are three major points discussed herein:

1. The Canadian Department of Transport's frost protection design criteria are based on the 10-yr average freezing index. It is the U. S. Corps of Engineers' practice to use a 10-yr maximum index or the average of three coldest years in 30 years as a design criterion. For Canadian conditions, a comparison was made (Fig. 25) for the 10-yr average and the 10-yr maximum freezing indices. The ratio of 10-yr maximum over 10-yr average freezing index is between 1.5 (FI.1000) and 1.2 (FI.4000).

2. It is the Canadian Department of Transport's design procedure to determine the minimum combined flexible or rigid pavement structure thickness (wearing surface, base and subbase) on the basis of approximately half the expected frost penetration.

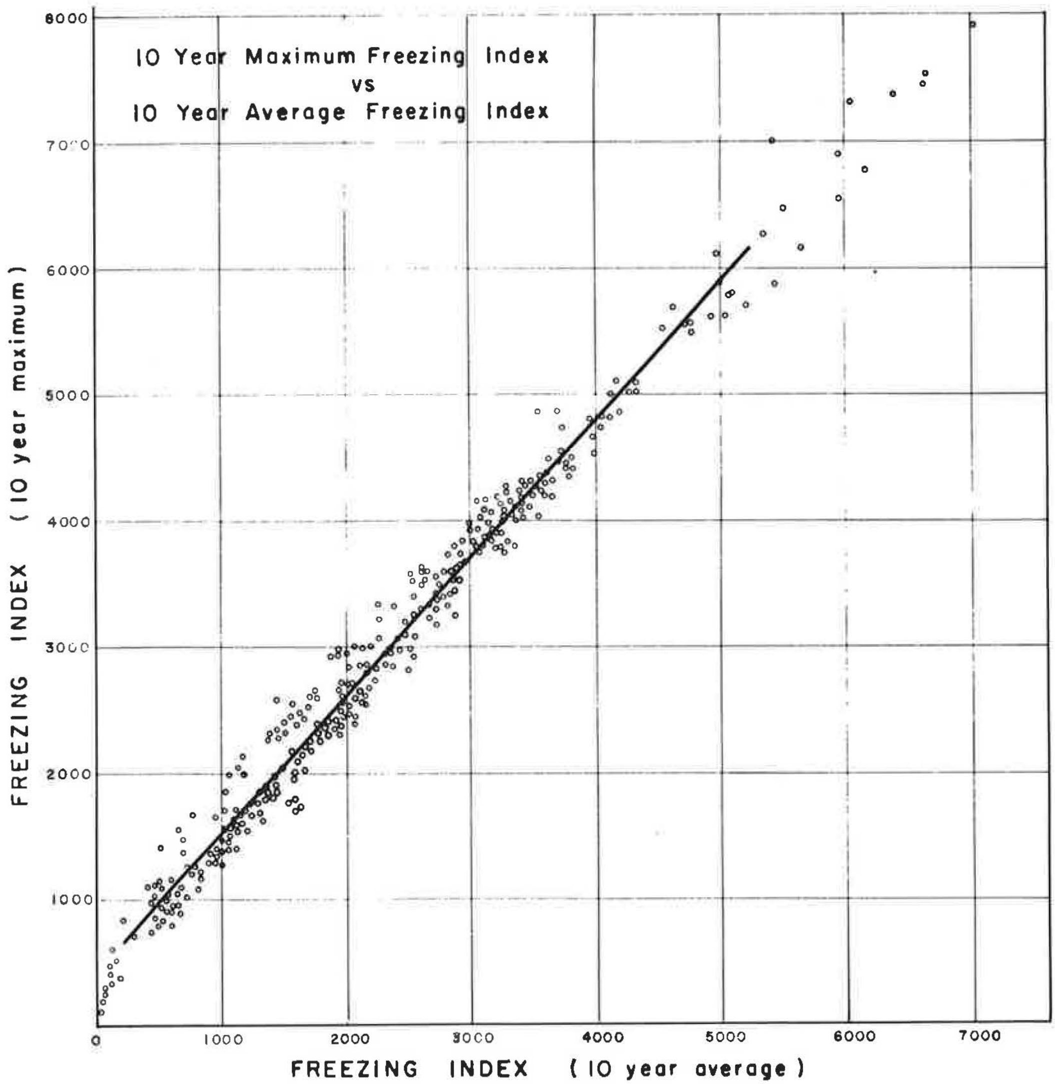

Figure 25. Department of Transport freezing indices for various Canadian meteorological stations.

This thickness is determined from the Department of Transport's design freezing index and the correlation shown in Figure 26.

3. It is the Canadian Department of Transport's design procedure to determine the thickness of necessary pavement structures on the basis of subgrade strength established by repetitive plate load tests (subgrade in equilibrium moisture conditions). Such tests are generally performed during the summer and fall. The design value used is the fall strength reduced by a spring load-carrying capacity reduction factor. The spring reduction factor is not considered to allow for the actual maximum strength reduction during the spring. Because the load-carrying capacity is a function of a number of repetitions of loading, a limited degree of overloading during the spring period is considered permissible. When the freezing index is higher than 500 (10-yr average) and no actual spring strength test data are available, subgrade fall load-carrying capacity values are reduced by silty clay and clay soils, 15%-45%; silt, very fine sand, and all frost-susceptible combinations of both, 45%-50%; medium and coarse sand, 10%; and gravel, 0%.

The actual spring reduction factor chosen within the range given above will depend on the performance of the existing pavements, the uniformity of the subgrade soil, moisture conditions of the subgrade, and the height of the ground water table. The most reliable source of information is the regular condition reports received on the condition of the pavement in question. Examples of such condition reports for flexible and rigid pavements are given in Figures 27 and 28.

In accordance with the U. S. Corps of Engineers' design procedure, silty clay and clay soils (CL & CH) are designated as F3 and F4 soils for which the Corps of Engineers' design charts give maximum frost protection.

Condition reports for 52 Canadian airports have been examined where the subgrade is silty clay or clay soil. The condition reports on these sites indicate that pavement distress due to frost damage is rarely experienced when the subgrade is uniform and the pavement thickness is sufficient to meet the minimum thickness requirement (Fig. 26).

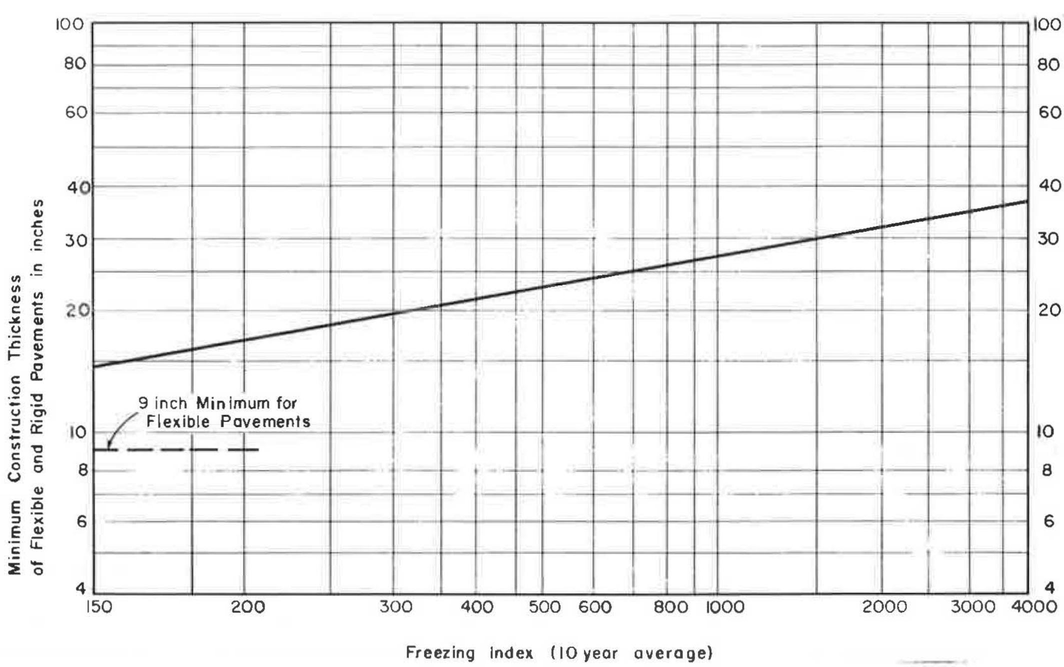

Figure 26. Minimum depth of frost protection for flexible and rigid pavements.

TABLE 5

COMPARISON OF U.S. CORPS OF ENGINEERS AND THE CANADIAN DEPARTMENT OF TRANSPORT DESIGN METHODS FOR FLEXIBLE AND RIGID PAVEMENTS[1]

Pavement Type	Design Method		Pavement Thickness (in.)				Total Cost ($ million)	Comparison of Costs (%)[2]
	Type	Agency and Criterion	Asphalt or P.C.C.	Crushed Base	Granular Base	Total		
Flexible	Strength	Department of Transport	4	12	27	43	4.476	100
		U.S. Corps of Engineers	4	12	57	73	6.856	153
	Frost	Department of Transport	4	12	18	34	3.766	100
		U.S. Corps of Engineers:						
		Complete protection	4	12	139	155	13.336	354
		Limited subgrade penetration	4	12	89	105	9.386	248
		Reduced subgrade strength	4	12	37	53	5.276	118[3]
Rigid		Department of Transport	10	6	18	34	6.078	100
		U.S. Corps of Engineers[4]	15	15	—	30	7.49	124

[1] Design aircraft = DC-8; 240k at 168 psi (DOT), and at 121 psi (USED).
[2] Dept. of Transport cost = 100%.
[3] Percentage based on DOT strength design equals 100%. Reduced subgrade strength design would not be used, as normal strength design requires greater thickness.
[4] Reduced subgrade strength.

AIRPORT "A"

RUNWAY 10 - 28 TAXIWAY -- APRON --

		NONE	MINOR	MAJOR	SEVERE
HAIR	Cracking Type	X			
LONGITUDINAL (Inc. Joints)	Cracking Type		X		
TRANSVERSE	Cracking Type		X		
CHICKEN WIRE (Approx. 3")	Cracking Type	X			
ALLIGATOR (Approx. 6")	Cracking Type	X			
LESS THAN 1/16 inch	Crack Widths		X		
LESS THAN 1/8 inch	Crack Widths		X		
LESS THAN 1/4 inch	Crack Widths			X	
STRIPPING		X			
RAVELLING		X			
RUTTING		X			
DISTORTION		X			
LONGITUDINAL	Deformation	X			
TRANSVERSE	Deformation	X			
SKIN PATCHES		X			
DEEP PATCHES		X			
SUB GRADE SETTLEMENT			X		
FROST HEAVE		X			
SURFACE ROUGHNESS			X		

IN YOUR OPINION THE GENERAL CONDITION IS:-

	100 %			
A			VERY GOOD	CHECK ONE
	80 %			
B		X	GOOD	
	60 %			
C			FAIR	
	40 %			
D			POOR	
	20 %			
E			VERY POOR	
	0 %			

DRAINAGE (4) Imperfectly drained soil, good surface drainage. Surface and sub-drainage are in good working condition.

REMARKS Worst section of cracking is between 10 end and N-S Taxi.

DATE 22 Feb. 1960 OBSERVER R. Tracy

Figure 27. Department of Transport flexible pavement condition report.

AIRPORT B

RUNWAY 07-25 TAXIWAY - APRON -

		NONE	MINOR	MAJOR	SEVERE
CORNER	Cracking	X			
EDGE	Cracking	X			
LATERAL	Cracks	X			
LONGITUDINAL	Cracks		X		
SCALING & SPALLING		X			
JOINT STEPPING OR FAULTING		X			
CONCRETE DISINTEGRATING		X			
PUMPING		X			
LOSS OF JOINT FILLING		X			
SUBGRADE SETTLEMENT		X			
FROST HEAVE			X	See remarks	
SIDE SLIPPAGE			X		

IN YOUR OPINION THE GENERAL CONDITION IS:-

	100%			
A			VERY GOOD	CHECK ONE
	80%			
B		X	GOOD	
	60%			
C			FAIR	
	40%			
D			POOR	
	20%			
E			VERY POOR	
	0%			

DRAINAGE (4) Imperfectly drained soil. Subdrainage in fair working condition. Surface drainage in good working condition.

REMARKS Differential frost heave at construction joint Sta. 103+00.

DATE February 19, 1960 OBSERVER S. Patton

Figure 28. Department of Transport rigid pavement condition report.

TABLE 6

COMPARISON OF U.S. CORPS OF ENGINEERS' AND THE CANADIAN DEPARTMENT OF TRANSPORT DESIGN METHODS FOR FLEXIBLE AND RIGID PAVEMENTS[1]

Pavement Type	Design Method: Type	Design Method: Agency and Criterion	Pavement Thickness (in.): Asphalt or P.C.C.	Crushed Base	Granular Base	Total	Total Cost ($ million)	Comparison of Costs (%)[2]
Flexible	Strength	Department of Transport (field in place CBR = 2.6, 15% SRF)	4	12	45	61	5.90	100[3]
		U.S. Corps of Engineers (lab soaked CBR = 2.3, 95% Dens.)	4	12	65	81	7.49	127[3]
	Frost	Department of Transport (minimum total thickness)	4	12	18	34	3.76	100
		U.S. Corps of Engineers:						
		Complete protection	4	12	139	155	13.34	355
		Limited frost penetration	4	12	89	105	9.38	250
		Reduced subgrade strength	4	12	47	63	6.06	103[4]
		72-in. total thickness	4	12	56	72	6.76	180
Rigid		Department of Transport (frost protected)	12	6	16	34	6.68	100[3]
	Frost	U.S. Corps of Engineers:						
		Complete protection (concrete strength f = 605 psi)	9	6	140	155	15.35	230
		Complete protection (f = 510)	11	6	138	155	15.94	239
		Limited frost penetration (f = 605)	9	6	90	105	11.38	170
		Limited frost penetration (f = 510)	11	6	88	105	11.99	179
		Reduced subgrade strength (f = 605)	16	6	10	32	7.75	116
		Reduced subgrade strength (f = 510)	18	6	12	36	8.67	130
		Reduced subgrade strength (f = 605, 015% steel)	14	6	10	30	8.95	134
		Reduced subgrade strength (f = 510, 015% steel)	15	6	12	33	9.79	146
		72-in. total thickness (f = 605)	9	6	57	72	8.76	131
		72-in. total thickness (f = 510)	11	6	55	72	9.38	140
		72-in. total thickness (f = 605, 015% steel)	8	6	58	72	9.59	143[3]
		72-in. total thickness (f = 510, 015% steel)	9	6	57	72	10.09	151[3]

[1]Design aircraft = DC-8; 315k at 168 psi.
[2]Department of Transport Cost = 100%.
[3]These designs would probably be used.
[4]Percentage based on DOT strength design equals 100%. Reduced subgrade strength design would not be used as normal strength design requires greater thickness.

Using the two different design methods, a parallel design analysis has been performed for a typical Canadian airfield constructed in 1958. The data given in Table 5 are self-explanatory and point out the considerable difference in pavement thickness requirements and construction costs of the two methods.

It should be pointed out that in the comparison, military and civil requirements are included which might not be fully comparable. There is also considerable difference between U. S. Military and Canadian Civil traffic density.

DATA SHEET

1. Assumed design aircraft.—DC-8 240k at 168 psi D.O.T. and at 121 psi USED
2. Subgrade soil.—CL (clay, silt and stone)
   CBR (measured) = 2.2 (soaked, undisturbed)
   Subgrade fall value × 16.1k (derived from unsoaked CBR tests)
   Spring reduction = 15%

Modulus of subgrade reaction, k $\approx$ 125 pci (derived from unsoaked CBR tests—CBR = 3.9)
Moisture content, Wn = 25% (measured)

3. Freezing Index.—2,736 d.d. 10-yr avg and 3,580 d.d. 10-yr max.
4. Base Course.—Moisture content, Wn = 7% γd = 130 pcf
5. Unit Weights:
   Granular base—γd = 130 pcf
   Crushed stone base—γd = 140 pcf and
   Asphalt—γd = 150 pcf
6. Total area of pavement surface = 8,110 × 10^3 ft?
7. Estimated costs:
   Cost per ton of granular = $1.80
   Cost per ton of crushed = $2.15
   Cost per ton of asphalt = $5.50
   Cost per cubic yard of concrete = $15.34
   Cost per lineal foot of construction joints = $0.20
8. Total footage of construction joints.—Based on all previous construction = 1,000,000 linear ft.

DATA SHEET

1. Design aircraft.—DC-8 315k at 168 psi
2. Subgrade soil.—CL (F-3 frost group)
   Lower quartile point field in place CBR = 2.6
   Lower quartile point remoulded soaked lab CBR = 2.3 (compacted to 95% mod. AASHO density)
   Horizontal variability of subgrade soil conditions taken to be slightly variable.
   D.O.T. spring reduction factor of 15%
   Moisture content 25%
3. Freezing index.—2,736 d.d. 10-yr avg, 3,580 d.d. 10-yr max.
4. Base course properties.—(For determining depth of frost penetration) Density 130 pcf, 7% moisture
5. Unit weights:
   Granular base - 130 pcf
   Crushed stone base - 140 pcf
   Asphaltic concrete - 150 pcf
6. Total area of pavement.—8.11 million sq ft
7. Material costs:
   PCC slab (including cement) = $15.34 per cu yd
   Asphaltic concrete (including bitumen) = $2.50 per ton
   Crushed gravel = $2.15 per ton
   Granular base = $1.80 per ton
   Concrete joints = $0.20 per lf
   Reinforcing steel = $0.15 per lb
8. Total length of concrete joints.—1,000,000 ft.

O. L. STOKSTAD, Design Development Engineer, Michigan Highway Department.—A significant feature of this paper by Linell, Hennion and Lobacz is that it describes design practices for building pavements which will provide uniform service without seasonal load restrictions. It was not too many years ago that highway engineers in areas of seasonal frost accepted spring load restrictions as inevitable. Slowly, as experience and knowledge were gained concerning the use of various soil materials, the selection and processing of free-draining granular material has permitted the economical construction of pavements for all-season use by any design axle load.

Eighteen years ago, when development of the techniques described was started, the first undertaking was to convert earlier highway experience into techniques which would be adequate for airfield needs. This objective has apparently been accomplished without sacrificing significance for the highway engineer. Procedures described not only satisfy airport needs, but they satisfy highway requirements imposed by Michigan soil and climatic conditions quite well.

After 18 years of study, no chemical treatment for frost action has worked its way into standardized practices. Methods described rely on the control of drainage and the selection of suitable construction materials as a means for controlling the detrimental influence of frost action on the character of foundation support. The examples given for both flexible and rigid pavement designs under conditions of frost range widely from a frost index of 600 to 3,000.

Of particular interest to highway engineers is the fact that airport pavement studies involve a much greater range of axle loads than loads to be carried by highway pavements. This fact eliminates the need for extrapolating when using airport criteria concerning axle load weights to be carried. The repetition of axle loads is another matter. In dealing with this subject, highway engineers talk in terms of millions, and airport engineers think in terms of thousands.

To compensate for this difference in load repetition, it has become customary in this area when using U. S. Engineering Department design criteria, to assume that airport wheel loads and highway axle loads are equivalent insofar as pavement strength requirements are concerned.

The authors are to be complimented on the thoroughness with which procedures are described. The paper shows why U. S. Engineering Department manuals serve as excellent guides in developing local pavement design and construction procedures.

K. A. LINELL, F. B. HENNION and E. F. LOBACZ, Closure—The authors wish to thank O. L. Stokstad and G. Y. Sebastyan for their excellent discussions. Mr. Stokstad's observations on the development of frost design technique bring up several interesting points. One of the problems which confronted engineers in converting earlier highway experiments or experience to the design of pavements for military aircraft was spring load restrictions. It was obvious that restrictions could not be placed on military aircraft operations. Therefore, design criteria had to be developed to provide pavements that would accommodate the design load during the several weeks in the spring when the thawing subgrade soils were at their minimum strength. The solution appeared simple—anticipate the amount of traffic that would be applied during the period of subgrade weakening and provide sufficient thickness of non-frost-susceptible base and subbase material to prevent over stressing the weakened subgrade soil. The nub of the problem lies in determining the strength of the pavement components—base, subbase, and subgrade material under variations of temperature, moisture, and soil composition in relation to the effects of load and load repetitions. These factors provided an interesting problem which is still consuming considerable time and effort.

Chemical treatment to preserve strength of soil during periods of thawing has been studied for a number of years. Various chemicals have been found effective in reducing the detrimental effects of frost action. The problem which remains unsolved is the development of a procedure for effectively dispersing and retaining the chemicals in the soil.

The authors were pleased to note that the procedures presented correlate quite well with those of Michigan, a State that has played a leading role in the development of frost design criteria for highways.

Mr. Sebastyan's discussion on the differences in frost protection requirements for airfields under the design procedures presented, and those of the Canadian Department of Transport have been reviewed with interest. Although the authors do not have the detailed design procedures of the Canadian Department of Transport at hand, it appears that basically the differences result from different assumptions of traffic density and the rather stringent requirement of surface smoothness for jet aircraft, especially military jets, incorporated in the Corps of Engineers' requirements.

At first thought it might seem that the relatively colder climate in much of Canada, as compared with the major part of the United States, might possibly be responsible for some of the differences in practice. Long periods of steady, intense cold are, for example, less destructive to pavements with respect to accumulative thaw weakening effects than are climatic conditions involving frequent intermediate cycles of freeze and thaw. The longer thaw weakening period which occurs in areas of deep frost penetration is probably less damaging to pavements than multiple shorter periods in which weakening is concentrated at shallower depths. Also, less ice lens growth per unit depth may be experienced when frost penetrates quite rapidly through the upper part of the subgrade in a very cold region, as compared with the lensing which may develop in the more southerly frost areas where freezing temperatures may barely penetrate the upper layers of the subgrade and advance at very slow rates. Study of freezing index data shows, however, that the ranges of freezing conditions which the design procedures are aimed at in both countries (with the inclusion of Alaska) are not greatly different. Therefore, the procedures should be comparable in this respect.

Highway Pavement Design in Frost Areas in Sweden

FOLKE RENGMARK, Geological Department, National Road Research Institute, Stockholm, Sweden

After a short introduction about the climatic and geologic conditions in Sweden, the classification of the soils into groups differing in susceptibility to frost is described. The fundamental methods for protecting roads against harmful effects of soil freezing are summarized. The bearing capacity in the spring break-up varies from year to year. Consequently, a special bearing value cannot be determined that can be used for highway pavement designing. This must be founded on non-varying factors, especially the type of the subsoil. A table is given for pavement design for roads with traffic of more than 1,000 heavy vehicles daily. Finally, special frost damages, frost cracks, which are quite common in the northern parts of Sweden, are described.

• IN VIEW of the climatic and geologic conditions in Sweden, soil freezing is a very important factor in road construction. In autumn, the rainfall is rather heavy, and this contributes to raising the ground water table to a high level before the beginning of the winter. The winter is cold, particularly in the northern parts of Sweden where the depth of frost penetration on the snow-cleared roads is sometimes about 2.5 m or more. Figure 1 shows the mean freezing index in Sweden, expressed in degree-days (C^0-d).

By far the greatest part of the bedrock in Sweden consists of rocks having an average granitic-quartzitic composition (granites, gneisses, leptites, porphyries, etc.). Disintegrated rock soils are not met with in this country because the products of rock disintegration were removed by abrasive action of glaciation, which occurred in this region during a very late geologic epoch. At the same time, parts of the undisintegrated, sound bedrock were also crushed. These products of mechanical crushing and their restratified sediments constitute the present Swedish soils. On account of what has been said previously, these soils consist mainly of products formed by decay of rocks having a granitic composition.

In some parts of Sweden, however, the bedrock has a different composition where softer rocks (limestone, schist, sandstone) are preponderant. The soils in these areas exhibit corresponding changes in composition.

CLASSIFICATION OF SOILS

Inorganic soils are classified into two main groups, viz., assorted and unassorted soils. These soils are divided into subgroups in conformity with the following particle size scale:

Boulders		>20 cm
Stone	Large stones	20- 6 cm
	Small stones	6- 2 cm
Gravel	Coarse gravel	20- 6 mm
	Fine gravel	6- 2 mm

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

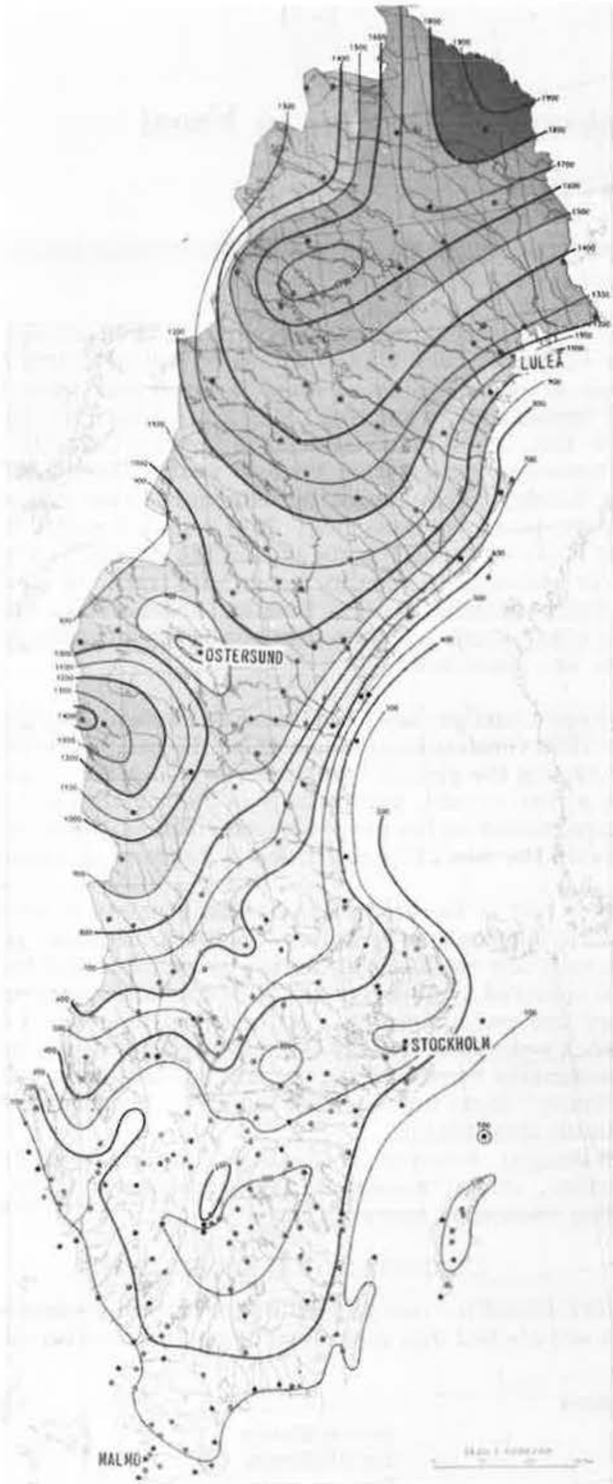

Figure 1. Mean freezing index, degree days (C°-d) for Sweden.

Sand	Coarse sand	2 -0.6 mm
	Medium sand	0.6 -0.2 mm
Mo	Coarse mo	0.2 -0.06 mm
	Fine mo	0.06 -0.02 mm
Silt	Coarse silt	0.02 -0.006 mm
	Fine silt	0.006-0.002 mm
Clay		<0.002 mm

The assorted soils are usually classified according to particle size distribution and clay content. This classification comprises the following distinct types of soils: gravel, sand, coarse mo, fine mo, silt, light clay, light medium clay, heavy medium clay, heavy clay, and very heavy clay.

The unassorted soils consist of moraines, which are classified according to the predominant particle size and clay content. This classification distinguishes between the following types: gravelly, sandy, moey, silty, clayey moraines, and moraine clays. In addition, there is a non-pronounced type, normal moraine, in which no particle size group is preponderant. The various types of moraines are classified on the basis of the clay content and the grading curve. The graphs shown in Figures 2 and 3 are used as an aid in classification.

FROST-SUSCEPTIBILITY OF SOILS

Soil freezing affects inorganic soils to a very high degree by causing a greater or smaller increase in the water content of the soil during the freezing process. This in turn gives rise to a greater or smaller reduction in the bearing capacity of the soil during thaw. However, a small number of soils are not affected in the aforementioned respect by freezing. In consideration of these characteristics, the inorganic soils can be classified according to their frost susceptibility.

I. Non-Frost-Susceptible Soils.

This group comprises the soils which are not liable to frost heaving in the course of the freezing process, and are therefore not softened during the thawing process.

II. Moderately Frost-Susceptible Soils.

This group comprises the soils which are normally subject to frost heaving in the course of the freezing process, and are exposed in this connection to moisture flow towards the boundary of the frozen zone, with the result that a certain quantity of excess water is fixed in the soil. However, these phenomena develop to a relatively great extent only when the rate of freezing is low or when the depth to the ground water table is small. During the thawing process, the soils belonging to this group are more or less softened by the liberated excess water according to the conditions under which the freezing process has taken place (high or low ground water table, slow or rapid soil freezing).

III. Highly Frost-Susceptible Soils.

This group comprises the soils in which the moisture flow towards the boundary of the frozen zone during the freezing process is considerable under normal conditions and very great if the ground water table is high. When the excess water fixed during this process is liberated in the course of thawing, it generally causes a great reduction in the bearing capacity of the soils belonging to this group. Because these soils are highly susceptible to conversion to a liquid condition, even a relatively small quantity of water can give rise to a considerable decrease in bearing capacity.

To sum up, the inorganic soils can be classified according to their frost susceptibility as follows:

Assorted soils	Frost-susceptibility group
Gravel Sand Coarse mo	I

Assorted soils	Frost-susceptibility group
Fine mo	III
Silt	III
Light clay	III
Light medium clay	III
Heavy medium clay	II
Heavy clay	II
Very heavy clay	II

Unassorted soils	Frost-susceptibility group
Gravelly moraine	I (-II)
Sandy moraine	II (-I)
Normal moraine	II
Sandy moey moraine	II
Moey moraine	III
Silty moraine	III
Clayey moraine	II (-III)
Moraine clay	II

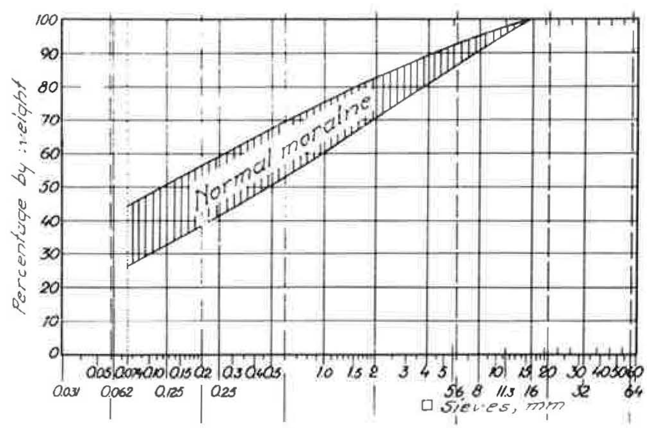

Figure 2. Grading of ordinary moraine soil (normal moraine) common in Swedish archaic rock areas. After G. Beskow.

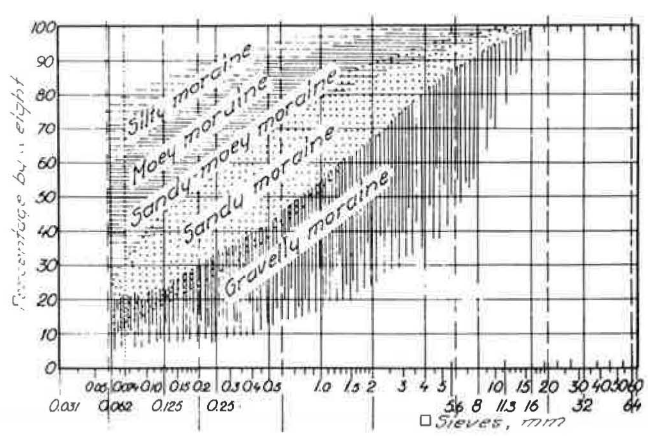

Figure 3. Grading of moraine types differing from the well-graded normal moraine by dominance of special fractions. After G. Beskow.

It follows that most inorganic soils in Sweden are frost-susceptible. Moreover, the geographic extent of the non-frost-susceptible soils in Sweden is relatively small. Therefore, soil freezing is of great importance in road construction.

PRINCIPAL METHODS FOR PROTECTING ROADS AGAINST HARMFUL EFFECTS OF SOIL FREEZING

The water absorption which takes place during the process of freezing results primarily in frost heaving, which can render a road surface so uneven as to cause trouble to traffic. This happens in areas where the soil conditions are non-uniform, chiefly in areas of transition from rock to frost-prone soil, and also at culverts. When thaw begins in the spring, the increased water content of the soil due to freezing gives rise to reduction in the bearing capacity of the road. Because the increase in the water content which takes place during soil freezing is the primary cause of the harmful effects produced by soil freezing on roads, various principal methods have been devised with a view to reducing or preventing the absorption of water due to moisture flow towards the frost line. These methods are briefly outlined (in the main according to G. Beskow) as follows:

I. Methods for Leveling Non-Uniform Frost Heave.
 A. Insertion of sand wedges in areas of transition from rock to frost-susceptible soil, and wedge-shaped tapering of culvert insulation.
 B. Insertion of V-shaped heat-insulating layer under the subbase of a road for the purpose of preventing frost cracks.

II. Methods for Reducing or Preventing Frost Heave.
 A. Methods for retarding or preventing the freezing of frost-susceptible soils.
  1. Spreading of non-frost-susceptible inorganic soil on top of the frost-susceptible subgrade. This method augments the thickness of the road construction, and hence increases the bearing capacity of the road.
  2. Spreading of heat-insulating materials (bark, peat) on top of the frost-susceptible subgrade. At the same time, this method reduces the depth of frost penetration.
  3. Replacement of frost-susceptible soil (excavation) with non-frost-susceptible soil.
 B. Methods for reducing or preventing water absorption from below during soil freezing.
  1. Breaking the capillary connection with the overlying backfill made of frost-susceptible soil by means of porous insulating layers (sand), and with the aid of impervious insulating layers (asphalted felt).
  2. Increase the depth of the ground water table below the carriageway by lowering the ground water table by deep drainage, and by raising the carriageway level by spreading materials on top of the existing road.
  3. Reduction of the thickness of the water films adsorbed on soil particles by load application (banking up), and by chemical stabilization.

III. Methods for Reducing or Preventing Harmful Effects of Excess Water Liberated During Thaw.
 A. Increase of the bearing capacity of the carriageway by augmenting the thickness of the base and the subbase. On gravel roads, this method can be supplemented with adjustment of the composition of the gravel course on the carriageway.
 B. Methods for removal of water.

Insulation by means of sand layers, which cut off the capillary flow, and lowering of the ground water table by deep drainage have formerly been used on a large scale, in addition to other methods, for prevention of damage due to soil freezing. For economic reasons, frost-susceptible soils from the subgrade were employed as backfill on the insulating sand layers.

In order to illustrate the efficiency of this method, it may be mentioned that a test road section (Fig. 4) was constructed in 1950 on National Main Road No. 13, which handles heavy and dense traffic. A layer of highly frost-susceptible silt (morainic soil

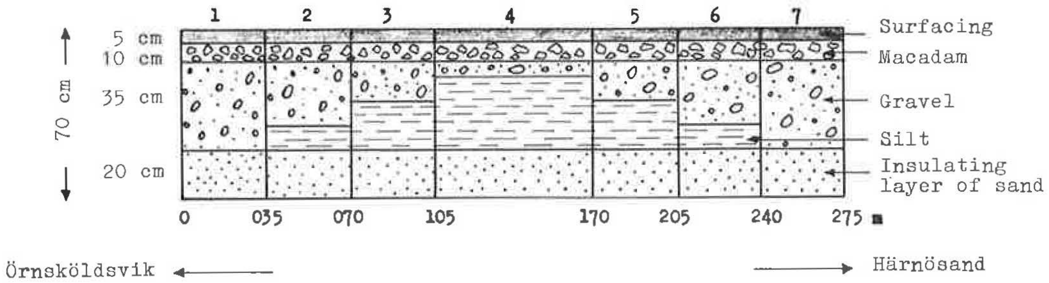

Figure 4. Road construction of the test road at Bjästa, Sweden.

was normally used) varying from 10 to 30 cm in thickness was placed as backfill on an insulating sand layer 20 cm in thickness. For the rest, the road construction comprises a gravel subbase, a broken stone base course 10 cm thick, and an asphalt surfacing 5 cm thick. The total thickness of the road construction is 70 cm. The insulating layer is made of sand with a well-assorted composition (Fig. 5). However, its content of fines varies within relatively wide limits. The subgrade of the test road section consists of silt.

After the construction of the test road section, samples have been taken at different times, distributed over several seasons, in order to check the water content of the

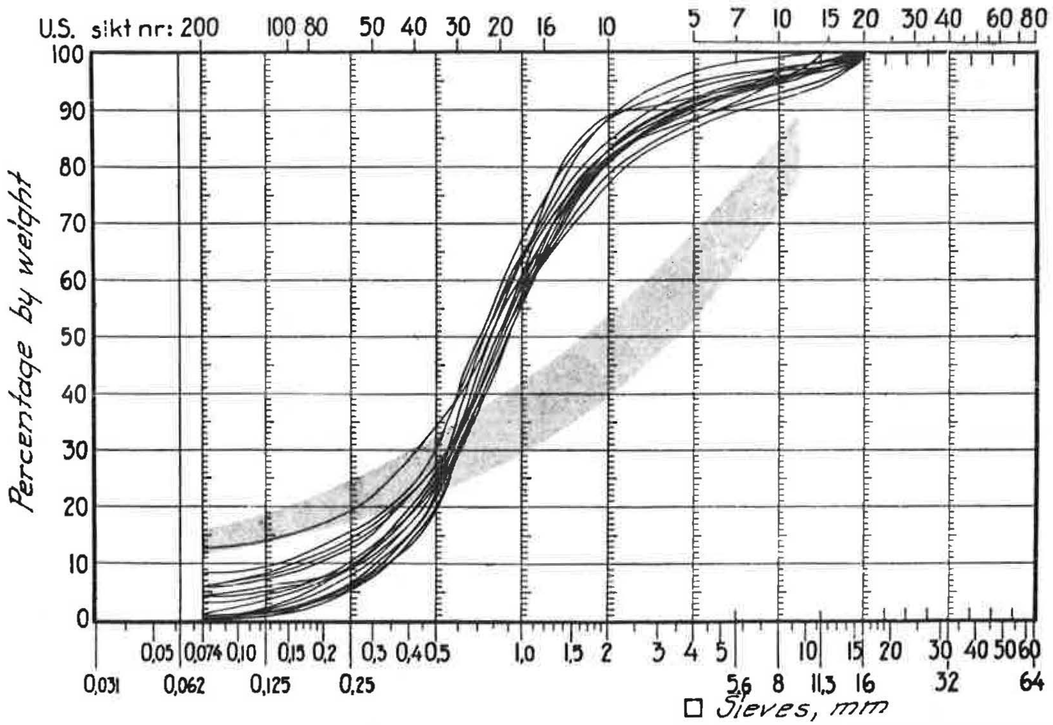

Figure 5. Grading curves for sand to the insulating layer in the bottom of the road construction of the test road at Bjästa.

insulating layer and the superposed silt layer employed as backfill. The water content of the insulating layer varied from 0.7 to 1.6 percent. The water content of the silt layer, which averages about 20 percent or slightly higher, has decreased in the course of several years by a few percent in comparison with the water content at the time of construction of this test road section. No increase in the water content was detected when these layers were frozen. The test road section had to carry heavy and dense traffic in the course of the past 12 years. Nevertheless, the test road section did not exhibit any frost damage. It may therefore be stated that the insulating layer used on this road has produced the intended effect of completely cutting off the capillary flow, with the result that the increase in the water content of the backfilled silt layer due to soil freezing was entirely prevented.

It is important to observe that the insulating sand layer should be placed on a certain definite level above the bottom of the side ditch so that the water cannot infiltrate the insulating layer. Some failures have occurred owing to soil flow solifluction from the slopes into the side ditches, with the result that water percolated into the insulating layer. Therefore, this method is now used on small roads only, whereas the base and subbase of other roads, particularly those which are surfaced, consist wholly of materials which are not frost-susceptible.

DETERMINATION OF TOTAL THICKNESS OF SURFACING, BASE, AND SUBBASE

The reduction in the bearing capacity of the frost-susceptible soils which takes place during thaw is dependent not only on the character and density of traffic, but also on many other factors, primarily on the type of soil, rate of freezing, ground water table, type of climate, heat insulation due to snow cover, and topographic conditions. Most of these factors can vary not only during the same freezing season, but also from one freezing season to another. The bearing capacity during the thawing period at the same point on a road or in the adjacent area can therefore vary considerably from one year to another, and also from time to time during the same period of thawing. Accordingly, it is difficult to obtain from measurements a correct actual value of the bearing capacity of a frost-susceptible soil which can be used as a basis for determining the total thickness of the surfacing, the base course, and the subbase. In particular, when new roads are to be constructed, the determination of the bearing capacity of a frost-susceptible subgrade is of little importance, because the value which is observed at that time will perhaps not agree with the bearing capacity of the soil at the time when the construction of the road is completed. Furthermore, the heat-insulating effect of snow cover can completely prevent freezing. The value of the bearing capacity observed under such conditions gives wholly misleading information on the true bearing capacity.

For practical purposes, it is therefore necessary to estimate the bearing capacity during the period of thawing on the basis of the non-varying factors, such as the type of soil and the features of the area where the road is to be built. On the basis of these factors and previous experiences concerning the requisite total thickness of the surfacing, the base, and the subbase on various soils, five different design tables have been prepared to take into account the traffic density and the type of wearing course. The requisite total thickness of the surfacing, the base, and the subbase is given in Table 1, which applies to roads with bituminous surfacings handling a traffic of at least 1,000 heavy vehicles (lorries and buses) per day during the period of thawing.

The total thickness of the base course and the bituminous surfacing should be 25 cm. The base course consists either of macadam or crushed gravel. If made of macadam, its surface is bound with bitumen as a rule. If made of gravel, its grading curve should run between, and conform to, two limiting curves. These limiting curves imply that 0 to 10 percent should pass the 0.074-mm sieve, 15 to 45 percent the 2-mm sieve, and 45 to 95 percent the 16-mm sieve.

The subbase consists of gravel, sand, or some other material which is not frost-susceptible. These materials should not lie nearer to the road surface than is indicated in the design table. Broken stone and the like may also be used, but in such cases a

TABLE 1

TOTAL THICKNESS OF SURFACING, BASE AND SUBBASE

Type of Soil (in subgrade or embankment)	Frost Suscep-tibility Group	Height of Road Surface Above Ground Level (cm)	Minimum Total Thickness of Surfacing, Base, and Subbase (cm)
Gravel Very gravelly moraine	I	–	–[1]
Sandy gravel Gravelly sand Very sandy moraine	I	–	25
Sand and coarse mo	I	–	35
Gravelly moraine Sandy moraine Normal moraine	II	Less than 30 or in cuts	60
		30 or more	40
Sandy moey moraine Clayey gravelly moraine Clayey sandy moraine Clayey normal moraine Moraine clay	II	Less than 40 or in cuts	70
		40 or more	50
(Clayey) moey moraine (Clayey) silty moraine Fine mo Silt Lighter clays	III	Less than 70 or in cuts	80
		70 or more	70
Heavier clays: Dry crust	II	Less than 40 or in cuts	70
		40 or more	50
Heavier clays: Very soft clay	II	–	90
Peat and mud	I	–	100

[1] The surfacing can be laid directly on this material if it meets the specifications for base courses.

filter layer of sand, 15 cm thick, should be inserted under the subbase if the subgrade consists of clay, peat, or some other highly frost-susceptible material.

FROST CRACKS

In recent years, longitudinal frost cracks (Fig. 6) have appeared mostly in new-built roads, especially in the northern parts of Sweden. These cracks are usually formed fairly early in the frost period, i.e., when the roads have a high bearing capacity. The cracks increase in width and length as the frost penetrates into the subgrade. Gen-

Figure 6. Frost crack in a road near Ersnäs.

erally, the cracks run along the approximate center line of the road or on either side of this center line.

As found from investigations, a necessary condition for the formation of frost cracks is that the road be built on a subgrade which is liable to frost heave and that the depth of frost penetration at the center of the road be greater than on the sides. As has also been shown, this uneven frost depth is due to the fact that the insulation due to snow cover at the center of the road is much less than on the sides. Consequently, the depth of frost penetration at the center of the road is greater than on the sides (Fig. 7). The depth of frost penetration at different times has been determined by the aid of a simple frost depth indicator (Fig. 8) described by R. Gandahl in the Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering. This indicator is filled with a blue-colored indicating solution. When the frost penetrates the ground, this instrument reacts so that the color of the solution changes to colorless and transparent in the frozen zone. The difference between the frozen and unfrozen parts is distinct (Fig. 8) and the depth of frost penetration is easily read from the graduated indicator tube.

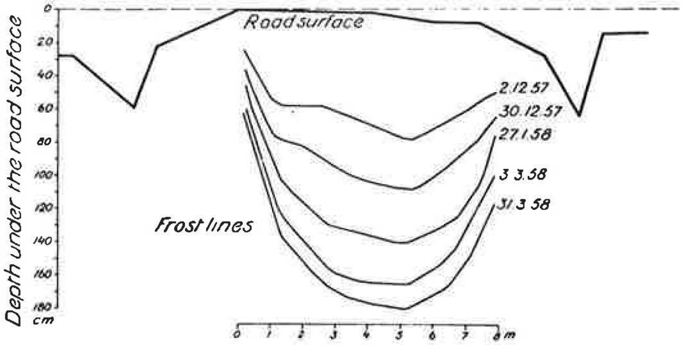

Figure 7. Frost lines at different dates during the freezing period 1957-58 in a road near Rutvik.

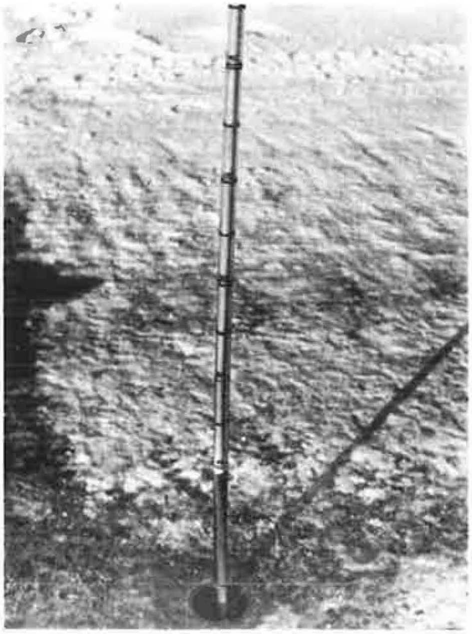

Figure 8. Frost indicator (model Gandahl) in the position of observation. In this case the frost depth is 141 cm.

A frost heave that is greater at the center of the road than on the sides may cause bending and tensile stresses in the road construction, and may thus involve the risk of cracking. This risk seems to be particularly great when the subbase consists of coarse-grained gravel, which does not retain any water in the pores, and therefore lacks adequate tensile strength in both frozen and unfrozen condition. In order to show this, the strength of some soil materials in frozen condition has been determined. The equipment used for this purpose is shown in Figure 9. The grading curves of the materials under test are reproduced in Figure 10, and the test results are given in Figure 11. It is seen from these graphs that the coarser material, Type 1, which did not contain any fine-grained particles, did not exhibit any strength in a frozen condition because no water was retained in the coarse pores, thus the material was not rendered cohesive during the freezing process.

In the tests on the other samples, the strength first increased with the increase in the water content up to a maximum and then decreased until it reached the strength of the ice itself. The maximum strength of the samples first increased as the content of fines became higher, and then decreased after the content of fines had reached a certain definite value. The highest maximum strength was observed in the case of Type III material, whose grading curve is in the main close to the ideal composition of gravel-wearing courses. For comparison, it may be mentioned that a test specimen made of concrete, whose composition corresponded to that of a concrete pavement mix, had a strength which was about half as high as that of Type III material.

In spite of the fact that a very high strength of soil materials in frozen condition can be obtained by adjusting their composition, this strength is probably not sufficient to prevent the formation of frost cracks in roads on account of the high stresses which are produced in the road construction when the frost heave is uneven. This adjustment of the composition can only reduce to some measure the liability to crack formation. To increase the total thickness of road construction to such an extent as to prevent frost crack formation is probably not recommendable for economic reasons (cracks have occurred in roads whose total thickness of construction was 1.6 m). Because frost crack formation is primarily due to the fact that the depth of frost penetration at the center of the road is greater than on the sides, it is possible to prevent crack formation by either increasing the depth of frost penetration on the sides by thorough snow clearing, or by reducing the depth of frost penetration at the center by using a heat-insulating layer. The latter method has proved to be particularly efficient.

On some Swedish test roads, a V-shaped layer of bark (or peat) has been provided under the subbase (Fig. 12). The thickness of the bark layer at the center of the road is 25 cm. During the frost season of 1960-61 when the freezing index at the time of maximum depth of frost penetration was 1,322 degree-days (C^{0}-d), the maximum depth of frost penetration on four road sections equipped with bark layers varied from 170 to 200 cm. On the other hand, the corresponding maximum depth of frost penetration on intermediate road sections, which are of a conventional type comprising a sand subbase and a gravel base course, varied from 230 to 260 cm. The difference in the maximum depth of frost penetration between the center and the edges of the road on the test sections with bark layers was negative. The greatest value of this difference was -13 cm.

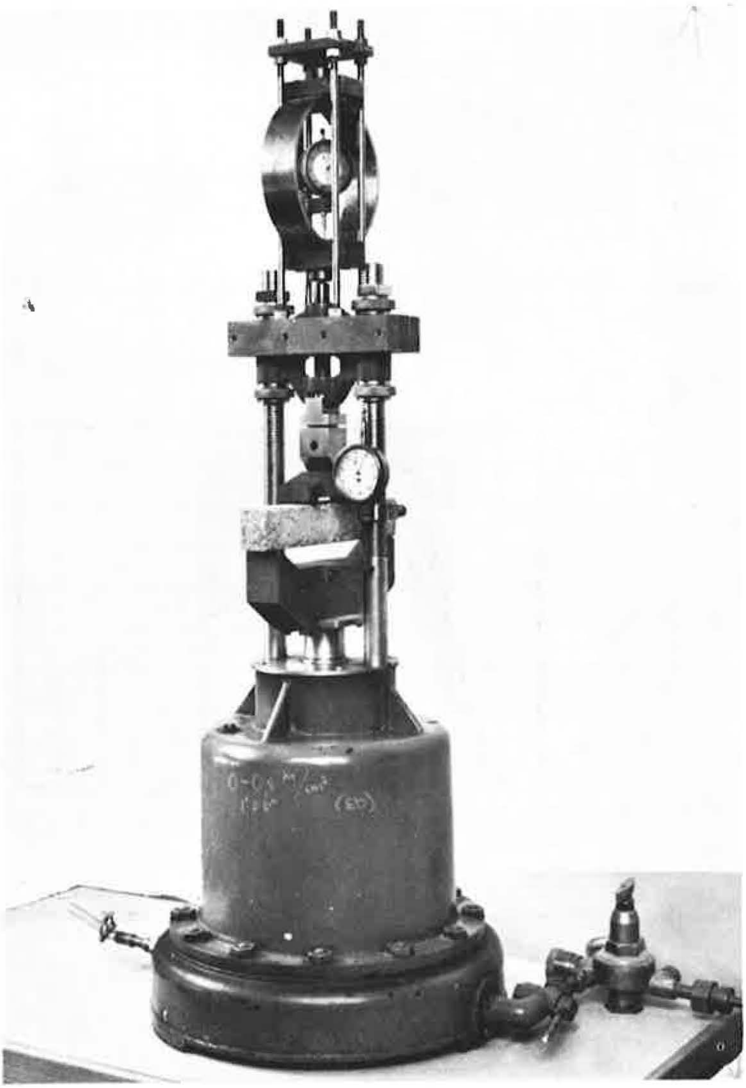

Figure 9. Pressure apparatus for determining the bending-tensile strength of soil samples in frozen conditions.

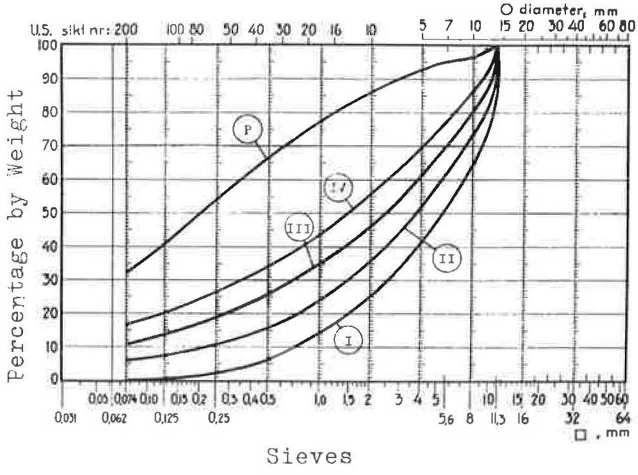

Figure 10. Grading curves for soils tested with respect to the bending-tensile strength in frozen condition.

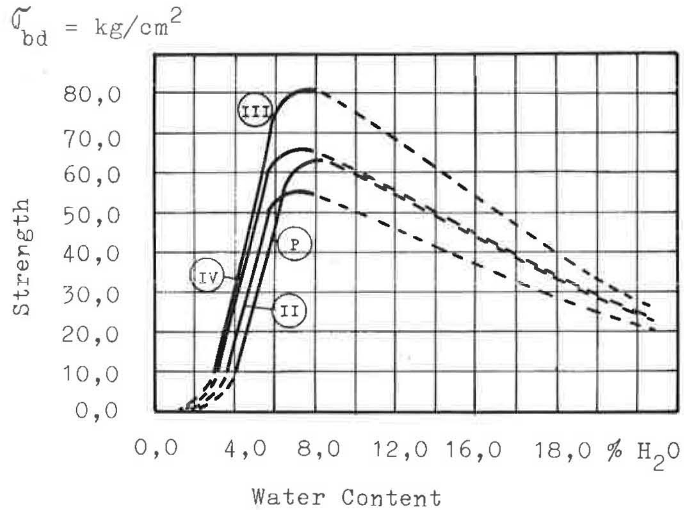

Figure 11. Bending-tensile strength for frozen soils in relation to moisture content.

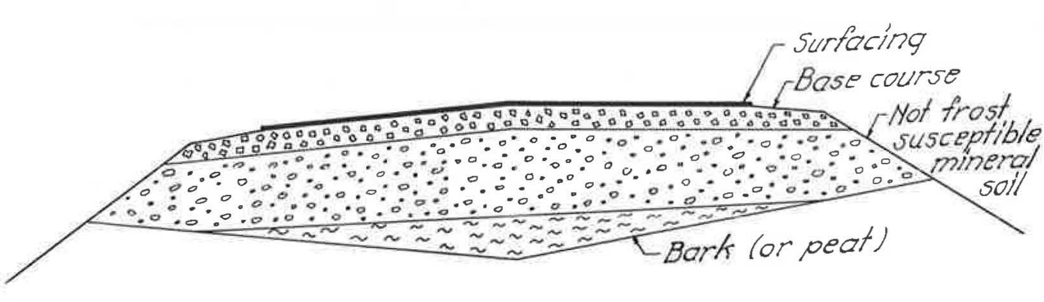

Figure 12. Road construction with V-formed layer of bark. In the center of the road, the thickness of the bark layer is 25 cm and the thickness of the remaining part of the road construction is 80 cm.

This means that the maximum depth of frost penetration at the center of the road in the present case was 13 cm smaller than at the edges of the road. The corresponding value for one of the test sections without bark layers was +20 cm, that is to say, the maximum depth of frost penetration at the center of the road was 20 cm greater than at the edges. It should be noted that no frost cracks have formed on the road sections with bark layers during the frost seasons of 1960-61 and 1961-62, whereas frost cracks have occurred during the latter frost season on road sections without bark layers.

Appendix

After the preceding paper was prepared the National Swedish Road Research Institute proposed the following new tables for pavement design.

Type of wearing course			Gravel	Bituminous surfacing					Concrete
Design table			1	2	3	4	5	6	7
Daily traffic volume (no. of vehicles)									
Heavy traffic (lorries and buses) during the period of thawing			<50	<50	<250	<1000	<3000	>3000	
or									
Total average daily traffic in summer (June - Aug.)			<500	<500	<2500	<10000	<30000	>30000	
Type of soil (in subgrade and embankment)			Minimum total thickness of surfacing, base and subbase, cm						
A	Gravel Sandy gravel Gravelly moraine Sandy moraine	 I[1] I I	15	15	20	25	30	35	25
B	Gravelly sand Sand Coarse mo	I I I	20	25	30	35	40	45	25
C	Gravelly moraine Sandy moraine Normal moraine	II[2] II II	30	40	50	60	70	80	60
D	Sandy moey moraine Clayey moraine Moraine clay	II II II	40	50	60	70	80	90	70
E	Moey (silty) moraine Fine mo and silt Lighter clays Heavier clays (dry crust)	III[3] III III II	50	60	70	80	90	100	80
F	Heavier, very soft clays Peat and mud	II I	60	70	90	100	110	120	80[4]

[1] I = not frost-susceptible soil.
[2] II = moderately frost-susceptible soil.
[3] III = very frost-susceptible soil.
[4] Concrete is not recommended on peat and mud.

Discussion

I. C. MACFARLANE, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Canada.—One of the methods recommended by Mr. Rengmark for reducing or preventing frost heave is the spreading of heat-insulating materials, such as bark or peat, on top of the frost-susceptible subgrade. He points out that this also reduces the depth of frost penetration. It is interesting to observe that Skaven-Haug in his paper on protection against frost heaving on the Norwegian railways (1) also refers to this unusual use of peat. He shows very clearly that a layer of wet, compressed peat overlain by a dry bearing layer is extremely frost retarding. It also considerably reduced the depth of excavation required in the removal of frost-susceptible soil and subsequent replacement with non-frost-susceptible material. Miyakawa reports that this technique has also been followed in Japan with some success (2).

In Canada, this technique has rarely, if ever, been attempted. Rather, peat is generally considered to be somewhat of a liability, and great pains are usually taken to remove it from a proposed road right-of-way. The suggestion of deliberately incorporating a layer of peat into a roadway subgrade may be met with considerable scepticism by many engineers.

In the course of an investigation in northern Ontario to evaluate the performance of roads over muskeg (3), the writer observed that roads floated directly on the muskeg exhibited much less damage from the effects of frost action than did adjacent sections of road over clayey and silty soils. Even further north—in northern Alberta, British Columbia and the Territories—frost-susceptible soils and consequent difficulties are encountered to a considerable extent in the construction of the "roads to resources," which essentially are secondary roads designed for a low density of very heavy loads.

In many areas non-frost-susceptible soils are difficult to locate. One wonders if in cases such as this—and for secondary roads in general on frost-susceptible soils—it would not result in lower maintenance costs in the long run if the Scandinavian practice of incorporating a layer of wet, compressed peat into the roadway were followed. A corollary of this would be deliberately to site roads over muskeg areas where feasible rather than over inorganic frost-susceptible terrain.

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1. Skaven-Haug, S., "Protection Against Frost Heaving on the Norwegian Railways." Geotechnique, Vol. 9, No. 3, pp. 87-106 (Sept. 1959).
2. Miyakawa, I., and Koyama, M., "On the Residual Frost Subgrade Underneath the Select Fill in the Alleviation Practice for Frost Damage." Soil and Foundation, Vol 3, No. 1, pp. 10-18 (Sept.1962).
3. MacFarlane, I. C., and Rutka, A., "An Evaluation of Pavement Performance over Muskeg in Northern Ontario." HRB Bull. 316, pp. 32-43 (1962).

F. C. BROWNRIDGE, Special Assignments Engineer, Ontario Department of Highways.—Mr. Rengmark's report is interesting to highway engineers in Ontario because of the similar environment problems and his empirical approach to pavement design. He describes several experiments in highway construction in frost areas which have not been tried here.

Although Sweden lies entirely north of latitude 56^0, or approximately that of Hudson Bay, its temperature is decidedly moderated by its maritime character, its relatively low elevation and the influence of the Gulf Stream. The combination of these factors result in a freezing index for lower Sweden which compares with that of southern Ontario.

EMPIRICAL PAVEMENT DESIGN

In Table 1, Mr. Rengmark gives the minimum total thickness design for flexible pavements built on various types of subgrade soil, classified as to their frost susceptibility. Comparing this with the current Ontario design for the same conditions, the latter was from one to seven inches thicker with the greater depths required for the Group III soils. For Group I, the additional depth was from one to three inches, and for Group II, from three to six inches greater. When a crushed-rock subbase material is used in Sweden, an additional 5.9 in. of sand filter layer is employed where the subgrade is either clay, peat or silt. For these cases, the total pavement thicknesses would be approximately the same.

USE OF SAND-INSULATING LAYERS TO CUT OFF CAPILLARY FLOW

The use of cut-off sand layers for the test sections of National Main Road No. 13, (Fig. 4) appears to have been effective. There was concern over the possible saturation of the silt used for the embankment due to the entrance of surface run-off. On inquiry, Mr. Rengmark stated that the full width of the road surface was paved.

Unless granular material was very scarce we would hesitate to recommend this method of construction in frost areas. It is noted in his last paragraph that some failures have occurred and that present practice utilizes non-frost-susceptible bases except for minor roads.

FROST CRACKS

Mr. Rengmark refers to the recent occurrence of longitudinal frost cracks appearing mostly in new construction and especially in the northern parts of Sweden. His investigations show a necessary condition for their formation is a frost-susceptible subgrade with the frost penetration greater at the center than at the sides. The risk of cracking from unequal heaving appears greater when the subbase consists of coarse-grained gravel with little fines, and has low strengths in a frozen or unfrozen condition.

In northern Ontario, transverse cracking has occurred on new construction in a very irregular pattern. On the same contract and for the same design, cracks have been noted at 1,500 to 2,000 ft spacing for one area, while on another section they have been observed as close as 200 ft apart. Several years ago, adjoining Provinces and the more northern States were circularized in regard to transverse cracking in flexible pavements. Replies indicated that although these had been observed, they were not considered a serious enough problem to justify any concerted action, although all agreed they were related to severe frost conditions.

In Ontario, it is economically unfeasible to design for the full depth of frost penetration and the complete elimination of frost heaving. The general practice is to use non-frost-susceptible materials and sufficient depth so that the subgrade soil will not be over-stressed during the critical spring period. Frost heaving is, therefore, accepted with the design aimed at obtaining uniform heaving and preventing differential heaving as much as possible. Experience indicates that more effective and economical results can be obtained by the control of moisture through efficient drainage than by the removal of frost-susceptible soils or the use of increased depths of granular subbases.

FROST CRACK INVESTIGATION IN ONTARIO

Mr. Alex Rutka, Materials and Research Engineer, has furnished information on a long-term investigation of bituminous pavement cracking in Ontario which started in 1961. It identifies the specific conditions of pavement, granular bases, subgrade soil, moisture and temperature regime, frost-depth penetration, etc.

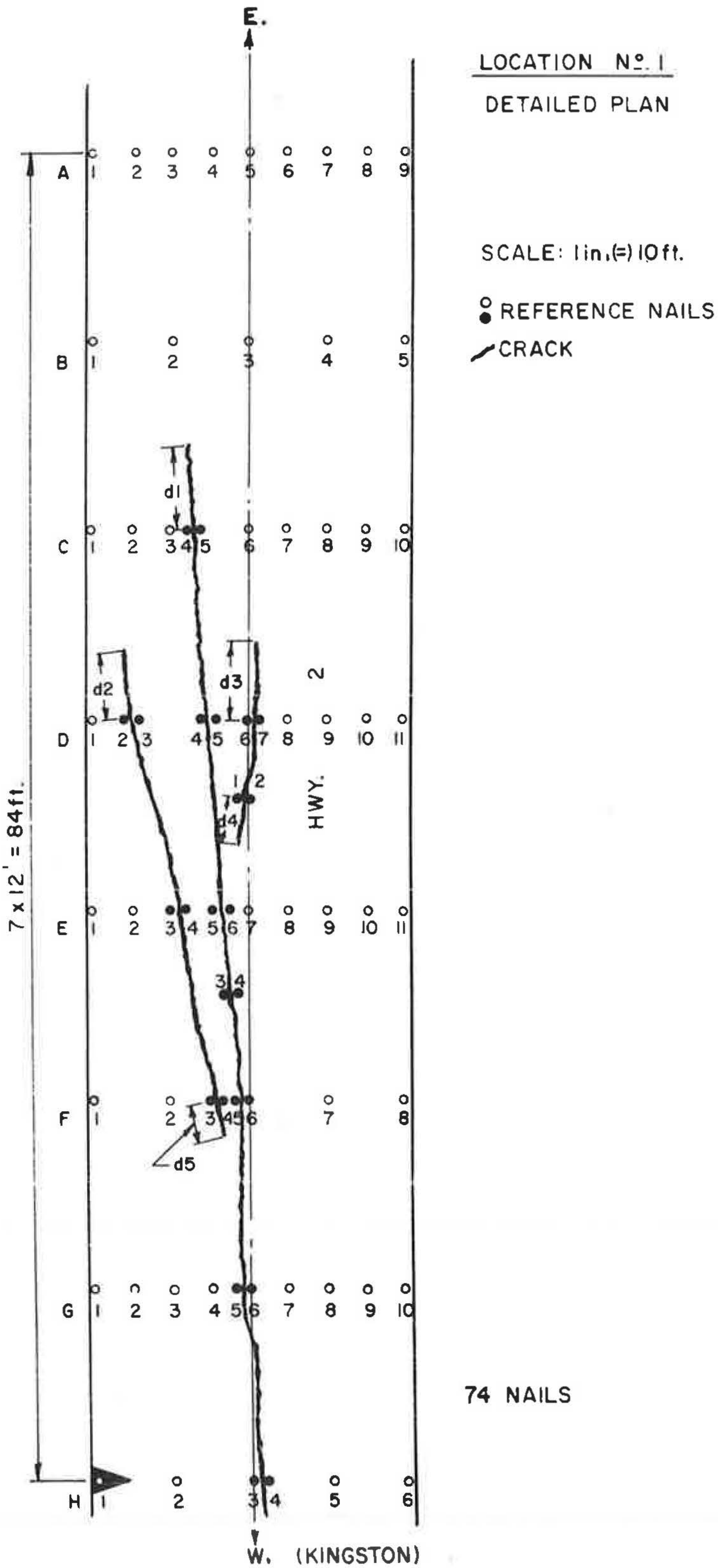

Figure 13. Typical installation of bench marks and reference nails.

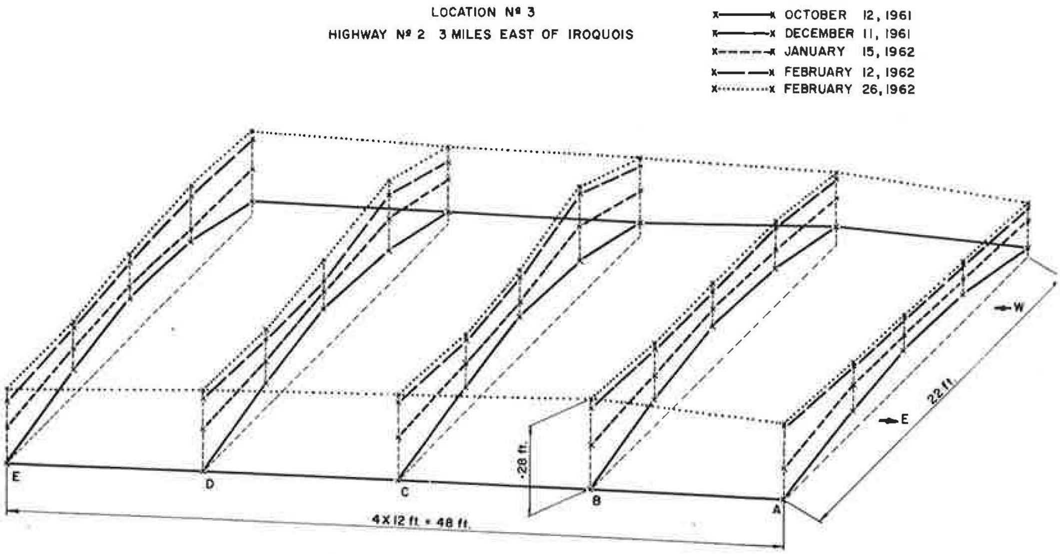

Figure 14. Vertical movements of pavement surface.

Twenty test sections, each of 100 ft length, were selected as representative of transverse, longitudinal, slippage, shrinkage and alligator cracking. The selection varied for geographic and topographic conditions, as well as sub-soil, moisture and traffic intensities. Permanent bench marks were installed and reference nails driven into the pavement in a pre-established pattern at each site (Fig. 13). The elevation of each reference nail and the distance between those shown as full black dots were taken on a weekly or fortnightly basis. The elevation records are plotted on a three-dimensional graph (Fig. 14).

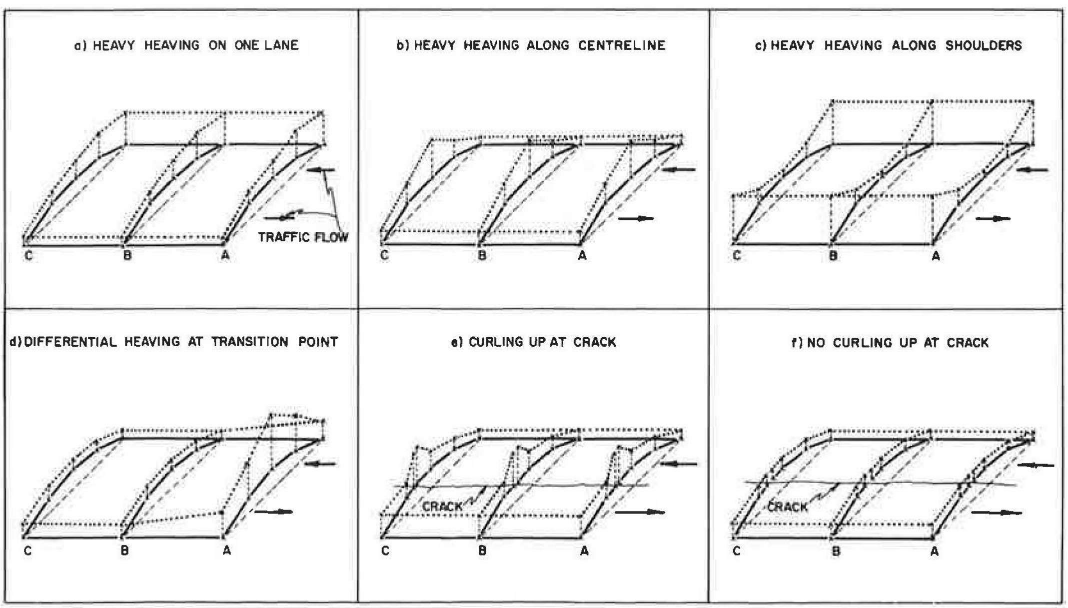

Figure 15. Patterns of vertical movement.

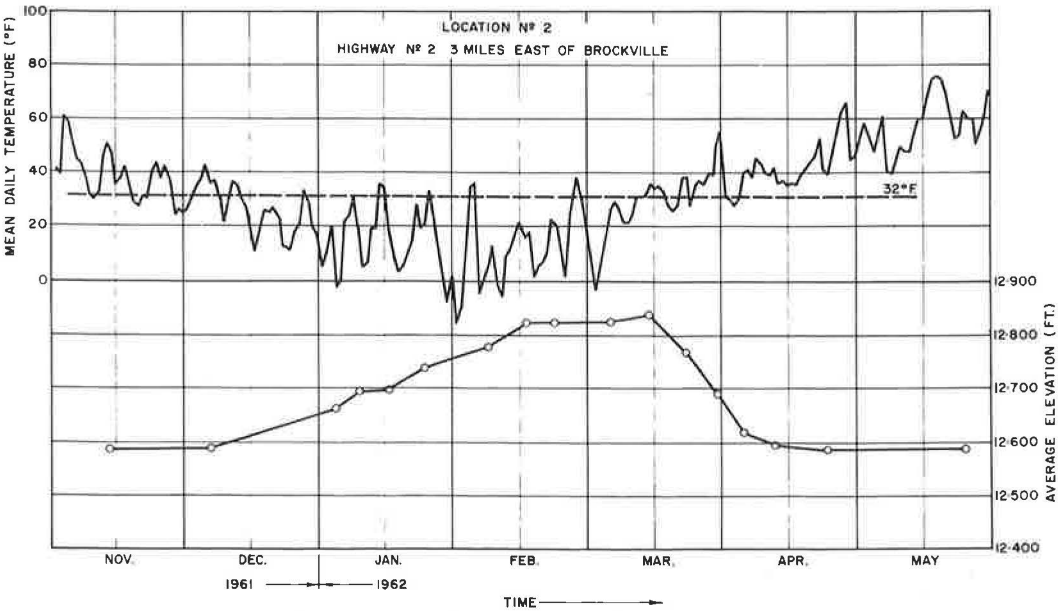

Figure 16. Average elevation vs daily mean temperature.

Although the investigation is not yet complete for all the test sites, characteristic patterns of surface movements have been observed. These patterns with the probable causes are shown in Figure 15.

Temperature and precipitation data are being obtained from weather stations in the vicinity of the test sections. An attempt will be made to analyze these data with observed surface movements. One such graph is shown in Figure 16. An analysis of graphs of this kind promises information on frost heaving concerning the role of temperature and the time-lag between temperature changes and the resulting vertical movements. The heaving process starts about two weeks after the temperature drops below the freezing point; is gradual in spite of significant temperature variations within the freezing range; is reversed at almost the same time the temperature again rises above 32 F in the spring; and the surface quickly settles down to its original level in about a month's time.

WILLIAM J. RAMSEY, Sr. Geologist, Division of Materials & Tests, Nebraska Department of Roads.—The report by Mr. Rengmark is an excellent summary of Sweden's highway design practices in frost areas, and this writer wishes to congratulate him for his endeavor. It was noted in his report that Sweden has experimented with the construction of an insulating layer of sand near the surface of a silt-clay subgrade. The result was the placement of a thin layer of silt-clay material over a sand material immediately below the base course. In Nebraska, a similar type of construction was detrimental to the performance of the flexible pavement.

A project of this nature, which is located on Nebraska Highway 44 between its junction with U. S. 6 and Kearney, Neb., is used as an illustration.

Prior to recent reconstruction, this road was last graded and surfaced in 1941. The flexible pavement surfacing consisted of a 3-in. × 27-ft soil stabilized base course and armor coat (Fig. 17).

In 1959 a subgrade survey was conducted. From this investigation, the project was divided into the following two sections of different soil types:

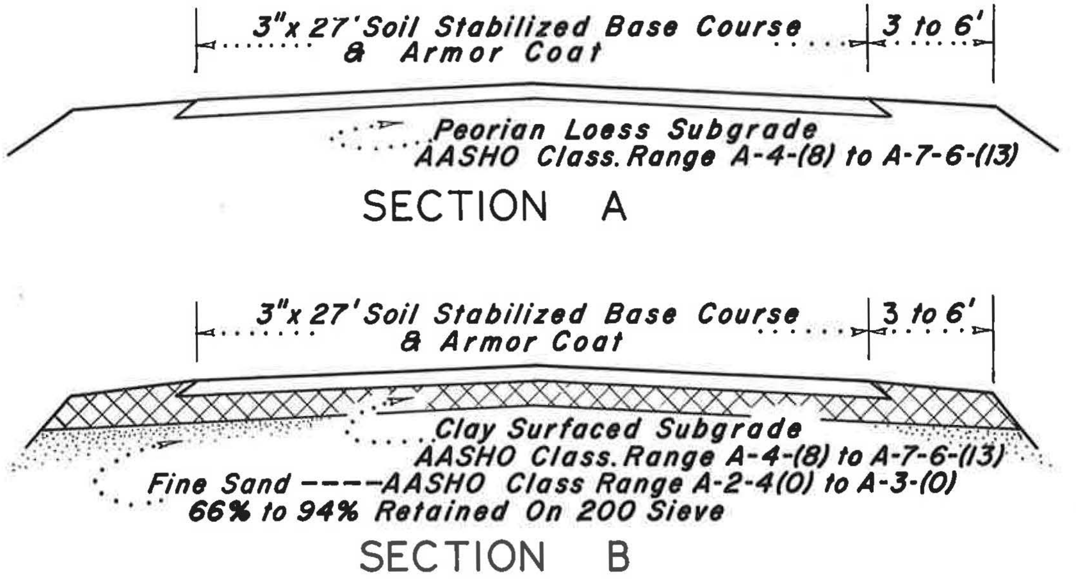

Figure 17. Typical cross-section of 1941 construction.

(A) The subgrade of the south 6½ miles consisted of silt-clay soils of Peorian loess origin (Fig. 1).

(B) The north 5 miles traversed a sandhill and alluvial sand section in which small pockets of Peorian loess and alluvial silts and clays were encountered. A layer of cohesive soil was used as a clay-surfacing material and had been placed over the sand subgrade (Fig. 1). This layer was approximately 4 in. thick.

Table 2 gives the minimum and maximum range of the engineering characteristics of the soils encountered during the subgrade survey. It should be noted that the Peorian loess subgrade material in section A and the material used as clay surfacing in section B have engineering characteristics that are similar.

TABLE 2

RANGE OF ENGINEERING CHARACTERISTICS OF SUBGRADE SOILS

Plasticity Index	Hydrometer Analysis		Sieve Analysis % Retained		AASHO Classification
	Silt 0.074-0.005	Clay -0.005	No. 10	No. 200	
		Section A (Peorian loess subgrade)			
7-21	36-53	18-41	0-7	1-23	A-4(8) to A-7-6(13)
		Section B (Clay-surfaced subgrade)			
5-21	38-59	14-28	0-3	2-27	A-4(8) to A-7-6(13)
		(Dune & alluvial sand)			
NP	3-20	2-8	0-10	66-94	A-2-4(0) to A-3(0)

During the field investigation, it was observed that considerably more maintenance was required in section B than section A. A review of the maintenance records which show the patching and repair accomplished between 1941-58 confirmed this.

From these records, Figure 18 was prepared. Note in section A that the length of patching per mile or portion thereof varies from approximately 700 ft (mile 1 to 2) to 4,700 ft (mile 6 to 6.5), or an average of slightly over 2,000 ft per mile. However, in section B the length of patching varies from about 7,400 ft between mile 11 and 11.5 to 13,700 ft between mile 8 and 9. This section has an average length of patching per mile of approximately 11,000 ft.

In the course of the field investigation, observations of the total thickness of the bituminous material were made (Fig. 19). These measurements include those sections where only an armor coat surfacing was encountered, as well as those sections which had bituminous patches applied. It is apparent that a thicker build-up of bituminous material has occurred in section B than section A. The average thickness of the bituminous material in section B is over 3 in., but the average thickness in section A is less than $1\frac{1}{2}$ in.

Admittedly, the 1941 design of this project was not adequate for today's traffic. However, because the flexible pavement design in both sections is the same, the following conclusions seem to be justified on the basis of the data presented:

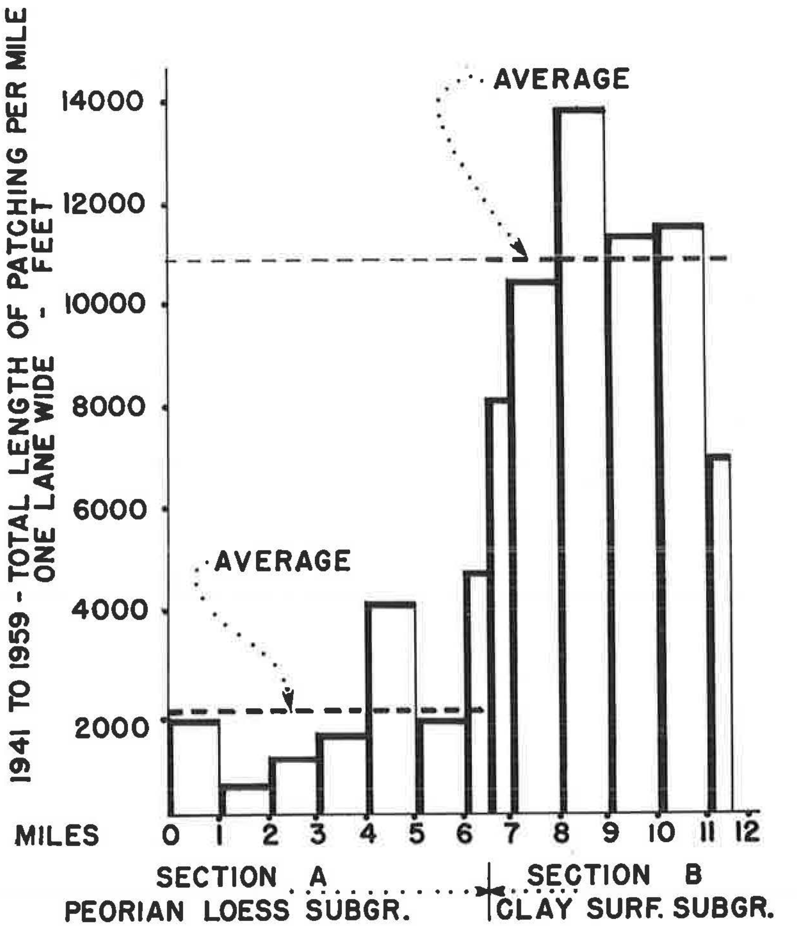

Figure 18. Total length of patching for each mile.

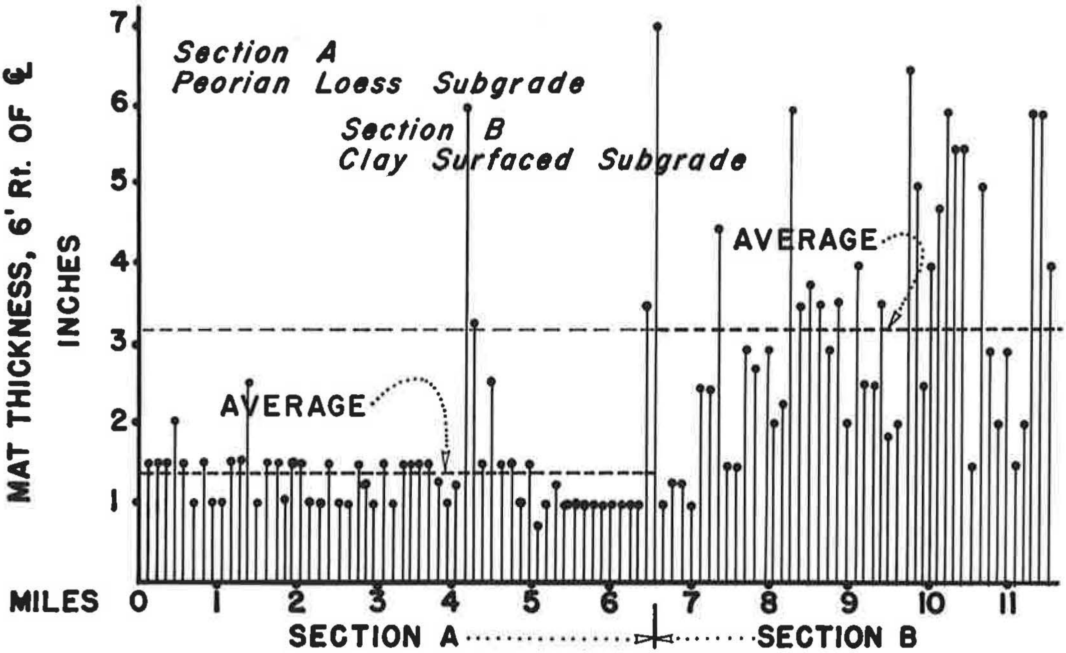

Figure 19. Borings showing thickness of bituminous surfacing in 1959.

(1) The placement of a silt-clay layer over a sand subgrade has a detrimental effect on the serviceability of the flexible pavement.

(2) Any given cohesive soil is less favorable for use in the subgrade if underlain by sand.

FOLKE RENGMARK, Closure—Ramsey reports on an investigation of Nebraska Highway 44 and on damages caused on this road. Considering the thickness of the road construction, it is surprising that no more serious damages have arisen. According to Table 1, the thickness of the road construction by Swedish standards along section A should be at least 70 cm, and 35 cm along section B.

The conclusions Ramsey draws are much too general. This is shown by the results reached at the experimental road at Bjästa. This road was built to try a method of breaking capillary connection with insulating sand layers. For this experiment, silt was used in the road construction to make the test harder. Of course, the purpose was not to test the silt for road-making purposes.

His statement that clay-surfacing materials have to be removed prior to construction of flexible pavements is completely in agreement with the opinion in Sweden.

Design of Swiss Roads Against Frost Action

A. VON MOOS, Consulting Geologist, Lecturer, Swiss Federal Institute of Technology, Zürich, Switzerland.

This paper describes the topography and climate of Switzerland in relation to road design. Development in highway technology, particularly during the last 20 years, is traced. Methods for assessing road stability on the basis of soil type, ground water conditions, and frost penetration are discussed. Empirical curves for pavement design are shown.

• SWITZERLAND, in the center of central Europe, occupies a surface of 41,295 km^2. The greatest extensions of this small country are only about 220 km in the north-south direction and 348 km in the east-west direction. More exciting is the vertical scale, from an elevation of 193 m above sea level at the surface of the Lago Maggiore in southern Switzerland, to 4,634 m above sea level at the top of the Dufourspitze in the Monte Rosa Massif near Zermatt. These differences are an indication of the intense vertical structure of this country.

The climate of Switzerland falls between the atlantic-oceanic climate of western Europe and the continental climate of eastern Europe. The precipitation is about 120 cm/year on a 40-year average. Rather dry parts are found in the deep valley of the Rhone in the Wallis with an average of 60 cm of precipitation per year, whereas the Alps of the Bernese-Oberland and of the Canton of Claris are rather wet, with an average of 320 cm/year and more. Precipitation is fairly well distributed over the whole year with some predominance in the South during spring and autumn, and in the North during summer. The mean annual temperature varies, depending on the elevation and the latitude, between +10 to 12 C in southern Switzerland and near the Lake of Geneva to about +1 to +2 C on the highest alpine roads. It is typical for the country that even in the South there may be several frost periods in some years. Therefore, frost action in roads must always be considered.

ROADS AND ROAD STUDIES

Switzerland, with its population of 5,429,061 has at its disposal a network of 50,000 to 60,000 km of roads for motor traffic. Some of the present roads date back to the time when the Romans occupied the country, and others to the Middle Ages. Most of the roads, however, have been built within the last 100 years by municipalities and cantons. Although the greater part of them are located in valleys and plains, there is still a considerable number that cross the mountains of the Alps and the Jura. Design, construction and maintenance of this rather dense network of roads are administered by the public works departments of municipalities and cantons.

At the present time, Switzerland plans a new network of 1,680 km of so-called National Roads, most of which will be open to motor vehicles only. In this case, it is the Federal Government that is responsible for planning and the co-ordination of this national undertaking. The Federal Government will contribute, on the average, 83 percent of the cost; the balance to be paid by the cantons and the municipalities. In July 1962 only 136 km of these National Roads were completed and 206 km were under con-

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

struction. By the end of 1966, it is hoped that about 677 km (about 40% of the total) will be in use.

The design of the roads which have been constructed by the municipalities and the cantons has varied a great deal and depends on local tradition, financing methods, and designers. In order to develop more basic design information, the Swiss Federal Institute of Technology at Zürich and the Ecole Polytechnique de l'Universite de Lausanne are doing research work, especially on frost problems. Many of their papers are published in the Proceedings of the International Society of Soil Mechanics and Foundation Engineering, the Association Internationale Permanente des Congres de la Route, or in the periodical of the Association of Swiss Road Engineers, entitled "Strasse and Verkehr."

STUDIES ON FROST PROBLEMS

Damage due to frost action including thawing in the subsoil of the roads in Switzerland has been recognized for many years. Many case records have been published (4-6, 10-12). The theoretical aspect of the problems has been treated by Ruckli (8) and

TABLE 1

FREEZING INDEX, MEAN ANNUAL TEMPERATURE AND MEAN TEMPERATURE OVER THE DURATION OF FROST PENETRATION

No.	Measuring Stations	Height Above Sea Level (m)	Exposure of Meas. Station[a]	FI_{10} Avg. Freezing Index of 10 Successive Years (°C-days)	FI_{30} Avg. Freezing Index of 3 Coldest Winters in 30 Years (°C-days)	Mean Annual Temp. of 10 Successive Years (°C)	δ_0 Mean Annual Temp., Cold Year 1929 (°C)	δ_S Mean Temp. over Duration of Frost Pen.[b] (°C)
1	Aarau	406	T	96.7	297.5	9.24	7.9	-4.8
2	Adelboden	1,345	H	226.0	468.9	6.04	5.0	-5.5
3	Airolo	1,170	H	171.9	326.3	6.57	6.2	-3.6
4	Altdorf	456	T	61.3	223.4	9.63	8.8	-4.6
5	Andermatt	1,442	T	564.6	872.7	3.50	2.9	-6.5
6	Basel	318	Te	93.7	292.3	9.80	9.7	-4.7
7	Bellinzona	236	T	21.6	65.9	11.98	11.4	-2.4
8	Bern	572	Te	117.4	320.9	8.91	7.9	-4.4
9	Château-d'Oex	994	H	259.9	484.1	6.44	6.1	-4.5
10	Chur	609	T	113.9	318.7	8.92	8.2	-4.3
11	Davos-Platz	1,561	H	623.7	872.8	3.42	2.8	-6.8
12	Engelberg	1,018	T	257.2	509.8	6.03	5.4	-5.5
13	Frauenfeld	432	Te	119.1	333.1	9.02	7.9	-4.6
14	Fribourg	670	H	127.1	323.9	8.58	7.6	-5.9
15	Genève	405	K	48.2	173.5	10.74	9.7	-4.2
16	Glarus	503	T	154.9	391.0	8.28	7.1	-6.8
17	Interlaken	568	T	83.3	229.4	8.89	7.9	-4.2
18	Kloten	440	T	116.2[c]	289.6[d]	8.60	7.2[e]	-3.6
19	Kreuzlingen	445	H	115.9	326.3	8.87	8.3	-4.4
20	La Brévine	1,060	T	375.5	654.2	4.87	4.6	-6.8
21	La Chaux-de-Fonds	990	K	187.1	422.6	6.95	5.9	-3.7
22	Lausanne	589	H	69.2	206.0	9.99	9.2	-4.6
23	Luzern	497	H	91.8	277.2	9.65	8.4	-5.9
24	Neuenburg	487	H	82.2	240.8	9.71	9.0	-5.2
25	Sargans	510	H	104.8	317.7	9.51	8.6	-6.1
26	Sarnen	474	T	88.9	278.4	9.19	7.6	-5.4
27	Schaffhausen	448	H	115.2	358.1	8.96	8.0	-4.3
28	Schuls	1,253	H	483.0	736.1	5.42	4.3	-6.3
29	Sitten	549	H	71.6	210.4	10.52	9.8	-3.3
30	Solothurn	470	H	99.3	283.8	9.28	8.3	-4.0
31	Splügen	1,500	H	527.0	788.3	3.89	3.3	-6.5
32	St. Gallen	664	until 54:H later :T	170.0	401.6	7.63	6.9	-4.8
33	St. Moritz	1,853	H	632.8	338.7	2.59	0.5[e]	-6.7
34	Zermatt	1,610	T	505.3	593.2	3.91	3.7	-5.4
35	Zürich (MZA)	569	H	108.6	288.0	9.26	8.5	-6.3

[a]Exposure of measuring station: T = valley bottom; H = mountain slope; K = round hilltop; Te = terrace.

[b]Mean value calculated from the freezing indexes and the respective number of days of the 3 coldest winters in 30 years.

[c]Mean from 9 years, of measurements since 1949 only.

[d]Freezing index of the coldest winter in 10 years (1955/56).

[e]Mean annual temperature in 1956.

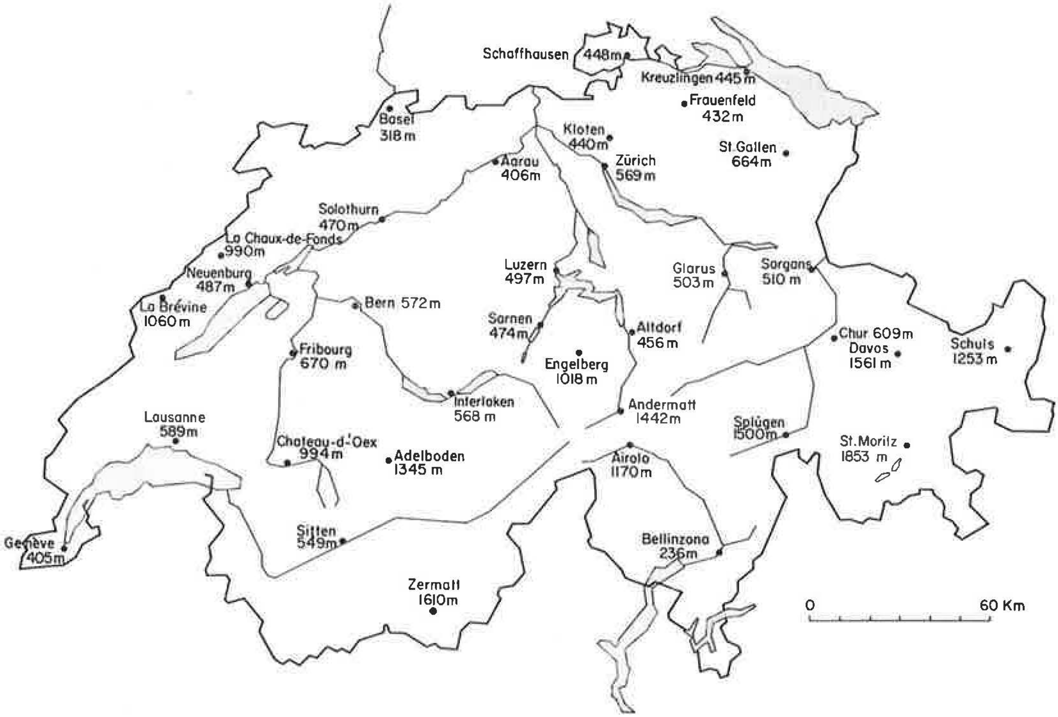

Figure 1. Distribution of the places in Switzerland where the freezing index, the mean annual temperature and mean temperature over the duration of frost penetration has been calculated (Table 1). Altitude above sea level is in meters, after Schnitter and Zobrist.

more recently by Balduzzi (2), who worked out a theory of the development of ice lenses and constructed very effective apparatus for the measurement of frost penetration, frost heaving and the heaving forces in the laboratory.

The Soil Mechanics Laboratory of the Swiss Federal Institute of Technology at Zürich calculated the mean annual temperature, the mean temperature during the frost periods and the respective frost indexes of 35 locations in Switzerland (9). The results are given in Table 1, and the locations together with their elevation in meters are shown in Figure 1.

On this basis, the same laboratory calculated the theoretical maximal frost penetration for the above locations for a sandy gravel during the three coldest winters over periods of 30 years and of 20 years (9). The calculations are based on the theories of Aldrich and Paynter (1), using the modified Berggren formula. These results (Table 2) represent a welcome basis for the practical design of roads in Switzerland whose base is normally constructed of sandy gravel.

To get more information, the Association of Swiss Road Engineers sponsored in recent years the construction of test fields at Zürich 569 m a.s.l., Payerne 460 m a.s.l. in the Western Central plain of Switzerland and Davos 1,561 m a.s.l. in the Eastern Alps of Switzerland. These stations measure the frost penetration under different surfaces (concrete and flexible pavement of various thicknesses). These measurements are underway and have already given very helpful results for the future design of roads. At the same time, they offer a welcome comparison for different temporary measurements of frost penetration under airfields and roads in Switzerland.

Balduzzi (2, 3) published results of his studies on some typical fine-grained soils in the frost laboratory, especially their reaction under freezing and thawing with regard to heaving and shear strength. The recommendations in the following section were based on these results.

TABLE 2

CALCULATED DEPTH OF FROST PENETRATION IN BASE MATERIAL FOR 35 SWISS TEMPERATURE OBSERVATION STATIONS

No.	Station	Height Above Sea (m)	X_{30} (cm)	X_{10} (cm)
1	Aarau	406	129	71
2	Adelboden	1,345	189	125
3	Airolo	1,170	143	103
4	Altdorf	456	107	54
5	Andermatt	1,442	288	225
6	Basel	318	117	69
7	Bellinzona	236	46	27
8	Bern	572	133	78
9	Château-d'Oex	994	180	129
10	Chur	609	131	77
11	Davos-Platz	1,561	265	238
12	Engelberg	1,018	194	133
13	Frauenfeld	432	135	79
14	Fribourg	670	139	85
15	Genève	405	90	47
16	Glarus	503	157	96
17	Interlaken	568	112	66
18	Kloten	440	128	77
19	Kreuzlingen	445	131	78
20	La Brévine	1,060	231	171
21	La Chaux-de-Fonds	990	170	108
22	Lausanne	589	102	58
23	Luzern	497	124	70
24	Neuenburg	487	111	65
25	Sargans	510	132	75
26	Sarnen	474	128	69
27	Schaffhausen	448	140	77
28	Schuls	1,253	247	189
29	Sitten	549	94	55
30	Solothurn	470	122	70
31	Splügen	1,500	269	213
32	St. Gallen	664	158	100
33	St. Moritz	1,853	320	250
34	Zermatt	1,610	246	207
35	Zürich	569	127	78

X_{30}: average depth of frost penetration during the 3 coldest winters in 30 years. Calculated from:

FI_{30}: average freezing index for the 3 coldest winters in 30 years;

δ_0: mean annual temperature during the cold year 1929;

δ_s: mean temperature over the duration of frost penetration.

X_{10}: average depth of frost penetration during 10 years. Calculated from:

FI_{10}: average freezing index of 10 successive years;

δ_{00}: mean annual temperature of 10 successive years;

δ_s: mean temperature over the duration of frost penetration.

These calculated values of X_{30} and X_{10} are valid for the materials: GW to GP;

$$\gamma d = 2.1 \ t/m^3; \ w = 4 \text{ percent}$$

Studies by the same laboratory encouraged the possibility of stopping the formation of ice lenses in some soils by treatment with cement.

DESIGN

During the last 20 years, the Association of Swiss Road Engineers has created eleven working committees to establish standards or norms which usually are accepted by the public works departments of the municipalities, cantons, and the Federal Government for their respective road construction.

Design for Bearing Capacity

The association of Swiss Road Engineers has published two recommendations for the design of roads regarding bearing capacity.

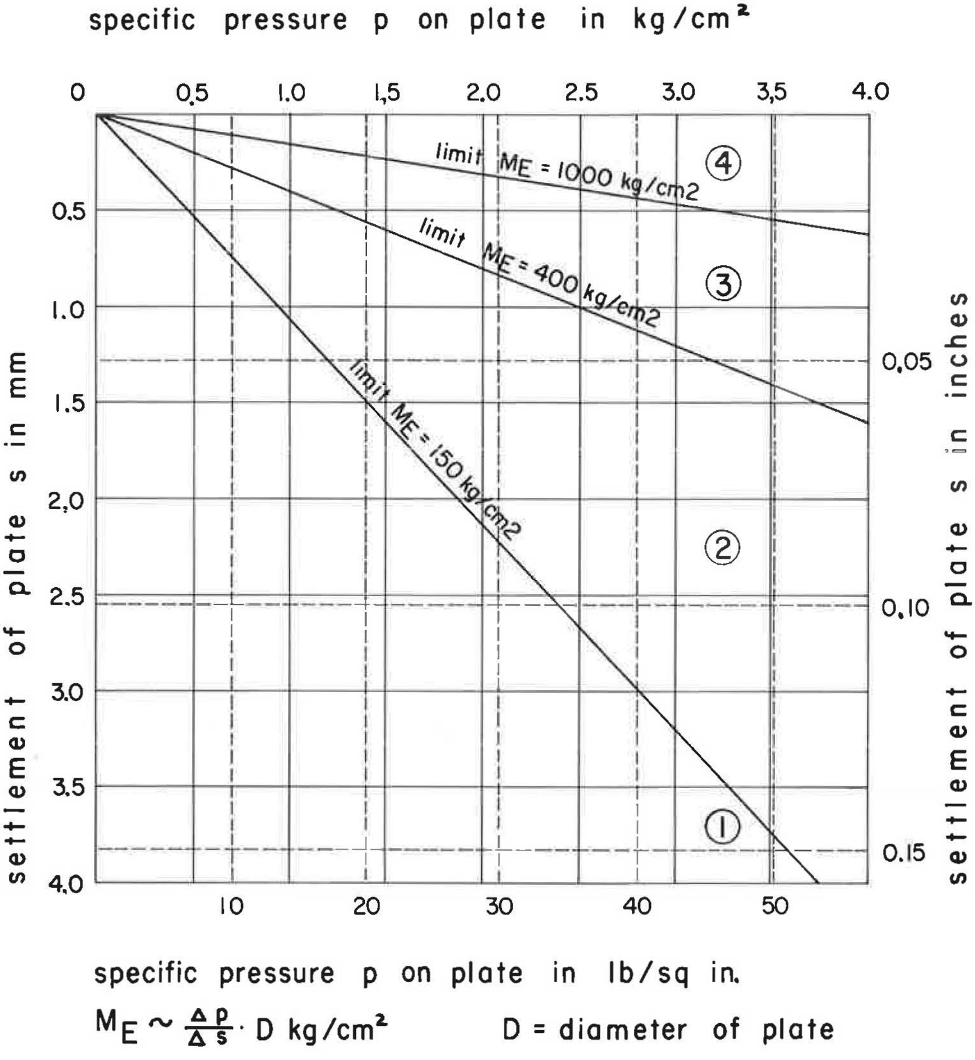

Figure 2. Diagram of the necessary bearing capacity of the different layers of a road. 1 = unfavorable subsoil, 2 = favorable subsoil or compacted subgrade, 3 = subbase, and 4 = base, after Norm SNV 40 317/1953 of the Swiss Association of Road Engineers.

Plate Bearing Method. - Norm SNV 40 317/1953 describes a design method based on the measurement of the bearing capacity by a plate bearing test (plates of 200 cm^2 and 700 cm^2). To control construction, the results of measurements on different parts of the road structure with the 200-cm^2 plate are plotted in Figure 2. If a measured M_E falls outside the required zone, the compaction should be continued or the method of compaction changed. If M_E falls within the zone, then the job is ready for the next layer.

Before the surfacing is started, the compaction should be checked on top of the base with a 700-cm^2 plate and the result compared with Figure 3. If the point falls into zone B, the work is in order. If it falls into zone A, the compaction should be continued or the method changed. The zones of Figures 2 or 3 are empirical and were established after much discussion of experiences.

CBR Method. — The second recommended design for a road, based on the well-known amd still discussed field CBR test, is treated in Norm SNV 70 315/1959 (first edition SNV 40310) and (SNV 40315/1953).

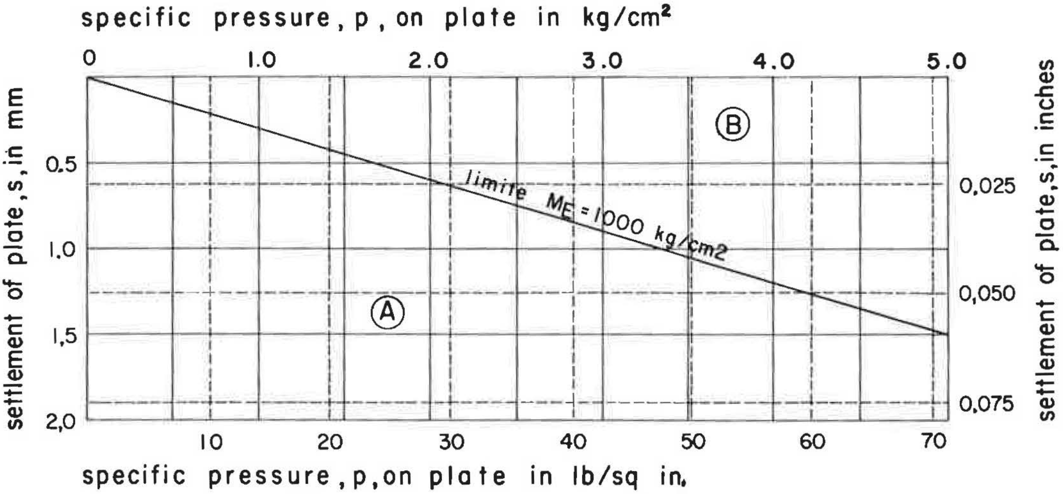

Figure 3. Diagram of the necessary bearing capacity on top of the base, after Norm SNV 40 317/1953 of the Swiss Association of Road Engineers.

Frost Considerations

As a basis for further discussion within the Association of Swiss Road Engineers, the author (7) made a suggestion for the design of Swiss roads against frost action which is given in Table 3. This suggestion has been generally superseded by the use of standard classification systems.

Use of Classification Systems

In order to unify the nomenclature of soils, the Association of Swiss Road Engineers and some other Swiss organizations agreed after long discussion to accept the Unified Soil Classification System (USCS) (see Norm SNV 70 005/1959). This was a great help in further work.

To accommodate differences in opinion and get, as quickly as possible, a generally acceptable method of design against damage due to freezing and thawing, the Association of Swiss Road Engineers issued 1957 Norm SNV 40 325 based on the classification of soils by both the USCS and the U.S. Public Roads Administration Classification.

The road design is based on Table 4 and Figure 4. The designer may classify the soil in accordance with either the USCS or the U.S. Bureau of Public Roads Classifi-

TABLE 3

PROPOSAL FOR THE DESIGN OF SWISS ROADS AGAINST DAMAGE DUE TO FROST ACTION
DIMENSION OF SURFACING, BASE AND SUBBASE AFTER VON MOOS 1956

Soil Groups	Depth of the Groundwater Table	Flexible Pavements		Concrete Pavements	
		Normal Road Frequency[a]	Expressway Frequency[b]	Normal Road Frequency[a]	Expressway Frequency[b]
Coarse stone, gravel-sand, sand, material smaller than 0.02 m/m less than 3% of the total sample	No influence	Design after bearing capacity only, see Norm SNV 40' 317/1953 and SNV 70' 315/1959			
Coarse stone, gravel-sand, sand, material smaller than 0.02 m/m between 3 and 15% of the total sample	More than 200 cm under the surface of the road				
	Between 0 cm and 200 cm under the surface of the road	50 cm[c]	60 cm[c]	35 cm[c]	45 cm[c]
Silt, clay, gravel and coarse stone, material smaller than 0.02 m/m over 15% of the total sample	More than 200 cm under the surface	60 cm[c]	70 cm[c]	50 cm[c]	60 cm[c]
	Between 0 and 200 cm under the surface of the road	70 cm[c]	80 cm[c]	60 cm[c]	70 cm[c]
Peat, lake marl, clays		Design should be based on special studies			

[a]Less than 6,000 vehicles per day.

[b]More than 6,000 vehicles per day.

[c]Addition for elevation above sea level in meters: from 600 to 1,000, 10%; 1,000 to 1,500, 20%; over 1,500, 30%. Reduction on embankments: 10 cm, addition in cuts: 10 cm.

TABLE 4

DESIGN OF THE ROAD CONSTRUCTION ACCORDING TO NORM SNV 40'325 OF THE SWISS ASSOCIATION OF ROAD ENGINEERS 1957

Class	Soil Group	Soil Classification System		Design	
		USCS[a]	BPR[b]	Good Groundwater Conditions	Bad Groundwater Conditions
a (no frost sensitivity)	Well and poorly graded gravel-sands or sands with little or no fines	GW, GP SW, SP	A-1, A-2 A-3	Only if bearing capacity according SNV 70'315/ 40'317	
b (little frost sensitivity)	Well and poorly graded silty or clayey gravel-sands or sands	GM GC	A-1b	Zone 1 (Fig. 4)	Zone 2 (Fig. 4)
c (medium frost sensitivity)	Silty or clayey sands, inorganic clays of high plasticity, organic clays of medium to high plasticity	SM, SC CH OH	A-2, A-4 A-6, A-7 A-7	Zone 2 (Fig. 4)	Zone 3 (Fig. 4)
d (heavy reaction to frost)	Inorganic silts and very fine sands with slight plasticity; inorganic clay of low to medium plasticity	ML, MH CL	A-4 A-5	Curve 4 (Fig. 4)	Curve 4 (Fig. 4)
	Organic silts and organic silty clays of low plasticity	OL	A-7-5		

[a]USCS: Unified Soil Classification System, U.S. Dept. of the Interior, Bureau of Reclamation 1953

[b]BPR: U.S. Bureau of Public Roads Classification, HRB Proc., 25:375 (1945).

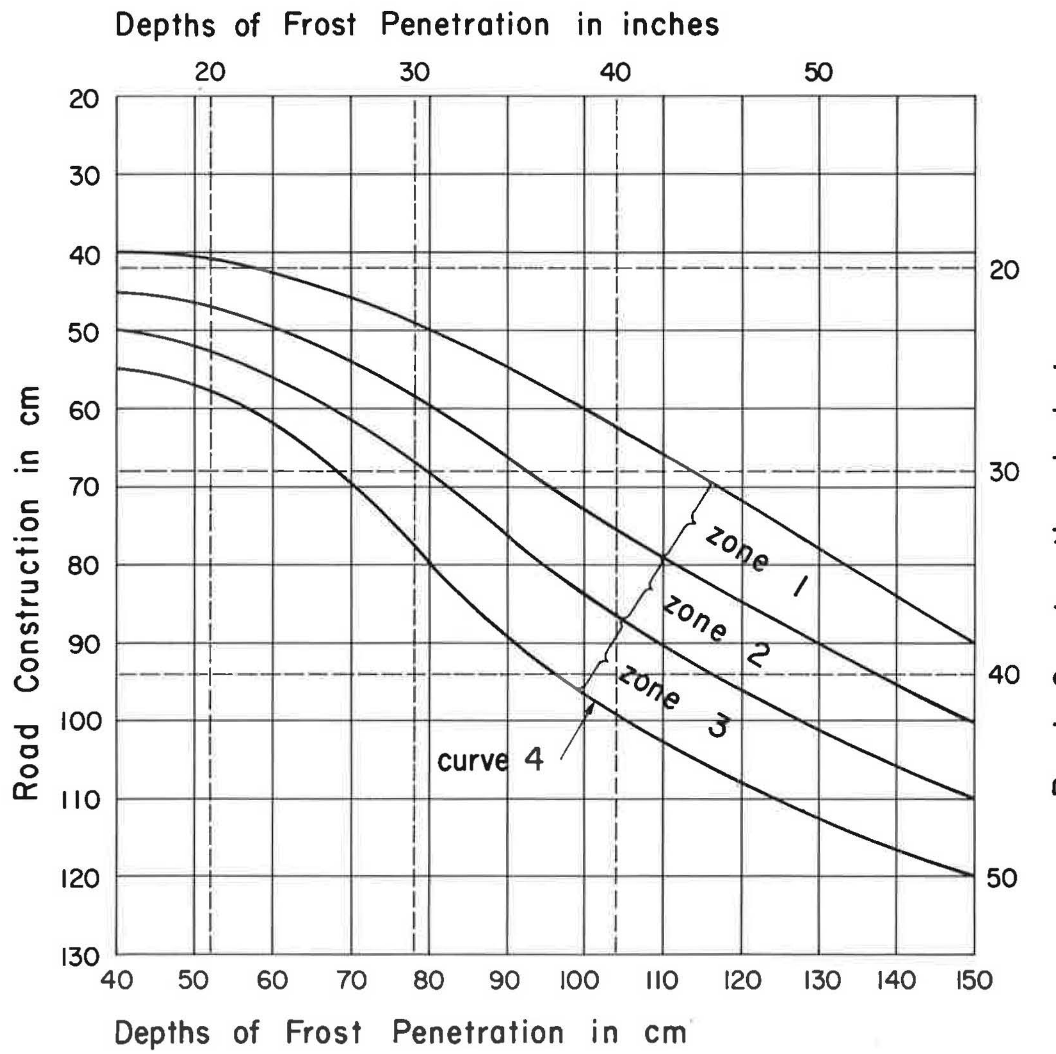

Figure 4. Thickness of the road construction in frost areas in case of thin surfacing, after Norm SNV 40 325 of the Swiss Association of Road Engineers.

cation. Soils are termed frost resistant if parts smaller than 0.02 mm constitute less than 3 percent by weight, and frost susceptible if this content is more than 3 percent. Ground water conditions are considered good if the ground water level is deeper than three times the maximum frost penetration. In the absence of more detailed information, the maximum frost penetration may be estimated from Table 5, but a more reliable estimate can usually be made from Table 2.

This method of design has been widely applied but must be revised in the near future. Experience has indicated that some reductions in design thickness may be possible.

Institute of Technology Method

In 1959, the Soil Mechanics Laboratory at the Swiss Federal Institute of Technology suggested their own rule for the design of roads using the following three groups based on the USCS. Grouping a material must be based on the field classification during the

TABLE 5

AVERAGE FROST PENETRATION

Location	Frost Penetration (cm)
Southern Switzerland, Central plain and Alpine valleys up to 600 m above sea level	50-100
Jura and Alps 800 to 1,400 m above sea level	80-140
Exposed parts of Jura Mountains, Alps over 1,400 m above sea level	120-200

investigation of the subsoil of the road. In critical cases, a sample should be studied in the laboratory.

Group 1

GW, GP, SW, SP--soils which, under the influence of the penetration of the 0 C isotherm, experience neither a heave nor a loss of bearing capacity during thawing. These roads can be designed for bearing capacity only.

Group 2

ML, MH, partly SM and GH—soils which, under the influence of the penetration of the 0 C isotherm, show a considerable heave. During the thawing process a sudden total loss of bearing capacity will follow. These soils should be removed and replaced by GW or SW material. The depth to which the material is to be replaced should equal the frost penetration in the GW and SM material. Frost penetration for some 35 places in Switzerland in different parts and elevations are given in Table 2.

Group 3

CH, OH, OL, CL, partly SC and GC—soils which, during the penetration of the 0 C isotherm, show a small heave due to a small heaving pressure. During the thawing of these soils, the bearing capacity is only reduced a little. These roads should not be designed for full frost penetration, but for only a reduced bearing capacity during thawing.

As a rule, roads which have to be built on soils of the latter group should be constructed of material of high resistance so that the pressure of the rolling vehicle is spread out over a wide area. In these cases, it is often very effective to stabilize the subsoil or the base by treatment with cement. This treatment of the base provides a good working surface for the mechanical compaction of the succeeding layers of the road. This treatment might also be used to change the upper part of a subsoil consisting of a silty or a clayey gravel or sand into a less frost-and thaw-susceptible material. As a rule, good drainage of these soils is very effective.

CONCLUSION

In Switzerland there is still no universally agreed method to design roads against frost action.

The working committee of the Association of Swiss Road Engineers has suggested a revision of their Norm SNV 40 325 regarding design of roads against damage due to freezing and thawing. This will be done in co-operation with the Soil Mechanics Laboratory at the Swiss Federal Institute of Technology, especially with Dr. F. Balduzzi, taking into account the new experiences in laboratory, field, and especially the results of the AASHO Test. The trend is towards a design based on the principle of maintaining a sufficient bearing capacity during thawing; by soil stabilization with cement for example. In all cases, a reduction in the total thickness of the road is possible, and this results in a more effective and economic way of building new roads.

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Discussion

W. H. PERLOFF, JR., Assistant Professor of Civil Engineering, Ohio State University, Columbus— The author presented a most interesting and informative description of frost action in Switzerland and Swiss practices of design against detrimental effects. Consideration of frost effects clearly plays an important role in pavement design throughout Switzerland. The problems which the Swiss face are similar to those encountered in the United States, because the maximum frost penetrations observed in Switzerland are of the same order of magnitude as those observed in the New England and northern midwest States.

It is of particular interest to compare Swiss design practices with those in use in this country. Dr. von Moos indicates that pavement components for Swiss roads are designed on the basis of bearing capacity, independent of frost effect. Field check of construction methods is performed with a plate bearing test or the field CBR method.

These methods seem to offer distinct advantages over the compaction criteria often applied in this country. Because the primary function of the soil components of a pavement system is to bear the loads imposed on them, a load-test technique of construction control seems reasonable.

The Swiss approaches to design, on the basis of frost considerations, are similar in many ways to methods used in the United States. The method suggested by the Association of Swiss Road Engineers appears to be similar in concept to that recommended by the U. S. Army Corps of Engineers (17). The Swiss method appears to yield somewhat more conservative values for total thickness of the paving system than the U.S. Corps of Engineers' approach. This can be illustrated by an example: For a frost penetration of 40 in. in soils which have little to medium frost sensitivity, von Moos' Table 4 indicates a total road thickness of approximately 35-40 in. For the same frost penetration and soil (groups F-3, F-4, Corps of Engineers' designation), a combined thickness of approximately 25-35 in. is indicated. This applies to the limited subgrade frost penetration method of design. The Corps of Engineers reduced subgrade strength design would generally give an even smaller thickness of road construction.

The method suggested by the Swiss Federal Institute of Technology is also similar in concept to the Corps of Engineers approach. However, no specific values are given to permit comparison. On the basis of this discussion, this writer agrees with Dr. von Moos that the present Swiss design practices are quite conservative and that a reduction of the total thickness of roads is possible and desirable. The indicated trend is toward the use of a "reduced bearing capacity during thaw" method. Perhaps the United States practice in this regard may be of assistance to the Swiss as they formulate their own criteria for design of roads against frost action by this method.

WM. P. HOFMANN, Director, Bureau of Soil Mechanics, New York State Department of Public Works. — This discussion and comparison of methods is based on the practices of the New York State Department of Public Works.

From the author's statistics, it appears that the climates of Switzerland and New York are somewhat similar, except that the freezing indexes of the northern third of the State are considerably greater than those calculated for the higher measuring stations in Switzerland.

All of New York State, except a small area south of the Allegheny River, was covered by the last continental glacier. In general, the topography is either rolling or mountainous, but the subsurface conditions are quite complex, exhibiting extreme variations in small horizontal and vertical distances.

Dr. von Moos indicates that, although Swiss highway engineers have not as yet agreed on a method of pavement design, the methods presently in use are based on a classification of the subgrade soil in accordance with either the Unified or AASHO System, a consideration of the depth to the water table, and the depth of frost penetration.

It is extremely difficult and impractical to accurately predict future subgrade behavior under frost action in areas of complex glaciation with any reasonable amount of subsurface exploration and laboratory testing. It is equally impractical to design satisfactory and yet economical pavements in glaciated areas by inserting laboratory test results into formulas for pavement thickness. Replacement of frost-susceptible soils with clean, granular materials to the full depth of anticipated frost penetration is generally extremely uneconomical, particularly in areas where these materials are scarce or not available.

In areas of complex glaciation, terrain and environment become the most important factors; not the texture of the major component of any deposit. For example, in New York the most critical differential pavement heave problems occur in cuts through kame, outwash and esker terrains generally composed of clean, granular materials. The stratification, permeability, topography and the presence of silt layers and lenses in such deposits produce great problems in differential heave. Cuts through folded rock formations produce differential heave problems of similar magnitude.

For many years, the New York State Department of Public Works has used a method of pavement design that is based on an evaluation of the performance of existing pavements serving under similar terrain and environmental conditions. This method is not as primitive or archaic as it seems at first glance. The State has many thousands of miles of highways on all types of terrain under wide ranges of environment. They constitute full-scale models readily available for observation. However, the essential requirement of this method is the judgment of an engineer thoroughly experienced in the design, construction and performance of pavements on any terrain under any environment.

The following points are valid assumptions when designing highway pavements to serve in areas of complex glaciation and significant seasonal frost action:

(1) All surface and shallow subsurface drainage facilities are generally inoperative during freezing and thawing, shallow culverts and underdrains are frozen, and the ditches blocked with snow.

(2) Consequently, most subgrades and subbases are saturated during freezing and thawing, regardless of the position of the water table.

(3) Generally, all pavements heave measurable amounts, regardless of any reasonable or economical preventive measures. Every effort should be made to obtain uniformity of subgrade and pavement conditions so that the heave is sufficiently uniform to minimize adverse riding qualities of and damage to the pavement surface.

(4) The thickness of the pavement section should be governed by the reduced bearing capacity of the subgrade during the spring thaw, and not necessarily the depth of frost penetration. Because bearing capacity during the thaw is known to vary considerably from year to year, the effect over a span of several years can only be estimated by observations of the performance of existing pavements serving under similar terrain and environment.

Frost Design Practice in Canada

MALCOLM D. ARMSTRONG and THOMAS I. CSATHY, Respectively, Principal Research Engineer and Project Soils Engineer, Ontario Department of Highways

This paper discusses the methods being used by Canadian highway engineers to combat frost action. It was compiled from information supplied by the two Federal agencies concerned with pavement design for roads and airfields and the highway departments of the Provincial Governments.

A description is given of the varied climatic, soil, and traffic conditions and of the type and extent of frost damage encountered. A survey of frost design and construction practices for flexible pavements, rigid pavements, and highway structures is given. The present standards of frost design practice are reviewed, and problem areas requiring research are indicated.

•CLIMATIC CONDITIONS in Canada are such that frost action is a problem of major importance for highway engineers throughout the country. The aim of this paper is to describe the considerations given to frost action by the various agencies concerned with the design of highways and airfields.

Twelve agencies have contributed to the preparation of this paper, including the Federal Departments of Transport and Public Works, and the Highway Departments of the ten Provinces.

With minor exceptions each Province is responsible for all highway construction within its boundaries. The Federal Department of Public Works is responsible for the design and construction of highways in the National Parks, the Northwest Territories, and the Yukon Territory. The Federal Department of Transport is responsible for the design and construction of the principal airfields throughout the country.

ENVIRONMENTAL CONDITIONS

Climatic Conditions

Canada is the world's third largest country and thus it is not surprising that it is a land of many climates. Stretching through nearly 90° of longitude, it also extends in a northerly direction from latitude 42° to within a few hundred miles of the North Pole. Practically all the country is subject to seasonal frost and approximately one-third lies within the regions of discontinuous or continuous permafrost (Fig. 1). The design principles discussed in this paper refer specifically to areas of seasonal frost.

Canada has six climatic regions, the characteristic features of which are briefly summarized.

The Southeastern Climatic Region takes in Southern Ontario and Quebec, along with the Maritime Provinces of New Brunswick, Nova Scotia, Prince Edward Island, and the island portion of Newfoundland. This region is the most densely populated and the climate is very similar to that of the Northeastern United States. The winter is shorter than it is in the adjoining northern regions, and the summer (in Ontario, at least) may be quite warm. The Great Lakes have a moderating effect on the climate in Southwestern Ontario, but this effect is scarcely noticeable in Southeastern Ontario and in Southern Quebec. Temperatures in the interior of New Brunswick are similar to those

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

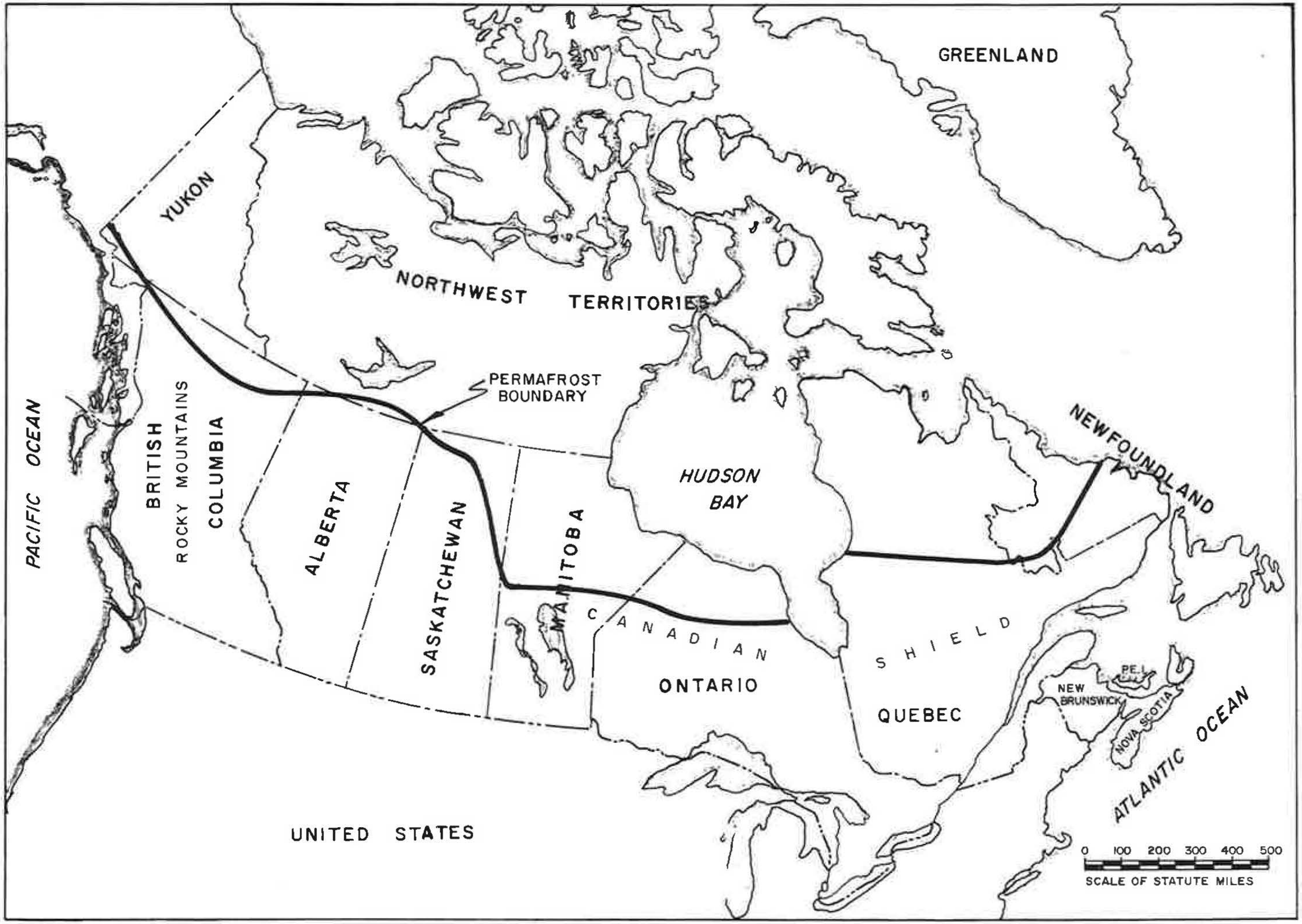


Figure 1. Permafrost boundary in Canada.

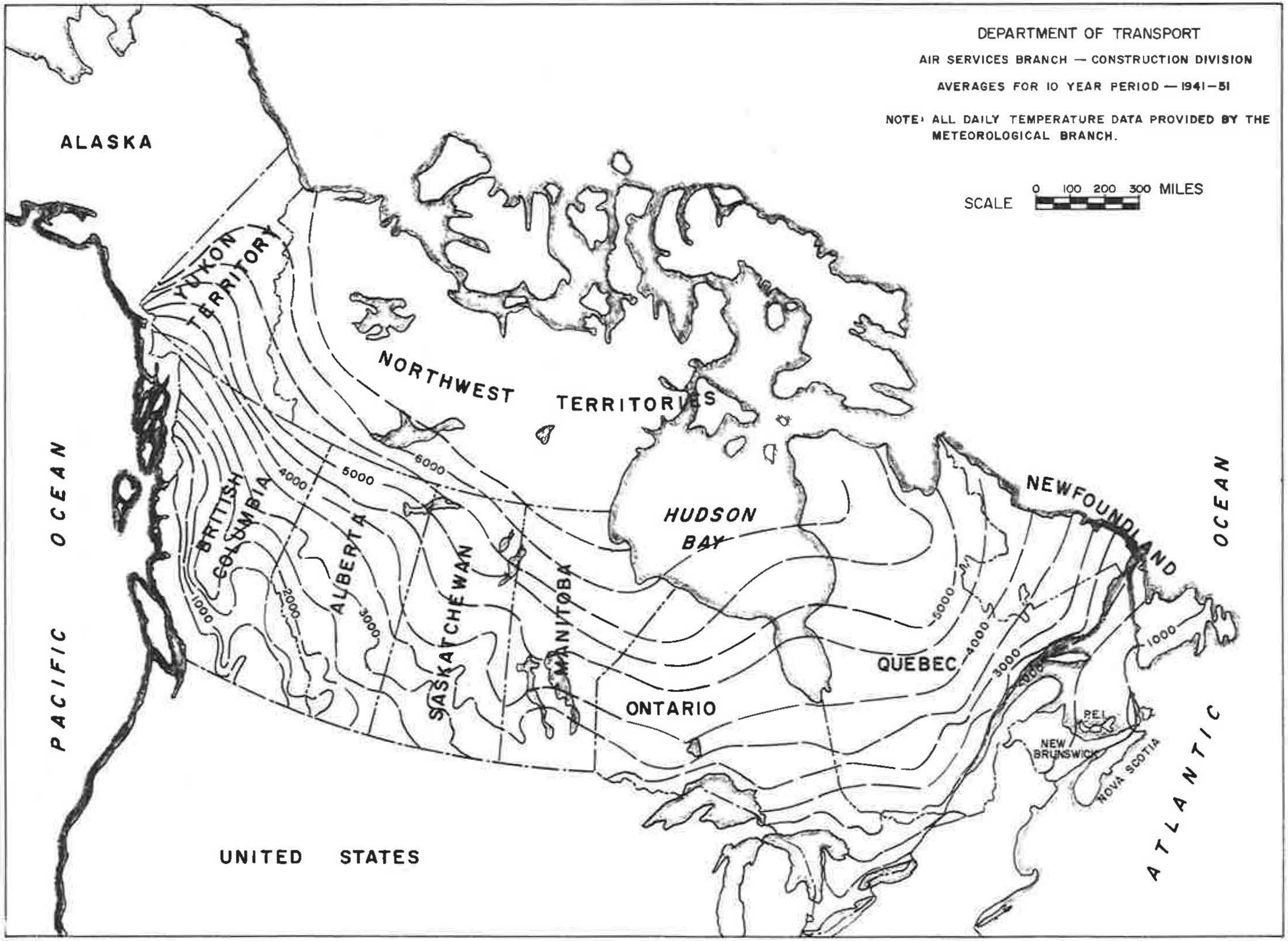


Figure 2. Freezing indexes in Canada.

TABLE 1

CLIMATIC DATA FOR THE PROVINCES OF CANADA[a]

Province	Temperature (°F)									No. of Frost-Free Days	Freezing Index (degree-days)	Frost Penetration (ft)	Precipitation (in.)					
	Mean Annual	Mean Daily				Extreme Lowest	Extreme Highest	2.5% Design Value[b]					Mean Annual Total[c]	Mean Annual Rain	Mean Seasonal Rainfall			
		Jan.	Apr.	July	Oct.			Wint.	Sum.						Wint.	Spr.	Sum.	Fall
British Columbia:																		
Coast	40-50	30-40	40-50	55-65	40-50	$\underline{30}$-0[s]	90-100	0-10	70-75	200-275	0- 500	0- 3	50-100	40-100	5-40	5-30	5-20	5-40
Int.	35-45	0-30	35-50	55-70	35-45	$\underline{65}$-$\underline{30}$	95-105	$\underline{30}$- 0	75-90	50-150	500-4,000	2- 8	15- 50	10- 30	1-10	2- 6	3- 6	3-10
Alberta	30-40	0-10	35-45	60-65	35-45	$\underline{65}$-$\underline{40}$	100-105	$\underline{40}$-$\underline{30}$	80-90	150-250	2,000-4,000	4- 8	15- 20	10- 15	0- 1	1- 3	5-10	2- 3
Saskatchewen	30-40	$\underline{10}$-10	30-40	60-65	35-40	$\underline{60}$-$\underline{50}$	100-110	$\underline{40}$-$\underline{30}$	85-90	150-250	2,500-4,500	6- 8	15- 20	10- 15	0- 1	1- 3	5- 8	2- 3
Manitoba	25-35	$\underline{10}$- 0	25-35	60-65	30-40	$\underline{60}$-$\underline{50}$	95-110	$\underline{40}$-$\underline{30}$	80-85	125-175	3,000-6,000	4-10	15- 20	10- 15	0- 1	1- 3	6- 8	2- 3
Ontario:																		
North	30-40	$\underline{5}$-10	25-35	60-65	35-45	$\underline{60}$-$\underline{45}$	100-105	$\underline{40}$-$\underline{20}$	80-85	125-175	1,500-4,500	6- 8	25- 35	20- 25	0- 1	2- 5	8-10	5-10
South	40-50	10-20	35-45	65-70	45-50	$\underline{45}$-$\underline{20}$	100-105	$\underline{20}$- 0	85-90	200-250	500-1,500	2- 4	30- 35	20- 30	1- 3	5- 8	8-10	6-10
Quebec	30-40	$\underline{10}$-10	25-40	55-65	35-45	$\underline{60}$-$\underline{40}$	95-100	$\underline{30}$-$\underline{10}$	75-85	150-200	2,000-4,000	4- 8	35- 40	20- 30	0- 2	2- 6	10-12	7-10
Newfoundland	30-40	10-20	25-35	55-65	40-45	$\underline{40}$-$\underline{20}$	85- 90	$\underline{20}$- 0	70-80	200-250	500-1,500	2- 4	35- 50	25- 40	2-10	4-10	10-12	8-12
New Brunswick	35-45	10-20	35-40	65-70	40-45	$\underline{40}$-$\underline{30}$	100-105	$\underline{20}$-$\underline{10}$	75-85	80-120	1,500-2,000	4- 6	35- 40	25- 35	2- 5	5- 8	10-12	8-10
Pr. Edward I.	40-45	15-20	35-40	65-70	45-50	$\underline{30}$-$\underline{20}$	95-100	$\underline{10}$- 0	75-80	200-250	1,000-1,500	2- 4	35- 40	25- 30	2- 5	5- 7	10-12	8-12
Nova Scotia	40-50	20-25	35-40	65-70	45-50	$\underline{30}$-$\underline{20}$	90-100	$\underline{10}$- 0	75-80	200-275	500-1,000	2- 4	40- 50	30- 40	5-10	7-10	10-12	8-12

[a] Underlined values are negative.
[b] The value at or below (above) which 2.5 percent of the January (July) hourly temperatures occur.
[c] Rainfall + 0.1 snowfall.

in Quebec, but in the other Maritime areas both the summers and the winters are modified to a great extent by the ocean. Precipitation is ample in this region.

The Prairie Climatic Region includes the settled southern half of Alberta, Saskatchewan, and Manitoba. The climate of this area is characterized by extreme differences between summer and winter temperatures and relatively low annual precipitation. Temperatures in any season fluctuate widely from day to day and from day to night. About one-half the total precipitation falls during the summer months.

Proceeding westward the third and most complex climatic region of Canada is encountered. This is the Cordillera which includes part of British Columbia and the Yukon Territory. In this region of mountains, plateaus, and valleys, altitude is usually more of a climatic determinant than latitude. The rugged terrain of the country makes it almost impossible to map the climate accurately but, in general, precipitation decreases eastward from the Pacific region, especially in the lee of the successive mountain ranges. On the other hand, temperature variability and severity increase as the mountain region is traversed to the east. In this region the daily temperature variations are greater than anywhere else in Canada. Summers in the southern interior mountain valleys are hotter than any location in the Prairies, but the northern portions are much colder.

The Pacific Climatic Region consists of the islands and the narrow coastal belt of British Columbia, which varies in width from a few miles up to 100 miles. Temperatures rarely drop below 0 F in the winter or rise above 90 F in the summer. With a winter season maximum, this region is rainier than any other in the country.

The Northern Climatic Region might well be called sub-Arctic. Bordered on the north by the Arctic Tundra, this region includes the sourthern part of the Northwest Territories, the northern half of Alberta, Saskatchewan and Manitoba, northern Ontario, most of Quebec, and the mainland part of Newfoundland. The region consists of sparse, lightly-treed barren land in the north and the more heavily forested native boreal forest in the south. Appreciable snow cover lasts for more than one-half the year, especially in the northeastern section. Extremely low temperatures occur every winter throughout most of the northwestern section, and very high temperatures may occur in summer. Precipitation is light in the northwest; in fact this section is subhumid, but ample in most of the southeastern portion. The Central Quebec portion has a very high snowfall each winter.

The sixth climatic region, the Arctic, lies in the upper parts of the permafrost area and is not included in the scope of this paper.

Figure 2 is a freezing index map for Canada. Other characteristic climatic data for the individual Provinces are briefly summarized in Table 1. The values are average figures for general information, and are representative of those parts of the Provinces where an appreciable amount of highway construction has been carried out.

Soil Conditions

Most of Canada's soils are of glacial origin. The most recent period of glacial action, the Wisconsin, was the most important, but there are some soils in the Yukon that belong to an earlier period.

The only place where nonglacial soils have significance is the western Yukon Territory, although they also occur in Alberta and Saskatchewan. Alluvial soils of the postglacial period are to be found only in the Fraser (British Columbia) and MacKenzie (Northwest Territories) River deltas.

The Wisconsin glaciers blanketed great tracts of land with an unsorted mixture of materials that are commonly known as till. This mantle is usually referred to as ground moraine. Glacial till is generally a very compact mixture of materials varying in particle size from clay to boulders, the actual composition depending on the type of terrain. Large areas are also covered with end moraine or hummocky moraine. These are broadly similar to ground moraine in composition, although generally more sandy or gravelly. Glaciofluvial deposits are also widespread in Canada. A further important feature is the vast extent of clays deposited in the great glacial lakes and in the encroachments of the sea. Both the lacustrine and marine clays are usually sensitive.

In the Western Provinces of British Columbia, Alberta, Saskatchewan, and Manitoba the thickness of the ground moraine ranges from a few feet to over 400 feet. The end moraines are normally subdued, and are often only traceable with difficulty. The predominant terrain feature is the wide-scale hummocky moraine. The till soils are normally resorted and contain variable quantities of silt, sand, and gravel. The tills in the Rocky Mountain region are particularly variable, ranging in texture from clayey to gravelly. Cretaceous shales occur in a strip several hundred miles wide to the east of the Rocky Mountains. A large portion of Saskatchewan is underlain by the Bearpaw shale. There are important lacustrine sand, silt, and clay deposits in southern Saskatchewan and Manitoba. Alluvial deposits and loess-type soils are confined to small areas.

In Ontario and Quebec, the predominant soil type is glacial till. The end moraines are usually less than 10 miles wide, but many tens of miles long. A characteristic feature of the terrain is the frequent occurrence of large eskers. The till is mostly sandy or gravelly, only occasionally clayey. Clay tills are found in southern Ontario and in the St. Lawrence River Valley. There are large areas of marine and lacustrine clay deposits in the Ottawa and St. Lawrence valleys and around the coast of Hudson Bay. In general the soil conditions are extremely variable, even over short distances, due to the fact that both Provinces have been fully glaciated.

Glacial deposits are also predominant in the Atlantic Provinces of Newfoundland, New Brunswick, Prince Edward Island, and Nova Scotia. In Newfoundland, the most common subgrade material is a silty till, but there are some areas of marine sediments on the west coast. There is an abundance of water throughout the island, and the ground water table is often very close to the surface. The subgrade soil types in New Brunswick range from rocky tills to clayey tills, and in general the ground water is fairly near the surface.

Highway location within the mountainous National Parks is controlled predominantly by the existence of valleys and alpine passes carved by glacial and river action. Soil conditions are extremely variable over short distances in these areas. The main subgrade soil types are bedrock, glacial and fluvial granular deposits, and a third group comprising glacial tills, very silty gravels, and pockets of pure silt. Although surface drainage is seldom a problem, subsurface drainage is often critical due to the erratic nature of drainage conditions in the mountainous areas.

One important aspect of the common occurrence of glacial soil features, (such as moraines, eskers, and kames) is that they provide an excellent supply of granular materials for highway construction throughout most parts of the country. Except in the areas of the heavy lacustrine and marine clays, granular material is usually available within economical haul distances.

The most dangerous soil types from the frost point of view are the lacustrine and alluvial silts and the resorted glacial tills in the Western Provinces, soils with high short-term capillarity (such as loams and silts), and soils that undergo large changes in bearing capacity with changes in moisture content (such as plastic clays in Ontario and Quebec, and the silty or loamy glacial tills of the Maritime Provinces).

Highway-Operating Conditions

It is naturally the aim of every highway agency in Canada to provide at least a primary system of roads that need not be restricted in the spring because of the weakening of the subgrade due to frost action. The various Provinces differ in their financial ability to achieve this and, in fact, all the Provinces place restrictions on some of their roads in the spring period. In general, all the newly constructed roads on the primary system are designed to remain unrestricted; e.g., the Trans-Canada Highway. However, except in British Columbia, Alberta, and Ontario, the extent of such recent construction is relatively small, and thus the whole of most of the provincial road systems are still subjected to spring restrictions. As time goes on, however, all Provinces will have more and more continuous highway routes that will remain unrestricted.

Restrictions are normally envisaged for all secondary roads and for at least some of the intermediate roads. In most Provinces, restrictions call for half-loading in the spring, but in some Provinces (for instance, British Columbia), loads are limited to a proportion of the full load, depending on the actual measured loss of pavement strength.

Legal limitations during the normal nonrestricted seasons are such that in most Provinces the maximum single axle load is 18,000 lb, the maximum tandem axle load is 32,000 lb, and the maximum load per inch of tire widths is 500 to 600 lb. For instance, in Saskatchewan the following load limit regulations apply: 500 lb per 1-in. width of tire on any wheel or wheel group 18,000 lb per single axle when the axles are more than 8 ft apart, 32,000 lb per axle group when the extreme axles of the group are less than 8 ft apart, 44,000 lb per axle group when the extreme axles of the group are more than 8 ft but less than 20 ft apart. In addition to these limits, there are regulations for maximum gross weights for the various vehicle types.

The volume of traffic on the main trunk routes varies from 1,000 to 40,000 AADT and in certain exceptional cases runs as high as 70,000 AADT.

Four traffic categories are distinguished in the Department of Public Works' practice, based on the daily number of cars, heavy trucks, and buses. Traffic conditions within the mountain parks presently fall into the "medium" category during the summer tourist season, which means that more than 2,000 cars and 50 heavy trucks and buses use the roads each day. Roads being built in the Territories are of relatively low category, and only a small percentage of them have been paved.

On airfields built by the Department of Transport the design aircraft loadings vary from an 11,000-lb single wheel load to a 230,000-lb multiple wheel gear load.

FROST DAMAGE AND CORRECTIVE MEASURES

Heaving Damage

Most Provinces acknowledge the fact that the depth of frost penetration is too great to permit them to construct highways that will be entirely free of frost heave in the winter. Their efforts are therefore directed towards minimizing the detrimental effects of frost heaves. Due to higher standards of modern construction, severe frost heaves occur almost exclusively on older roads that have underdesigned pavements. A uniform heave of 10 in. and a differential heave of 5 in. are regarded as severe.

The most common critical subsurface conditions with respect to frost heaving arise in silty subsoils where the water table is within capillary reach of the frost front. Pockets of silt in glacial tills, or pockets of very fine sand, are also frequently the source of trouble. Grade-points at transitions from cut to fill are liable to become locations of differential heaving, as are areas where there are abrupt changes in the type of embankment material, or cuts having stratified soil conditions. Depressions in underlying ledge rock or in other impervious layers, and excess hydrostatic pressures in the subgrade associated with side-hill cuts also generate heaving conditions.

Bad heaving conditions may develop in areas with heavy showfall if all the snow is pushed onto the shoulders. The snow banks, sometimes several feet in height, act as insulators so that no heaving occurs at the shoulder. The bare roadway, however, has no insulation and deep frost penetration takes place, often accompanied by a heave of several inches with a maximum near the centerline. This results in "backbreaking" stresses and a substantial crack adjacent and parallel to the centerline.

The intensity of heaving depends to a considerable extent on temperature and precipitation conditions. Frost action is normally at its worst in a winter following a very wet fall, and heaving will be accentuated if the descent of the the frost line is slow (according to Alberta experience). In addition, Ontario experience shows that repeated freeze-thaw cycles at the beginning of the cold season will produce large heaves. New Brunswick, on the other hand, reports that most of the heaving seems to occur in the latter half of the winter season, probably when the frost line has reached its maximum penetration. The adverse effect of freeze-thaw cycles is stressed by the experience of the Department of Public Works in the area of the Mountain Parks. Very warm dry winds (commonly referred to as "Chinook") can change the ambient temperature by as much as 60^{0} within a 6-hr period. Such changes usually occur about four times each winter.

Among the airfields for which the Department of Transport is responsible, serious frost damage is restricted to the older pavements which were probably not constructed to present standards. The heaves reported do not generally exceed 4 in., although in one case a heave of 10 in. was reported. The diameter of the area affected varies from 2 ft or less up to 50 ft or more in some cases. On some airfields only a single small area of heaving is reported, whereas on others heaving is reported to be general over the whole field. Of 140 airfields investigated only 24 showed significant frost heaving, and these were in areas having freezing indexes ranging from 400 to 6,600 degree-days. Some of the more serious cases of heaving occur in regions with freezing indexes around 1,000 degree-days. Frost heaves are most often associated with silty subsoils, and though a high water table appears to be conducive to heaving, it is not a prime factor. In several cases, pronounced heaving has been reported in pavements where the water table was more than 10 ft below the surface. Cases have been reported where painted areas of the airfield pavements heaved more than adjacent unpainted areas.

As soon as differential heaves develop, "bump" signs are installed by all Provinces, but actual maintenance work is kept to the minimum consistent with safety. In certain cases, severe differential heaves are eased by placing cold mix patching adjacent to the heaves. This patching is removed with a grader the following spring, as the heaves subside. Heaves are normally recorded on the highway profile in February or March each year for several years before, and for 2 to 3 years after, reconstruction. On roads not scheduled for reconstruction, corrective measures are taken during the summer, in the form of excavation and backfilling with non frost-susceptible granular material, provision of adequate drainage, removal of boulders, etc.

The usual construction practice for Department of Public Works' project is to lay a 3-in. thick asphaltic base course (150 to 200 pen. asphalt cement) within one year of completion of the subgrade, subbase, and base courses. The final pavement is not normally applied until two to three years later or as warranted by traffic volumes and weights. Between these stages of construction the road is kept under close observation to detect any areas of differential frost heaving or other structural deficiencies. Frost heaves are marked and recorded, the height of heaving is measured and the effect of differential heaving on driving comfort is noted so that proper measures can be taken during the summer or fall period.

Thawing Damage

The worst aspect of frost action on Canadian highways is the weakening of the subgrade during the thawing period. Older pavements with inadequate design thickness show considerable damage during the spring and require extensive maintenance; this is particularly true of flexible pavements. Concrete pavements are generally not seriously affected by thawing damage, except for the occasional occurrence of pumping.

According to general experience, thawing damage is dependent on the strength of the pavement system. An extensive study of the seasonal variations in pavement strength has been carried out under the auspices of the Canadian Good Roads Association. All Provinces have participated in this study for which the Benkelman beam deflection test was used to record the variations in strength of sections of pavement chosen to be representative of all soil and traffic conditions. Reductions in load-carrying capacity of the order of 40 percent were reported from Alberta, 50 percent from Ontario, and 30 to 60 percent from New Brunswick.

The soil and moisture conditions that are critical from the heaving point of view are also critical as far as thawing damage is concerned. The worst conditions are again encountered when the water table is high and the subgrade soil consists of silty glacial tills, loam tills, clay loam tills, varved clays, or organic silts. High seasonal loss of subgrade support is reported from the clay-plain areas of Southwestern Ontario. The general experience is that the greatest losses are encountered when a wet fall is followed by a winter with many cycles of freezing and thawing without much snow, and then by a rapid thaw during the spring. If the thaw takes place before the ditches are clear of snow and ice, then conditions become even worse because dispersal of surface and melt waters is very slow. Warm spring rains and high ambient temperatures that cause rapid thawing contribute to heavy thawing damage. In Saskatchewan it has been found that the reduction in load-carrying capacity during the spring period is dependent on the amount of rain during the two preceding years. Some pavement sections that required spring load restrictions at one time have been found to be in less need of restrictions following periods of low rainfall and vice versa.

Spring reduction in load-carrying capacity due to the thawing of frozen subgrades is also the worst aspect of frost action in the airfields maintained by the Department of Transport, and here again the effect is worse on flexible pavements than on rigid pavements.

Both the heaving and thawing phases of frost action often lead to the cracking of the pavement surface. It is an important yearly maintenance task to fill the cracks to keep water out of the base and subgrade. Inasmuch as some of these cracks extend into the subgrade, some sand or similar material is used to minimize the amount of asphalt required for crack filling. Areas of extensive cracking are individually investigated and the corrective measures adopted normally involve excavation and backfilling together with the improvement of the surface and subsurface drainage conditions.

Figure 3 shows an example of corrective treatment applied in British Columbia at frost heave locations. The length and width of the excavation will be dictated by the size of the frost heave and the soil conditions. The excavation, which normally varies from 3 to 4 ft in depth, must extend to one shoulder, each end is cut to a slope of 12 to 1, and the floor is sloped towards the shoulder to ensure proper drainage. The excavation is then backfilled with clean granular material.

The depth of the excavation is normally between 3 and 5 ft in Ontario (with both ends of the excavation tapered off at a slope of 20 to 1); at least one-half the depth of frost penetration according to New Brunswick specifications, and 4 to 6 ft (or down to the lower boundary of the frost susceptible soil) in the practice of the Department of Public Works. Adequate drainage is provided from the bottom of the excavation, which is then backfilled with select granular material.

Other Types of Frost Damage

This section discusses types of frost damage that are of secondary importance as compared to the usual heaving or thawing damage along the main routes of highway. The most common examples are deformation and dislocation of structures, erosion of slopes and shoulders, and surface icing.

Almost all Provinces report differential heaving, resulting in cracking of the pavement surface, at culverts and occasionally at other structures. There have been recorded instances of frost heave lifting bridge piers, but the deformation and dislocation of structures is normally caused by the movement of the fill adjacent to the abutment rather than by the movement of the structure itself. This movement is controlled in recent years by the use of non-frost-susceptible fill material adjacent to structures and by compacting to standard Proctor density to minimize settlement and movement due to freeze-thaw conditions. Complete icing of culverts also occurs occasionally.

The erosion of sideslopes and ditches due to the loosening effect of freeze-thaw cycles is a considerable problem in several Provinces, especially where the predominant soil type is a silt. Severe erosion problems are encountered in the glacial till areas as well. Simple measures are taken to combat sloughing; for example, in New Brunswick, slopes are either sodded or covered with a gravel blanket 6 to 12 in. thick. In Ontario, slopes are normally stabilized either by vegetation or by flattening the slope and are rounded at the top to reduce erosion. Newly seeded slopes are protected with a hay mulch and a light covering of bituminous material to protect the surface. Other means of soil stabilization are pegging, wire-meshing, placement of granular sheet or topsoil. Cut-off ditches are provided where needed and catch-basins may also be installed. Rock slopes are inspected in the spring and pieces loosened by freeze-thaw cycles are removed. On roads built by the Department of Public Works, rock falls due to frost action in excavated solid rock backslopes in mountainous areas are a constant problem. These falls seldom extend to the travel lanes but are a menace

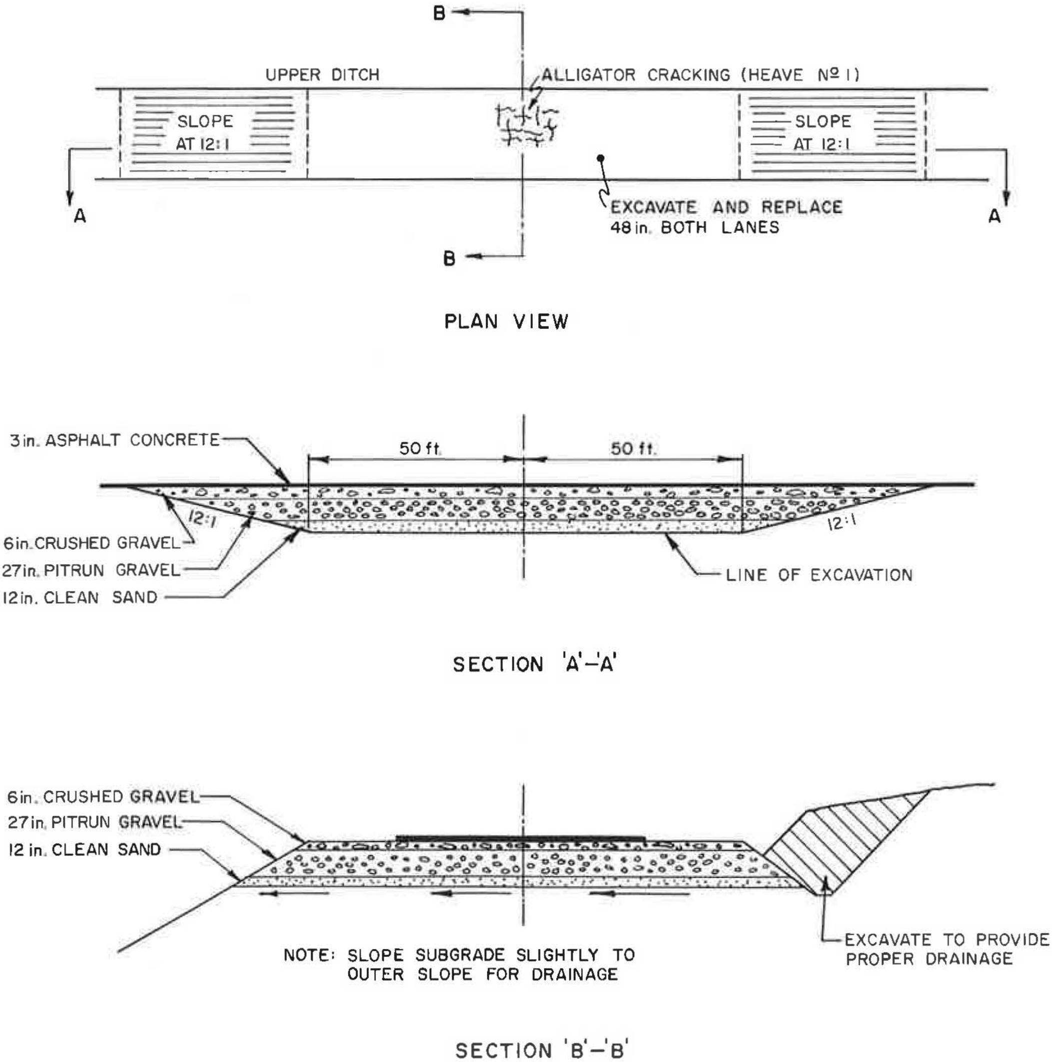


Figure 3. Treatment of a frost heave area in British Columbia.

to effective ditch drainage. No solution, besides the rather impractical step of cutting flatter backslopes, has been tendered for this problem. The erosion of common cuts due to frost slides has been handled effectively by grass seeding and extensive installation of horizontal drains.

Surface icing due to freeze-thaw temperatures in the presence of snow is reported to be a considerable problem in the Mountain National Parks, in New Brunswick, and to some extent in Manitoba. Icing conditions are normally reduced to a tolerable level by spreading salt or sand on the road surface.

On airfields built by the Department of Transport some spalling of portland cement concrete surfaces and deterioration of asphaltic concrete surfaces is evident. This difficulty has been overcome in recent years by more rigid quality control measures during construction. It has sometimes been observed that a relatively sudden, large drop in temperature (say, from 60 to 20 F in 24 hr) has caused cracking in asphaltic pavements. Frequently these are along joints, or coincide with cracks in the underlying pavement in the case of overlays. Such cracks have been observed to extend from the paved area into the adjacent graded area, indicating subgrade contraction. This cracking appears to be more prevalent in granular than in cohesive soils. Structures (such as manholes and catch-basins) extending below the pavement layer are especially prone to heaving. In this condition they constitute one source of damage to snow-removal equipment. In addition, they may fracture concrete slabs at the surface or break subsurface drains.

FROST DESIGN PRACTICES EMPLOYED BY THE VARIOUS AGENCIES

Throughout Canada the depth of frost penetration is such that it is impracticable to provide pavement thicknesses that would prohibit frost penetration into the subgrade. Such thicknesses would far exceed the requirements for bearing capacity, thus construction would definitely be uneconomical. Efforts are therefore confined to minimizing the undesirable effects of frost rather than eliminating frost action altogether.

In this section consideration is given to the three basic factors in the mechanism of frost action, and specific reference is then made to the design practices of the individual agencies.

Temperature Factor

The normal practice of the Provincial highway departments is to consider an average frost penetration depth, based on general experience, for design purposes. Throughout the country, this value ranges from 2 to 8 ft. Except for Northern Ontario, freezing indexes are not normally used to determine expected frost penetration. It was found that designs based on freezing indexes were not economical because the calculated depths of frost penetration were much greater than those normally used in practice.

The Federal Department of Public Works found from field studies that the depth of frost penetration did not depend exclusively on temperature conditions but varied markedly from point to point according to the soil type, the degree of exposure, and the elevation of the site under consideration. As a result of detailed studies made in Banff, this authority evolved a series of empirical relationships that can be used to determine the depth of frost penetration at a particular site.

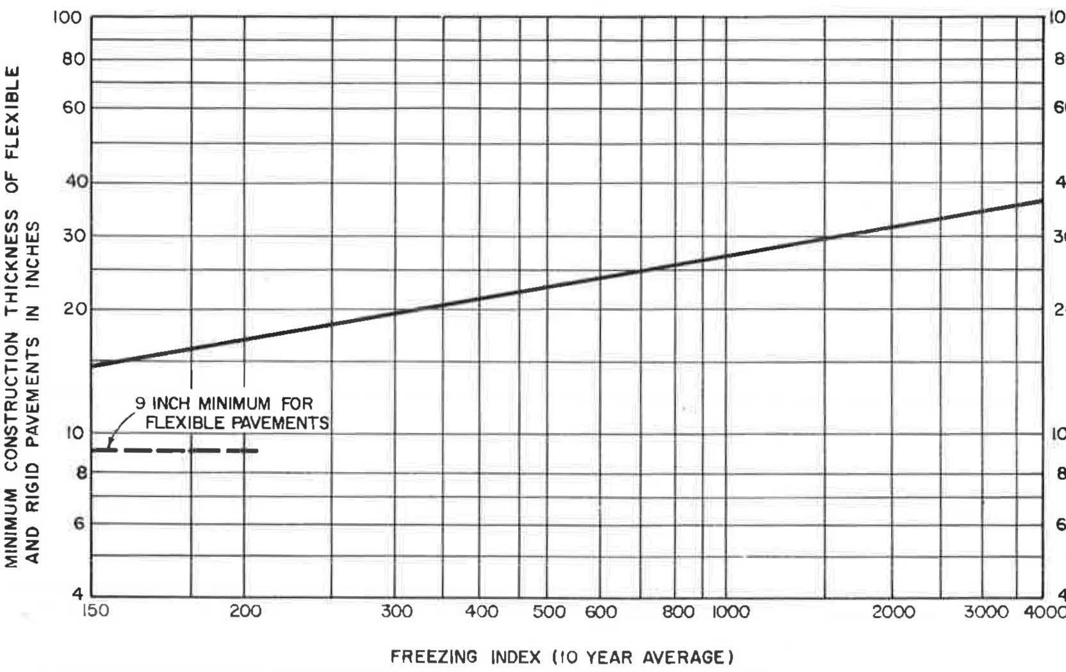


Figure 4. Department of Transport minimum depth of frost protection for flexible and rigid pavements.

Since the early 1950's the Department of Transport design procedures call for a minimum depth of pavement structure (pavement, base, and subbase) of approximately one-half the average depth of frost penetration, regardless of strength requirements. This minimum depth is determined normally from the 10-year average freezing index for the site, according to Figure 4. This method has been found to be effective in reducing frost damage to a minimum.

Soil Factor

All except one of the Canadian highway agencies base their frost design on the degree of frost susceptibility of the subgrade soil as established by various criteria based essentially on particle-size distribution and Atterberg limits. The exception is the Province of British Columbia which makes no direct use of the soil characteristics in estimating the frost susceptibility of the subgrade. In this Province, pavement design is based on the results of deflection tests made on the surface of existing pavements at different times of the year. The tests carried out in the spring period reveal the weakening effect of the thawing process, and this weakening is taken into account in the Province's design method. Identification tests are carried out on the subgrade soils, however, because the seasonal deflection studies are made only on a relatively small number of control sections, and the estimation of the loss in strength of a particular section of pavement in the spring involves the selection of a control section on the same type of soil.

All the other agencies make use of the results of the sieve analysis, liquid limit, and plastic limit tests to identify the frost-susceptible soil types. Two Provinces (Alberta and Saskatchewan) make use of the U.S. Corps of Engineers' system of classification. The Department of Public Works uses the criteria of Beskow and Casagrande; Manitoba, Ontario, and New Brunswick have related their frost experience to the proportion of silt and clay in the soil. Quebec, Nova Scotia, and Newfoundland also base their assessment on the amounts of fine material in the subgrade soil.

None of the agencies reported the use of laboratory freezing tests in their evaluation method.

In the National Parks, the Department of Public Works requires that A-4 or finer soils be kept at least 3 ft below the final pavement level, and that all organic material be removed from the subgrade. A-6 or finer soils are kept 5 ft below the pavement level, whereas all A-2 to A-4 and cleaner granular soils are selectively placed in the top 2 ft of the subgrade (they are also used as cushion over rock excavation).

In Manitoba, soils with clay fractions less than 20 percent and combined silt and sand fractions over 60 percent are considered frost susceptible. Some borderline soils with clay contents as high as 30 percent may be considered frost active.

Ontario, New Brunswick, and Newfoundland make use of empirical design charts in which the minimum thickness of nonsusceptible granular cover is obtained directly from the particle-size characteristics of the subgrade soil. The Ontario chart (Table 2) is based on the silt and clay fractions, and gives design information for flexible and rigid pavements. The Newfoundland chart (Table 3) relies on the percentage passing the No. 200 sieve. The practice in New Brunswick is to place a 1 to 2 ft layer of nonsusceptible material on soils with silt content between 35 and 50 percent. When the silt content exceeds 50 percent, a 3-ft thick layer of nonactive material is required together with a 6-in. thick filter layer to prevent fine-grained soil from working up into the overlying granular materials. Experience has shown that when the silt content of granular materials reaches 6 to 8 percent, and the moisture supply is ample, heaving is significant and thawing is accompanied by loss of density and bearing capacity. When certain clay loam and loam tills contain mica in small sizes (passing the No. 200 sieve), the frost susceptibility of these soils appears to be increased and their recovery is delayed.

The criteria used in Quebec are shown in Table 4. In general, any soil with more than 8 percent passing the No. 200 sieve is regarded as potentially frost susceptible. In Nova Scotia, the only property used to assess frost susceptibility is the amount of material passing the No. 200 sieve.

TABLE 2

ONTARIO DESIGN THICKNESSES

Soil Type	% Silt[a]	% Very Fine Sand and Silt[a]	Minimum Granular Cover[b] (in.)	
			Flexible Pavement	Rigid Pavement
Granular:				
Base course			0	0
Subbase course			4- 9[c]	4[c]
Intermediate	0- 40	0- 45	12-21	9
	40- 50	45- 60	18-24	9
	50-100	60-100	24-36	9-12
Clay[d]:				
Nonvarved			18-24	9
Varved			18-36	9-12

[a]Grain-size limits for silt are 0.005 to 0.05 mm, for sand 0.05 to 0.1 mm.
[b]Range in thicknesses covers various classes of roads.
[c]Required for stability and not for frost protection.
[d]If percentage finer than 0.005 mm is greater than 30, material is classified as clay.

TABLE 3

NEWFOUNDLAND GRANULAR BASE THICKNESSES[a]

Frost Susceptibility Classification	% Passing No. 200 Sieve	Thickness of Granular Base (in.)
Non susceptible	0- 6	3[b]
Moderately susceptible	6- 8	6
	8-10	9
	10-12	12
Susceptible	12-15	15
	15-19	18
	19-24	24

[a]It is assumed that 3 in. of Class A granular base-course material will always be used, the remaining thickness to consist of Class B granular base-course material.
[b]Leveling course.

TABLE 4

QUEBEC FROST-SUSCEPTIBILITY CRITERIA

% Passing No. 200 Sieve	% Silt and Fine Sand	Frost Susceptibility Classification
0 - 10	0 - 20	Non susceptible
10 - 30	20 - 40	Susceptible
>30	>40	Very susceptible

The Department of Transport uses a zoned particle-size distribution diagram (Fig. 5), in conjuncture with pavement condition reports and data concerning the ground water conditions, to estimate the probable spring loss in bearing capacity when actual spring load-test information is not available. This diagram gives a load reduction factor which is used in the Department's pavement design method (1).

Most agencies recognize the risk involved if boulders are introduced in fill materials or are left in the upper layers of cuts. Ontario, for instance, does not permit boulders over 6 in. in diameter within 36 in. of profile grade. New Brunswick excludes boulders over 8 in. in diameter from the top 24 in. of the subgrade; Manitoba allows none over 4 in. in diameter in the top 12 in.; and Newfoundland requires that all borrow material be smaller than a 6-in. maximum size. The Department of Transport requires that all boulders be removed from the top 24 in. of the subgrade.

Moisture Factor

All agencies recognize the important effect of ground water on frost action, and all except two require that the subgrade surface shall be at a specified minimum height above the ground water table. The two exceptions are British Columbia, where the minimum height requirement is replaced by the specification that ditches shall go down to the bottom of the base gravel, and Newfoundland, where frost design is based exclusively on the control of the soil factor.

As an alternative to the minimum height requirement, certain Provinces call for the use of a granular or clay capillary cutoff layer in circumstances where this treatment is more economical.

The ground water table is normally kept at allowable level by means of side ditches. Subdrains are not generally used by any of the highway agencies except to cut off seepage flows. The Department of Transport employs subdrains along all paved surfaces to drain the base and subbase layers. In some Provinces, the general practice now is to construct full-width granular subbases and bases to facilitate drainage.

British Columbia

The design of flexible pavements in British Columbia is based entirely on studies of the bearing capacity changes that occur on existing pavements at different times of the year. No direct account is taken of the varying frost susceptibility of the subgrade soils because this is reflected in the seasonal fluctuation of bearing capacity.

The method of design involves the seasonal study of load-bearing capacity of a number of control sections, using the Benkelman beam field test to measure the deflection of the pavement under a standard axle load. These deflections are converted, using a Department of Transport relationship between the Benkelman beam deflection and the plate-bearing capacity for a 30-in. diameter plate, to equivalent load-carrying capacities. The records of the seasonal fluctuations in load-carrying capacity are used to estimate the loss in bearing capacity that will take place on the section of pavement under consideration between the time at which the tests are made and the worst period in the spring. Once the minimum bearing capacity of the pavement has been estimated, the MacLeod equation $T = K. \log P/S$ is used to calculate the thickness required to reduce the estimated worst deflection to the maximum permissible deflection for the particular type of road. A detailed description of this procedure is given in the Appendix.

Experience has shown that the design thicknesses derived by the preceding method are very similar for frost-susceptible and non-frost-susceptible subgrades.

In British Columbia the validity of specifying a minimum depth to the water table is questioned, as heaves have been observed in fills 10 ft and even 15 ft high. In this Province, ditches are required to go down to the level of the bottom of the base gravel.

The selection of subgrade materials is not normally practiced because the pavements are designed for the actual soil conditions existing at the site. If the subgrade is very poor, its weakness will be revealed by load tests and the thickness of structure will be increased accordingly.

Compaction control requires proof-rolling of the upper 2 ft of the subgrade using a

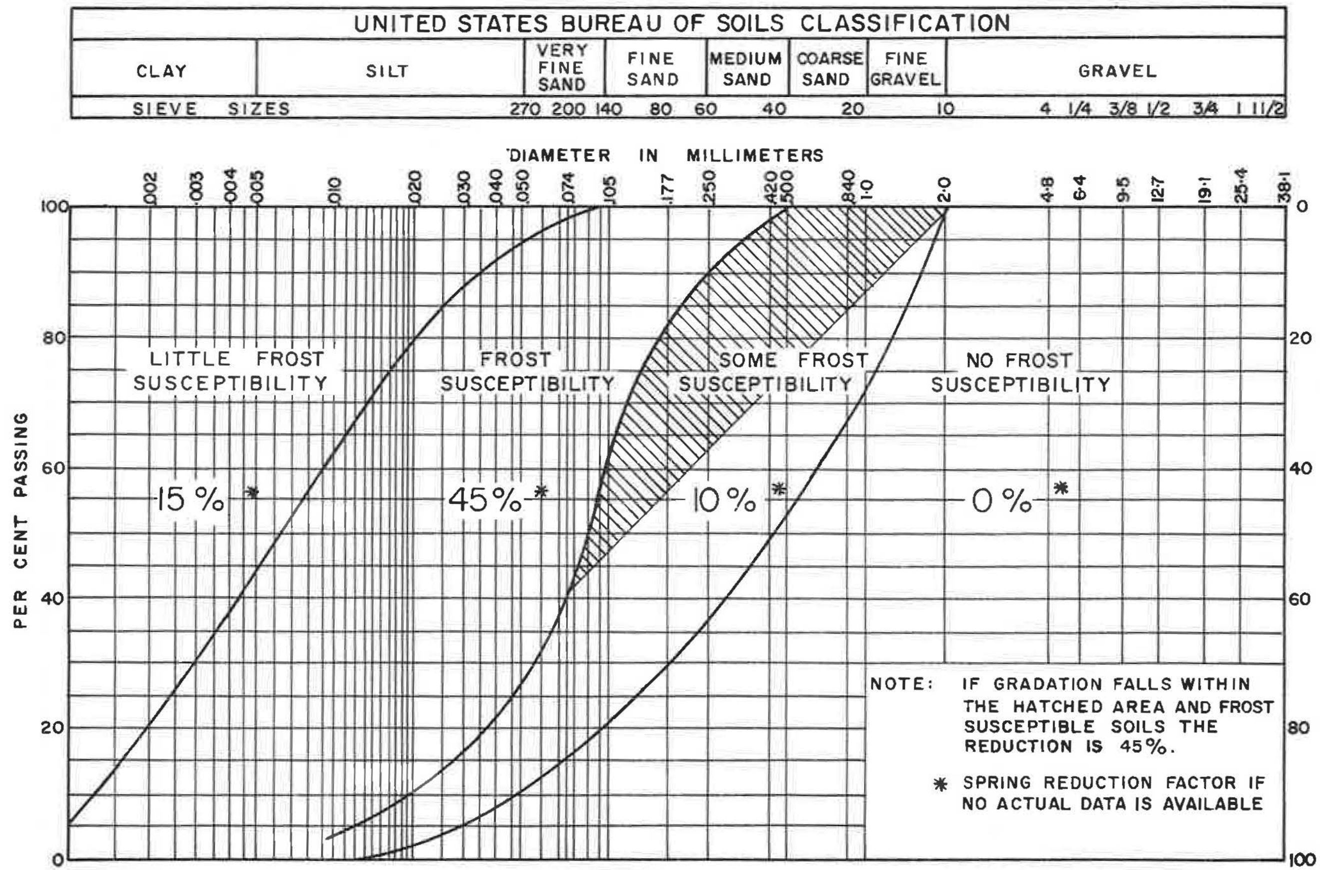


Figure 5. Department of Transport spring reduction factors.

50-ton roller. In the course of this testing weak spots that may be revealed are either recompacted or, if this does not achieve the desired strength, the trouble spots are replaced with better material. Construction is discontinued in freezing temperatures and, as a protective measure, crossfalls of 0.02 ft per foot are maintained on all earthworks to facilitate runoff.

Alberta

In Alberta wet and frost susceptible materials are usually replaced with select clay or granular material, and the ends of the excavations are sloped to prevent abrupt changes in the frost susceptibility characteristics of the completed subgrade. This replacement of susceptible or wet material is not always practicable, because in large areas of the Province no alternative materials are available. Under these circumstances some success has been achieved in minimizing frost action by special ground water control measures.

Perforated pipe subdrains or filter layers are used in cut sections and in subsurface excavations at traffic interchanges. Subdrains are also used to cut off seepage. Wherever possible these drains are installed below the frost line. It is now general practice to employ full-width granular bases to facilitate drainage of the pavement layers. Ditch bottoms are 5 ft below subgrade level.

Compaction requirements are as follows: fill materials, 95 percent of standard Proctor density; subgrade (top 12 in.) and base materials, each 100 percent of standard Proctor density.

Saskatchewan

In Saskatchewan a minimum of 4 ft of non-frost-active soil is specified for fills placed on frost-active foundations, or in cases where the water table is less than 10 ft below the ground level. In urban areas or in other circumstances where elevations are controlled (e.g., at level crossings), the top 4 ft is excavated and replaced with non-frost-active backfill. It is recognized that even the non-frost-active soils (including most base and subbase materials) support frost heaving to some extent and thus some heaving is more or less inevitable. It is attempted, however, to avoid or minimize differential heaving by making soil conditions as uniform as possible; e.g., by the use of gradual transitions at all discontinuities in the subgrade.

To facilitate drainage, grade lines are set 2.5 ft above the natural ground level. Grades are normally set 5 ft above the ground water level, but on silty soils this height is increased to about 8 ft. In some cases a granular cutoff layer placed above the water table might be required as an alternative; the thickness of such a layer would depend on the capillarity of the cutoff material. As another alternative the silty material may be excavated and replaced with suitable backfill soil. Subdrains are employed sometimes to improve drainage. In certain cases they are installed above the frost line and are expected to function during the warmer seasons only.

In bases and the top 12 in. of the subgrade, 100 percent of standard Proctor density is required, and 95 percent is required in the remaining fill material. The placing of material in thin layers is rigorously controlled and no work is permitted on the base course when the temperature falls below 35 F.

Manitoba

Frost-susceptible soils are generally replaced to a depth of 3 ft with selected granular material, clay, or other nonsusceptible soil. The backfill is placed in 6- to 8-in. thick layers, compacted to 95 percent of AASHO standard density. Grades are set at least 4 ft above the water table, and in silty soils this is increased to 5 ft.

Side ditches are used with backslopes and sideslopes at 4 to 1, and the ditch bottom is sloped away from the subgrade. The width of the ditches is between 25 and 30 ft, depending on the right-of-way and earth quantity requirements. Earthwork is discontinued when the soil begins to freeze, and no paving is permitted when the temperature falls below 40 F.

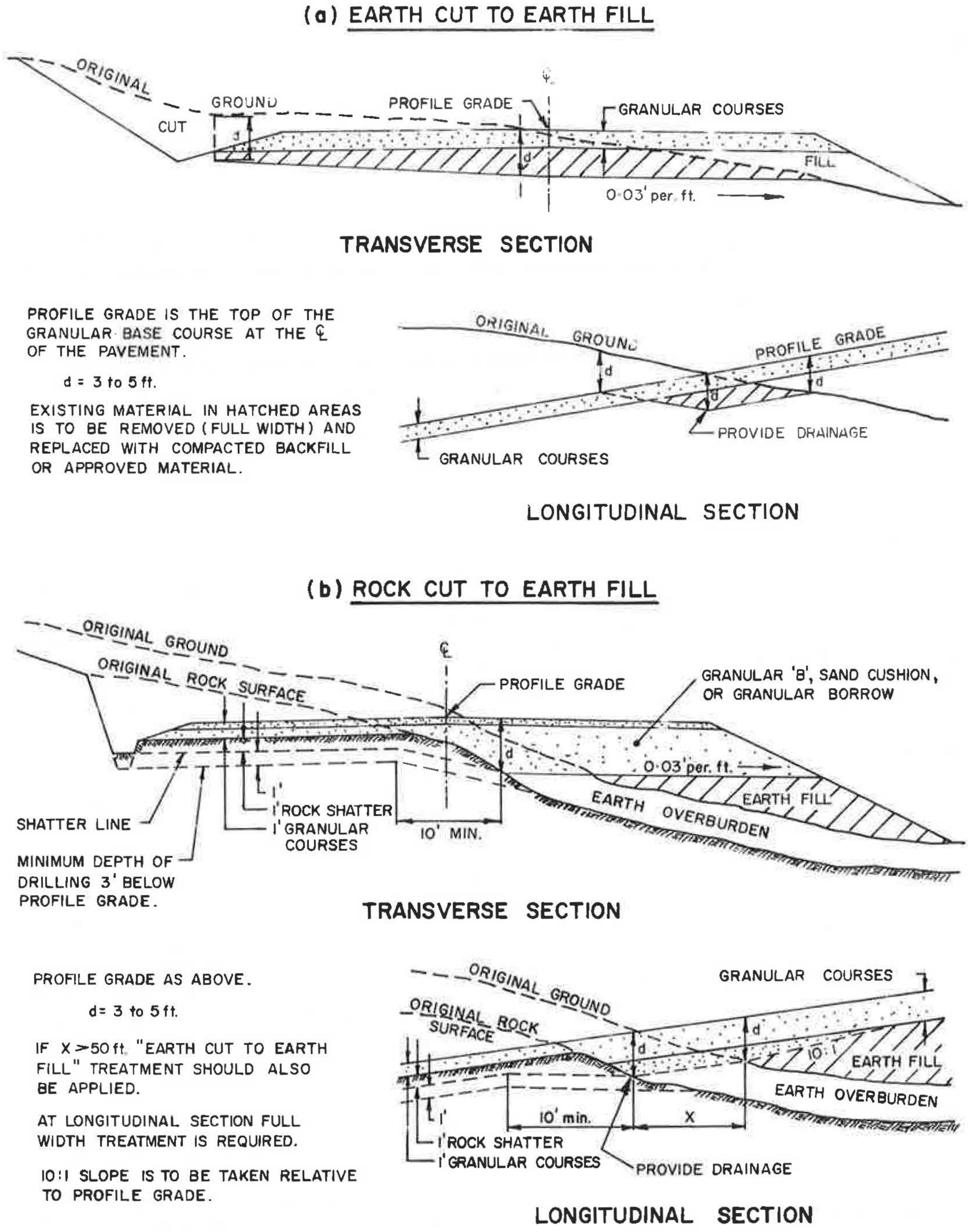
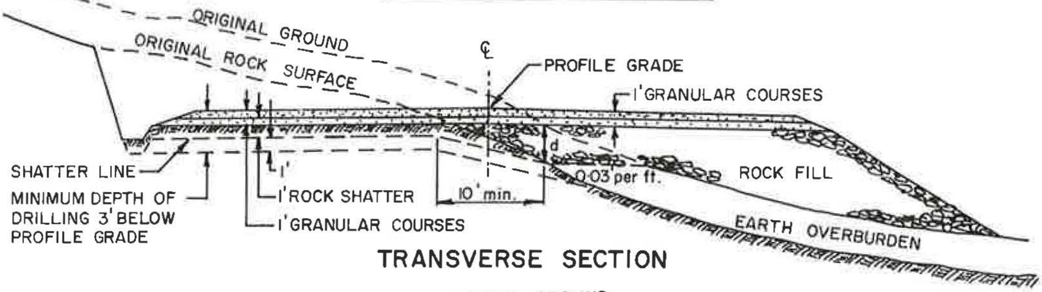


Figure 6. Ontario treatment of transition sections.

(c) ROCK CUT TO ROCK FILL



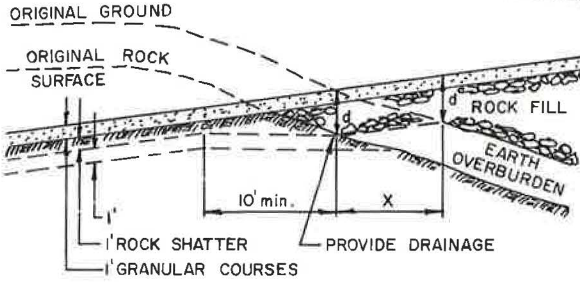
TRANSVERSE SECTION

PROFILE GRADE IS THE TOP OF THE GRANULAR BASE AT THE ℄ OF THE PAVEMENT.

d = 3 to 5 ft.

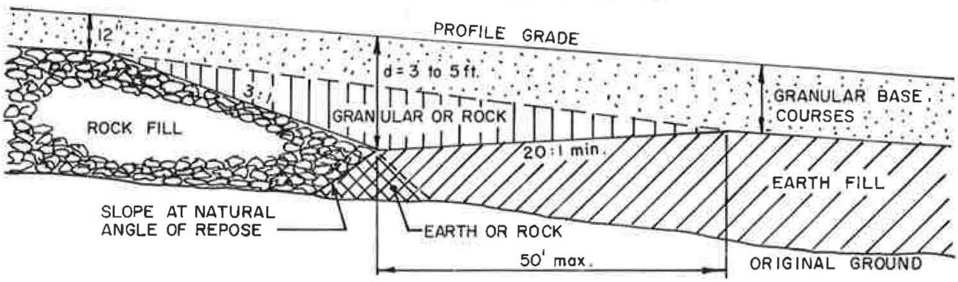
AT LONGITUDINAL TRANSITION, FULL WIDTH TREATMENT IS REQUIRED.

IF X>50 ft., "EARTH CUT TO EARTH FILL" TREATMENT SHOULD ALSO BE APPLIED, EMPLOYING ROCK BACKFILL.



LONGITUDINAL SECTION

(d) EARTH FILL TO ROCK FILL



(e) EARTH FILL TO GRANULAR FILL

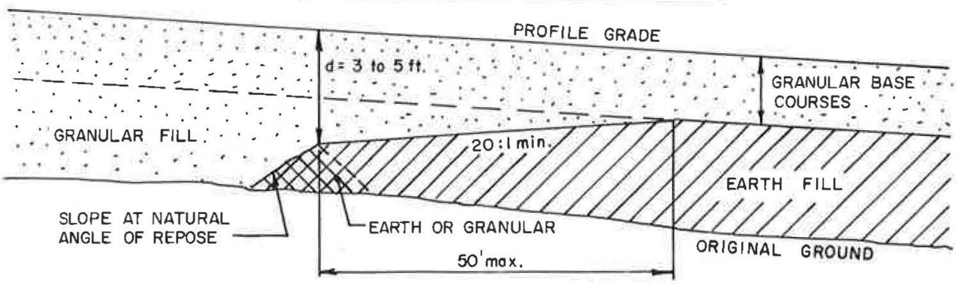


Figure 6. Continued.

Attempts have been made to use calcium chloride and sodium chloride to reduce the frost susceptibility of soils. The evaluation of these tests has not yet been completed and the method has not been accepted for general practice.

Ontario

Grades are kept at least 4 ft above the ground water table and 1 to 4 ft (or more) above ground level, depending on the drainage characteristics of the subsoil. There is a tendency to set the grade 3 ft above the ground line regardless of internal drainage characteristics, for snow control purposes. Subbases are generally placed full width and side ditches are constructed at least 18 in. below subgrade elevation. When this is not possible (e.g., due to property restrictions), perforated pipe subdrains are placed.

Pavements are usually designed for a thickness of about 0.5 to 0.75 times the depth of frost penetration. The minimum thicknesses of granular material to be used on subgrades of the various types are shown in Table 2. Though the thicknesses given by this table are used as a general guide, special account is taken of the following factors in each particular case: depth of frost penetration in the area, frost-susceptibility of the subsoils as revealed by by special investigations, susceptibility of subsoils to large changes in bearing strength, adverse moisture conditions and poor drainage, topography, and type of road.

Transition points between cut and fill require special treatment if differential heaving is to be avoided, and a series of standard treatments have been evolved for use under the various combinations of circumstances which are met in practice. These standard treatments are shown in Figure 6.

Compaction requirements for earthworks call for a minimum of 95 percent of the maximum AASHO density. For granular base and subbases, 100 percent of the maximum AASHO density is required. Construction is normally suspended in early December and no paving is permitted when the temperature falls below 35 F.

Quebec

The general rule in Quebec is to ensure that no frost-susceptible material lies within 4.5 ft of the final grade. The frost-susceptible material is either excavated and replaced with nonsusceptible material or the gradeline is raised to provide the minimum clearance.

In areas of high water table a cutoff blanket of nonsusceptible sand or gravel is used, and a sand cushion is placed at the base of embankments between the natural soil and the embankment material in cases where the subgrade is a fine-grained soil. It is now general practice to employ full-width granular construction. Side ditches are carried down to a level 18 in. below subgrade.

Newfoundland

In Newfoundland highly frost-susceptible materials are removed from the subgrade but, contrary to the general practice in the other Provinces where the replacement material is usually a selected granular material, the material removed is replaced with soil similar to that of the rest of the subgrade. The use of granular backfill material is not favored, as this is thought to produce differential conditions in the subgrade which will result in differential frost heaving. The edges of excavations are sloped back gradually to provide transitions between the different sections of the subgrade.

Rigorous compaction control is not exercised partly due to shortages of trained personnel, but also because an interval of at least 12 mo usually elapses between subgrade preparation and paving operations, during which time weak points will normally be discovered and treated. Full-width granular base-course construction is used and specifications require that all granular material should contain between 3 and 6 percent of material passing the No. 200 sieve. The thickness of granular base courses is based on the silt content of the subgrade (Table 3), and so far, the performance of the roads constructed to these requirements has been quite satisfactory.

Economic circumstances curtail freedom to choose borrow material, but when borrow materials are used they are chosen with the lowest susceptibility characteristics consistent with economy. The placing of frozen materials is not permitted.

Prince Edward Island

The general principle of frost design in Prince Edward Island is to remove highly frost-susceptible soils or to replace them with materials of better quality. In cuts, the subgrade is undercut by a minimum of 18 in. to ensure a homogeneous subgrade. The natural water table is normally lowered by deep ditching. Sand layers are sometimes used between the subgrade and the granular base in areas of high water table. Where rock strata intersect the subgrade and would lead ground water to it, the rock is shattered by drilling and blasting. Soil cement is being used on an expanding scale to improve bearing capacity and to minimize differential frost heaving.

Nova Scotia

Frost design practice in Nova Scotia is very similar to that in the other Maritime Provinces. Grades are set 4 ft above the water table and full-width granular construction is generally used. The criteria for base course materials are that not more than 10 percent should pass the No. 100 sieve and not more than 5 percent should pass the No. 200 sieve. The steepest slope used in earthworks is 2 to 1 and the minimum ditch width at the bottom is 4 ft.

New Brunswick

In New Brunswick the depth of frost penetration is considered a check on the upper limit of pavement thicknesses. Grades are set 4 ft above the water table and frost susceptible materials are usually removed and replaced with selected granular material. At grade points, the subgrade is excavated to a depth of 2 ft. The cut is carried forward toward the fill at a minimum gradient of 0.5 percent, and toward the cut at a gradient parallel to the slope of the natural ground or to maintain a depth of cut of 2 ft beneath the gradeline.

Subgrade soils are compacted to a minimum of 95 percent of standard Proctor density and granular materials are compacted to 100 percent of standard Proctor density. Paving is discontinued when the ambient temperature falls below 40 F.

Department of Public Works

Grades are set 3.5 ft above the water table in cuts and 4.5 ft in fills. Ditches are kept at least 1 ft below the subgrade level in areas of frost-susceptible soil and high water table. Compaction standards call for soils to be compacted to 95 percent of standard Proctor density, with the top 6 in. of the subgrade compacted to 100 percent. Sand filters are used on top of subgrades containing saturated silt soils.

In cuts, frost-susceptible soils are excavated and replaced with suitable nonsusceptible material. Where subexcavation is not required and a granular subbase is provided, the subgrade soil is scarified to a depth of 6 in., and recompacted to 100 percent Proctor density. If no granular subbase is placed, the top 12 in. of the subgrade is treated in a similar manner.

Full-width granular construction is the normal practice. Granular materials are restricted to a maximum of 8 percent passing the No. 200 sieve and are compacted to 100 percent of standard Proctor density. The paved surface includes the shoulders.

During construction, a 6-in. crown is maintained on earthworks to facilitate drainage. Embankment material is not placed in a frozen state nor on a frozen surface.

Department of Transport

The Department of Transport flexible design procedure for airfields assumes that the bearing capacity of a pavement surface is proportional to that of the subgrade, so that the spring reduction in subgrade bearing capacity is a very important design consideration. The effect on flexible pavements is much more severe than on rigid pavements. In the rigid pavement design procedure employed (Westergaard, central loading condition), the effect of the subgrade strength on the strength of the pavement is more complex but, in general, a 50 percent reduction in the subgrade strength will only reduce the load-carrying capacity of the slab by approximately 20 percent. However, there is evidence that the subgrade or base has a greater influence on the load-carrying capacity of a concrete slab than is apparent from the Westergaard theory.

In the case of rigid pavements the thickness of the pavement structure is almost invariably governed bv the frost protection requirements, whereas in the cases of flexible pavements, the pavement depth is governed by frost in the case of light aircraft loads or strong subgrades, and by strength conditions in the case of weak subgrades or heavy aircraft loads. Where the existing subgrade consists of a nonsusceptible granular soil, the minimum depth requirements may be waived.

Grading requirements call for the removal of all frost-susceptible material to a depth of 3 ft below the pavement surface. If such materials occur in isolated pockets, removal to greater depths may be required by the engineer. Where changes in subsoil conditions occur, transitions are made as gradual as possible.

All fills are compacted to a minimum of 90 percent of modified Proctor maximum density, and the top 5 in. of cohesive soils or the top 12 in. of granular soils are compacted to a minimum of 95 percent of modified Proctor density. In cut sections, the soils are scarified and recompacted in a similar manner. Subbases are compacted to 98 percent and bases to a minimum of 100 percent of the modified Proctor maximum density.

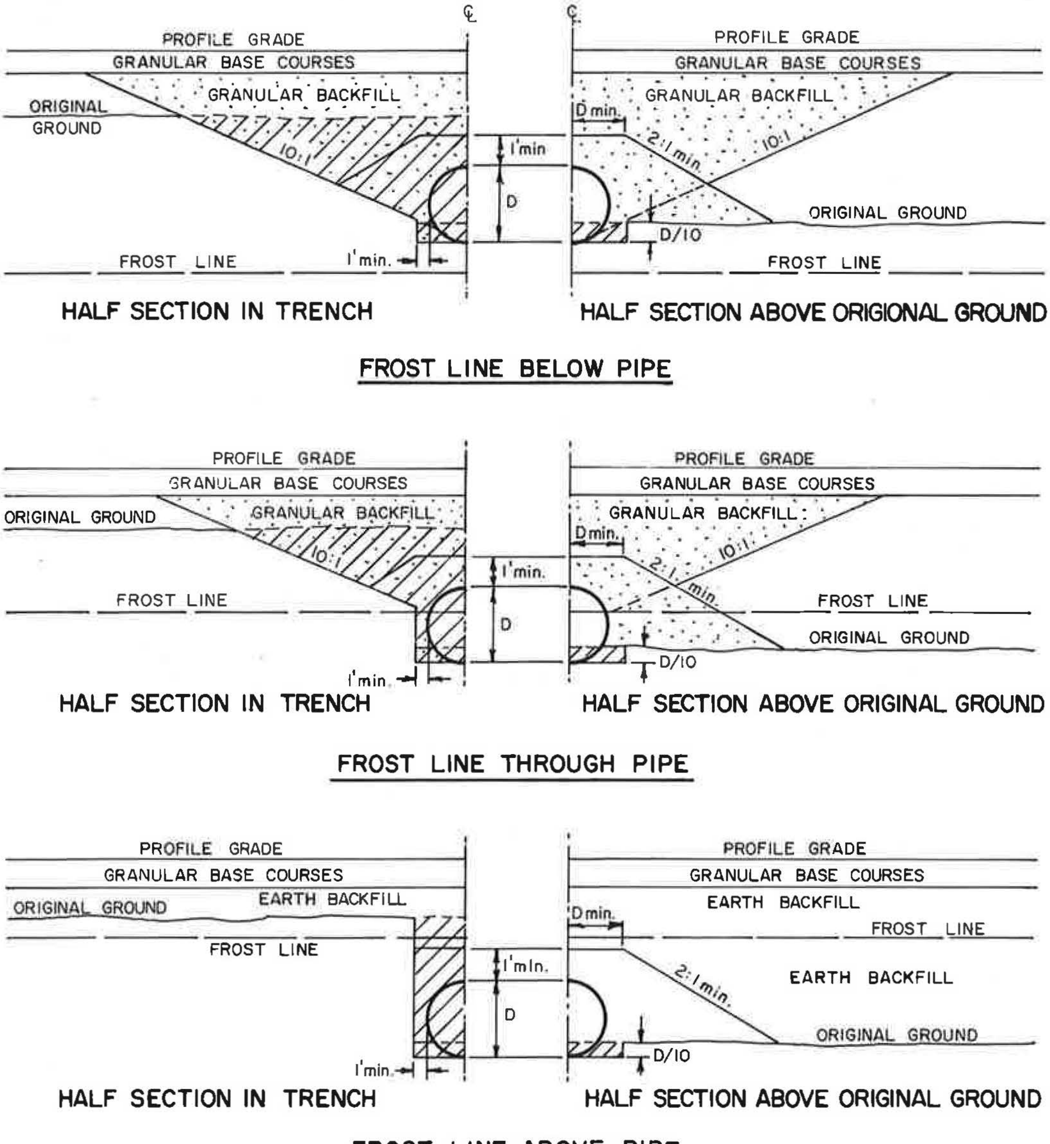
Except in clean granular soils, perforated metal drains are laid along all pavement edges to drain the base and subbase. If possible these pipes are placed below the frost line and the trenches are filled with graded filter material. In rock cuts, it is common practice to remove some of the rock, replace it with granular material, and in addition, to shatter the surface of the rock to a depth of approximately 12 in. to facilitate drainage further.

The Department of Transport normally attempts to keep the finished pavement surface at least 3 to 4 ft above the ground water level. The selection of the minimum height depends on such conditions as climate, soil type, drainage characteristics, and economic considerations. When the depth of ground water table is less than 3 ft, the spring load reduction factor (Fig. 5) is increased by 10 percent on all soil types.

Special Treatment at Structures

In the cases of bridges, foundations are normally located below the frost line so that the bridge structure itself will not normally be affected by frost action. The main treatment at structures involves the use of nonsusceptible granular material as backfill. All agencies specify the use of such granular material and all stress the need to ensure that the adjacent fill is cut back at a slope to make the transition from the fill to the structure as gradual as possible, in order to minimize differential movements and heaving between the pavement and the structure itself. British Columbia, for instance, requires the use of non-frost-susceptible material in backfills within 50 ft of bridge abutments. New Brunswick requires the slope of the surface of the fill to be beveled back at the rate of 1 in 4. Ontario calls for slopes of 10:1 from the point where the frost line intersects the backface of the abutment.

Similarly to bridges, granular backfill is used to avoid differential heaving at culverts. The Department of Public Works specifies 8 in. of granular material below the invert of the culverts and the use of granular backfill to a point 1 ft above the top of the culvert. Where the culvert is placed below the frost line, the trench is cut back at a slope of 10:1 or to a maximum of 50 ft. The standard design practices adopted in Ontario are shown in Figure 7.



NOTES.

D = INSIDE DIAMETER OF PIPE. WHERE PIPE-ARCH IS USED SUBSTITUTE SPAN FOR D.

HATCHED VOLUMES TO BE EXCAVATED AND BACKFILLED.

GRANULAR BACKFILL, WHERE APPLICABLE, TO EXTEND ACROSS ENTIRE ROAD SECTION.

Figure 7. Use of granular backfill at culverts (Ontario).

SPRING LOAD RESTRICTIONS

Although new sections of the primary highway routes are being designed and constructed to the standards required to carry the maximum authorized loads throughout the year, some Provinces are still constructing secondary highways with the intention of applying load restrictions in the spring period. In addition to these new sections of road, there are large mileages of older roads that must be protected in the spring by limiting loads, and the various Provinces have evolved their own techniques for deciding when and where to apply these limits, and what the limits shall be.

In British Columbia the period of restriction and the extent of the load reduction is fixed on the basis of the results of deflection tests made on a number of specially selected control sections at frequent intervals throughout the year, and particularly in the spring period.

The deflection tests are made with the Benkelman beam, using the standard axle load and the standard test procedure of the CGRA (2). As the most important tests are made when the pavements are in their weakest condition and the results may be influenced by the size of the deflection bowl, the CGRA procedure for correcting this possible error is rigorously followed. Also, as the strength of asphalt pavements is affected by their temperature, the test results are converted to an equivalent deflection at 32 F, using a correction factor determined experimentally by the Department of Transport (0.001-in. deflection for every 5 F difference in temperature). On each of the control sections, which are chosen to be as representative as possible of the complete range of climatic, soil, traffic and drainage conditions in the Province, 10 deflection tests are made at randomly selected positions.

After correction for possible inaccuracy due to large deflection bowls and for temperature, the mean value for the control section is found, and to this is added 1.5 times the standard deviation of the 10 results. The resulting figure is a prediction of the deflection value which would be exceeded only in six tests in every hundred if an unlimited number of tests could be carried out; it is called the "restriction deflection."

Experience has shown that if pavements are to remain undamaged in the spring under normal vehicle loadings the maximum deflection obtained by the standard procedure should not exceed the "critical deflection" of 0.040 in. If the restriction deflection exceeds the critical deflection, vehicle loads are restricted to a percentage of the normally allowed maximum load which would produce a restriction deflection of 0.040 in. if applied to the pavement at the worst time in the spring period.

It is possible to calculate the permissible percentage of normal full load for a given pavement from a knowledge of the restriction deflection and the critical deflection using a Department of Transport relationship between Benkelman beam deflection and plate load capacity (3), but in practice it is read directly from Figure 8. The two values of deflection plotted on the chart and the zone into which the point falls indicates the permissible percentage of full load. As a general rule the permissible load is not reduced below 70 percent of full load on primary highways and below 50 percent on secondary highways.

In a somewhat similar manner, the results of Benkelman beam tests serve as basis to determine the time, duration, and location of the spring restrictions necessary in the Province of Alberta.

In Saskatchewan, as soon as the frost begins to leave the pavement, a number of Benkelman beam trucks are placed on continuous circuits to test the paved highways. These tests are continued well beyond the critical spring period. These results and the observations of the Maintenance Branch District engineers are the information on which load restrictions are based. The load restrictions are usually in effect by April 1 and removed by May 15. The deflection tests are taken on the weaker controlling sections between destinations for commercial traffic. The restriction limit is either 350 lb per inch of tire width (gross limit 72,000 lb) or 250 lb per inch of tire width (gross limit again 72,000 lb). Restrictions are not specifically related to climatic conditions. They are related to pavement type and soil type, in that the control sections are the weakest pavement sections between destinations.

Observations and past experience are used to predict the time of load restrictions

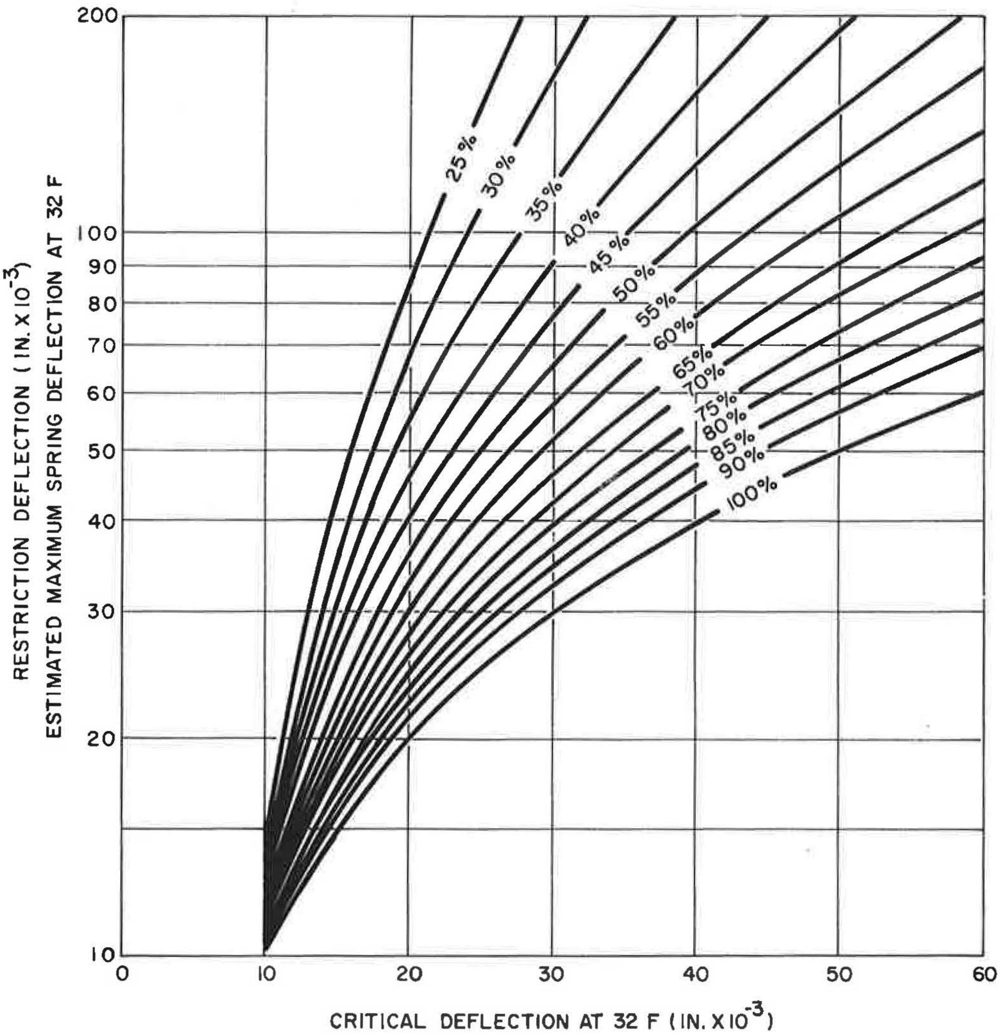


Figure 8. British Columbia load restriction chart.

in Manitoba. This is done in conjunction with Benkelman beam deflection data which has become an increasingly more important factor in placing of load restrictions. Restrictions are generally placed on highways in the northern part of the Province at a later date than those in the southern part, with the average period being April 1 to June 1. Roads other than Provincial trunk highways are generally restricted to 350 lb per inch of width of tire or 250 lb per inch of width of tire, and the maximum load is sometimes limited to 6,000 lb. No restrictions are imposed on concrete pavements.

Half-load restrictions are applied to certain highways in Southern Ontario and to most highways in the northern part of the Province. Most secondary highways and country roads throughout the Province are subject to spring restrictions. The actual sections of road to be restricted, the start of the restriction and the duration are decided by the Maintenance Branch on the basis of past experience.

The method of determining the needed load restrictions in Newfoundland is somewhat arbitrary at the present time, and is generally carried out on the basis of the experience of the District maintenance personnel. When restrictions are imposed, commercial vehicles are limited to one-half their registered gross loads. Half-load limit signs are posted on all roads affected and the public is informed of restrictions by the news media.

In New Brunswick, the duration of load restrictions and the time of lifting these restrictions are determined by the Benkelman beam deflection method. For purposes of weight restriction application the Province is divided into the southern and northern halves, between which there is a time difference of about 2 to 3 weeks both in imposing and lifting of weight restrictions. There are two main reasons for this: the difference in latitude, and the fact that the northern half is at a generally higher elevation than the southern half. Soil types also cause some difference in timing the lifting of the weight restrictions, because of the slow rate of strength recovery shown by certain soils.

Alberta, Nova Scotia, and Quebec also employ the Benkelman beam to measure the spring reduction of load-carrying capacity.

TRENDS AND RESEARCH NEEDS

Recent developments resulting in a better understanding of the complex phenomena of frost action contributed considerably to improved design and construction methods throughout the country. Research findings of such organizations as the CGRA, NRC, HRB, and certain universities are often quoted and gradually more and more used in the everyday practice. Each Province reports a major upgrading of design standards in recent years, and that the problem of frost action is generally being given more consideration during the investigation, design, and construction stages. As a result, gradually more and more applied research work connected with frost action problems is being carried out by the various highway agencies, which in turn will inevitably contribute to a better understanding of the underlying theoretical principles. Samples of present research activities and further needs follow.

In British Columbia major emphasis is being placed on improving the technique and interpretation of the Benkelman beam method in evaluating frost heave problems. Some consideration is being given to establishing a research project on injecting chemicals into the roadbed in frost heave areas.

Steps have been taken in Saskatchewan to correlate freezing index with the required thickness of non-frost-active material under the road surfacing. There appears to be considerable merit in this approach. On the basis of observations, tests on winter core-samples, and data reported by investigators outside the Province, the criteria for frost susceptibility have been improved. This improvement should continue with the evaluation of existing special design sections in frost-active areas. Further field and laboratory research is considered necessary to establish better design methods in areas of high freezing indexes.

Manitoba reports that frost heave measurements are being taken on several highways involving several different soil types, and are related to such factors as precipitation and temperature variations, as well as depth of frost penetration. Stabilization and treatment of susceptible soils may be tried although economical considerations do not usually allow this approach. The use of an insulation layer on the pavement may be attempted soon to ensure or assist frost escape from the bottom up. This would prevent the trapping of moisture above the frost line and the subsequent supersaturation of the roadbed. Calcium chloride has been injected in troublesome frost boils to reduce heaves. A study of soil type, drainage, pavement structure, and maintenance costs may be made to determine economics of wasting or treating frost-susceptible soils as opposed to simply maintaining the road by repairing it after partial failure. Clay mineralogy studies are being carried out using both differential-thermal analysis and X-ray defraction analysis, and it will be attempted to relate this information to frost susceptibility.

During the course of an Ontario research project, started in 1961, reference nails were installed at 20 representative highway locations throughout the Province. The elevation of these reference nails is checked on a weekly basis during the freezing and thawing seasons. These data are being analyzed with reference to the magnitude, timing, and pattern of pavement movements. It is also attempted to correlate pavement movements with the variation of climatic factors. Several smaller investigations are being carried out concerning the effect of soil type on the nature of frost action, involving the use of laboratory freezing cabinets. Field experiments have been conducted to evaluate the method of injecting lignosol into areas of bad differential heaving. Efforts are continued to establish reliable correlation between the degree of frost susceptibility and pore-size characteristics.

In Newfoundland non-frost-susceptible materials are in short supply, consequently the provision of granular base course materials is very costly, especially because these materials must have not more than 6 percent minus No. 200 material. It would be of great assistance if research could show a relationship between the critical minus No. 200 percentage and the physical properties of the minus No. 200 material. Perhaps the maximum of 6 percent could be increased without detrimentally affecting the road structure during the spring break-up conditions. So far, all minus No. 200 material has been considered as a uniform material, mainly because it seems to be reasonably uniform from a grain-size point of view. However, there is probably some significance in the shape of these particles, especially with regard to potential capillary action, and research in this direction might produce some very useful results.

The most notable change in recent design practice in New Brunswick is the use of the Benkelman beam deflection method for determining overlay thicknesses to rehabilitate old pavements. When the loss in spring bearing capacity of the road is known, this may be combined with the detailed deflection data to make a design that will provide a higher and more uniform spring bearing capacity. Research is needed in the evaluation of soil-cement as a base course layer, the maximum and minimum layer thickness, and effective methods of placing a seal coat on soil-cement. Research is also needed to devise a method of getting in-place densities in granular materials so as to give more reproducible results.

A substantial degree of success has been attained by using the present methods in the practice of the Department of Public Works, but is felt that further improvement and economy can be gained by paying more attention to better drainage of the highways. Soil types that are presently being excluded from the highway embankment could be used within the core of large fills if the postconstruction drainage conditions could be predicted and effectively handled. Stage construction should be planned wherever practical as a means of locating and correcting frost heave and structural deficiencies before the application of the final pavement. Present field research activities include the measurement of frost penetration, frost heaving magnitude and the rate, extent, and pattern of ice lensing. Such aspects as the effect of sanitary services on the thermal regime of the soil, the effect of snow cover on frost penetration, the variation of surface deflections due to variation of soil and temperature conditions, the effect of ambient temperatures on frost penetration, and the effectiveness of sulfide liquor treatment are also investigated. These investigations are still in the initial stages and the results will be published later. Further research should be carried out in connection with such aspects as the thermal conductivity of the different soil types, the effect of pore size on frost susceptibility, and the effectiveness of chemical treatments.

Studies of frost penetration by the Department of Public Works in the Banff area of Alberta resulted in the general equation:

$$D = a + b.\sqrt{F}$$

in which

D = frost penetration depth (in feet); and
F = cumulative degree-days below 29 F (air temperature).

The values of a and b depend on the type of the soil, drainage conditions, and the nature of the snow cover. It was found that a varied between 0.5 and 3.5, and b between 0.05 and 0.40, with average values of 1.3 and 0.12, respectively.

The most urgently needed improvement in the practice of the Department of Transport is a better method of determining the reduction in pavement load-carrying capacity during the thawing period. The most promising approach to the problem appears to be the present program of determining the actual strength reduction by field strength tests and the correlation of these reductions with relevant factors, such as subgrade soil type and condition, pavement thickness, and freezing index.

In the interest of economy, it will never be practical to eliminate entirely all heaving of the highway due to frost action. What is desirable and probably within economic and technological reach is the elimination of perceptible degree of differential heaving. A prerequisite to tolerable uniform heaving is a high degree of uniformity of the variables contributing to the phenomena. These conditions are rarely found in nature but should be attainable by selective disposition of subgrade soils, effective drainage, and over-all quality control of construction. The majority of data, criteria, and technology necessary to accomplish this is presently available, although further research and consequently a greater understanding of the mechanism of frost action will undoubtedly bring about a wider and more economical application of the necessary control measures.

ACKNOWLEDGMENTS

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Appendix

PAVEMENT DESIGN IN BRITISH COLUMBIA

The structural design of flexible pavements in British Columbia is based on the seasonal fluctuations in strength characteristics estimated from Benkelman beam deflection values. Obviously, it is quite impracticable to make seasonal deflection studies on every section of highway that comes up for design; therefore, use is made of a system of "control sections" distributed throughout the Province.

The control sections are chosen to be as representative as possible of the full range of soil, climatic, traffic, and drainage conditions existing in the Province, and the different types of road structure and maintenance practices. They are 1,000 ft long; 10 Benkelman beam tests are made on them at randomly selected points at weekly intervals throughout the year to provide a record of the seasonal fluctuation of pavement strength. The deflection tests are carried out using the CGRA standard axle load and test procedure (2); the individual results are corrected for possible measurement errors due to very large deflection bowls which upset the reference plane of the beam. The results are also converted to equivalent deflections at 32 F using a conversion allowance of 0.001 in. per 5 F evolved by the Department of Transport.

When the design of a project is under consideration, the proposed route is divided into 1,000-ft lengths and, using a table of random numbers, 10 deflection tests are made on each section. At the same time, tests are carried out on corresponding control sections and the results are used to estimate the loss in load-carrying capacity that each section of the project would undergo between the time of testing and the weakest

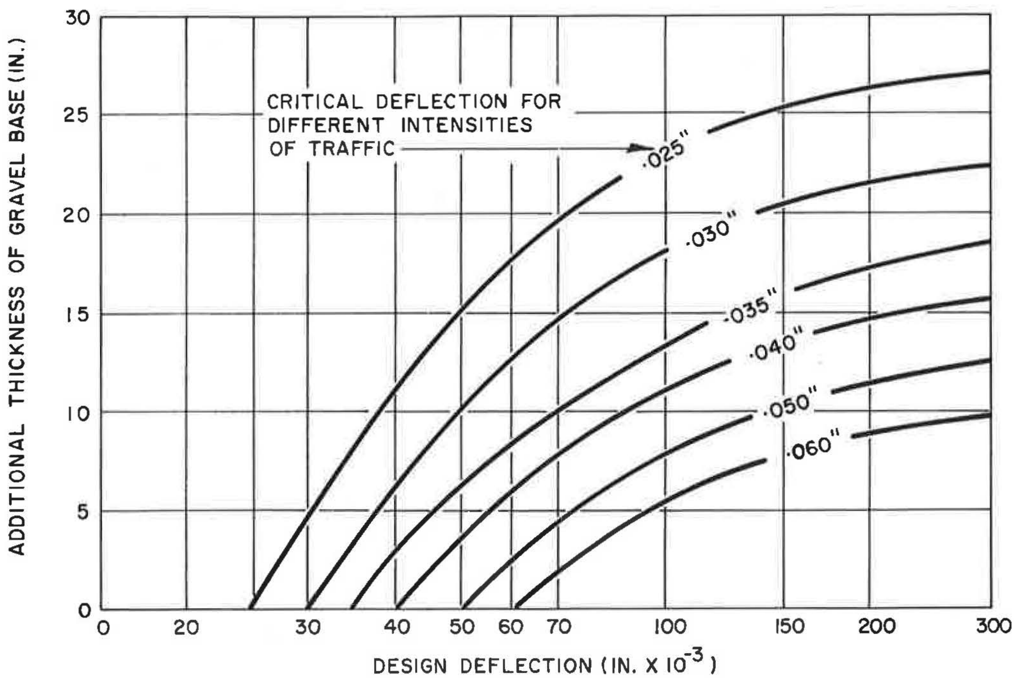

Figure 9. British Columbia pavement design chart.

time in the spring. The conversion from Benkelman beam deflection values to load-carrying capacity is made using an experimental relationship derived by the Department of Transport from extensive testing of airfield pavements (3).

Each of the test results on the individual sections of the project is converted to an estimated worst spring value and the mean for each section is calculated. To each mean is added an amount equal to 3 times the standard deviation of the 10 converted spring values to obtain an estimate of the deflection that would probably only be exceeded in one test out of a thousand if an unlimited number of tests could be carried out in each section. This estimated worst deflection is used as the design deflection.

Experience in British Columbia has shown that the maximum permissible deflection (or critical deflection) must not exceed 0.037 in. for heavily trafficked roads, 0.045 in. for medium or average traffic, and 0.056 for light traffic.

A design chart devised by Wilkins (4) is available (Fig. 9) which enables the extra thickness of granular material required on each section of the project to be determined quite readily. As an example of the use of this chart, if the design deflection of one section is assumed to be 0.090 in. and the estimated traffic is such that the maximum permissible deflection (or critical deflection) is 0.045 in., then the chart is entered on the horizontal axle at a value of 0.090 in. and a vertical line is followed until the curve for 0.045-in. deflection is encountered. From this point, a horizontal line is drawn to intersect the vertical axis and the additional thickness of granular base is read off. If an asphaltic concrete surfacing is to be placed, it is taken as equivalent to twice its thickness of gravel. So that in this example, where the extra depth of gravel required is 10 in., if an asphaltic concrete surfacing 3 in. thick is contemplated, the net thickness of gravel required would be 10 - 6 = 4 in.

The design chart is based on the Department of Transport relationship between Benkelman beam deflection and the load capacity of a 30-in. diameter plate, and the McLeod design formula $T = K. \log P/S$, in which T is the required thickness of gravel; K is equal to 35 (a constant for highways); P is the required bearing capacity of the pavement (equivalent to the Benkelman deflection permitted for the given traffic conditions); and S is the actual bearing capacity of the existing pavement (equivalent to the estimated worst spring deflection).

Discussion

K. N. BURN, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Canada—The authors are to be complimented on the excellent manner in which they have produced their very comprehensive report. Because of its wide scope, it was probably not possible to include detailed information on field studies of frost heave and subsequent damage. Some measurements made a few years ago at the site of the Building Research Centre in Ottawa might prove interesting and useful in this connection.

Early in the spring of 1955 as snow began to melt, longitudinal cracks in the surface of a pavement were discovered in a cut section of a road leading down to a basement level service area. The pavement had obviously heaved badly, so points from which to measure surface elevations across the road were quickly established and surveys conducted periodically until thawing of the roadway had ended.

The subgrade material is a post-glacial clay known as Leda clay which often contains up to 45 percent silt-size particles. It is weathered to a depth of 10 to 12 ft and is frequently fissured to about 20 ft. The roadbed was prepared by first compacting this material. An 8-in. base of graded coarse material was placed and compacted, followed by a 4-in. subsurface layer of bituminous concrete and a 1½-in. wearing surface.

Traffic is light and slow moving over this service road but it is occasionally used by quite heavy vehicles. It is kept clear of snow during the winter, but because it is low-lying, the area has a tendency to trap drifting snow and this is piled high on either side of the roadway. During the winter preceding these measurements, the roadway was plowed off-center with the result that the western side was always covered with snow but the eastern side was bare beyond the pavement.

The summer of 1954 was only slightly drier than normal, but autumn rains as usual returned the soil to a saturated condition. Snowfall was nearly normal (86 in.) and the freezing index of 1,700 degree-days was quite close to the 65-year mean.

Figure 11 shows the positions of the cross-sections that were observed in relation to the longitudinal cracks that appeared. The roadway is actually a ramp running from field level beyond the top of the figure to basement level in the large service area next to the building. The small inset cross-section is for that at B where the surface of the road is about 7 ft below field level.

Figure 12 shows pavement elevations for four surveys made during April for all three sections. The maximum heave measured in this manner was 8 in. in the center section. (It probably was somewhat greater than that because other plots of frost heave against time indicated that a maximum occurs early in March. Measurements made the following winter again indicated a maximum of approximately 8 in., but this followed an exceptionally dry summer and fall.) Measurements of the width of the crack in the pavement at each section are shown to illustrate how it closed as the subgrade thawed out.

Eight inches appears to be quite large, but conditions for ice lensing were obviously just right. Free water must have been supplied to the freezing soil beneath the roadway through the fissures in the clay which was kept from freezing by the deep insulating cover of snow on either side.

During the same winter measurements of heave were also made on the surface of an adjacent roadway including the point where it crossed a heated service tunnel. The upper surface of the tunnel is just at subgrade level and the road itself was constructed in the same manner as the one previously described. A number of points were established on the centerline of the road, but heave with time is plotted for only four of them in Figure 13: the point that heaved the most at 6 ft from the centerline of the tunnel (Curve C), a point over the center of the tunnel where the measured heave was the least (Curve B), a point 50 ft from the tunnel (Curve A), and a point over a culvert that exhibited an unusual time-heave plot.

Near the tunnel there was little snow insulation but water was readily available to the zone of ice lensing because heat losses from the tunnel kept the subsoil unfrozen and supplied with melting snow. Movement of moisture to the freezing zone would be

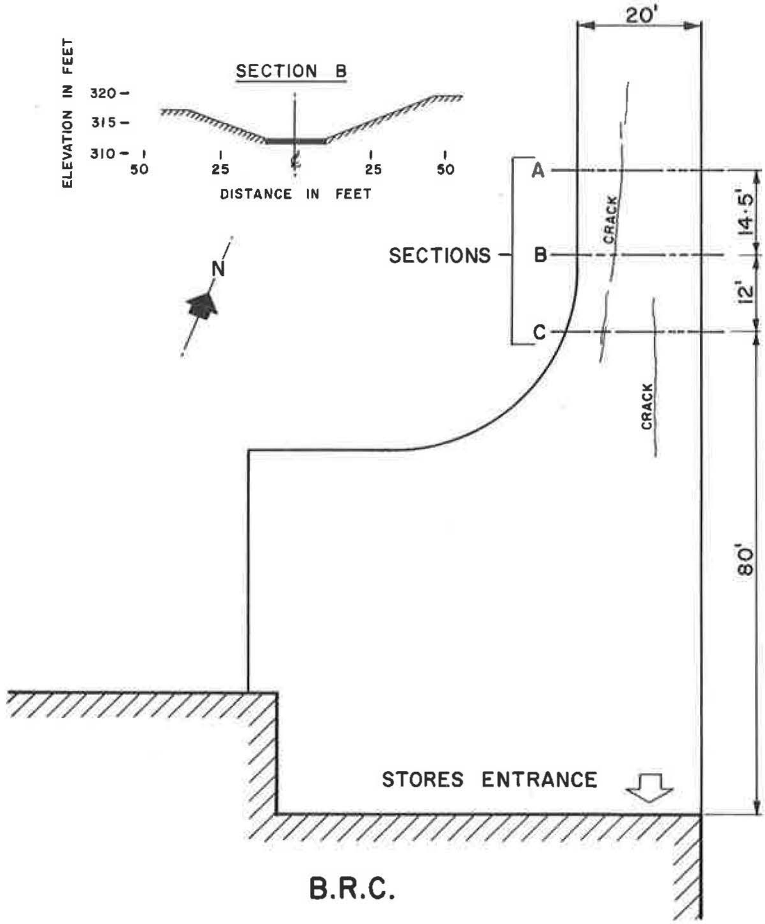

Figure 11. Plan of rear of Building Research Centre showing position of cracks in roadway and sections studied.

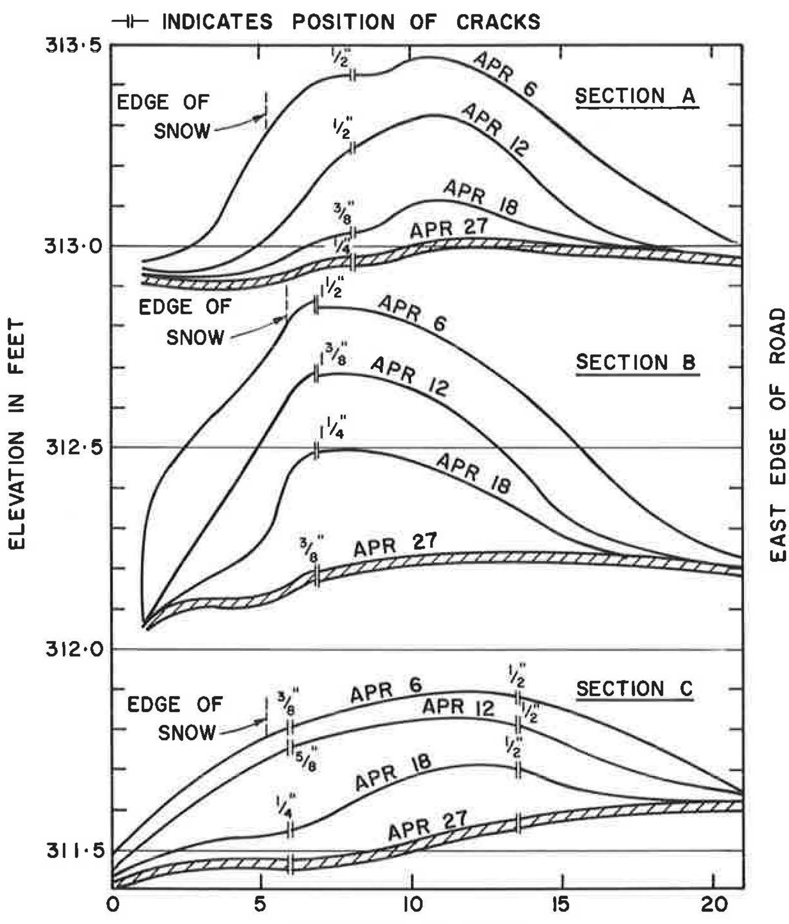

Figure 12. Settlement of road surface during thaw, April 1955, at the Building Research Centre.

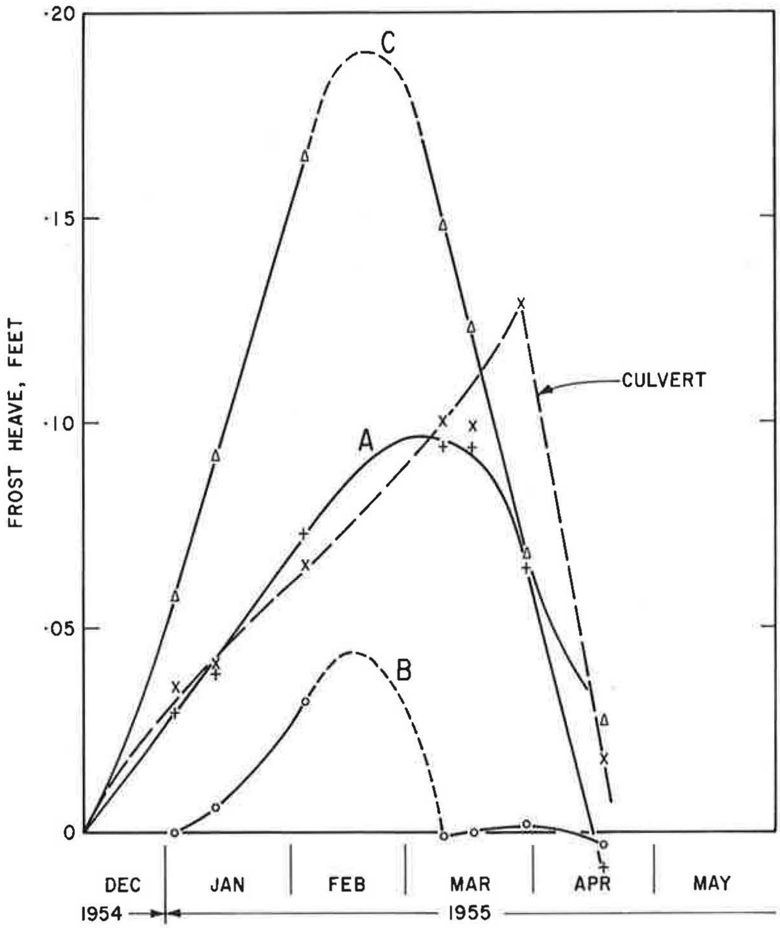

Figure 13. Measured frost heave of a road over a heated tunnel.

aided by the high thermal gradient existing between the tunnel and the zone of ice lensing. This differential heave along the centerline of the road, occurring as it does over a very short distance, very quickly resulted in cracking of the pavement on each side of the tunnel and parallel with its axis.

Frost heaving at two widely separated culverts on same roadway was also measured. Here the road stands about 3 ft above field level and the two culverts of corrugated steel beneath it are 1½ ft in diameter. Road fill was all compacted crushed stone above natural ground surface with the inverts of the culverts at the same level. Because drainage of this flat area is almost non-existent during the winter, no free water can be fed to the soil around the culverts and the measured heaves over the centerline of these culverts and points 5 ft away from them in the center of the road are practically identical with the heave measured at a point well removed from such influences. When the first thaw occurs, usually in March, water does become available at the culverts in gradually increasing quantities and although the unaffected road surface is starting to come down, ice lensing and heave continue to increase in the vicinity of the culvert, until it is overcome by the general thaw. The time-heave curve for one of the two culverts shown in Figure 13 illustrates the unusual condition in early spring. This was the case at both culverts and the preceding explanation may account for the common belief that frost heaving is especially severe just before breakup.

Preventive Measures to Reduce Frost Action on Highways in Finland

O. A. TAIVAINEN, Professor in Civil Engineering, Faculty of Technology, University of Oulu, Oulu, Finland

The report describes the methods used to diminish or prevent the detrimental effects of frost in Finland. The leveling of the non-uniform frost heaving came about by using wedges on the border of rock cuts, where embankments and cuts meet, where soil changes, and between both sides of culverts. Prevention or reduction of frost heaving was brought about by replacing the frost-susceptible soil with non-frost-susceptible soil or increasing the embankment on frost-susceptible soil. Absorption of the groundwater table into the base course was prevented by using an insulating course, lowering the groundwater table deep drainage, ditching of the road or by chemical stabilization. Vertical drain with gravel fill has also been used to reduce the suction of water into the border zone of freezing and to evaporate water from the base and subbase course. This report also describes some aspects to be considered in the construction of frost-resistant roads.

•CLIMATIC and geological conditions and the local circumstances in Finland, between the 60th and 70th latitude, are important factors to be considered when planning new highways and roads. The fall is usally very rainy so that the groundwater table is high when the frost begins. The winter is relatively long and cold. The mean freezing index is shown in Figure 1.

Finland belongs to the same primary granitic bedrock as the other Scandinavian countries. On top of the bedrock there are hard moraine or gravel ridges, but on lower places and on plains, sediments.

CLASSIFICATION OF SOILS

The Atterberg (Swedish) classification is used in Finland. The assorted soils are boulders, stone, gravel, sand, silt, and clay. The unassorted ones are in moraines.

The frost-susceptibility of the soils is almost the same as in Sweden. The soils are divided into three groups: non-frost-susceptible, moderately frost-susceptible, and highly frost-susceptible.

NATURE AND EXTENT OF FROST PROBLEM IN FINLAND

The climatic circumstances, the high groundwater table and the generally frost-sensitive soil all make the freezing problems difficult to handle throughout the country. The depth of the frozen earth layer is about 4 to 6 ft in South Finland and about 7 to 10 ft in North Finland. The old highways, which have only a thin base course, freeze badly every spring. About 65 percent of the highways and roads are frost-susceptible. In the spring the traffic must often be restricted. There have been restric-

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

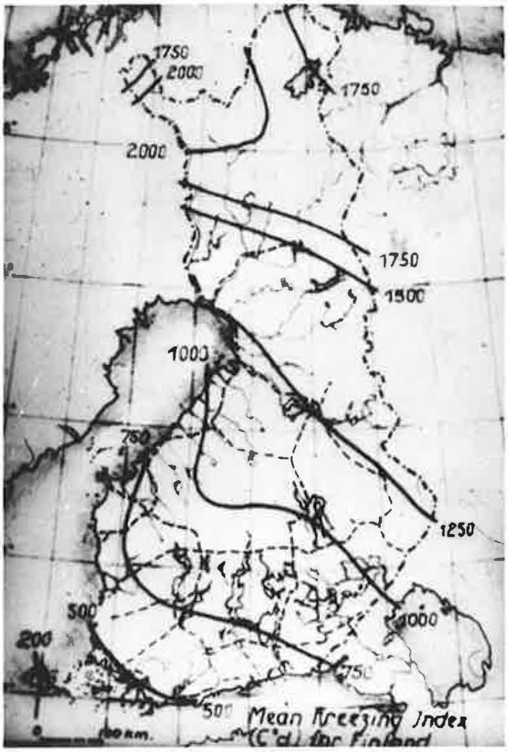

Figure 1. Mean freezing index for Finland. South, 500; Central, 1,000; and North, 1,250 to 2,000 degree-days (C x days).

tions on 25-40 percent of the highway routes. In 1955, during a very difficult spring, 47.3 percent of the roads were untrafficable. On these roads, there have been load restrictions to only 3, 6 or 8 tons. The greatest problem is the spring thaw in Central Finland, where about 80 percent of the roads were untrafficable some years ago. The most severe traffic restrictions (about 90 percent of the highways) were in the northernmost part of the country. Traffic has been restricted for periods of three weeks to two months, but some highway districts (in Central Finland) have had traffic restrictions until the end of June.

The frost heaving is of many different grades. On old highways, where the subbase is highly frost-susceptible soil, the heaving can be 2 ft or greater (Figs. 2 and 3). On the highways built during the last ten years, only non-frost-susceptible soils were used. On these roads, there were either no freezing problems, or only a few frost heavings of about 1 to 3 in.

In the spring, these new highways did not lose their capability to carry heavy traffic. From the viewpoint of the driver, the most difficult heavings appeared at the ends of rock cuttings or frost-susceptible earth cuttings, or when the frost heaving suddenly changed with respect to its quality or quantity.

PRESENT METHODS TO DIMINISH OR PREVENT DETRIMENTAL EFFECTS OF FROST

Frost heave is harmful to vehicles, causes breaks in the pavement, causes the subbase to soften, and reduces the bearing capacity of the road.

Use of Wedges

The methods of leveling the non-uniform frost heaving by using wedges are shown in Figures 4 through 9. Because the last few winters were mild, there was an opportunity to study the use of wedges for the first time. The author examined these few experiences with frost heaving due to the effect of the wedges and calculated the depth of the frost (according to Watzinger) using a surface correction factor, $\mu = 0.94$. The frost heaving was also calculated. The highest change of grade was assumed to be 0.3 percent. According to this study, the depth of the wedges was not sufficient to eliminate frost heaving greater than these maximum values more often than once every few years. The dimensions of the presently used wedges and those recommended are given in Table 1.

Use of Non-Frost-Susceptible Soil

The old road material was removed to a depth of 3 to 5 ft and replaced with gravel. On the new roads, the result was satisfactory, but on the older roads, it did not always improve the situation. During cold winters, the freezing may have penetrated even deeper into and below the base and subbase courses and caused larger cracks in pavement (about 2 in.) than before the soil was replaced. On old, narrow, steep sloping

Figure 2. Spring breakup on gravel roads in Central Finland.

TABLE 1

WEDGES AGAINST NON-UNIFORM FROST HEAVING

Type and Location of Section	Wedge Today					Wedge (After Taivainen)				
	Depth (m)	Top of Cut		Top of Earth Cut or Embankment		Depth (m)	Top of Cut		Top of Earth Cut or Embankment	
		Length (m)	Bottom Grade	Length (m)	Bottom Grade		Length (m)	Bottom Grade	Length (m)	Bottom Grade
Rock cut; highly-suscept. earth cut:										
S. Finland	1.7	5.8	1:4	18.0	1:20	1.7	5.8	1:4	27.0	1:30
N. Finland	1.7	5.8	1:4	18.0	1:20	2.25	8.0	1:4	43.5	1:30
Highly-suscept. earth cut; non-suscept. embankment:										
S. Finland	1.7	15.0	1:20	—	—	1.7	27.0	1:30	—	1:30*
N. Finland	1.7	15.0	1:20	—	—	2.25	43.5	1:30	—	1:30*
Highly-suscept. earth cut; frost suscept. embankment:										
S. Finland	1.2	5.0	1:10	8.0	1:20	1.2	15.0	1:30	15.0	1:30
N. Finland	1.2	5.0	1:10	8.0	1:20	1.2	15.0	1:30	15.0	1:30

*Down.

highways, which have been widened by soil filling on both edges, the part of the highway which was on top of the old road did not heave, but the edges of the road may have been raised to cause longitudinal cracks in the pavement.

Increasing Embankment on Frost-Susceptible Soil

Normal thickness of the base and subbase on frost-susceptible soil is now 0.80 m accord-

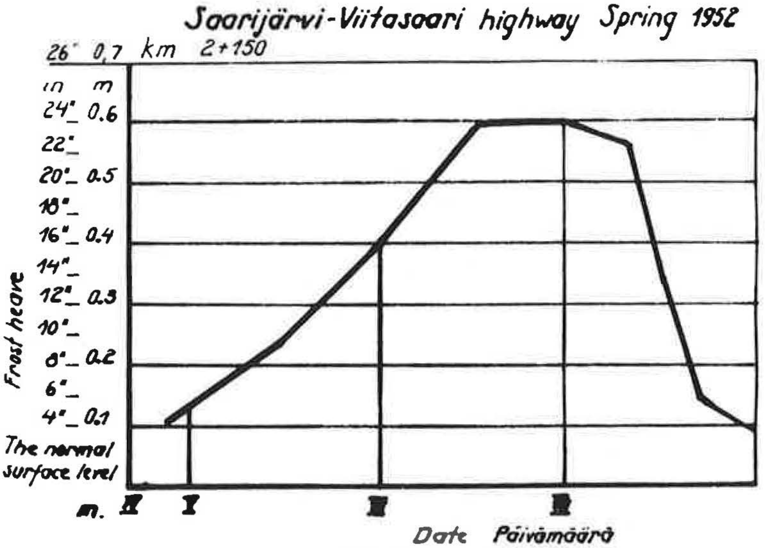

Figure 3. Frost heaving, Saarijärvi-Viitasaari highway, Spring 1952.

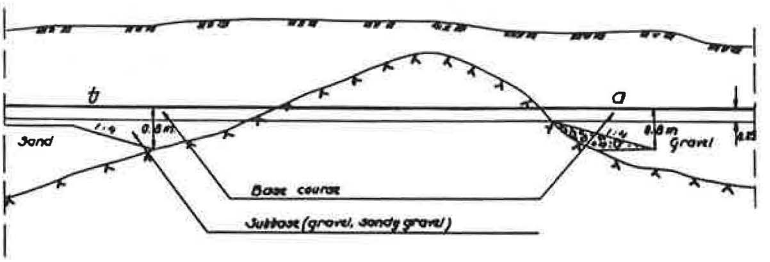

Figure 4. Wedge between rock cutting and non-frost-susceptible soil.

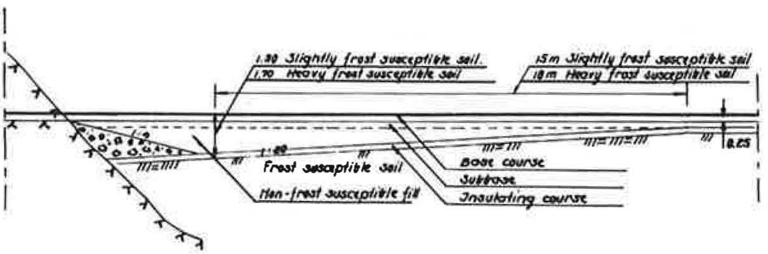

Figure 5. Wedge between rock cutting and frost-susceptible soil.

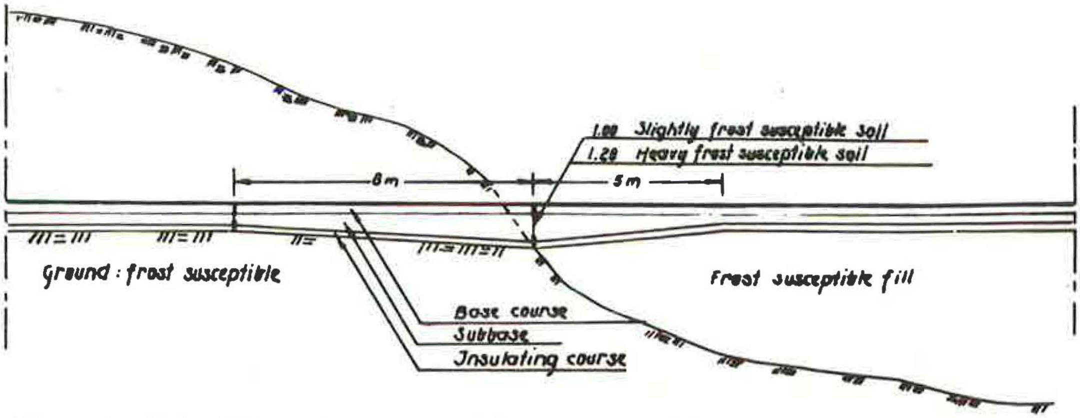

Figure 6. Wedge Between frost-susceptible earth cutting and frost-susceptible embankment.

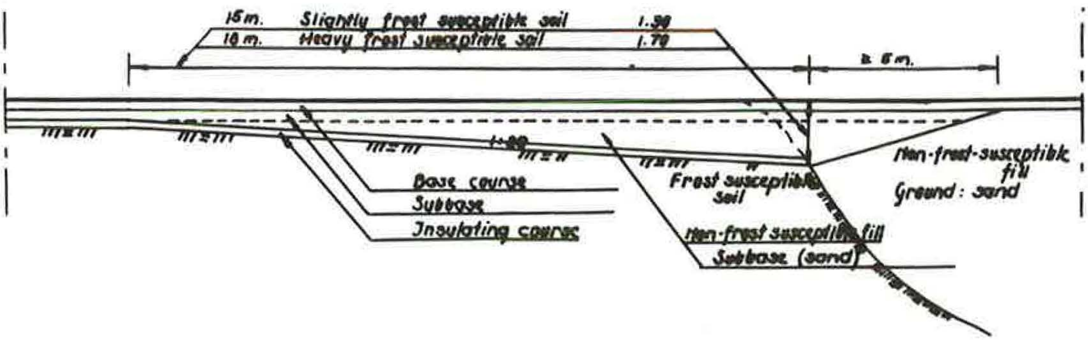

Figure 7. Wedge between frost-susceptible earth cutting and sand ground.

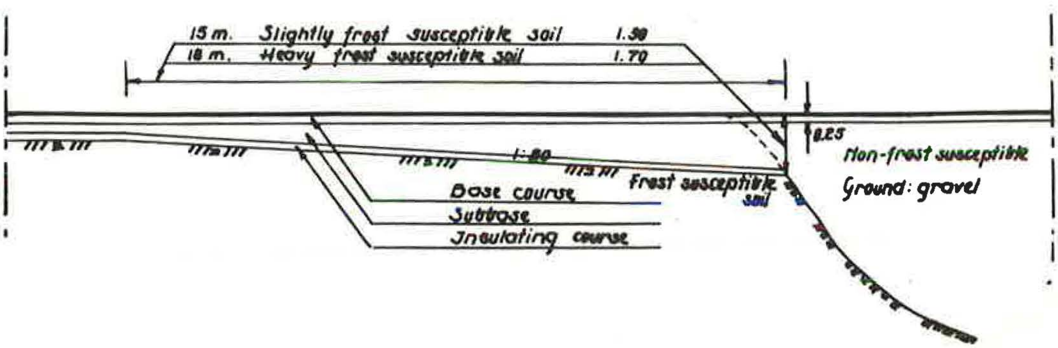

Figure 8. Wedge between frost-susceptible earth cutting and grav–

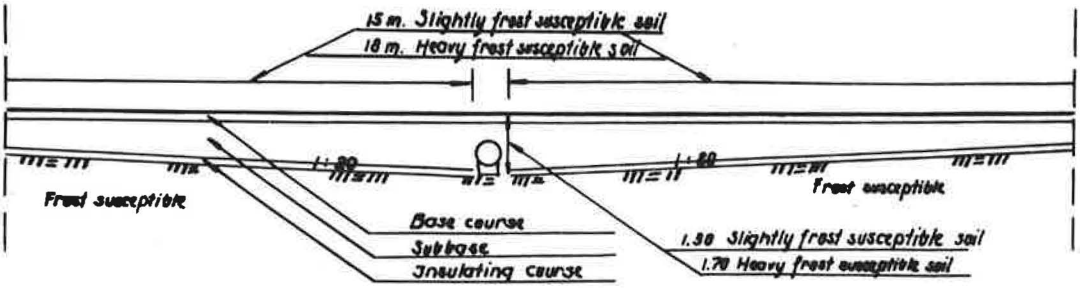

Figure 9. Wedge between both sides of a culvert.

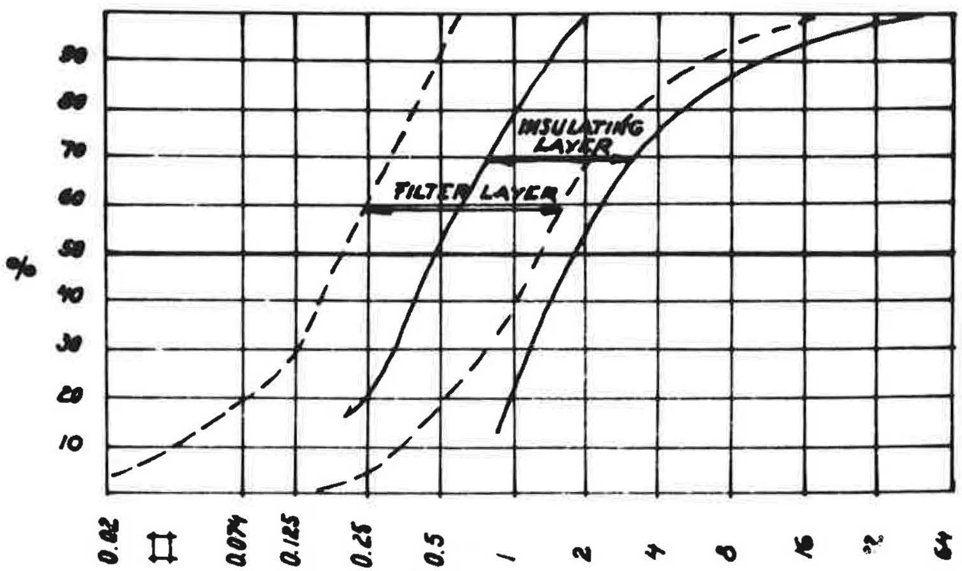

Figure 10. Use of insulating layer in highway.

ing to specified standards. However, the highways have generally been built to 1.0-meter thickness. Consequently, frost heaving and cracks due to the freezing have been lower and fewer. Frost heaving was studied on these roads and the conclusion was that, allowing frost heaving of 3 in., combined thickness of the base and subbase should be 2 ft, 8 in. (0.80 m) (F = 1,100 deg-days) in South Finland, 3 ft (90 cm) (F = 1,340 deg-days) in Central Finland, and about 5 ft (145 cm) (F = 2,100 deg-days) in North Finland. Under these conditions, large frost heaving will occur only once in every ten years.

Use of Insulating Layer

An insulating layer was used to interrupt the capillary contact in the base of the highway. The grain size distribution of the sand should be as shown in Figure 10, and it should not contain rocks bigger than 2 in. If such sand is not available, gravel may be used instead. A filler course containing 2-in. thick non-frost-susceptible moig sand (Figure 10) was made under the gravel course. The capillarity of the moig sand must not exceed 3 ft and should not contain rocks bigger than 2 in. The thickness of the insulating layer was 6 to 8 in. (15-20 cm), sometimes up to 16 in. (40 cm).

The insulating layer had a reducing and smoothing effect on frost heaving, and during the spring thaw, it increased the bearing capacity, but only if the layer was really dry and above the level of the water table. This layer was especially effective during mild winters.

Lowering Groundwater Table Through Deep Drainage

Drains were used to some extent to lower the water table. They were especially useful on roads built on slopes. Earlier, they were used under the open ditch but were clogged up in a few years. Drains have also been built under the shoulder. However, deformations occurred in the shoulder when the fill was not compacted enough. According to the instructions of the Administration of Roads and Waterways (1954), the drain has to be placed half-way between the edge of the road and the ditch (Fig. 11).

The depth of the drain is at least 5 to 8 ft (1.5-1.8 m) depending on the consistency of the soil, whether moderately or highly frost-susceptible.

The drains have reduced the frost damage to some extent, the frost heaving has been smaller, and the mutual differences have been evened out.

This is especially the case in mo soils. However, drains have not been successful in every case in avoiding frost heaving. If the ground soil is finer than 0.02 mm, drains have been used in some special cases under the road bed, but cracks in the pavement have generally occurred in these cases.

In planning drains, one must make arrangements for removing the water from underneath the road. In some cases, water remaining in the drains has caused frost heaving.

Ditching of Road

In ARW's instructions for dehydration of the water table, a depth of the roadside ditches of 1.0 to 1.1 m is required to dry the base and subbase courses and to avoid frost effects. On sloping roads an intercepting ditch is made on the upper side of the cutting. This leads the running surface water away. The absence of ditches in old roads caused strong freezing, and if traffic was maintained on the road in the spring, the road was completely untrafficable.

Chemical Stabilization

In South and Central Finland, experiments were made with lim
zation of the ground. Use of these methods proved successful i
water. This made the ground stronger and provided a better fou
of the upper layers. The insulating layer became thicker and the
reduced. Cement stabilization was used if more than 50 percent o
finer than 0.02 mm.

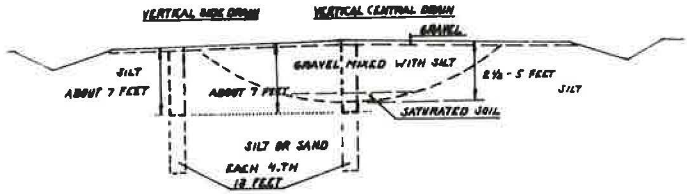

Figure 11. Placement of a drain.

Use of Vertical Drains

Badly freezing old highways were improved by building vertical drains. In the freezing spots of the highway, holes were drilled 8 in. in diameter and about 7 ft deep at a distance of 10 to 13 ft (3 to 4 m) from each other along the centerline of the road, or about 1 m from the road edge. The holes were filled with gravel or with a mixture of $CaCl_2$ or NaCl and gravel, in the ratio 1:1, 1:2, 1:3, and 1:4. A few of the holes terminated in the silt, but others penetrated to the region below it.

With vertical drains, Finland succeeded in keeping the frost heaving within only one-half of the value without drains. It was reduced to 8 to 16 in. (20 to 40 cm) from 20 to 32 in. (50 to 80 cm). Central vertical drains prevented the softening of the base course. Moving of the surface level as a result of the traffic and the slurry structure of the mo sand in the spring is also eliminated. Only one field was an exception. The drains were only 3 ft (90 cm) deep, and strong moving of the road surface due to traffic was observed in the spring. The frost heaving was 16 to 20 in. (40 to 60 cm).

The side vertical drains were also useful. The earlier commonly-occurring frost breaks were avoided when the holes were extended to the water-impervious layer. A few places were unsafe for traffic when the drains did not reach the water penetrating layer. The efficiency of the drains was decreased after a few years, because the silt often entered the gravel material. Gravel mixed with sodium chloride or calcium chloride, rather than gravel alone, has been proven to be a good material for filling the holes because it keeps the drains from freezing for a longer period of time. However, the practical significance of this difference is not clear. The primary importance of gravel filling is to provide the possibility for air to penetrate the subbase course. This reduces the suction of water into the border zone of freezing. Water can also evaporate from the base and subbase courses.

CONSIDERATIONS IN CONSTRUCTION OF FROST-RESISTANT ROADS

From experience it is known how harmful and dangerous the effect of the freezing of the base and subbase of the roads can be. The original road-ground must be absolutely level and sloping towards the sides. Longitudinal wheel tracks must be eliminated. If the ground is uneven and weak, it is advantageous to stabilize it with lime. The ground plane and the different beds can then be well compacted.

The water must not be allowed to run parallel to the road under the road bed. It has to be lead to the ditches.

If a road has to be constructed through frost-susceptible soil areas, the previously existing cross ditches should be filled with identical soil, not gravel or sand. The fill should be compacted effectively.

When an old road has to be widened and raised, the old embankment should be removed and planed over the total width of the new road. In North Finland, particularly, these have been harmful experiences with roads. Where only the edges of renewed roads were raised, longitudinal cracks appeared. Unusually steep edges are dangerous for the traffic.

Rocks in the ground below the road bed have been raised by frost heaving and cracked the pavement. The same condition resulted where pieces of telephone poles were left in or below the road bed.

When the rocks and other objects were removed unequal frost heaving, and a non-uniform surface level resulted. In those cases, the best way to repair the road, is to break the rocks and, if necessary, to fill the hole with gravel.

In the combined soil and rock cuts, harmful, uneven frost heaving and cracked pavements were observed. In such a case, normally the soil is replaced completely. If this is impossible, the soil is removed to a depth of 4 to 5 ft (1.3 to 1.5 m). This has already been shown to cause a great improvement.

Clay and other frost-susceptible soils often tend to accumulate on the bottom of rock cuts. Often, pits are found in which the water will gather. Frost heaving may have occurred in those places. To prevent this, the rock is blasted to a depth of about 3 ft (1.0 m) below the surface level of the road to be built. This method has given rather good results.

REFERENCE

1. "Specifications for the Building of the Transition Wedges." Administration of Roads and Waterways, Finland, Helsinki (1956).

Discussion

A. R. JUMIKIS, Professor of Civil Engineering, Rutgers, The State University, New Brunswick, N. J. —To characterize the severe conditions under which Finland must build its roads, a short recourse to the paper under discussion was felt to be desirable.

Frost Penetration and Heaves

In Finland, frost penetrates the glacial soils from about 4 to 6 ft in the southern part of the country, and from 7 to 10 ft in the northern part. Frost heaves have been observed 2 ft in magnitude and even greater.

Frost-Susceptibility of Soils

About 65 percent of Finland's so-called "old roads" are frost-susceptible, and during the spring thawing season, about 25 to 40 percent of the roads are closed to traffic to avoid "spring break-up" and to let the thawed soils dry out.

In severe years, 1955, for example, 47.3 percent of the roads were impassable. In certain years, about 80 percent of the roads in Central Finland were out of commission. Sometimes about 90 percent of the roads in Northern Finland had to be subjected to severe traffic restrictions time-wise, as well as load-wise.

Time-wise, depending on latitude; viz., severity of winter, roads had to be closed to traffic for a period of three weeks to two months to allow the roads to dry out. Sometimes roads were closed to traffic until the end of June.

This recourse obviously brings to the fore that coping with frost action on highways is a problem of national importance not only in Canada, the United States, or Europe in general, but in Finland in particular, especially if one considers Finland's relatively small population (estimated at 4,477,300 in 1960) and thus, the relatively small number of taxpayers as compared with the United States, for example. Everyone is aware that road-building is a very expensive enterprise.

REMEDIAL MEASURES

Sand Wedges

The paper describes the use of sand wedges in soil at transitions from cuts to fills, at contacts between various types of soil of different thermal properties, and at culverts. The experience with the sand wedges, however, is not wide. The author of the paper writes that the depth of frost penetration into the sand wedges was calculated theoretically, and finds on that basis that the wedges are somewhat too small (Table 1). However, it would also be desirable to measure the actual frost-penetration depths in these wedges and to check the performance of a series of variously sized wedges under similar climatic environment.

In the same connection, there are certain factors in highways and soil engineering which can be studied theoretically for the purpose of orientation in the various factors and frost-action phenomena. However, many things must be studied and verified experimentally in the laboratory as well as in the field. For example, in Table 1, Columns 6, 7 and 8, the wedges which are calculated by "using a surface correction factor μ = 0.94 in. may not be of the proper size after all. The size of the wedge and surface correction coefficients should be established from experiment, observations, and actual performance in the field.

However, no systematic data on the performance of sand wedges to remedy differential frost heaves on roads are presently available.

The wedge problem also points out the need in highway engineering for reliable thermal coefficients of soils, pavements and other highway materials. Some other questions also arise in this connection: what is the desired density, porosity, and permeability of such a heave-reducing wedge of soil? Likewise, it is of practical interest to know the moisture content and the porosity of the wedge before and after freezing, how to get rid of the excess moisture from the wedge, and how to restore the desired density and porosity after the spring thaw in order to be ready for the next fall and frost. Everyone knows that research costs time and money. It is hoped that in the future, after some severe winters, the Finns will have more actual qualitative and quantitative performance data on roads in frost areas than those given in the report under discussion.

Methods for Reducing Frost Heaves

The author lists several known methods which can be applied to reduce frost heaves. However, none of the described methods reflects a definite opinion on whether these methods are effective, or are the standard practice provided by the local specifications for building roads in that country, or whether they have been used as a sort of pilot field experiment. It would be interesting to know whether Figures 4 through 9 and Figure 11 are those reflecting the Finnish specifications for the construction.

Replacement of Frost-Susceptible Soil

One of the remedial methods mentioned in the paper is that of replacing frost-susceptible soil with a non-frost-susceptible one. This method essentially consists of removing the old material from the road by excavating it to a depth of 3 to 5 ft and replacing the excavated material with gravel. In discussing this method, the author makes an interesting but confirming remark; namely, that during cold winters, frost penetrated such a replaced soil deeper than before. This is in accordance with observations made by Austrian, Canadian and American highway engineers; namely, that frost in gravelly soils penetrates faster and deeper than in clayey soils, and that in spring, frost leaves gravelly soils sooner than clayey ones. Some engineers have even expressed concern over how deep one should go with backfilling gravelly soil. The deeper one goes, the deeper the frost penetrates (penetration governed by cold quantities in a given climatic condition, of course). However, such an increasingly gravelly backfill is expensive.

Does this not point out again that the thermal properties of soils would have to be studied, evaluated, and allowed to perform beneficially under freezing conditions?

Although thermal properties of soils have been studied to some extent in Sweden, the United States and Canada, it seems that more effort should be devoted to this subject. This problem is indeed a wide research topic in soil engineering.

Stabilization of earth roads by means of lime admixtures to surface material was also practiced in the Baltic States in the 1930's. The surfaces of such lime-stabilized roads were smoothed off in the spring and in the fall. In respect to service, these roads were satisfactory and relatively inexpensive.

Vertical Sand Drains

Relative to the use of vertical sand drains filled with gravel, mo (= silt) and salts, the following questions arise. Where does the water in such a drainage system go? Are the drains used as one-way or two-way drainage systems (relative to bottom and top)? Do calcium chloride and rock salt leach out? How long are chemicals efficient in the drains?

It seems that the description of the function of the drains does not satisfy some of the thermal aspects too well particularly if there is no lateral flux of heat in the soil underneath the road.

Further, if water can evaporate from the base and subbase courses, what has this to do with the vertical drain? According to the description, vapor would move upwards to the cold front and freeze, thus contributing to the formation of ice. Is vapor movement really a factor in soil moisture migration?

The writer's work on moisture-transfer mechanisms in silty soils on freezing shows that the vapor-transfer mechanism is an ineffective one for supplying soil moisture to the cold front (2).

Cracking of Pavements by Rocks

An interesting observation is made in the paper: upon freezing, rocks in the soil below pavements raise and bring about cracked pavements. Here the phenomenon of the effect of stress concentration underneath the pavement is brought clearly to the fore.

SUMMARY

The nature of frost damage to roads in Finland as described by the author is the same, for example, in Canada and the United States; namely, differential frost heaves in non-uniform soils at the contacts in cuts between two different types of soil with different thermal properties, at the transitions between cuts, fills, and at culverts; the breaking up of pavements; and spring break-ups during the seasons of thawing.

The report fulfills one definite function: it adds to the engineers' awareness of the damage factor to roads upon freezing.

The report also emphasizes, indirectly and directly, the need for more observations and knowledge on the complex freezing system soil-water-temperature.

Finally, the report under discussion may be characterized as supplemental information on how things are being done in Finland.

One should remember that in a country of very severe climatic conditions in respect to frost, highway construction and maintenance are expensive. For a country with a relatively small population, this is even a more difficult problem. One really has to look sympathetically on Finland's efforts to establish a good network of roads under very severe climatic conditions.

Reference

2. Jumikis, A. R., "Effective Soil Moisture Transfer Mechanisms Upon Freezing." HRB Bull. 317, pp. 1-8 (1962).

HAMILTON GRAY, Department of Civil Engineering, The Ohio State University, Coumbus, Ohio — The development of automotive transportation in Finland, as is true in other continental areas, has of course lagged somewhat behind that of the United States. The problems relative to frost action described in this paper recall to mind similar ones that have plagued American highway engineers working in the northern states for more than a generation. The similarity in analysis and treatment between Finnish and American practice is striking and indicates that the concepts of frost action as outlined by Beskow, Taber, and others are essentially sound and can form the basis for practical solutions.

It may be well to reassert the four conditions that must be fulfilled in order to bring about frost heaving in the soil. They are (1) a surface temperature below the freezing point; (2) the availability of water which can migrate toward the freezing zone; (3) the presence of a frost-susceptible soil, that is, one which will promote the growth of ice segregation and support the migration of water; and (4) a downward progression of the frost line which is compatible with the rate of moisture migration. Extremely rapid freezing in certain soils will result in reduced frost heave because of the inability of water to migrate rapidly toward the freezing zone.

Efforts to minimize the effect of frost action invoke modifications of the second and third conditions, because so long as no control can be exerted over climate, it appears impractical to alter the surface temperatures or to control the rate of freezing.

The restriction of traffic (up to 90 percent on occasion in the northern part of Finland) and the amount of untrafficability (amounting to as much as 80 percent in the center of Finland) represent figures which seem extreme to Americans who depend heavily on highway travel. On the other hand, the magnitude of the problem is reflected in the freezing index which amounts to as much as 3,600 degree-days F in the northern part of the country, and except for the southwest coastal areas, the entire country seems to experience freezing indexes of the order of 1,000 or more degree-days. The observation that old roads placed on highly frost-susceptible material have heaved as much as 2 ft also emphasizes the seriousness of the problem in Finland. Many other problems associated with frost action are encountered in Finland.

Figures 12-20 show some of the more common problem encountered in northern latitudes of the United States. Perhaps some of the situations appear to have little connection with roadways, but they do illustrate certain basic principles and demonstrate the omnipresence of frost action in high latitudes.

Engineers working in these northern latitudes have frequently observed that boundary markers are moved from their proper position. The phenomenon is essentially similar to the raising of rocks from below the road bed, which has proved bothersome in Finland. Similar frost action can be found very close to home. In Figure 12, a low set of front door steps, the outer edge of which rests on concrete cylinders embedded to a depth of 4 or 5 ft in the earth is raised. The opposite edge is hinged to the sill of the house. Adfreezing and associated expansion of the upper layers of the soil have lifted the concrete posts so that the steps tilt inward toward the house. As winter progresses, it may become impossible to open the storm door. This difficulty usually disappears with the advent of warm weather. However, many times the concrete posts do not settle back to their original positions, but retain a residual upward displacement after each winter. Consequently, after several years it may be impossible to open the storm door even in the summer! The only solution is to beat the tops off the concrete posts, or better, to pull them out of the ground altogether and set the outer edge of the steps on flat slabs which rise and fall with the surface. At least the steps will return to the initial position each time the ground thaws completely.

Figure 13, also taken close to home, shows the effect of lack of snow cover on the surface temperatures. In this case, the concrete sidewalk and adjacent turf were lifted 6 to 8 in. above the general surface of the surrounding lawn. This picture was taken in late March when the snow had disappeared, but the ground was still frozen almost completely. With the advent of warmer weather, the thawing permitted the sidewalk and adjacent turf to subside to a position commensurate with the general level of the surrounding lawn. Of course, the problem was introduced when the homeowner insisted on clearing the snow from the sidewalk. This allowed the frost to penetrate to greater depths because the surface was exposed to lower temperatures than the turf which was covered by several inches of insulating snow. Consequently, heaving beneath areas exposed to lower mean surface temperatures was pronounced.

The author indicated that most of the frost heave problems are concentrated near the ends of cuts made in rock or frost-susceptible materials. This, of course, reflects the

Figure 12. Frost action close to home.

Figure 13. Effect of lack of snow cover on a sidewalk.

Figure 14. Road surface raised due to freezing.

element of differential frost action accompanied by differential frost heaving.

The use of wedge-shaped volumes of frost-resistant material at the transitions between cuts and soils which are not frost-susceptible, or between rock cuts and frost-susceptible soils as well as in the vicinity of culverts, represents an economical means of reducing one of the most serious consequences of frost heave; namely, the uneven road surface which endangers vehicles moving at reasonable speeds.

Culverts have always given trouble because they provide an additional cold boundary to a part of the supporting structure of the roadway. This results in a more extensive zone of freezing with consequent differential heave.

A close inspection of the road surface (Fig. 14). should serve to convince an automobile driver that any effort to maintain a speed of 30 mph on this surface would provide a hair-raising experience, and be fraught with danger not only to the vehicle springs.

Figure 15 shows a low retaining wall which has obviously been thrown out of position, in this case, partly by the lateral freezing of earth against the vertical surface of the wall, and partly perhaps by greater penetration of frost on one side of the wall than on the other. Figure 16 shows a concrete headwall which was been cracked, presumably by forces accompanying differential heaving, and also perhaps by forces exerted by the culvert itself on the headwall. Figure 17 shows a similar headwall which has been shoved far out of its original position and the culvert has risen so that its upper surface projects above the level of the gravel shoulder. Many culverts exhibit a small annual increase in elevation and eventually appear at the road surface just like the rocks to which the author referred. At such times, it becomes necessary to dig out the culverts and replace them. However, long before they appear at the road surface, they lose most of their hydraulic function, because the elevation of the invert has become too great to discharge water at the proper time. The author did not indicate the depth to which the wedges should extend below the bottoms of his culverts. Circulation of cold air through the culverts, of course, produces freezing along the bottom as well as along the top of the culvert. It would be interesting to have figures which show

Figure 15. Low retaining wall thrown out of position because of freezing.

Figure 16. Concrete headwall cracked by differential heaving.

Figure 17. Headwall shoved from original position.

Figure 18. Shoulder elevated above pavement proper.

Figure 19. Damage caused by excessively heavy traffic.

Figure 20. Damage caused by excessively heavy traffic.

the necessary amount of frost-resistant material which should be placed beneath the culvert invert.

Finally, the desirability of continuing the frost-resistant base and subbase materials beyond the edges of the pavement and through the shoulders can be emphasized by Figure 18 in which it appears that the shoulders have been elevated to a greater extent than the pavement proper. This resulted in sags, or in some level sections, in the ponding of water on portions of the pavement. Obviously, at that time of year when daytime temperatures are above freezing but nighttime temperatures fall well below that point, the presence of this water will present a serious skid hazard for vehicular traffic in the evenings.

Figures 19 and 20 show the extent to which a road can be damaged when excessively heavy traffic is allowed to use it. The upheaved portions of the roadway are locally termed "mud volcanoes" and develop when excessive wheel loads squeeze the subbase material from the region beneath the load. Nothing short of complete rebuilding will remedy this type of situation. The damage resulted from illegal operation of heavy trucks after the road had been posted for limited loading.

It has long been realized that lowering the water table could more or less proportionally reduce the amount of frost action. Finnish experience seems to substantiate this, because generally the absence of side ditches which would serve to drain the base and subbase courses leads to unfavorable behavior. It is implied that the use of such ditches is invariably successful in diminishing the amount of frost action.

With buried side drains carried to depths approximating twice those of open ditches, success is not always attained. It is surmised that in some cases these drains may become clogged and fail to function, or that the groundwater table is so low the drains do not appreciably reduce its elevation. It is pertinent to point out that some people are so convinced of the efficaciousness of drainage that sub-drains have been installed to depths of several feet in situations where the groundwater table was actually a considerable distance below the drains. Therefore, the drains were unable to perform any useful function. It is not often that such a remedial measure is adopted without first ascertaining the prevailing conditions, but it has happened.

The use of circular vertical drains spaced at intervals is an interesting development, because in the majority of cases, the lower ends of these drains do not terminate within a stratum more permeable than the overlying soil. The explanation of their effectiveness is uncertain. Of course, if these vertical drains are filled with a salt as well as gravel, this will tend to leach into the surrounding soil and perhaps reduce the freezing effect. The author's suggestion that perhaps these drains serve to reduce the amount of soil suction above the water table may in effect represent the best available explanation of their value.

The paper finally emphasizes that uniform heave will result only when uniform conditions prevail; that is, when the soil conditions are uniform and the available water is present in a uniform way.

The significance of the paper would be enhanced if the author would provide a specific definition of "base course" and "subbase course" with particular reference to gradation of the materials.

Some clarification also appears to be warranted in connection with the need for wedges of frost-resistant material between rock cuts and non-susceptible soil.

Influence of Meteorologic Factors on Frost Damage in Roads

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• AN UNDATED map of the mail routes of approximately 1700 A.D., prepared by Johann Baptist Homann (1684-1720), shows, in general, almost the entire road network of the West German Republic of 1962. Shortly after more work was done on this road network by Napoleon, the traffic decreased to such an extent, due to the invention and construction of railroads, that the art of road building developed considerably slower than other branches of technology and became a science only recently. For example, until only a few years ago the term "Wege" (more primitive road, way, or path) was used in the traffic laws. The term "Strasse" (street, thoroughfare, higher type road) showed up in the traffic laws only in 1934.

Around the turn of the century, and especially during and after World War I, traffic naturally used the existing network of roads, which at that time was adequate, having been gradually improved to keep pace with the growing demands of traffic. When the first tar was used on roads in 1902, by Gugliametti, it was used more to provide a dust-free surface for the benefit of those living along the road and the other users of the road, than for the traffic itself.

With increasing weights and speeds, more and more failures developed. These failures were considered and treated as surface failures. The pavements were improved and strengthened, and later, when failures developed in these surfaces, they were again repaired as surface failures; that is, the surface was strengthened "because the traffic had again become heavier." The problem was looked upon, for decades, as a "skin" problem, and the man who knew how to build a good surface was considered a good road builder.

The importance of the base or foundation was not fully recognized for a long time because the technique of road building developed rather slowly as compared to the rapid increase in traffic.

About 100 years after the transport of freight had been diverted from the roads because of the development of the railroads, it began to be returned to the roads. The vehicles and their loads became heavier and heavier, and more numerous. The shortcomings of the road system showed up not only in size, width and location, but also in extensive failures and breakup, clearly indicating that strengthening of the surface alone was insufficient and incorrect, and pointing to the importance of the base and subsoil. Only now was it recognized that the surface, base and subsoil form a unit, the parts of which are equally important in providing carrying capacity. The concept that the road is a unit construction with participation of the base and subsoil, and not just a skin over it, has just in recent years begun to find its way into the consciousness of road builders in its full scope.

However, before the importance of correct earth construction could be investigated and established—considering the properties of the noncohesive, cohesive, and stony soils used, type of construction, and amount of compaction necessary, depending on the type of soil and moisture contents—the most spectacular results of the influence of the base on the destruction of roads had already been recognized; namely, that in winter and especially early spring, surfaces of modern pavements were damaged or destroyed by deep breakup, which could not possibly be due primarily to failure of the surface.

After at first assuming that, with cohesive soils with which this was mostly ob-

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served, the original cause was softening due to the entrance of surface water, investigations by geologists, especially by Taber (1), of the permafrost in arctic and subarctic areas, in the 1920's, gave an indication of the true cause of this type of damage.

"Frostphenomena" was recognized as the process whereby the water in frost-susceptible soils rises up to the frost border during the outflowing of heat from the ground during the winter and freezes there, forming ice lenses. In the frozen zone there is somewhat more water than the soil contained before freezing. This process takes place in the subsoil, resulting in lifting of the pavement and, since this is influenced by various factors of different magnitude, the formation of cracking and distortion of the surface.

Traffic will not be delayed too much by this.

Only in spring, with the entrance of heat into the subsoil, does sweeping destruction of the frost-endangered road sections start. The ice lenses melt. The water that was sucked up during freezing cannot drain away fast enough, not only because the soil below is still frozen but also because of the lower permeability of frost-endangered soils and the surface forces of the soil particles.

The subsoil softens and the pavement alone cannot carry the heavy traffic loads after the softened subsoil has left it with no support.

The road is severely damaged. It is pushed down or, in case of heavy frost action, breaks through. The base is pushed down into the subsoil. Bumps and furrows form, and the soft subsoil flows up through the base and the torn surface.

For the development of road damage as a result of unequal deep freezing during the winter, which shows up as frost cracks, separation, waves, and boils, and which in the following is called "frost heave damage," the following three factors are necessary:

1. Frost-susceptible soil.
2. Water.
3. Frost.

The share due to frost heave damage is, however, small compared to the heavy destructive damage to the roads when the frost comes out of the ground. This shows up as considerable deformation of the surface, base, and subsoil, and the consequent development of cracks, depressions and breakups, and in the following is called "loading frost damage." This is also connected with another contributing factor—the traffic.

After these relationships were recognized, and especially after it was established that by the elimination of any one of the named factors, every one of the appearances of frost damage could be prevented, attention was turned primarily to the question of which soil can be considered frost-susceptible. During construction or repair of roads this factor is the easiest and safest to eliminate. The frost as a climatic factor cannot be influenced, and the prevention of its penetration of the subsoil, up to now, cannot be sufficiently influenced by any available means.

Drainage can keep water away from the subsoil and therefore reduce the extent of frost damage, but it cannot completely eliminate the water factor, because of the free surface forces of frost-susceptible soil types and, above all, the vacuum generated during freezing, which is great enough to hold a portion of the soil water against its natural slope.

To begin with, there was developed a number of frost criteria that established frost susceptibility, and its degree, by the gradation of the soil. The best known were the criteria of Beskow (2), Casagrande (modified by Dücker) (3), and Schaible (4). The Freiberger frost law (4) makes the "frost rise" dependent on the porosity of the soil, which in turn is a function of the gradation. It makes it possible, by use of a simple formula based on porosity and capillary rise, to predict the expected frost heave.

All of these criteria made it possible, by means of simple field or laboratory tests, to recognize soil types, which if encountered in subsoil should be treated with frost-preventative measures.

The best remedy is to remove these soil types from the construction and to replace them with frostproof soils, or to incorporate a frost-preventing layer. Chemical methods for frostproofing frost-susceptible soils are still in the experimental stage and economical processes have not yet been developed.

Building in of insulating layers, to prevent penetration of frost in the subsoil, did not lead to success because such layers (e.g., glass fiber blankets) lose their efficiency after a certain length of time, due to damage caused by traffic. Because their strength is insufficient, they are actually foreign bodies in the systematically constructed road design, and endanger its stability.

PURPOSE OF THE INVESTIGATION

During the search for criteria about the frost susceptibility of soil types, which, due to its importance for frostproofing new road construction comprised the greatest share of the frost investigation, the investigation of the other factors was considerably neglected.

The climatic influences, particularly, receive less than their fair share and in Germany were almost completely neglected. In other countries consideration was given only to the frost index, which is the sum of the negative (-0 C) temperatures; as well as mathematical calculations of the depth of frost penetration and water supply. The leading investigator in the latter area was Ruckli (7); the influence of water content was studied by the British (8, 9, 10), and the frost index mainly by American and Scandinavian researchers (11, 12, 13, 14).

In isolated cases the precipitation was also taken into consideration (4, 15), because no conclusive connection existed between the frost index alone and the frost damage.

There has been a recent report about a Russian meteorological station at the side of a road near Kiev, which is evidently intended more to investigate the connection between saturation of the subsoil and its carrying power than to investigate frost phenomena (16). The results are not necessarily applicable to other soil and climatic conditions.

After the questions about the frost-susceptible soils were answered, and the various frost criteria differed only a few percent for the cohesive soils, more attention was turned toward the influence of the climate.

During an investigation undertaken for the West German Federal Department for Traffic and the Federal Institute for Road Building, the author had an opportunity to build up a comprehensive research program. This program was started in 1953 and is still continuing because, as with all meteorological problems, many years are required to find the answers. Yet it was possible to answer an essential question well enough, after a shorter observation period, to suggest the use of the conclusions in practice, as follows:

During the construction of new roads and the frostproof rebuilding of existing roads, the safest solution is to avoid the use of frost-susceptible soils. Even with the extensive rebuilding program, the network of frost-endangered roads is of such size that this will require many years.

Until the completion of the frostproof rebuilding program, the affected roads must be protected every year during the spring thaw to avoid incurring unwarranted expenses (17).

Although here the frost-susceptible soil factor cannot be eliminated, the water factor can be partially reduced by proper maintenance of ditches; because the frost factor cannot be influenced, elimination of the frost-loading damage causing factor (traffic) is the only solution.

After the almost total breakdown of the road network of the West German Republic, because of the very extensive frost breakup during the spring of 1953, it was necessary to decide against the originally strong resistance of the road users.

For the first time, during the spring thaw of 1954, traffic regulations were put in effect that kept heavy trucks off of the endangered roads. The weight of the trucks that were denied the use of certain road stretches depended on the degree to which the road was endangered (18). These measures proved themselves, and have since been maintained annually.

An extensive information network about the traffic restrictions, established by the Federal Department for Traffic, and a timely edition of a map showing the frost-endangered stretches of road which is reworked every year by this author, have considerably reduced the opposition of the road users to these measures. The restrictions last for only a short while, whereas detours due to repair work on damaged roads would have been necessary far into the summer. These traffic restrictions require costly preparations before the beginning of thaw periods: for example, preparation of signs, detour signs, etc. Also it is necessary to keep repair materials and personnel ready for use on the roads not affected by the restrictions.

It is very important for the transportation industry to have a timely picture of the possible extent of the traffic restrictions for every spring so that corresponding adjustments can be made.

Therefore, the following question came up: Inasmuch as the severity of the frost damage to the roads is not the same after every winter, how can the degree of danger be predicted on the basis of the fall and winter weather?

THE INVESTIGATION

The treatment of this question was new from a meteorological as well as a road construction standpoint. The German weather service had the data about the individual weather factors, although for the years before 1953 they were rather incomplete.

The interaction of these factors with regard to the frost endangering of the roads could be studied only from a special grouping of these decisive factors for several years. It was recognized from the start that only during the processing of the data could the best selection, combination, and presentation of the factors be made. It was also recognized that the data used at the start could possibly be abandoned later on and replaced with others.

However, certain points of view were established initially, as follows:

1. To reach back to observations predating 1953 did not make too much sense, even if by doing so a quicker answer to this pressing question would be considered possible. The German weather service was established only on November 11, 1952, as the central office for the West German Republic, uniting the separate offices for the various occupied areas. For the previous years, therefore, data on the weather factors were erratic, incomplete, and of different conception. For example, measurements of ground moisture content were considerably neglected in some areas and quite advanced in others. Standardization of all the observations started only after the inauguration of the German weather service.

Above all, before 1953 the number of trucks was too low and the traffic was not sufficiently developed to make any definite conclusions about its influence on the frost damage before that time. Only after 1953 did traffic reach a magnitude sufficient for the purposes of this investigation.

2. A localized treatment of the question by means of examination of selected road sections appeared to show little promise for success. To obtain a complete picture of the annual variation in danger to the road network of the West German Republic, the number of road stretches selected for close observation would be too great for all to be visited during the short thawing period.

Experience has shown that the observations of road maintenance personnel are too individual to give an accurate picture. This also is discussed later. Above all, this would produce errors because of an immense number of very localized climatic influences that are dependent on local conditions and not on weather factors (19). Also, different influences are evidenced by changing subsoil, water ratio, and local construction methods.

Therefore, the investigation had to aim chiefly at the observation of weather data from larger climatic areas, and its relationship to the average degree of frost endangering of the roads in this area. This is important, because the question deals with large areas and not with local individual cases. To what extent this general knowledge can be used as a basis for conclusions about local conditions will not be known until later.

3. To answer the question, there should be no measurements necessary that require complicated procedures or instruments. It was imperative to find a solution that would require only such data as can be easily obtained at any time. Only then would it be possible for the smallest stations of the Department of Roads and Traffic (e.g., maintenance foremen or local traffic stations) to make timely predictions of the endangering of the roads in their areas, by means of available resources. These re-

sources could only be the simple measurement of air temperature and the weather data in the weekly bulletin of the German weather service (precipitation, average daily temperature, ground moisture) and the data available from the closest weather station or weather post.

WEATHER FACTORS INVESTIGATED

With the previously mentioned viewpoints in mind, the influence of the following factors was investigated:

1. Precipitation and ground moisture in fall and winter.
2. Ground moisture before the start of the frost periods.
3. The temperature pattern.
4. Frost pattern in the ground.
5. Degree of frost endangering during thaw.

Precipitation and Ground Moisture in Fall and Winter

Although it was expected that, due to the complicated water balance in the weather-affected ground layer, the measurement of precipitation alone during the fall and winter would show no indication of the frost endangering for the coming spring, it was still undertaken initially. Although shortly after the war the network of precipitation measuring stations was already quite extensive, in 1953 there were only a few stations for measuring ground moisture.

Besides, it was possible that, as far as precipitation goes, a dry fall or a wet one might show an influence on the degree of frost endangering of the roads in the spring.

Also, it was important to follow the precipitation during the thaw period, in order to find out if a rise in the water content produced by the melting of ice lenses, in connection with high precipitation during the thaw, caused a higher degree of frost endangering.

For this purpose the ground moisture alone was not suitable, because the weather stations do not perform this measurement in partially frozen ground.

Therefore, the daily precipitation at a group of climatically representative stations, with corresponding ground moistures, was recorded for the period from October to April for individual years. Figure 1 is typical of such a record.

To judge the influence of the precipitation during the thaw periods, tables were prepared of precipitation totals during the thaw periods for several years. In doing this, the sums for two different time periods were selected: (a) from the 1st to the 5th day after the start of thawing from the top, because during this period low-pressure areas with high precipitation are predominant, and (b) from the 4th day before, to the 10th day after the start of thawing from the top to include the influence of a longer time period. (Fig. 2 is typical.)

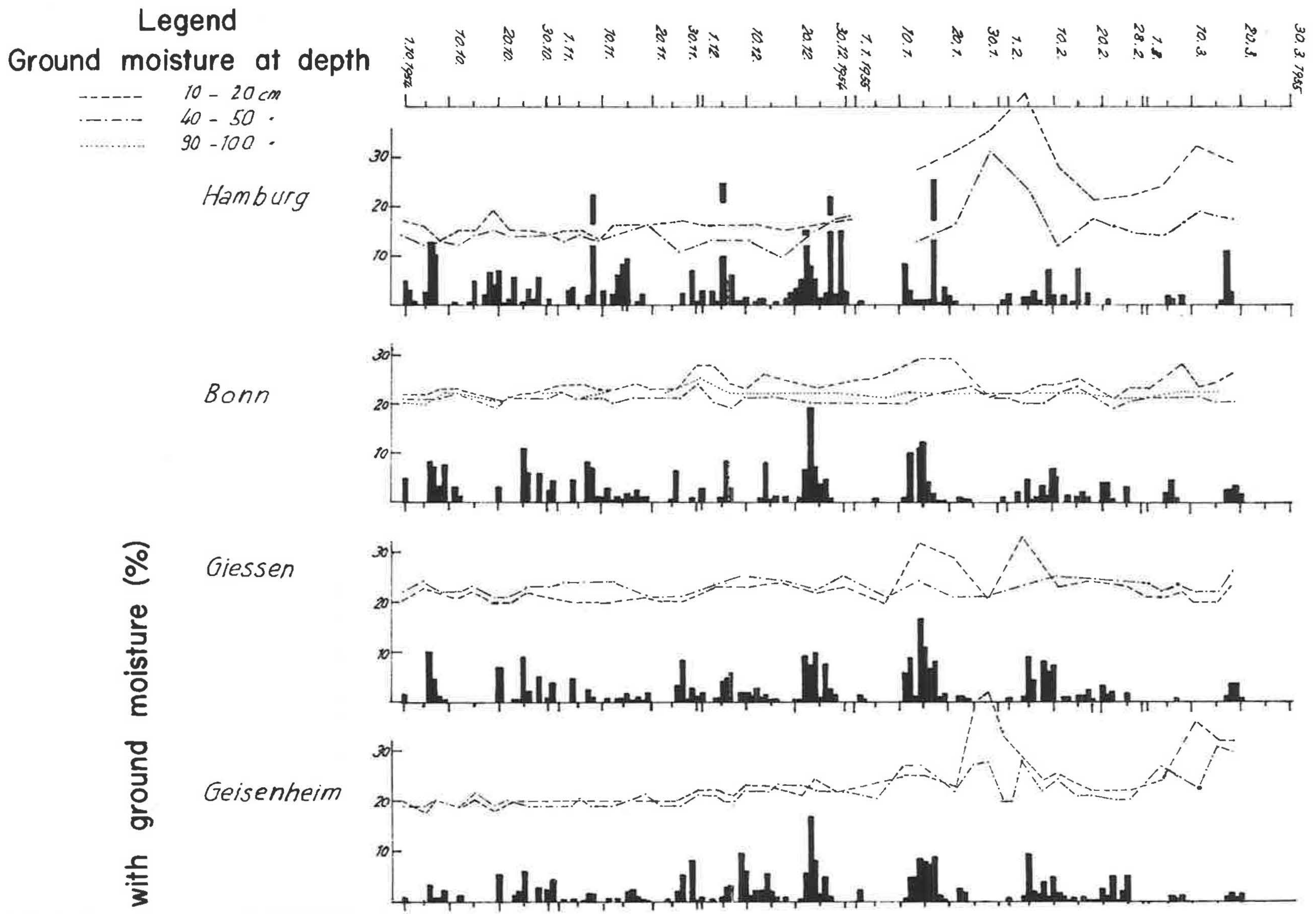
Ground Moisture Before Start of Frost Periods

In the attempt to use as many simple measurements as possible from as many points as possible, the recorded ground moisture contents of the German weather service were used, especially as the network of the ground moisture measuring stations became considerably more extensive during this investigation.

These values (moisture content) are determined twice each week by drying a soil sample taken from an uncovered and unworked area.

The moisture content is given, as in soil mechanics, as percent of the dry weight of soil. It is determined at a depth of 10 to 20 cm, 40 to 50 cm, and, at some stations, 90 to 100 cm, and is published in the German weather service weekly report.

Of course, the moisture content under an area having no growth will not agree with the moisture content under a pavement. However, it should be proper to use it as a criterion for the water penetration underneath the roads, because high moisture contents at the station test area correspond to higher water contents under the roads. Unfortunately, there are practically no comparative studies of the relationship. It is obvious that large-scale studies on this subject would be very expensive and would also cause traffic delays.



with ground moisture (%)

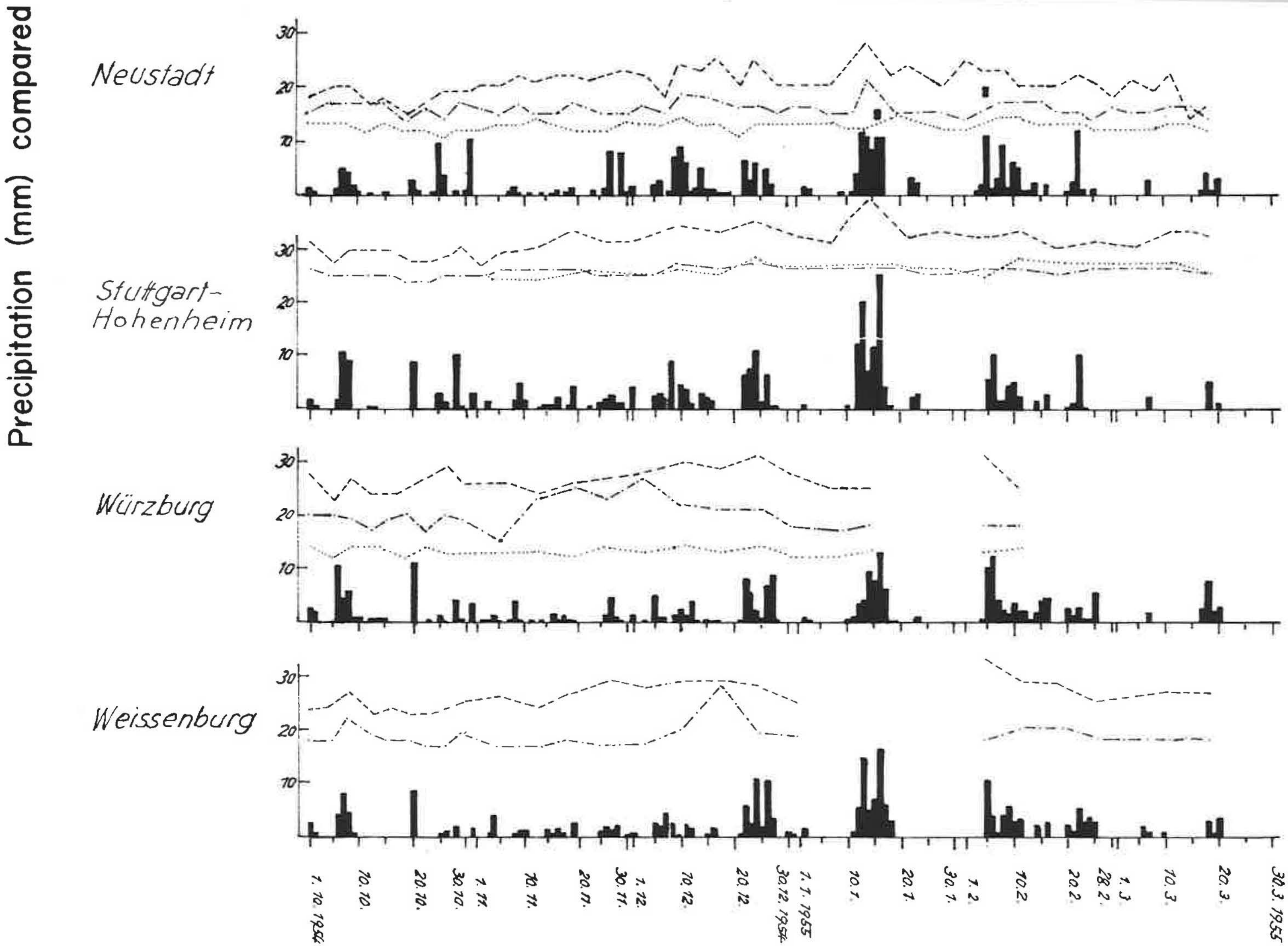


Figure 1. Precipitation and ground moisture, winter 1954-5.

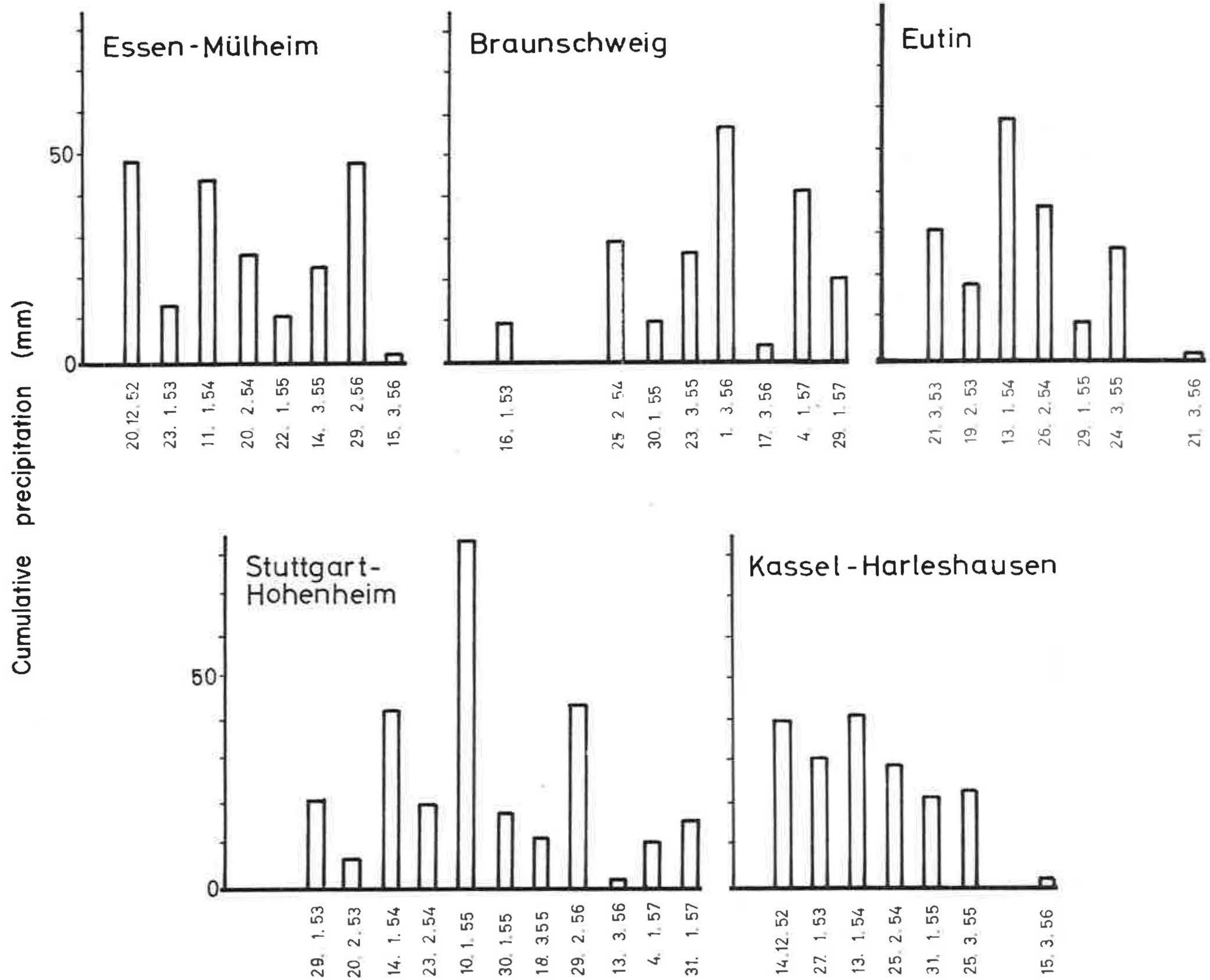


Figure 2. Cumulative precipitation at end of frost periods; 4 days before to 10 days after start of thawing from top.

There are few measurements of this type, and those were undertaken to clarify some very localized problems on short stretches of road. In a thorough investigation of frost damage to a first-class highway, Dücker (20) in March and October 1952 drilled cores of the frozen subsoil with a special apparatus. Unfortunately, no drilling was done at the side of the road. Results were compared only with precipitation.

A comparison of the moisture contents under roads and at the roadside, although not in connection with the frost problem, was conducted in Trinidad (9).

Only in 1959 was a systematic series of moisture content measurements started under and at the side of the Highway 29 test section at Grunbach, by means of neutron probes. The measurements were conducted by Behr, but the results have not yet been published. The investigations by Dücker showed high water contents under the road after large amounts of precipitation.

Both investigations show that, in cases of high moisture in the ground at the roadside, there is a corresponding high water content under the road. However, drying out takes place more slowly under the road than at the roadside, and the higher water content remains under the pavement somewhat longer.

Extending over longer periods of time, British investigations of test areas, where water contents were determined under concrete slabs, vegetation-covered ground and bare ground, by measuring the negative pore water pressure, established (8, 10) that (a) at a depth of 30 cm the water content of clayey soils under vegetation cover is lower than under roads, although only from May to October, and during a rainy summer this period is shorter; and (b) under bare ground the water content at the same depth is greater than under vegetation.

In another place in the British report to the XI Road Conference, it is stated that in cohesive soils the water content at a depth of 0.9 to 1.2 m is the same under roads as in the surrounding area.

A statement in the Russian report to the same conference, which sets the water content under roads as 86 percent of that under bare ground, is valid only for Russian prairies and their climate, and cannot be transferred to other conditions.

In regard to frost problems, one is on the safe side if the moisture content of bare ground is used for judging moisture under frost-endangered roads in fall and winter.

The depths investigated are also sufficient, because normal freezing takes place to a depth of 40 to 50 cm, whereas the investigators reach a depth of 90 to 100 cm.

The water contents, as determined by different stations in the West German Republic, show the amount of water in different soil types and, therefore, their absolute values vary from station to station.

An attempt was made to make the relative values of the degree of saturation of the soils independent of the soil type by introducing the "Mittleren Ausschöpfbaren Bodenfeuchtegehaltes" (or average moisture content), abbreviated MAB (21, 22). The determination makes it possible to compare the reports of different stations on the basis of relative figures, and to compare the existing water content with extreme values (23). That is, the difference between the repeatedly measured highest value and the repeatedly measured lowest value of ground moisture at a station is selected as the basis. This difference is called the MAB, assumed to be 100. The existing moisture content is shown as a percent of MAB.

Example: The water content at a station is determined to be 25 percent. The repeatedly measured upper and lower limits are 27 percent and 13 percent, respectively. The MAB = 27 - 13 = 14, and the measured moisture content of 25 percent represents (25 - 13)/14 × 100 = 86 percent of the MAB.

This method of calculation is somewhat analogous to the one used to establish compaction ratios of noncohesive soils, where the loosest and the densest arrangements are identified with the lowest and highest moisture contents, and the natural density with the existing water content.

After careful consideration, the moisture contents based on the dry weight of the soil, as well as the percent of MAB, were compiled for another series of climatically representative stations. For both values, lists of German weather service data were used. It was not possible to use the MAB values due to the fact that many stations had new sets of extreme values, which changed the basis of the calculations. Therefore, the

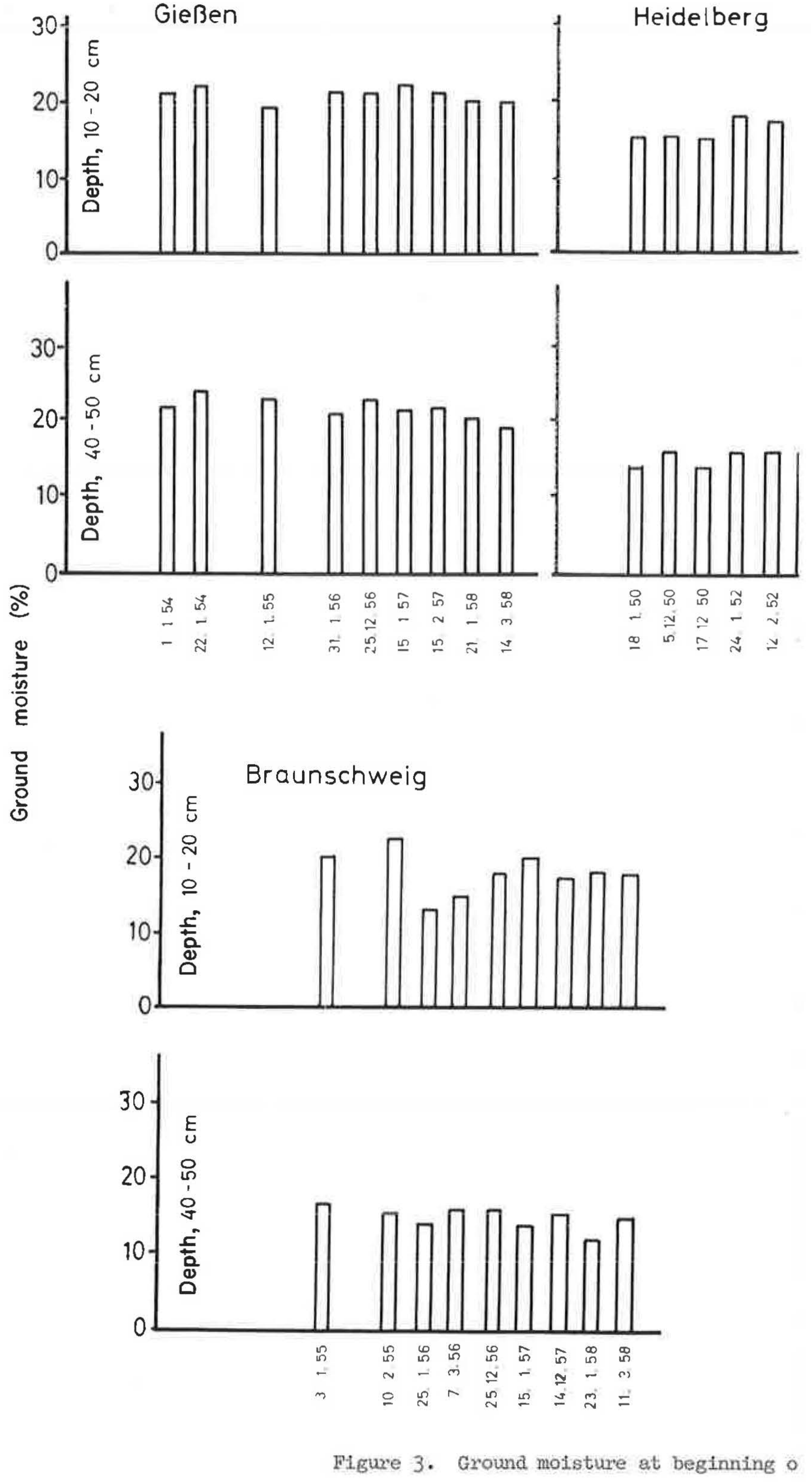
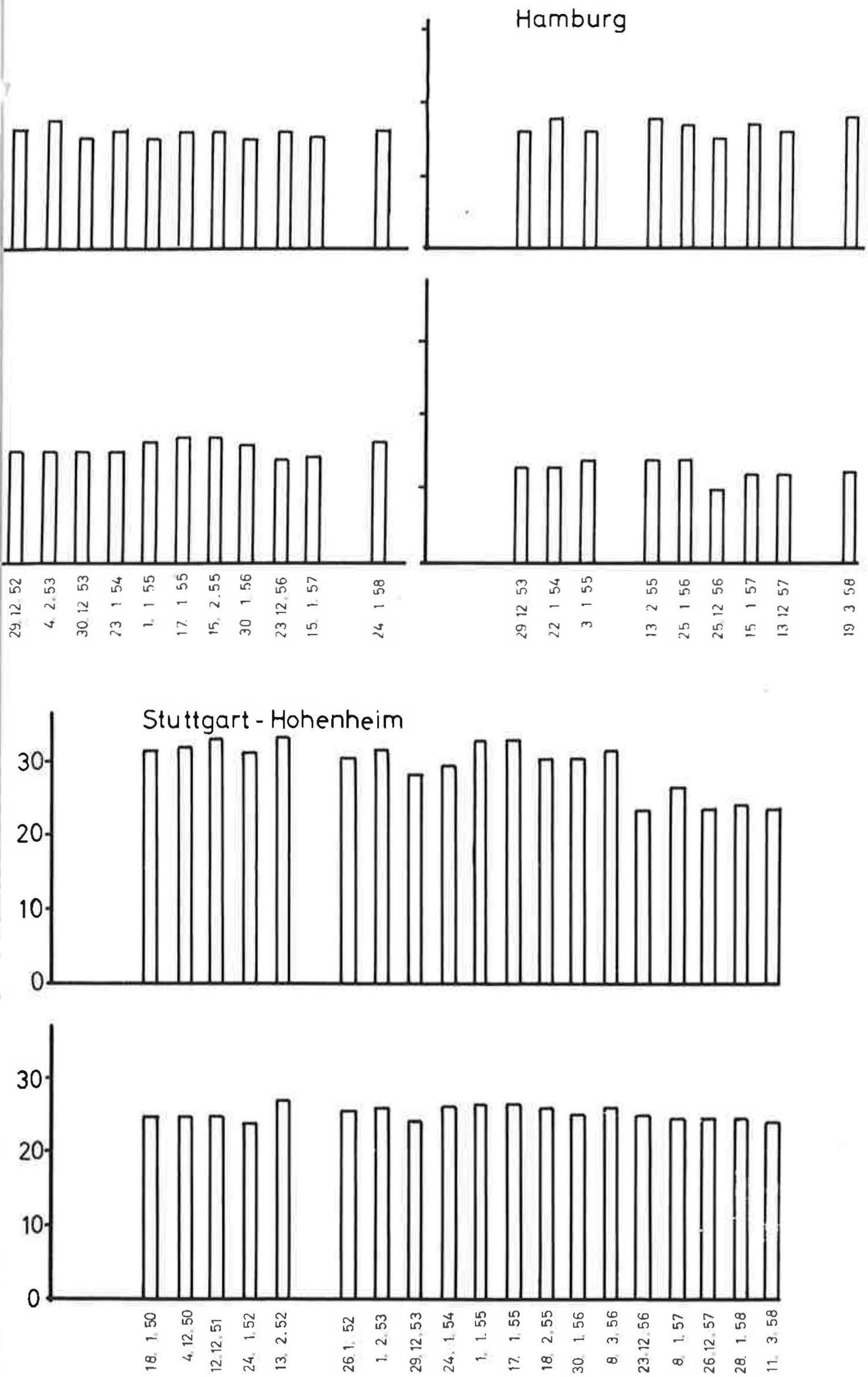


Figure 3. Ground moisture at beginning o



frost periods, in percent of dry weight of soil.

values for percent of MAB were calculated by using 1956 extreme values, and the percent of moisture on the basis of dry weight.

By use of these summaries, which represented the data from October to April for several years, the water contents and MAB at the start of the individual frost periods were determined and plotted as in Figure 3.

Temperature Pattern

The temperature pattern can easily be followed by the usual method of plotting the daily air temperatures (average, maximum, and minimum) above the date.

A considerably better illustration of the situation is obtained by using the temperature summation curve, which shows the sums of the daily temperatures in a time sequence. This curve is obtained by successively adding average daily temperatures, carefully observing the sign, and entering the results above the date shown on the horizontal axis. According to the customarily accepted method, cold is plotted upward and warmth downward; a climbing curve denotes frost, a horizontal curve denotes an average daily temperature of 0 C, and a falling curve means warm weather. The steeper the upward slope of the curve, the heavier the frost; the steeper the downward slope, the greater the warmth. Indentations pointing downward, followed by climbing of the curve, indicate temporary thaw periods. The sum of the average daily temperatures at the highest point of the curve is the "frost index."

Plotting of the curves was started every winter with the first significant frost period. The few megative (-0 C) average daily temperatues that showed up before this and were separated by extended warmer periods were disregarded, because in these cases the frost did not penetrate the pavement. The sum of the average daily temperatures was used in this investigation for the following reasons:

1. It is the best method for comparison of air temperatures and frost patterns because it is a measure of the warmth supplied to or withdrawn from the subsoil. The maximum and minimum temperatures, on the other hand, exist only for a short while, and their influence does not reach the deeper layers. For the same reason, no observations of radiation were made, because it affects the temperatures of pavement and base only. This is mentioned again in the discussion of the results.
2. The weekly bulletin of the German weather service contains average daily temperatures for a large number of stations. It also can be calculated easily from

$$T_A \text{ Air} = \frac{T_1 + T_2 + 2\,T_3}{4} \tag{1}$$

in which T_A Air is the average daily temperature, and T_1, T_2 and T_3 are the average temperatures at 7 a.m., 2 p.m., and 9 p.m., respectively.

Because the frost periods for different years start on different dates, a confusing picture is obtained if the curves for individual winters are plotted using the actual data. Therefore, in this investigation the start of the first extensive frost period in each year was assumed to be the zero point for the start of the curves. In other words, the temperature summation curves of the years under observation were all started from the origin of the coordinate system, which gave an obvious comparison of the frost patterns for the various years (Fig. 4). Only in this case, where a definite time is established for the start of the curve, is it permissible to assume a zero point for ΣT_A Air, because, in general, the temperature curves can be started on any selected day on which a sum of temperatures of the preceding days already exists. In those cases only a scale can be given for the temperature summation, but no zero point.

On the basis of temperature curves for individual stations for several winters, temperature curves for a large number of stations for individual winters from 1952/53 to 1960/61 were prepared (Figs. 5, 6, 7, 8, 9).

Frost Pattern in Ground

To establish the connection between the temperature pattern (as well as the temperature summation curves) and the depth of the frost penetration into the ground,

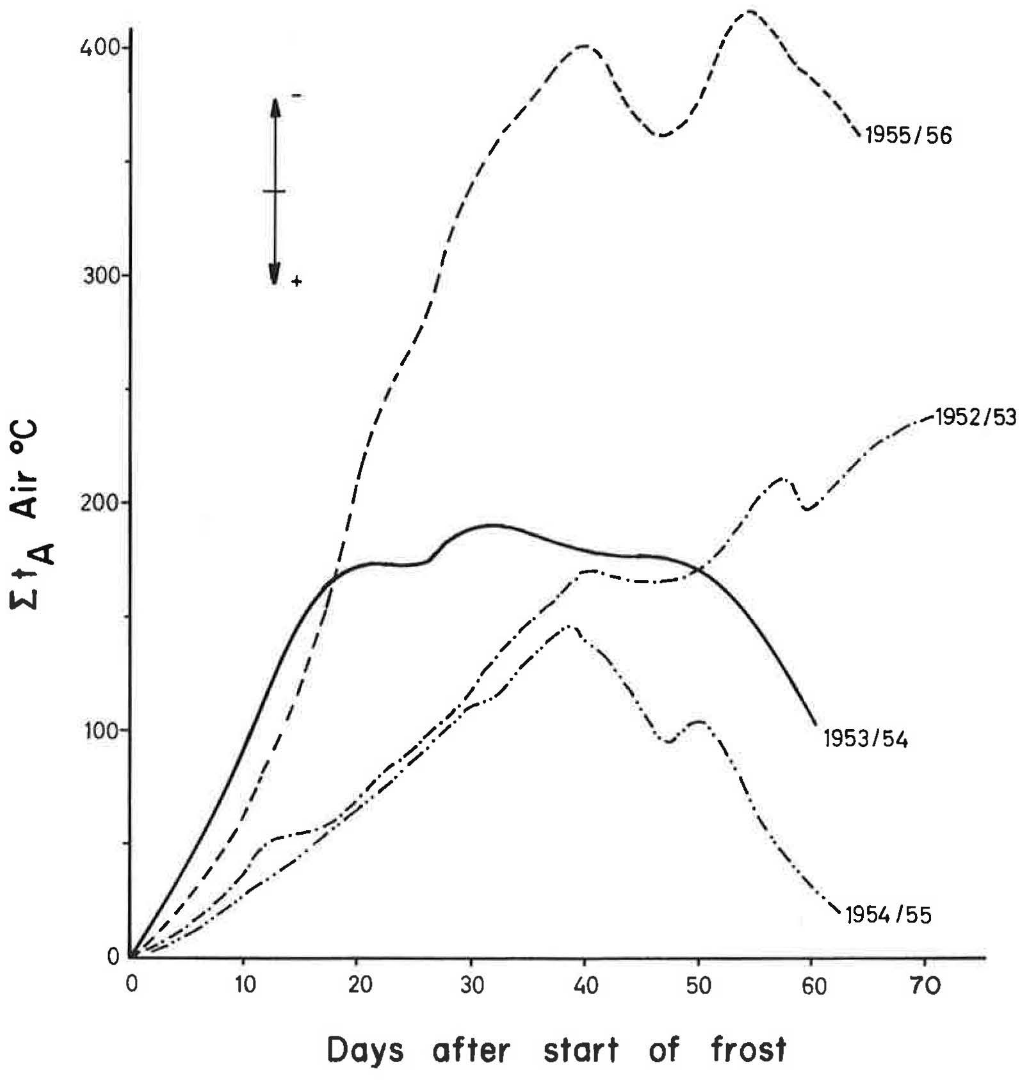


Figure 4. Adjusted temperature summation curves (Kempten).

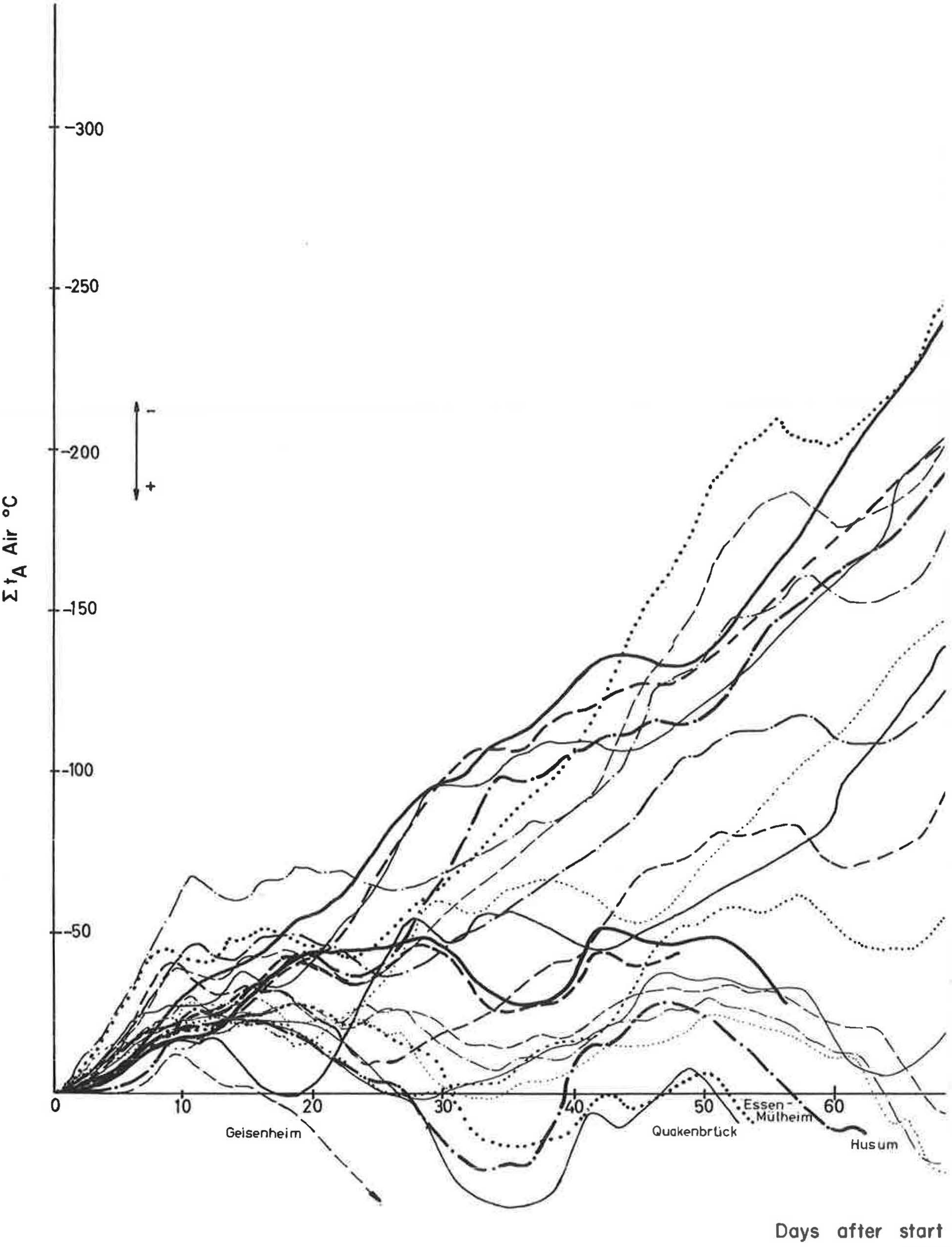
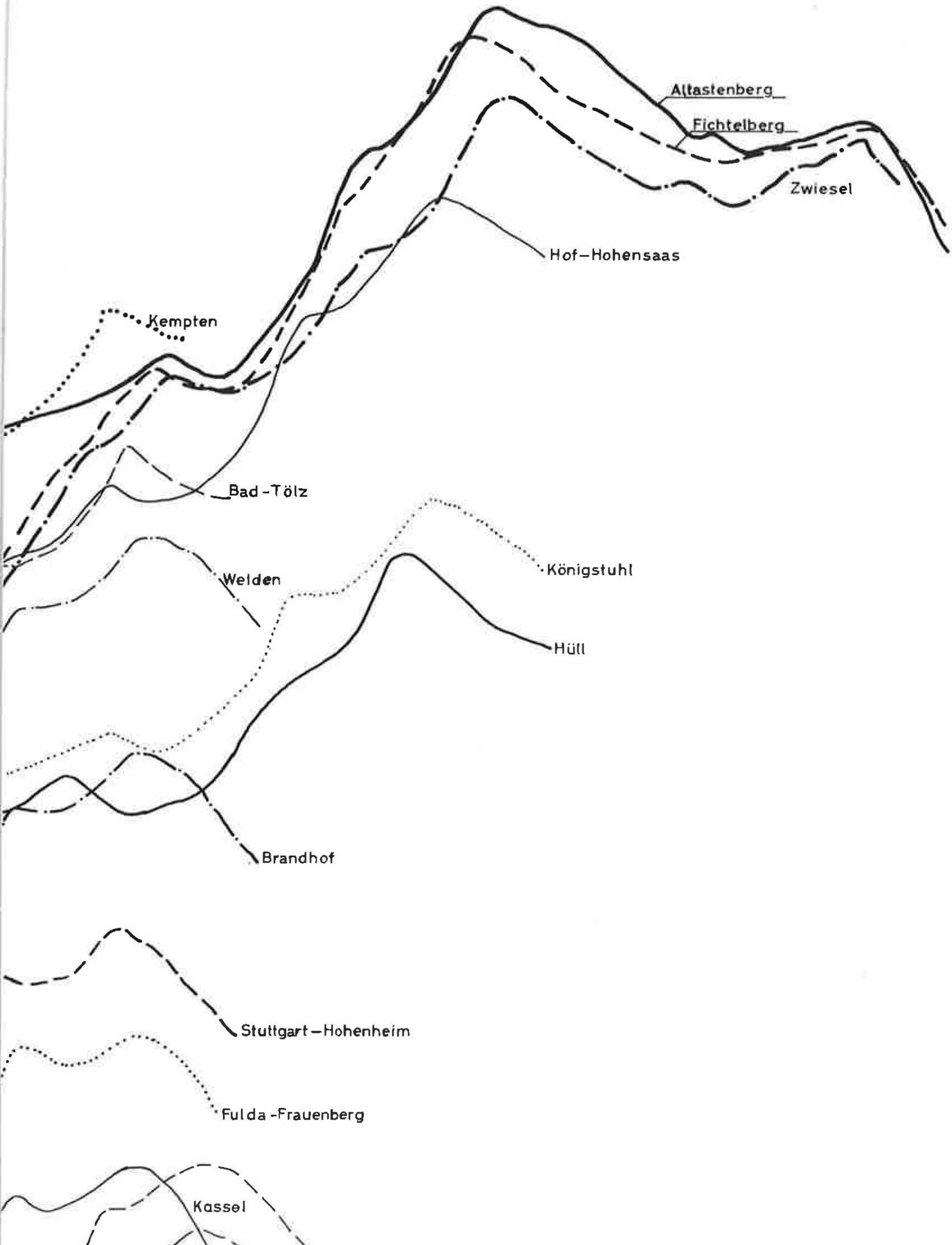


Figure 5. Temperature summation curves for winter 1952-3;



f frost

day = November 30 to December 28, 1952.

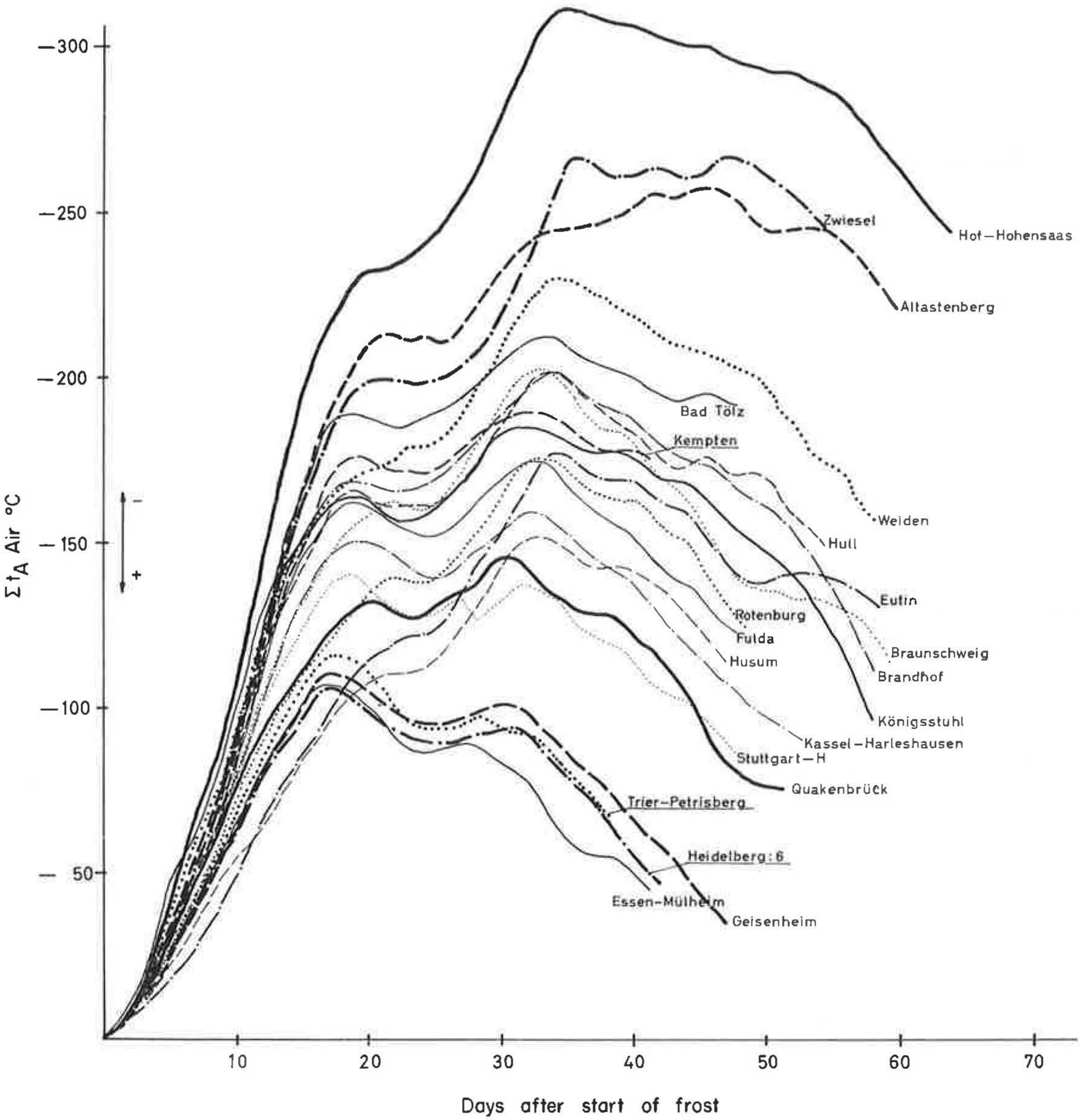


Figure 6. Temperature summation curves, winter 1953-4; frost periods, Jan.-Feb. (about Jan. 23, 1954).

special summaries prepared by the German weather service for general studies of frost problems by the Department of Roads were utilized.

These summaries contained the following weather data for a large number of climatically representative weather stations: Temperature sum curves; average, maximum and minimum air temperatures; daily precipitation; and line of equal ground temperature (isopleths).

Use of the isopleths for solution of this special question had certain limitations, as the ground temperatures used to prepare the isopleths were taken under fields not cleared of snow. This had considerable influence on the ground temperatures. Also, because the roads are kept clear of snow for traffic safety reasons, the frost penetration shown on the isopleth diagrams cannot correspond with the frost penetration under the streets.

An attempt was made to establish the influence of snow cover on the frost penetration by using diagrams for 6 winters at 26 weather stations.

It seldom happens that the same frost pattern shows up at a weather station in different years. At the same time, even with the same temperature patterns, in one year there was snow and in another there was not. That happened in only 4 cases with the 156 diagrams selected. Figure 10 is typical of the diagrams showing the influence of snow cover on frost penetration of the ground. Such diagrams not only confirm the strong influence of snow cover, but also seem to be interesting as general climatic information, of which type only the preparations of Kreutz, in a somewhat different form, exist.

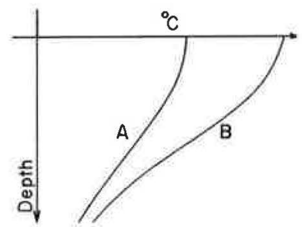
To eliminate the snow influence over a period of years, a network of stations to measure ground temperatures under road pavements was established. The main purpose of this network is to establish the required depth of frost protection measured in the different climatic areas of the West German Republic.

The measurements at these permanently snow-free stations were also used as required to show the ground temperature pattern in connection with the influence of weather factors on the frost endangering of the roads.

At these stations the temperature measurements are taken under a 4- by 4-m concrete slab 22 cm thick (Fig. 11). A concrete slab was selected because, with a given aggregate gradation and cement content, as nearly as possible, the same heat conductivity, heat capacity, and radiation properties could be secured for all stations. The 4- by 4-m size was selected to exclude the edge influence for measurements up to 1.70 m. The temperature at the first station, which was established at the autobahn maintenance station at Oelde, was measured by means of electrical resistance thermometers. Later installations used mercury-based temperature sensors, which by means of heat-compensated double copper tubing transferred the expansion or contraction of the mercury, by means of barograph-like cans, to the recording drums, where a continuous record of the temperature was obtained. Recording of the temperatures in the upper ground layers (to a depth of 0.65 m) was required because they showed definite fluctuations.

For organizational reasons, it would have been possible to make individual readings only on the hours established by the German weather service; 7 a.m., 2 p.m., and 9 p.m., because these stations were connected with the observation posts of the German weather service. First, the personnel of the posts are much better trained in handling delicate instruments than are the maintenance employees. Second, all other weather data, as well as the ground temperatures under the already mentioned bare areas and areas not cleared of snow, are measured at the stations, which could give some interesting comparisons.

The use of reading times that are relatively far apart could easily lead to misinterpretations, as shown in the following example. It is assumed that these curves, showing the temperature (pattern) as a function of depth, result from a series of measurements in the early morning, with A taken under road pavement and B taken under natural ground. Two interpretations are possible:



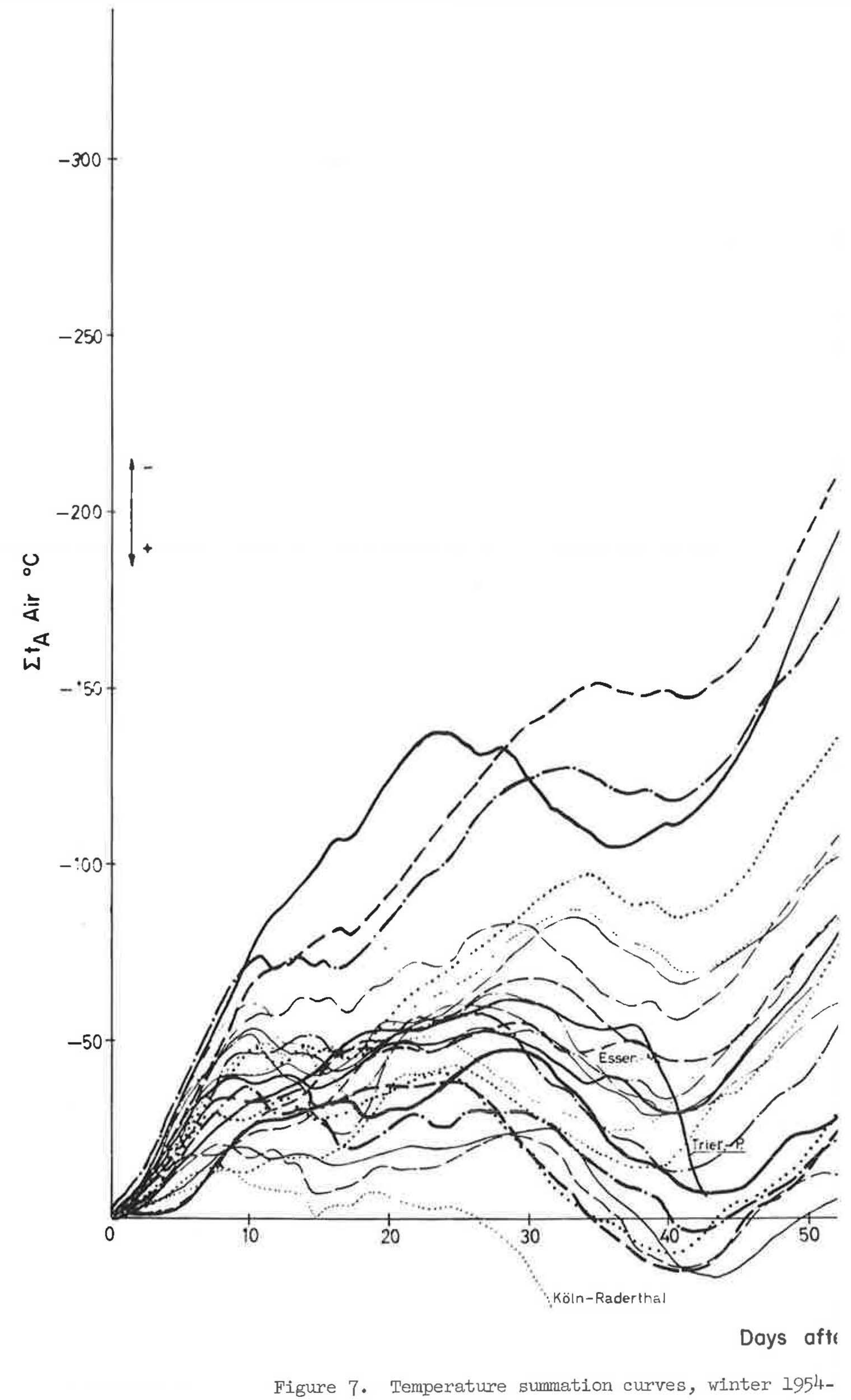
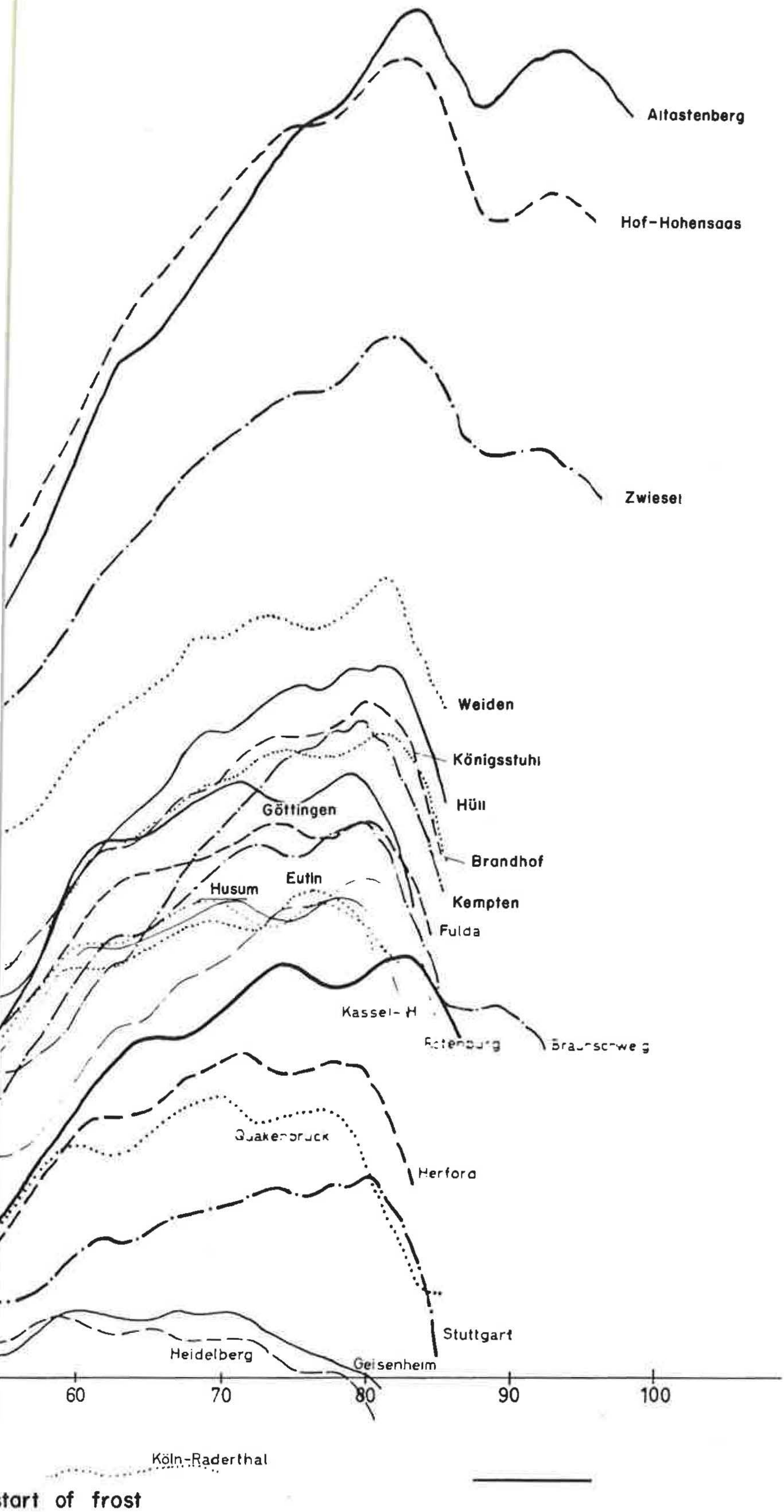
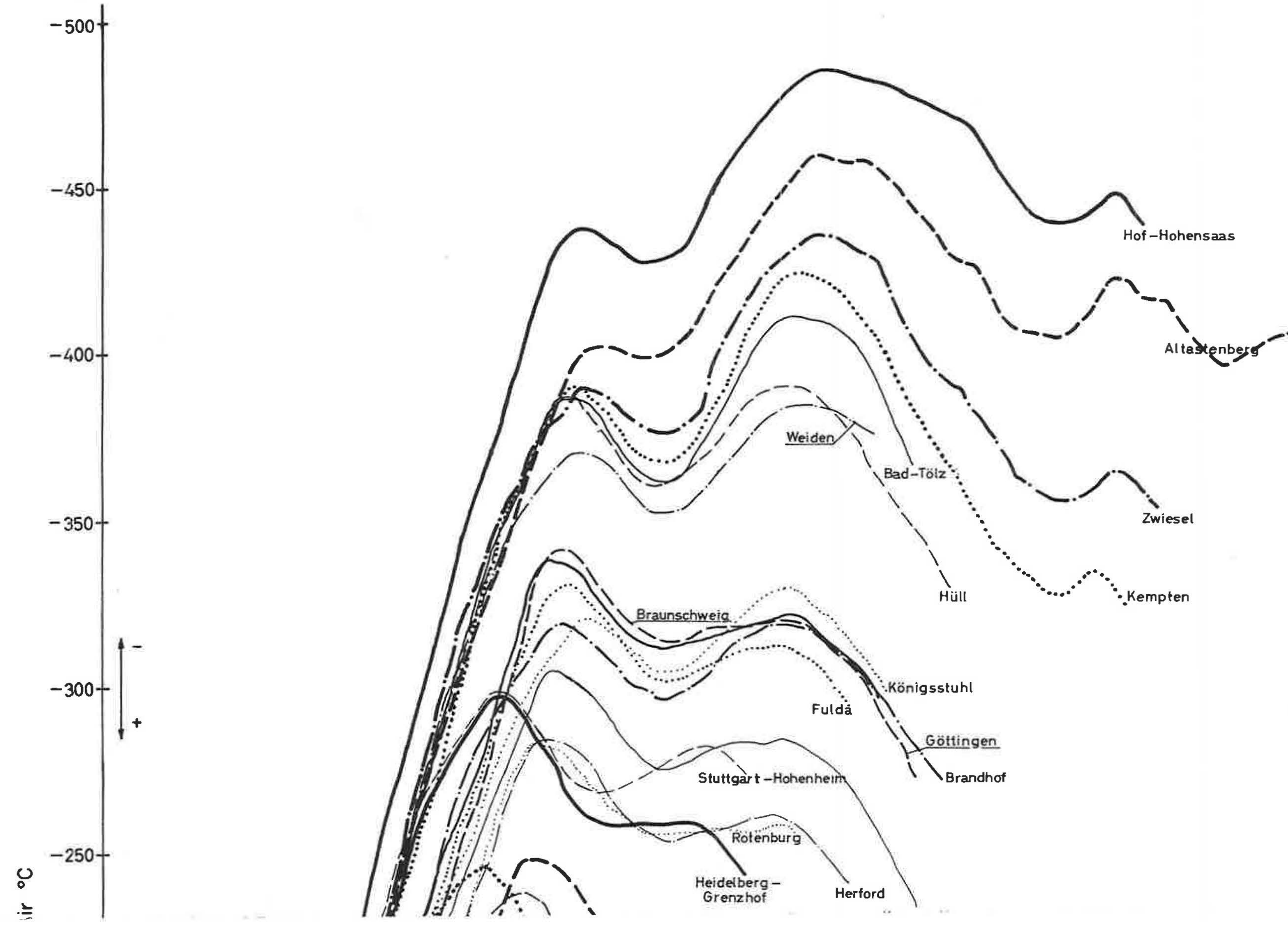


Figure 7. Temperature summation curves, winter 1954-



day = about December 11 to December 31, 1954.


-500
-450
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-300
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°C
Hof-Hohensaas
Altastenberg
Zwiesel
Kempten
Weiden
Bad-Tölz
Hüll
Braunschweig
Königsstuhl
Fulda
Göttingen
Brandhof
Stuttgart-Hohenheim
Rotenburg
Heidelberg-Grenzhof
Herford

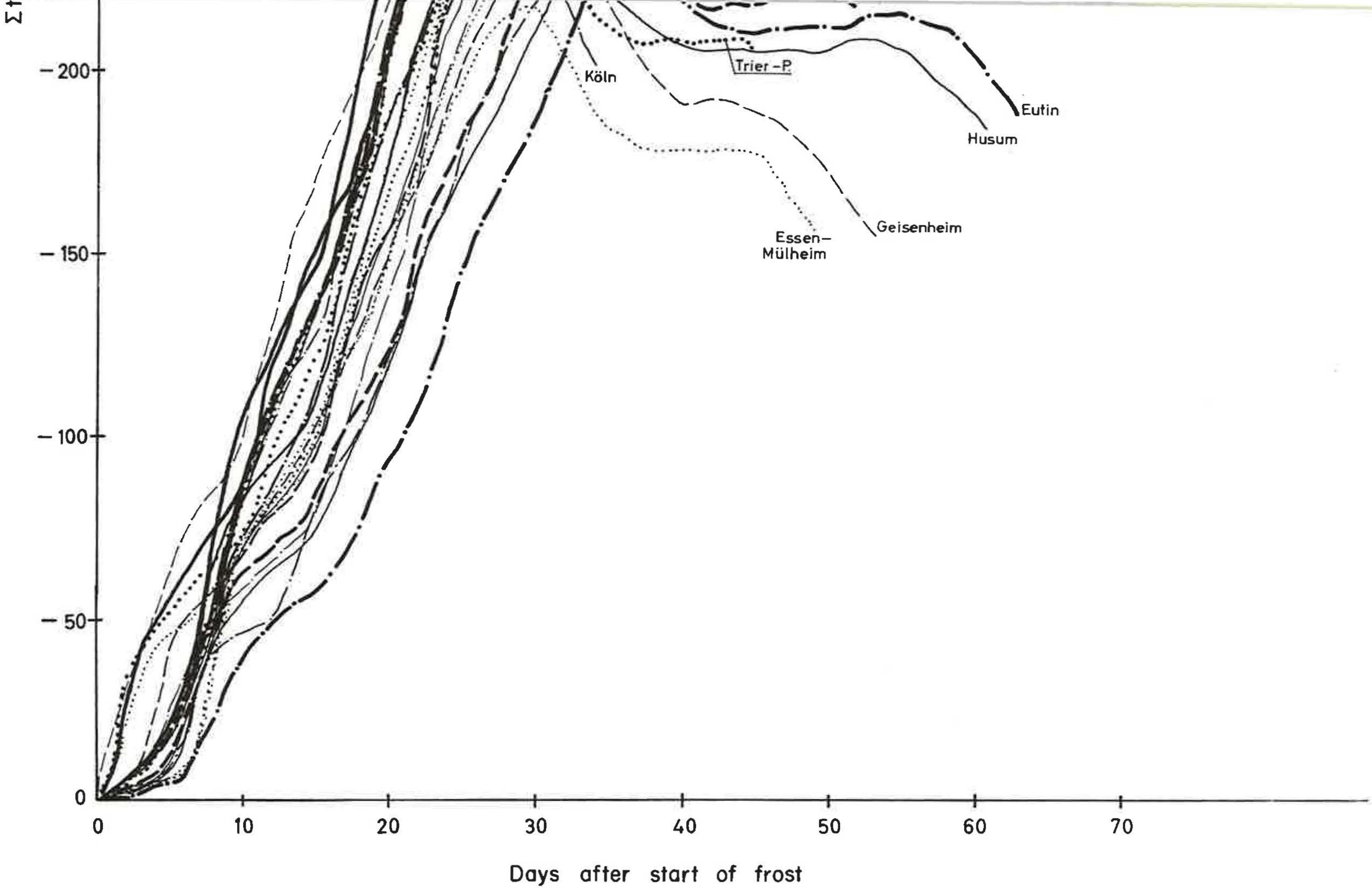

Figure 8. Temperature summation curves, winter 1955-6; 0 day = about January 24, 1956.

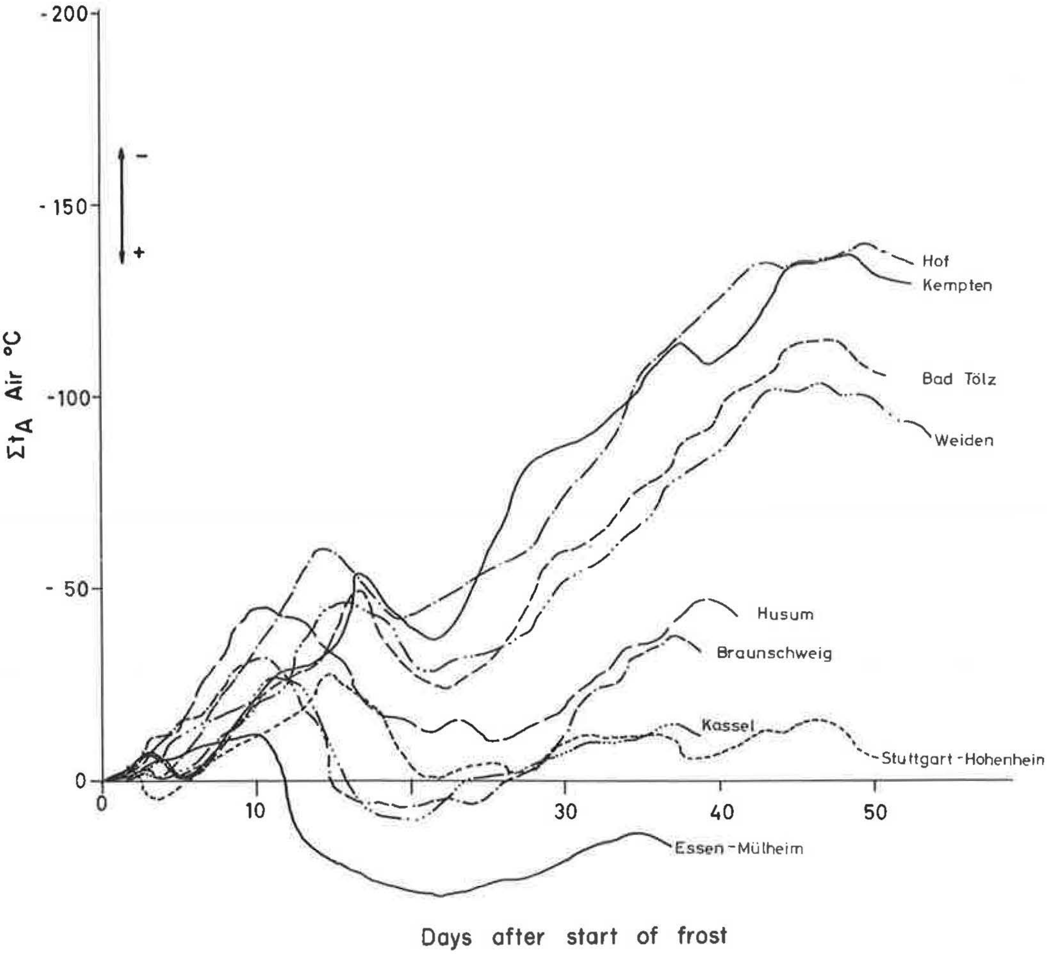

Figure 9. Temperature summation curves, winter 1958-9; 0 day = about Jan. 1, 1959.

1. The ground temperature under the pavement follows the air temperature faster than the temperature in natural ground. The temperature interval of A is larger than that of B. The cooling at location A during the night was greater than at B, but the heating during the day will also be greater. At the time of the recording, A has not yet caught up with B, but will shortly.
2. The ground under the pavement reacts more slowly than the uncovered ground. The temperature of A swings within a small interval. B has cooled more during the night, but at the time of the recording has already overtaken A.

Both interpretations, although directly opposite, are correct, and show the uncertainty of individual readings separated by relatively long periods of time.

Because of this, the temperature readings of the upper ground layers, to a depth of 0.65 m, are recorded on 7-day charts. Due to small fluctuations, it is sufficient to read the temperatures, for depths up to 1.70 m, once daily by means of a rod thermometer.

The recordings and readings are carried on continuously during the year to collect material for investigations which have no connection with the questions in this paper.

It was decided against using recording resistance thermometers because of the small gain in accuracy as compared to the increase in cost. More value was placed on a more extensive network than on a few slightly more accurate stations.

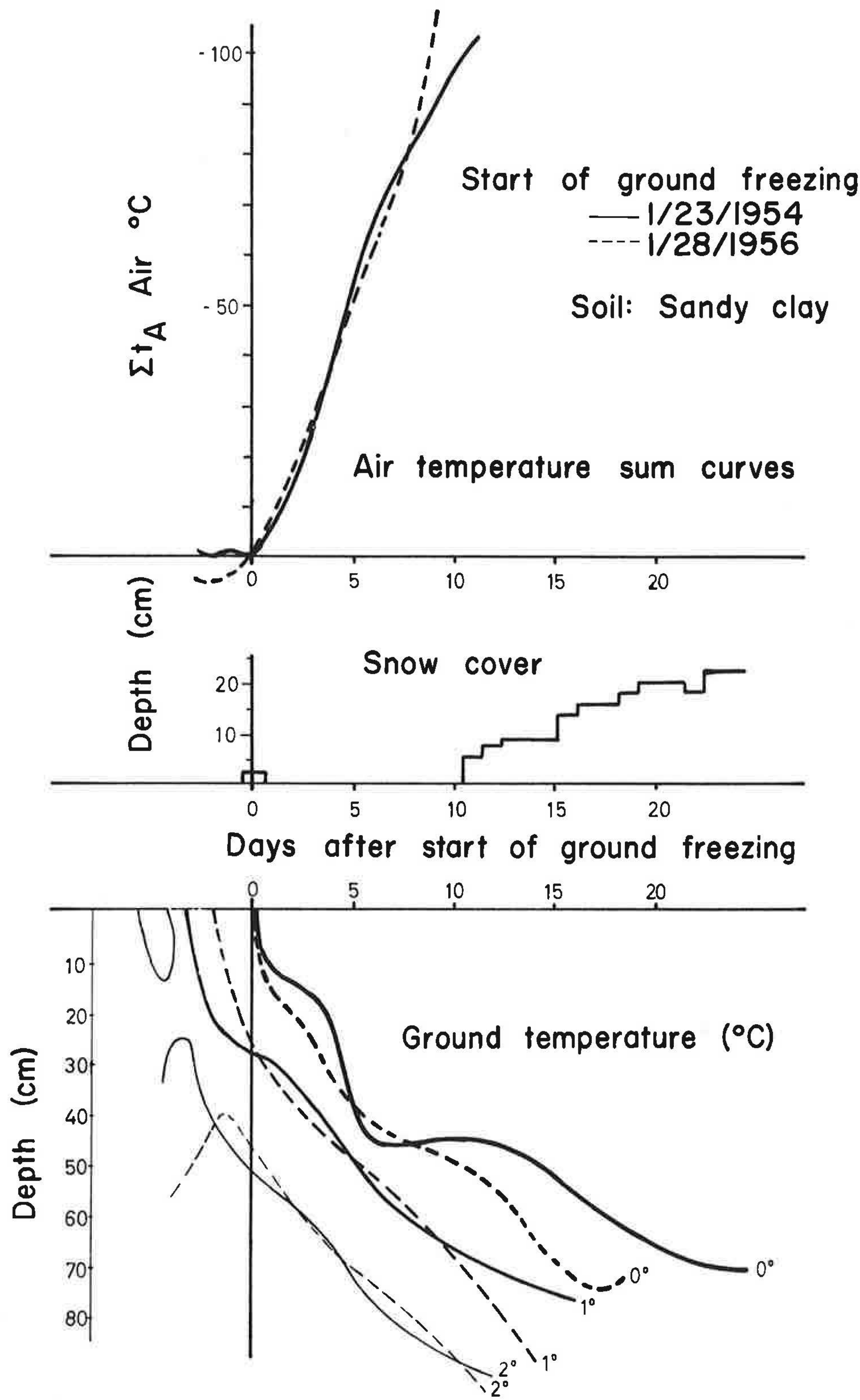

Figure 10. Depth of snow cover and frost penetration at Brandhof.

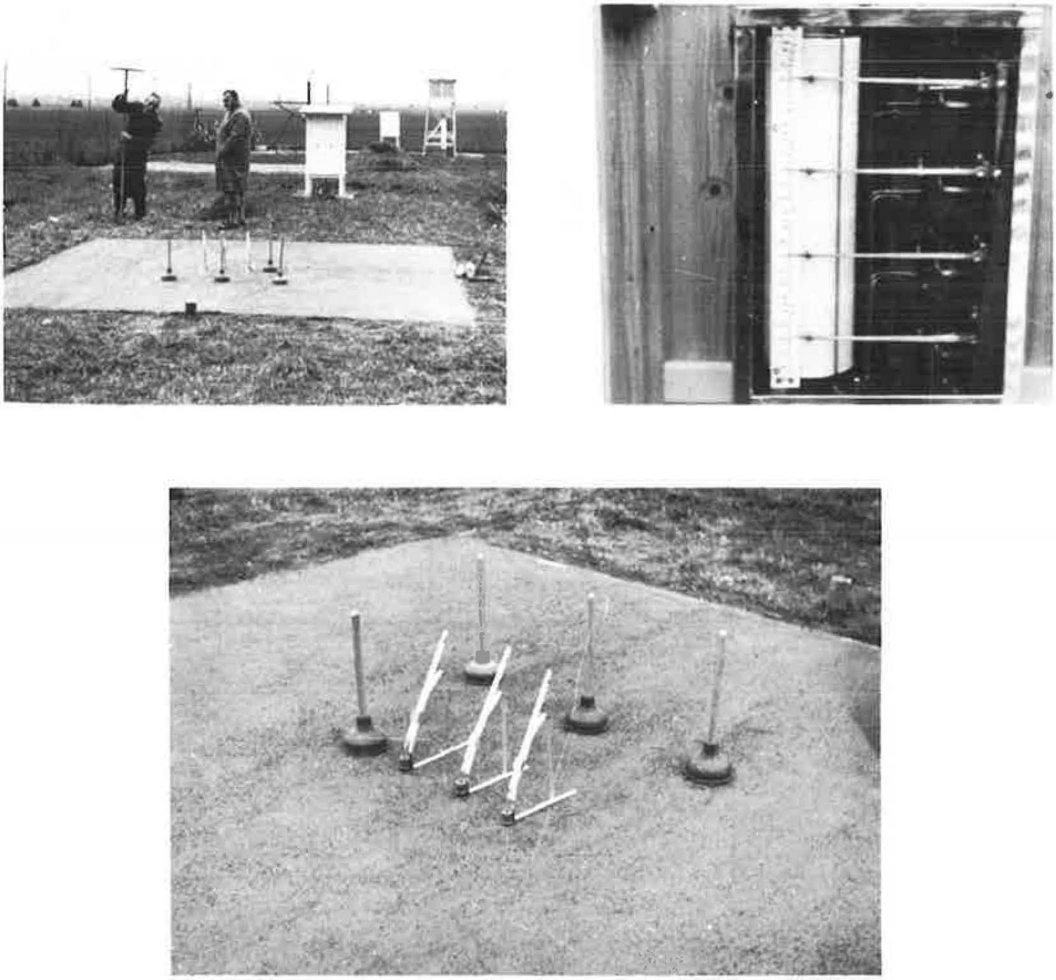

Figure 11. Ground temperature measuring station at Braunschweig - Volkenrode.

Later on, following a conversation between the author and the teaching staff of the geotechnical faculty of the college for traffic engineering at Dresden, a few ground temperature measuring stations were established at existing meteorological stations in middle Germany. Recent reports (16, 24) give no observation results, but are confined to generalities.

These stations, of course, have resistance thermometers as measuring elements. evidently without recorders, and are operated during the winter months only.

Degree of Frost Endangering During Thaw

Although it was possible in the investigation of previous factors to use measurements or prepare summaries of available figures, the degree of frost endangering of the roads during individual springs was considerably more difficult.

A personal inspection was impossible, due to the size of the network of frost-endangered roads (about 10, 000 km) and the relatively short duration of the thaw period.

The determination of the frost damage, on the basis of reports from the respective maintenance offices, is subject to strong individual influences. The essential distinction between slight depressions, extensive pavement deformations, and frost break-up is difficult, and the interpretation depends on the individual observer.

No doubt, the requested reports would lead, even if unconsciously, to a conflict in the mind of the observer between a desire for objectivity and subjective considerations. By reporting a somewhat exaggerated damage, it could be hoped that more money would be allotted for repair work. Underestimating the damage would create the impression of careful maintenance work on the road sections under the responsibility of the observer. These considerations would undoubtedly add more uncertainty to the reported conclusions about frost damage.

The extent to which such judgments are influenced by the individual is shown by a section of a frost damage map prepared by the Traffic Ministry with the cooperation of the author on the basis of the reports of individual road department offices in 1954. On this map, which shows the pavement conditions on December 1, 1953, as reported by the individual offices, the area of one particular office showed heavy frost breakups only, whereas in the neighboring areas all three types appeared, although there were no climatic or soil differences. Personal observations during an inspection tour that also included the area mentioned showed that the types of damage in this area varied the same as in the surrounding areas.

Disregarding this, the picture would still be distorted, even in a case of objective and correct observations, due to the construction work done to frostproof the roads during individual years.

Judging of the frost endangering on the basis of traffic restrictions onthe endangered road sections during the spring thaw was also impossible, as economic considerations played an important part in these restrictions. The difficulty on many seriously endangered pavement sections was that they could not be restricted without inflicting serious damage on the economy and supply of certain population centers. These sections were kept open to traffic without regard to road damage. A true picture of the degree of frost endangering by the number of kilometers of closed roads during individual thaw periods would result only if all endangered roads were subjected to restrictions. Even then, subjective judgments would have an influence. The difficulty of this problem is also expressed by Crawford and Boyd, who were working only in a considerably narrower field. They stated (15): "The most difficult problem, in investigations of this type, is the establishing of relative road conditions in a larger area, as well as differences from year to year." Therefore, they restricted themselves merely to descriptions like "very bad," "bad," "good," etc.

After the reasons mentioned indicated that absolute values should not be used, an attempt was made to set up some relative values. The damage to the frost-endangered road system in the catastrophic spring of 1953 was selected as a reference point in the various climatic areas, according to extent and severity of the destruction, and set at 100 percent. The damage in following years was compared with this base and expressed as a percentage of this maximum.

In individual years, frostproofed roads and roads that had traffic restrictions were excluded from the judging, so that only frost-endangered roads carrying traffic were considered.

Because a personal judgment of the whole road system by the author was not possible, the assistance of road builders in larger areas was requested. Their profession and experience guaranteed a less faulty judgment, and they did not suffer from the previously mentioned influences. The following deserve special recognition for their valuable contributions: Dipl.-Ing. Schleburg and Dipl.-Ing. Scheiblauer (Munich); Reg.-Baurat Trattner (Stuttgart); Reg.-Baurat Tharang (Koblenz); Reg.-Baurat Batsch (Kassel); Dipl.-Ing. Bode (Hannover); and Oberreg.-Baurat Jahn (Flensburg). The judgments of these gentlemen were compared and supplemented by observations of the author during inspection tours, and by results of frequent conversations with experienced truck drivers at truck stops. In this manner the investigation was carried on until the winter of 1956/57.

The figures for the years 1952/53 to 1956/57, on the one hand, already indicated a criteria. On the other hand, the rapidly increasing traffic threatened to distort the results. Also, due to other activities, the assistance of the previously mentioned road builders was not available after 1956/57. Therefore, in the following years the author's own observations only were used to check the correctness of the results. This is mentioned later in the discussion of the developed criteria.

Although the precentage of frost damage for individual winters was only estimated, the agreement among all observers was suprisingly good and the following damage estimates were established:

Winter	Relative Damage (%)
1952/53	100
1953/54	50
1954/55	100
1955/56	60
1956/57	30

It is once more emphasized that this deals with relative values only, whereas the base value of 1952/53 in individual climatic areas represents completely different absolute damage. For example: Although in one certain climatic area the existing damage was only one-half as severe as in another, the frost-endangering degree in both areas was assumed to be 100 percent, because it was unanimous with all observers that the damage in the spring of 1953 was the most extensive ever observed in their areas. Moreover, this deals with average values, and individual geologically or geographically different observation posts can show certain departures from those. For example: Kempten, 40 percent in winter of 1953/54 and 70 percent in 1955/56.

INTERPRETATION OF THE DATA

Precipitation and Ground Moisture

The amount of precipitation in the fall and winter does not permit any predictions about the ground moisture controlling frost endangering during January and February, which are the months when the most severe frost periods generally show up. This is to be expected because of the complicated water budget, which was expressed as a simple formula by Fischer (25), which in effect eliminates a calculation of the ground moisture variation on the basis of short-term precipitation observations. A considerable literature has grown out of this problem, because the individual factors are interdependent (26, 27, 28, 29).

Rain following dry periods at first replenishes the hygroscopic water and adsorped and entrapped water in the ground. In this way a light rain can increase the water content of the ground considerably. Heavier or longer precipitation following this does not raise the water content nearly in proportion to its intensity, because the retained water already exists in the ground, therefore the water coming from a heavy rain rapidly evaporates or runs down to ground water. These complex relationships clearly indicate that a ground moisture estimate on the basis of the precipitation only, is more than problematic.

An example from the author's diagrams shows this (Fig. 1): The dry winter of 1953/54 was compared to the wet winter of 1954/55. In the first, or the 6 weeks between the beginning of November to December 20, 1953, there was scattered and light precipitation, indicating a very dry fall. In the other winter, considerable heavy precipitation extended through the fall, and was also heavier during the winter months than in 1952/53. The measurements at all the stations included in this, however, showed that the ground moisture was equally high for both years in the critical months of January and February, when the deep penetrating frost periods occur. Heavier individual rainfalls influenced only for a short period the upper 2 dm of the ground. (The sharp rise in ground moistures at Giessen in 1953 was caused by changing the area being measured from humus soil to loess.)

The increase in the water content is easily recognizable after the frost periods, during which the ground moisture could not be measured or plotted. This increase is caused by the formation of ice lenses in the upper layers from water raised up from the lower layers.

After coming to the conclusion, on the basis of the described considerations, and on diagrams prepared by the author as well as those found in the literature, that the precipitation alone would not be sufficient to predict the ground moisture and the influence of water on the expected frost endangering, this factor was not pursued further.

Ground Moisture Before Start of Frost Periods

During investigation of the relationship between precipitation and ground moisture, it was indicated that the water content at approximately the start of frost in every year approached the same value, independent of the precipitation. The diagrams showing the natural water content on the day before the start of individual frost periods confirm this (Fig. 3).

The differences in water contents at the individual observation stations before the various frost periods are slight, even in the upper 2 dm that are more influenced by precipitation. At a depth of 40 to 50 cm they are even more equalized, so that it would not be considered feasible to make an estimate of the frost endangering of the roads in individual years on the basis of ground moisture.

There are several frost periods in nearly all the years under consideration. The ground moisture before individual frosts in one winter is practically the same. This is opposite to the prevailing assumption that the intervening thaw periods, with generally heavy rainfall, would raise the ground moisture before the next frost period.

The differences in water content between various stations (for example, the considerably higher ones at Stuttgart-Hohenheim as against the low contents at Hamburg or Heidelberg) are caused by different soil types in the measuring areas (silt at Stuttgart-Hohenheim, sand at Hamburg).

It only remained to investigate if these insignificant differences were not the largest possible fluctuations between the lowest and highest water contents of the soil types. This examination permits use of the MAB method, in which, as already mentioned, only the range is considered. Slight changes in the ground moisture will produce decisive differences, because the examined range between the lowest and the highest ground moisture is smaller, and the lowest water content established by repeated observation over a longer period of time has already been assumed to be zero.

The diagram of these MAB before individual frost periods also shows a nearly equal high degree of saturation at all stations. Here at a depth of 40 to 50 cm, which supplies the water for the frost zone, it is equalized and is between 60 and 100 percent. Irregularities (as for example at Hamburg, with values always below 100 percent, or other MAB that are over 100 percent) can be explained as results of incorrectly selected maximum or minimum values that were later adjusted according to German weather service observations. As previously mentioned, in the interest of an equal judgment, in this case the newly established base values should not be taken into account. The tolerances of the water content determination must be considered with these slight fluctuations in mind. It also must be considered that there is a possibility of slight changes in soil in the measuring area where the water content sample originates. It is assumed, for example, that this was the case for upper soil layers at Stuttgart-Hohenheim in 1957/58. In any case, neither the absolute water content nor the MAB's justify any conclusion about the expected frost damage. Even with the slight differences, it cannot be established that excessively high ground moisture existed in the highly frost-endangered winters of 1952/53 and 1954/55 and that very low ground moisture existed in some winters with little frost damage.

On the contrary, both methods of presentation permit recognition of the fact that, independent of the precipitation, approximately the same saturation of subsoil exists at the start of individual frost periods and, in fact, also after intervening thaw periods. Due to low evaporation, there is enough precipitation until January or February, even in dry falls, to produce this saturation.

The frost-endangering degree of roads is not influenced by precipitation differences or ground moisture at the start of or between frost periods.

Precipitation During Thaw Periods

A factor that can possibly influence the extent and severity of the frost damage during thaw is the rainfall during this time. By penetrating the weather-damaged leaky pavements the precipitation could soften up more of the soil under the road which is already rich in water due to the melting of the ice lenses.

The diagrams of the cumulative precipitation sums for the winters 1952/53 to 1956/57 for the short 4-day period during the thaw, as well as for the longer 14-day period (Fig. 2), show no evident relationship between frost damage and precipitation. For example, during the spring of 1953, which produced extensive frost damage, there was little or no rain during the time periods under consideration; but in the spring of 1954 and 1956, which showed little frost damage, there was considerable precipitation in many places.

The diagrams, which were prepared by using points in addition to those used in the ground moisture presentations, because the requirement of ground moisture measuring was dropped, show that heavy precipitation during the thaw periods has not increased the destruction and the absence of precipitation has not reduced it. There is no relationship between precipitation during thaw periods and degree of frost endangering.

The often-heard opposite opinion is probably strongly influenced by appearance. A frost-damaged road covered with water puddles shows the subsoil, that has pushed up through the pavement, in an additionally softened condition, and consequently appears to be more deeply damaged than the same road in a dry but otherwise identical condition (Fig. 12).

Figure 12. State road 229 at Halver (Section Remscheid - Lüdenscheid) on April 8, 1955.

Temperature and Ground Frost Pattern

Although differences in precipitation or ground moisture before, between, and after the individual frost periods do not constitute the basic cause for the variations in frost damage, there is evidence of a definite dependence of the degree of frost damage on the temperature pattern and the ground frost pattern that is connected with it. For the individual years, the temperature sum curves of the five winters, 1952/53 to 1956/57 (Figs. 5, 6, 7, 8) for which the frost endangering degree during thaw was estimated as 100 percent, 60 percent, 100 percent, 50 percent, and 30 percent, respectively, indicate the following:

In the winters of 1952/53 and 1954/55 that led to catastrophic road destruction in the following spring, the temperature sum curves in general show a flatter slope, which means that these winters were mild in the entire West German Republic, with smaller negative average daily temperatures. (The only exceptions in 1954/55 were the stations of Altastenberg and Hof, which were located in climatic extremes. The frost damage in the Hof area and in the Altastenberg area was less. This was probably the result of the longer duration of the steeper part of the curve during the first frost period which, at the other stations was probably too short to have much influence on the second longer one with a flatter pattern.)

In the winter of 1953/54 and 1955/56, with following moderate frost damage, and in the winter of 1956/57 with slight frost damage, the temperature sum curves climbed steeply. These are cases of frost periods of very low average daily temperatures, sharp frosts, and heavy winters.

This can be shown as distinctly for the winter 1953/54 as for 1955/56, if the second important frost period is plotted separately and the start of the curves for all stations is moved to the origin (Fig. 6). At stations which showed a flatter curve for the second frost period in winter 1956/57, the freezing time was too short to permit any significant frost penetration through the road surface. The first frost period at these stations, and both periods at all others, were so sharp that the frost damage following spring thaw was very slight.

The graphs indicate further that the winters of 1953/54 and 1956/57 with sharp frosts and slight frost damages show as extensive intervening thaw periods as do most of the stations in the mild winter of 1952/53 and all stations in 1954/55. The "frost-indexes" —the total sum of the negative temperatures--in the winter of 1953/54 are as high, and in the winter of 1955/56 considerably higher, than in 1952/53 and 1954/55. The frost damage is not in proportion to the frost index, as high or higher, but is just the opposite—considerably less extensive. The temperature pattern indicates the following:

1. The frost damage to the roads in spring depends on the negative temperature pattern. It is more severe with flatter inclination of the temperature sum curve connected with milder frost, and slighter with steeply inclined temperature sum curves connected with sharp frosts.
2. The intervening thaw periods are of just as little importance in this relationship as is the ground moisture.
3. Frost index is not a controlling factor of the frost damage in spring.

This, at first, surprising statement that sharp frost produces less damage can easily be explained by the relationship between temperature pattern, depth of frost penetration, speed of penetration, and pressure distribution in the ground, with the help of a sketch showing the principles (Fig. 13).

The ground temperature station at Cologne was selected as an example for this sketch and the mild winter of 1954/55, with very extensive frost damage, was compared to the heavy winter of 1955/56, with moderate damage.

For the pressure distribution, results of measurements made by the author at an experimental road on the land of the Department of Roads were used (30). These corresponded, principally in the shape of the curves, magnitude of pressures and theoretical calculations of the pressure distribution in the ground, to the later measurements done with more refined methods and calculations of pressure distribution (31, 32, 33). The pressure distribution under 5 cm of bituminous surface and 25 cm of crushed rock

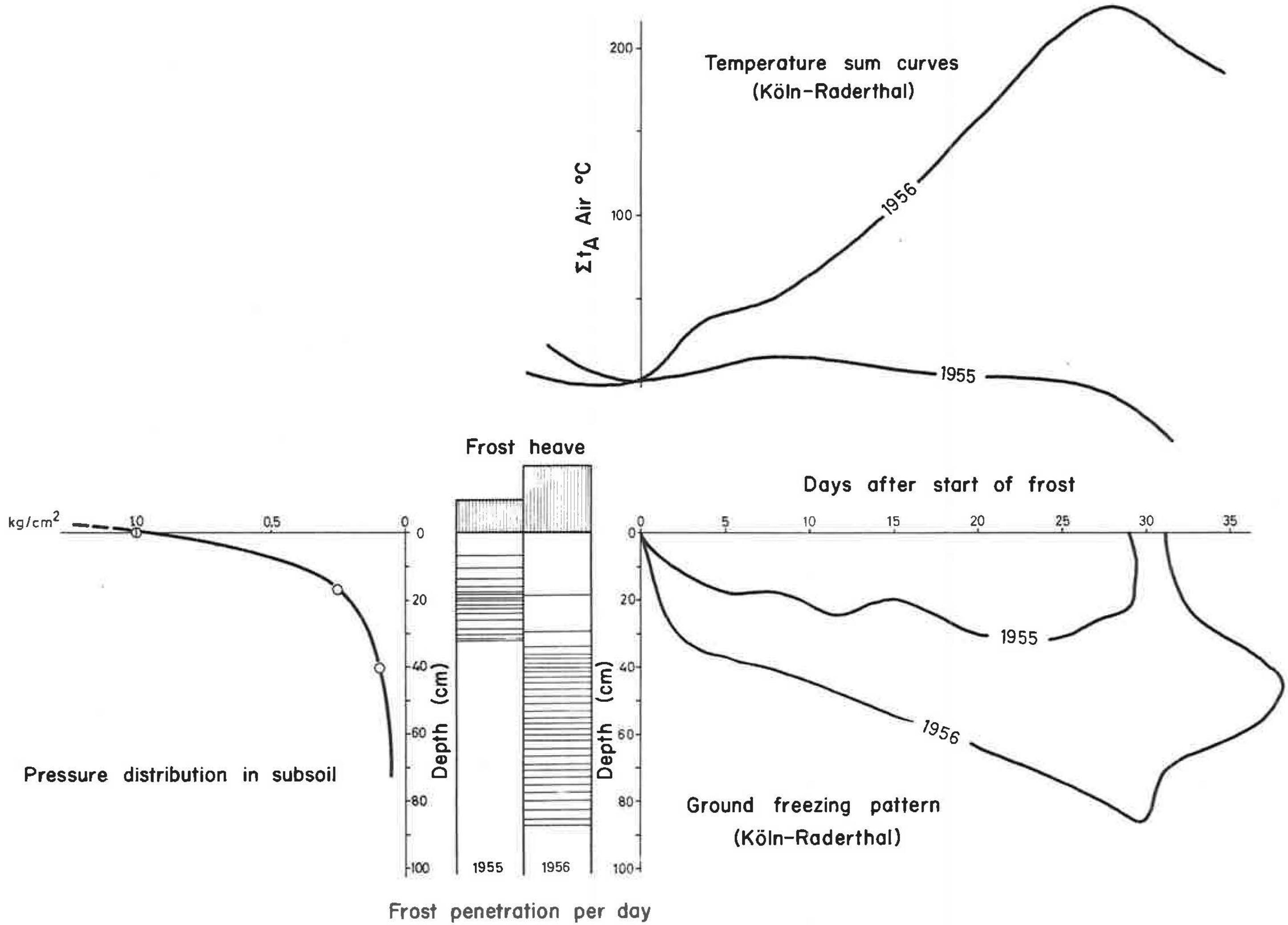

Figure 13. Relationship between temperature and ground freezing patterns, pressure distribution in subsoil, and frost heave.

base, which type of construction is often encountered on the old frost-endangered roads, was selected for the sketch. The static load was 2,500 kg on a round plate of 30-cm diameter.

An assumption for the interpretation of the processes is the independence of the suction forces from the temperature drop. The existing theoretical developments and resulting mathematical formulas clearly show this independence (7, 34). Extensive experiments to establish the magnitude of the suction forces produced no satisfactory results; at the same time, it did not contradict the previously mentioned independence (35). Earlier experiments by Beskow (2) showed equal frost heaves in equal time periods with considerably different temperature drops.

In his fundamental experiments about the degree of frost danger in various soils, Dücker (36) came to the conclusion that the frost heave depends on the freezing temperature, and that, at an air temperature of -15 C, more water is drawn up in a minute than at -10 C. However, both soil samples were subjected for the same length of time to the two freezing temperatures, which resulted in different frost penetration depths. Therefore, in regard to the unit frost penetration, the picture shifts, especially since Dücker established that equal amounts of water froze during equal periods of time. The amount of water drawn up was calculated at any time as uniformly distributed throughout the entire frost zone, which assumption did not affect the solution of Dücker's question. Actually, the water content in these experiments will be higher in the lower part of the frost zone than in the upper. The frost penetration is at first more rapid with lower freezing temperatures, but it slows down with depth because the liberated heat has farther to travel. Besides, the temperature at the lower limit of the frost zone is always 0 C. It is noteworthy that Dücker's experiments with most frost-endangered soils at the -10 C freezing temperature show, without exception, higher water contents in the entire frost zone than at -15 C, even when the water content is determined as the average of the whole frost zone. This agrees with the following considerations of the author. Also Ruckli, in his interpretation of these experiments, comes to the conclusion (7), that they do not contradict the concept of the independence of suction forces from temperature drop. According to oral information from Schaad (Zürich, April 28, 1959) he has arrived at the conclusion that, contrary to permeability and capillary rise, the suction forces depend on soil type only and not on temperature drop.

According to the mathematical development and the mentioned experiments, which were of course intended for another purpose, the assumption of equal drawing up of water in a time unit is justified.

Beyond that, consideration should be given to the known fact, that with sharp frost and resulting very rapid initial frost penetration, the time is insufficient to insure a timely supply of water to the frost zone, because of resisting forces caused by its movement. As a result, even very frost-susceptible soils at low freezing temperatures freeze homogeneously without forming ice lenses. Schenk (37) also attributes this partially to the higher viscosity of the water in this case.

Figure 13 shows that the frost penetration for the first few days, in the case of air temperatures of a few negative degrees represented by a flat temperature sum curve, is considerably slower than with low freezing temperatures (steep temperature sum curve). In the first case (1955) there was considerably more time available for each depth unit under the pavement (surface and base) to draw up water, than in the second case (1956). The comparison of the penetration depths by days, projected to the left of the ground frost pattern, shows that clearly. Thus, with mild frosts the water supply to the subsoil adjacent to the pavement is greater than with sharp frosts.

Of course, more water collects in the lower part of the frost zone in the second case (sharp frost) due to the less rapid advance of the frost at lower depths, and the absolute amount of water drawn up can be greater, resulting in higher frost heave than with mild frost.

An examination of the pressure distribution of traffic loads indicates that the higher water concentration, caused by moderate freezing temperatures, is in the area of higher pressures. With low temperatures, however, the water is in an area of very slight pressures, and considerably less water is drawn up to the high-pressure area. There is even the possibility of homogeneous freezing. The subsoil softens up considerably more in the critical high-pressure area during spring thaws after mild winters, and loses more of its carrying capacity than the same area would after a hard winter. In addition, following the deep frost penetration of a hard winter, the thawing out of the lower part of the frost zone, which has collected most of the water, starts considerably earlier than the upper part, due to the earth heat, and therefore a portion of this water can drain away in the unfrozen subsoil while the upper part of the frost zone is still frozen and able to carry loads.

These relationships cause the more extensive and severe frost damage during the thaw after a mild winter.

This of course does not take place with rigid pavements. With these, the heaviest damage takes place due to change of support conditions because of unequal frost heaving. These pavements can bridge the softened subsoil during the spring thaw considerably better due to their much wider pressure distribution. The rigid pavements, which were not frostproofed, are disregarded in this paper, due to their constituting such a small and diminishing portion of the frost-endangered road system.

FROST ENDANGERING CRITERION

These considerations show that the knowledge obtained from the temperature sum curves of the winters 1952/53 to 1956/57 should be generally applicable.

Next to the criteria for frost susceptibility of soils as a measure of the required protective steps (38, 39) in new construction and in rebuilding of roads, a criterion can be established, as a result of these investigations, which will make possible an estimate of the extent and severity of the expected frost damage of the non-frostproofed roads during the thaw periods, on the basis of the temperature pattern (Fig. 14).

The only requirement is to draw the temperature sum curves of the significant frost periods as in Figure 14. The position of these curves in the individual sections will indicate the frost damage to be expected under traffic load. If several severe frost periods, separated by thorough thaw periods, occur in one winter, they should be treated separately. For example, the first frost period in the winter of 1954/55 produced no damage due to the sharp frosts. The catastrophic condition of the roads in the spring of 1955 was caused by the mild frosts of the second period.

There is less damage if frosts are of such short duration that the frozen zone extends only a little below the pavement (surface and base) than with longer frosts which show the same slope for the temperature sum curve. The damage after the winter of 1956/57, for instance, was only slight, although the temperature pattern was entirely in the "moderate damage" area. For that reason, the K-line was added to the criterion. The expected damage will be a degree lower than indicated by the section if the maximum of the temperature sum curve is to the left of the K-line. Instead of severe damage, there will be moderate, or instead of moderate, only slight.

The temperature curves often start in one section and then cross into another. In these cases the damage will correspond to the sector which contains the longest portion of the climbing part of the curve, calculated in days. For example, if the longer climbing part of the curve is in the "moderate damage" sector, the frost has already penetrated rapidly so far that the following mild frosts, which cause the curve to move into the "severe damage" sector, cannot influence it any more. If the longer part of the curve is in the "severe damage" sector, a substantial amount of water has already been drawn up close to the base, and following sharp frosts can produce no moderation.

As opposed to these considerations, the conclusion that light damage is to be expected in the case of temperature sum curves in the lowest sector is based on the fact that the frosts of the temperature sum curves in this area do not penetrate under the base of the road and consequently can only cause damage due to frost-susceptible soils that have penetrated the base, or are present there because of the destruction of the base. Obviously, the changeover between sectors of the graphical criterion, and also the position of the K-line, is gradual and depends on the soil type and local climatic influences. It may happen, for that reason, that the criterion is overcome by the problems connected with the complex weather processes and, as in the case of weather prediction, occasionally in isolated areas reality deviates from the prediction.

The deviations will, however, be within narrow limits, especially in regard to soil types: In clayey soils, for example, frost penetrates slower than in silts and cohesive sands, and therefore more damage could be expected with clays than with the other types. The slower frost penetration with clays is, however, equalized by their lower permeability, which reduces the water supply to the frost zone. This is also stated in the previously mentioned "Freiberger Frostregel." Because this is chiefly a larger scale survey, the differentiation of the borderlines of the sectors according to soil types could be waived.

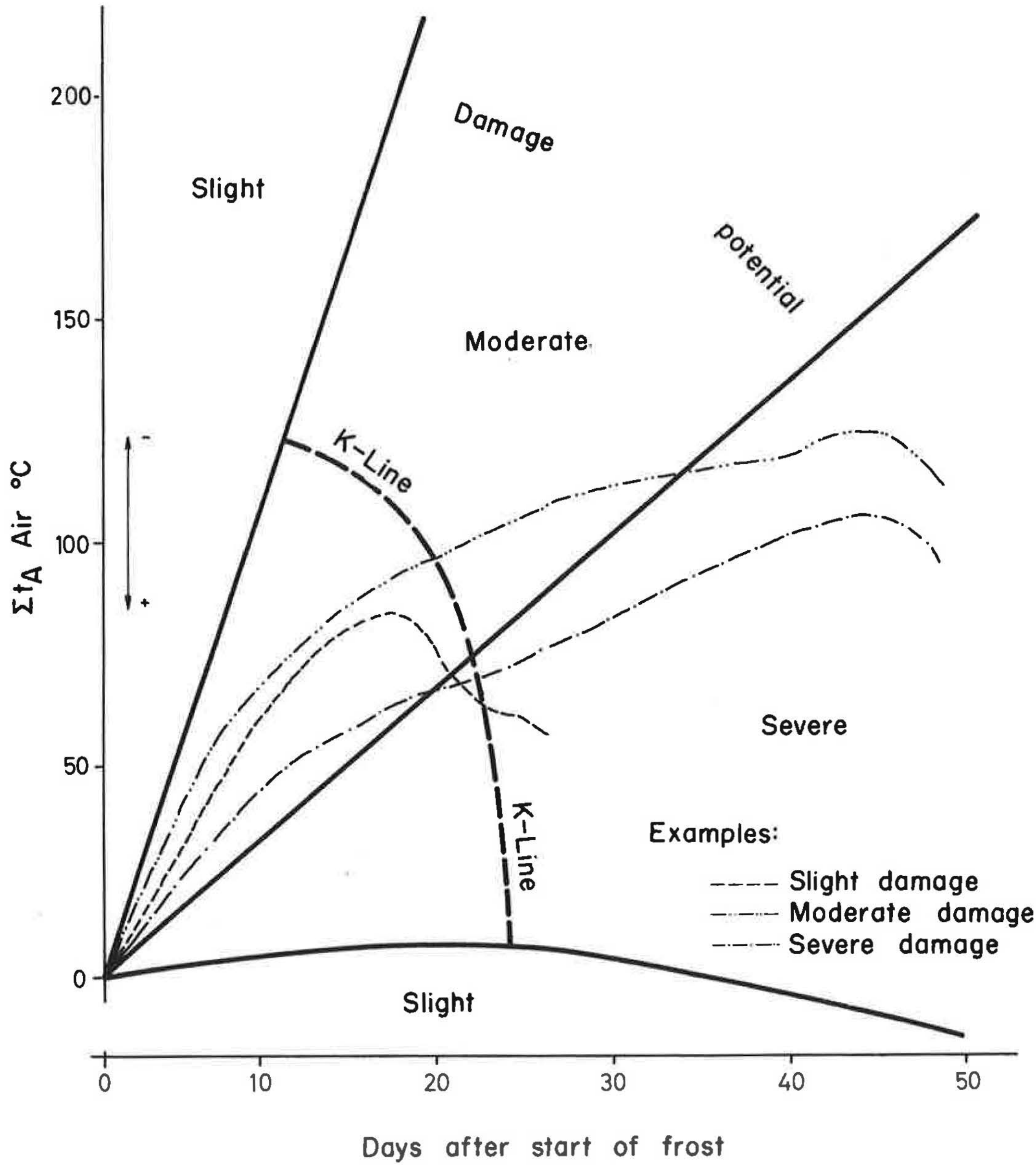

Figure 14. Locations of the temperature sum curves during freezing and the damage expected during thaw: (a) If the climbing part of the curve ends before K-line, next lowest endangering degree applies; (b) If the curve crosses into another sector, the damage will correspond to the sector which contains the longest portion of the climbing part of the curve calculated in days; (c) The frost periods of any winter are judged separately if there are significant intervening thaw periods.

CHECKING OF THE CRITERION

Several phenomena that have been treated as special cases in the literature, can be simply explained after it was established, as a result of the discussed relationships and investigations, that the basic cause of the spring destruction of the roads was not hard winters with extensive frost heaves, but the mild ones, independent of precipitation and ground moisture.

Schaible, for example, writes (40): "Despite a mild winter, with the maximum frost penetration of 30 to 60 cm which practically takes in the rock course only, the roads suffered extreme damage because of weakened and cavity-filled bases." This information confirms the developed criterion, according to which the damage took place not despite of but actually because of the mild winter. The slow penetration of the frost to the depth of 30 to 60 cm creates, according to Figure 13, the highest danger, because the thickness of the pavement and base of frost-endangered roads in general just barely reaches 30 cm. In another place Schaible reports (41) that, under equal conditions, roads in lower and sheltered locations show a tendency toward increased and earlier damage, or (4): "that frost and thaw damage in lower locations, despite milder frosts, appear more extensive and severe than in high locations with hard and long frosts, so that the majority of this damage takes place in low locations." This observation agrees with the determination discussed herein.

Schaible explained this with more intervening periods and more rapid thawing, and the damage of the mild winter of 1952/53 he explained by more precipitation. The criterion and the investigations about the ground moisture show also, in this case, that the damage was not in spite of but because of the mild winter, and not as a result of the precipitation and thaw periods. Likewise, oral information from Oberreg.-Baurat Schmiedinger (government of Schwaben; April 24, 1957) that the frost damage in the spring of 1957 was more extensive in the Augsburg area than in the area of Kempten, also points in this direction. The subsoil in both areas is frost-susceptible "molasse." The precipitation was higher during the fall and winter at Kempten, and the thaw periods in both areas were the same. The temperature curve at Augsburg was less steep than that at Kempten.

An investigation by Crawford and Boyd (15) is interesting in this connection; they both likewise came to the conclusion that the frost index alone is not decisive. Because they adhered to the popular point of view that heavy frost with resulting high frost heave is the basic cause of severe damage, they developed a damage index by multiplying the frost index by the water reserve from the precipitation of the last 30 days before the start of frost. This damage index indeed agrees with the actual damage of two winters, but not at all with the damage of the winters of 1952/53 and 1953/54 (see Table 1).

In the last winter the index for Calgary, as well as Ottawa, gave high values, which predicted severe damage. Actually, the condition of the roads was better than normal in the spring. In the winter of 1952/53 the index for Ottawa was lower, and for Calgary, due to a long dry spell, even zero. The condition of the roads in the spring was bad. Crawford and Boyd explain this disagreement by the fact that there was at first a hard frost during the winter of 1953/54, but the expected damage did not result because of the following mild frosts. This explanation, according to the developments reported herein, is less satisfactory. A truer interpretation is obtained if the temperature pattern from their investigation is compared with the criterion developed here (Fig. 15): The temperature sum curve for 1952/53 was flatter and led to damage, while the steeper curve of 1953/54 was very close to the borderline of the slight damage sector, and no damage resulted. The investigation by Crawford and Boyd not only confirms the developed criterion, but, by proper interpretation of the figures, it also shows the independence of frost damage from precipitation, especially in the example of the winter of 1952/53, as proved in this paper. For the rest of it, their observations agree with those of this author; namely, that the intervening thaw periods have no influence on the damage.

A further contribution to the confirmation of the knowledge obtained was the author's measurements with the "frostnail" and the "frost indicator" on the Fichtel Mountain Road (B 303) near Bischofsgrün and on State Road 27 near Tauberbischofsheim.

TABLE 1

WEATHER AND ROAD DAMAGE DATA FOR TWO CANADIAN STATIONS[a]

Winter	Frost-Thaw Cycles (no.)	Start of Frost Heaves	Frost-Index (°F-days)	Precip. During 30 Days Before Start of Frost (in.)	Damage Index, Dolch	Ground Moisture Accum. (in.)	Modified Damage Index	Road Condition in Spring
(a) Calgary, Alberta								
1948-49	13	Nov.16	2,421	0.37	896	0.30	726	–
1949-50	17	Dec.2	2,889	0.01	20	0	0	–
1950-51	14	Nov. 5	2,493	1.22	3,041	0.94	2,343	Worse than normal
1951-52	15	Oct. 15	2,313	2.16	4,996	3.80	8,789	Worst of investigated
1952-53	18	Nov.14	1,143	Tr	6	0	0	Worse than normal
1953-54	15	Nov.17	1,824	0.31	565	0.24	438	Better than normal
Avg.	15	Nov.11	2,180	0.68	1,589	0.88	2,049	–
(b) Ottawa, Ontario								
1948-49	11	Nov.28	1,269	4.17	5,292	4.00	5,076	–
1949-50	18	Nov.17	1,719	2.04	3,507	2.76	4,744	–
1950-51	13	Nov.21	1,491	3.76	5,606	4.00	5,964	Very bad
1951-52	13	Nov.1	1,557	1.50	2,336	0.51	794	Very good
1952-53	12	Nov.28	933	2.13	1,987	4.00	3,732	Bad
1953-54	14	Dec.15	1,449	2.56	3,709	3.41	4,941	Good
Avg.	13	Nov.23	1,403	2.69	3,740	3.11	4,208	–

[a]From Crawford and Boyd (15).

Both instruments, which were developed from the ideas of Dr. Dittrich of the Department of Roads, permit continuous measurement of the frost heave of the road surface, and the frost penetration (42, 43). In the winter of 1955/56, with hard frost, rapid frost penetration was measured to a depth of 95 cm at Bischofsgrün, and to 65 cm at Tauberbischofsheim. The frost heaves of 15 cm and 6 cm were correspondingly very high. Some frost cracking naturally occurred at Bischofsgrün. The damage in the spring of 1956 was negligible at both places due to the rapid frost penetration. In the spring of 1955, the frost heaving measured only 3 cm and 1.5 cm, respectively, due to slower frost penetration to approximately one-half the depth of 1956. Considerable damage resulted. (The figures for 1955 were obtained from the Bayreuth and Tauberbischofsheim area offices of the Department of Roads, because at that time the instruments had not yet been installed.)

The checking of the criterion, on the basis of the temperature patterns for the winters of 1957/58 to 1960/61, confirms its applicability. Of course, it was not possible to use the systematic establishment of the frost endangering degree as previously described. As explained, only limited judging was possible, based on the author's tours and spot check inquiries. Therefore, only a smaller number of stations was used for the graphs, starting with 1958/59 (see Fig. 9).

Winter of 1957/58

Because of the complicated pattern of the cumulative temperature curves for the entire winter of 1957/58 the first impression would be to expect heavy damage, according to the criterion. In reality only slight to moderate damage was reported. Closer scrutiny reveals three distinct frost periods during that winter. The first, around the end of the year, was so short and mild that it could produce no damage. The other two, which are especially noticeable in the case of stations with curves above the horizontal (time) axis, show that the curves are predominantly in the heavy damage section, but without exception end before reaching the K-line. This indicates only moderate damage, which is all that actually happened, the same as for locations where the curves crossed the K-line but were in the moderate damage sector. Slight damage was reported at stations where the curves were below the horizontal axis. This winter emphasizes the importance of the K-line and the necessity of examining the different frost periods individually.

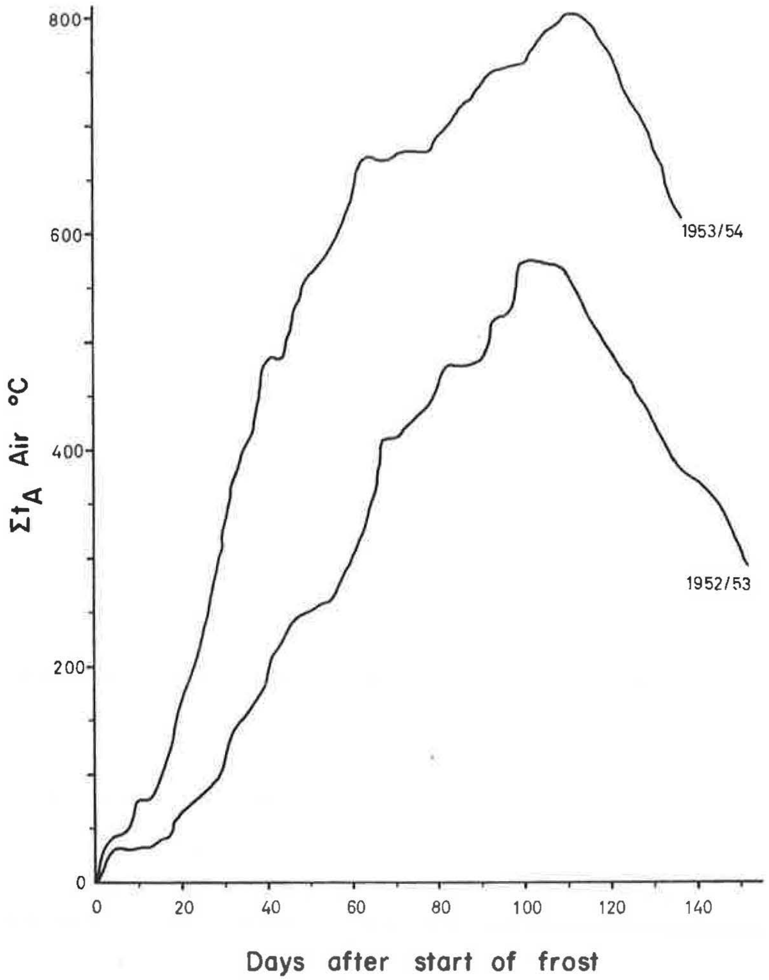

Figure 15. Temperature sum curves (from Crawford and Boyd, 15).

The temperature sum curves for the winters of 1958/59 to 1960/61 show the reason why the West German Republic has escaped extensive frost damage since 1956/57, and why this damage has been limited to narrow areas.

Winter of 1958/59 (Fig. 9)

If the first 3-day-long frost periods in 1958/59 are disregarded, the curves for the north German stations of Husum, Braunschweig, and Kassel are in the moderate damage sector for the first distinct frost periods. They end so far before the K-line that only slight damage occurred, in agreement with the criterion, and similar to the experience in Rhineland, corresponding to the curve for Essen-Mülheim, which was near the border of the lower sector of "slight damage." The second periods, which were partly in the severe damage sector and partly in the moderate, were again so short that generally slight damage occurred. In the entire north German area the damage was slight. It was different with the Bavarian stations, the curves of which, according to the criterion, by their length and inclination indicated moderate damage. According to the author's observations, in March 1959, of the condition of State Road 33 in the Black Forest, for sections near Triberg and Engen which correspond in temperature patterns to the Bavarian stations, and of the sections of State Road 30 between Ulm and

Ravensburg, and State Road 19 between Memmingen and Lindau, there was considerable damage. The temperatures at State Roads 30 and 19 would definitely have been milder than those at Kempten. That would definitely place the curves in the sector of heavy damage, which is what actually occurred.

Winter of 1959/60

In the winter of 1959/60 the frosts during the first frost period were, without exception, hard and short. Most of the temperature sum curves are located on the border between the moderate and slight damage sectors. They all end before reaching the K-line. The second frost period was likewise short everywhere. The temperature sum curves were, of course, somewhat less inclined and were located in the moderate damage sector. Inasmuch as they all ended before reaching the K-line, the next lower degree of endangering was to be expected, according to the criterion. Actually, the frost damage in the spring of 1960 was slight, which was confirmed by the author's tour of the same roads visited in the winter of 1958/59.

Winter of 1960/61

The temperature pattern of the winter of 1960/61 corresponded almost exactly with that of 1958/59, with two short frost periods in north Germany and a longer one with moderate freezing temperatures in south Germany. Only in Baden-Württemberg were the frosts longer than in 1958/59.

Again the frost damage in the north was slight, corresponding to the position of the temperature sum curves to the left of the K-line, whereas it was considerably more extensive in Baden-Württemberg and Bavaria. According to the weekly reports from Districts No. 5 and No. 6 of the Traffic Ministry, it was necessary to put extensive traffic restrictions into effect. The author's travels through the area of Kempten, and again on State Roads 19 and 30, in March 1961, showed that the damage in this spring was even more severe than in 1959. The continuous temperature sum curve for Kempten, with a long frost period, is actually located in the severe damage sector. The foreman of the repair group of the maintenance area of Ravensburg confirmed in April 1961, during this tour, that damage to State Road 30 was severe after the winters of 1958/59 and 1960/61, and very slight in the spring of 1960. These reports corresponded, as did the author's observations, with the respective position of the temperature sum curves in the criterion.

CONCLUSIONS

On the basis of the observations of the years of 1952 to 1957, a criterion was developed which made it possible to predict the degree of expected frost damage by observing the weather factors. This criterion, based on the temperature pattern, makes possible not only this prediction but also the explanation of phenomena that contradicted previous assumptions and were therefore treated as special cases.

An analysis of the winters of 1956/57 to 1960/61 based on the criterion agrees with the actual damage. It was proved to be correct in these five years. At the same time it was confirmed, during this period, that the temperature pattern alone makes the prediction possible, and therefore the degree of damage is independent of the precipitation and ground moisture. The temperature pattern during freezing thus permits prediction of the expected average frost damage in a larger area.

The criterion provides relative values for individual cases of short road sections. Here the degree of frost susceptibility of the soil, very local climatic conditions, and the construction of the road itself, play a more important part. However, the criterion can also be used in these cases when the known severest and most extensive damage for that road section is considered as "severe damage" and the criterion is standardized on this basis.

The evidence that led to the criterion, and was based on the observations made on the roads that were not frostproofed, is also in agreement with the frostproofing measures in new road construction. In this area there are a few more conclusions that will be mentioned briefly. The frostproofing is not attempted for the entire frost depth, but only to a point where the underlying frost-susceptible soil in softened condition will have sufficient carrying capacity for the pressures existing at that depth and where the frost-protective layer will effectively equalize the unequal frost heaves.

The evidence in this study indicates that the depth of the frostproofing must be the same in the various climatic areas. It is not necessary to increase it in high areas with predominantly hard frosts and rapid frost penetration. On the other hand, it cannot be reduced in milder areas, otherwise dangerously high water concentrations can take place directly underneath the thin frost-protective layers, due to the slow frost penetration. A reduction in depth is permissible only if there is evidence that the frost will never penetrate below the planned depth of the frostproofing layer. In regard to this, it should be remembered that frost penetration is more rapid in the noncohesive soils of the frostproofing layer than in cohesive soils.

This is not caused, as is erroneously assumed (44, 45), by different temperature or heat conductivity, but, as is well known, quite predominantly by the considerably higher water content of the cohesive soils. Due to this, the freezing liberates considerably more latent heat of fusion, which has to be carried away to begin with, and therefore the frost penetration is slowed down.

The following must be considered in connection with investigations of measures to prevent frost damage by building in of insulating layers: Measures of this type can slow frost penetration in frost-susceptible soils to such a point that the concentration of water would take place adjacent to the bottom plane of the construction. These layers can transform a non-damaging hard frost into a damaging mild one. From this standpoint they can be effective only if they completely prevent the frost from penetrating the ground.

In addition, materials of this type must blend into the systematic construction of the road, with respect to their durability and carrying capacity under static and dynamic loading.

Because the frost-protective layer, with its properties and carrying capacities as fixed in the ZTVE (39), is at the same time a load-carrying element of the construction, other methods of frostproofing frost-susceptible soils should be investigated to find out if cohesive soils under such treatments provide a carrying capacity comparable to that of a frostproofing layer.

SUMMARY

The answer to the question of whether the degree of frost damage expected for individual years on the still extensive network of frost-susceptible roads can be predicted by means of the weather patterns for the fall and winter, is not a contribution to the expansion of knowledge about the frost problems of the road only. Clarification of these relationships make it possible for the Department of Roads to have an idea of how extensive necessary traffic restrictions will be, to make corresponding preparations, and to plan measures for maintenance and restoration of the roads. It enables the transportation industry to anticipate scheduling of its transports during the critical thaw period of the subsoil.

The investigation of the decisive weather factors of precipitation and resulting ground moisture, and the temperature pattern, produced the following:

1. The precipitation before freezing, during intervening thaw periods, and during the final spring thaw, has no influence on the degree of frost endangering.

2. The moisture contents of frost-susceptible subsoils before the start of the frost periods are different for various soil types but are nearly equal each year for individual soils. Even relatively light precipitation in fall and at the start of the winter, up to the first frost periods, is sufficient to saturate the soil with water. Heavy precipitation does not change this degree of saturation at relatively shallow depths (40 to 50 cm). These conditions hold even more true for the subsoil underneath the road, because in summer drying out under the road is less than in natural ground. Consequently, no relationship could be established between precipitation and ground moisture and the degree of frost endangering. From the standpoint of water content the starting point every winter is the same.

3. On the contrary, an incontestable connection was established between temperature pattern and degree of frost endangering. After mild frost periods considerably more severe and extensive frost damage due to traffic loads is to be expected, than after hard winters with sharp frosts, provided the frost line in the mild winters extends under the road structure.

4. In general, frost heaves will be higher in hard winters. But the frost heave damage caused by that condition constitutes only a small part of the total frost damage. By far more predominant are the failures caused by traffic during the thaw. Thus, high frost heaves give no indication of the expected total damage.

5. A criterion (chart) was developed by means of which the degree of frost endangering during the thaw can be predicted according to the temperature pattern. It is based on the average daily air temperature, which is used to plot a temperature sum curve. The position of this curve in the sectors of the criterion chart makes the prediction possible. To make it possible for all interested groups to make this prediction without any complicated instruments or calculations, an attempt was made to use simple measurements that could be taken anywhere. The criterion was checked with the weather pattern and frost damage for five years. It permits the explanation of questions that were left open by existing investigations and therefore were treated as special cases.

6. In addition, the deliberations about the influence of weather patterns produced some conclusions about frostproofing measures in new road construction and improvement of old roads, especially about the independence of the depth of frostproofing from weather conditions in individual climatic areas and the uncertainty of the use of insulating materials in road construction.

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Discussion

HANS F. WINTERKORN, Professor of Civil Engineering, Princeton University—The purpose of the investigation reported in this paper was to find a method by which the areal extent and severity of frost damage on roads could be predicted from meteorological data collected during the fall and winter preceding the spring break-up. The better such a prediction, the better and more economical can be the planning of the restrictions and regulations which may have to be imposed on traffic during the thawing period to prevent destruction of the affected roads. The catastrophic dimensions of such destruction experienced by Germany on several occasions during the last decade are an eloquent testimony to the economic importance of the present study and its results.

The selection of meteorologic factors for closer study of possible significant correlation with severity and extent of frost damage was indicated by the normal availability of the pertinent data through the Weather Service; also, it was assumed that enough was known about the influence of such factors as soil granulometry, thermal and moisture conduction properties in function of moisture content, and bearing capacities at different granulometries, moisture contents and temperatures, to assay their general effect without resort to additional work. The latter may be true, but there is evidence in this paper, as well as in most of the pertinent English language literature, that highway engineers, although eagerly availing themselves of known physical facts and concepts, are still shying away from making proper use of physico-chemical knowledge and concepts that are of primary importance for the understanding of the phenomena involved in frost action on soils.

The meteorological factors investigated by the author were (a) precipitation and soil moisture content in fall and winter, (b) soil moisture at the start of the freezing period, (c) course of air temperature, (d) course of soil temperature and freezing progress, and (e) degree of frost damage observed on thawing. On the basis of his extensive and comprehensive data, he comes to the conclusions that for West German climatic and soil conditions the degree of frost damage depends primarily on the course of negative temperatures during the frost period. This course is graphically represented by plotting the cumulative negative temperature against time in days. Danger of frost damage is greater in the case of flat than of steep inclinations of the curves. The lesser damage caused by short freezing periods is taken care of by theK-line, which starts at about 25 days on the abscissa and moves to the left at increasing rate, ending at a point having approximate coordinates of 12 days and 120 C negative cumulative temperature. The graphical representation in Figure 14 gives the new criterion for expected frost danger.

In view of the comprehensive character and high quality of the work reported by Mr. Kübler, there is little or no room for quarrel with the justification and usefulness of the criterion for the purpose and conditions for which it has been developed. However, a few comments appear to be indicated, first with respect to the physical and physico-chemical phenomena on which the validity of the criterion is ultimately based and second with respect to its application to climatic regions in which the soil, contrary to the conditions in West Germany, cannot be assumed to be in an essentially moisture saturated state at the beginning of the freezing period.

In his conclusions the author states that the criterion based on the course of temperature permits elucidation of phenomena which run counter to the concepts held to the present date and which had to be considered hitherto as special cases. This statement may be correct as far as the majority opinion of highway engineers is concerned, but is certainly not true for the concepts of those that had performed serious scientific study on the frost damage problem and on the underlying causes. The criterion singles out as the most important factor the rate of heat removal from the system. The effect of this rate on the phenomena occurring in a cooling system, including the rate of formation of centers of crystallization and the rate and direction of crystal growth, has long been known in the field of mineral paragenesis and had been recognized and utilized in many technologies. Among the latter are the heat treatment of metals, glasses and ceramics, ultrarefining of metals and chemicals by zone melting, and food preservation by quick freezing processes, to mention only a few.

Taber (1) and Beskow (2), the pioneers in the field of soil frost research, as geologists and mineralogists were well aware of the importance of the rate of cooling on the final properties of a system produced or modified by crystallization; recognition of this fact is implicit in the methods developed for laboratory testing of frost susceptibility of soils.

Also, Kögler, Scheidig and Leussink (46) pointed out that the particular manner in which ice crystallized in soil was an expression of crysto-chemical phenomena. This was further emphasized by Winterkorn (47), who also pointed out the analogy existing between ice lens formation and the rhythmic flocculation responsible for "Liesegang's rings" and allied phenomena observed in geologic formations (48). In view of the cited and other available evidence, it is not surprising that the criterion is valid for the climatic area for which it has been developed; rather, the scientist wonders why it was not developed sooner.

The simplicity of the criterion derives from the fact that irrespective of the amount of precipitation, the subsoils in West Germany possess approximately the same degree of moisture staturation at the start of the freezing period. This, of course, limits its range of applicability in its present form. However, because it has a valid scientific basis, one should try to extend its usefulness and develop a more general criterion applicable to areas of different climates and soil water regimes. For this purpose, the actual degree of water saturation of the subsoils concerned must be taken into account. Simple climatologic methods of keeping track of the water balance in soils are available, due mainly to the efforts of Thornthwaite and his disciples (49); hence, the task of extending the present criterion should not be too difficult nor involve much additional expense. In the suggested extension of the criterion, one may have to draw on other available physico-chemical knowledge with respect to the condition of water in soils and the thermal and capillary properties of soils at various states of saturation and consolidation, as well as on pertinent knowledge with respect to the influence of climate on soils and highways (50, 51, 52, 53).

It is felt that utilization of presently available physico-chemical knowledge concerning soil-water systems and their response to application of temperature gradients, both above and below the normal freezing point of water, will greatly aid and speed up the development not only of a more widely applicable criterion of frost danger, but also of cheaper and more certain methods of preventing frost damage than hitherto available in cases when it is economically impossible to remove or replace frost-susceptible soils.

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EDWARD PENNER, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Canada—In the paper by Crawford and Boyd (15) the authors observed some correlation between the severity of road break-up in the spring and the accumulation rate of degree-days during the early part of the freezing period in the Ottawa area. The trafficability of a selected secondary road was rated as good, very good, bad, or very bad, during several critical thawing periods. The paper showed the accumulated degree-days of freezing as a function of time for two successive years, but it provided spring break-up ratings for four successive years. The curves for the two missing years and the road assessments for the four years have been added to their diagram in Figure 16.

For the two years when the degree-days of frost accumulated at a slow rate in the fall the results show the spring road condition was either "bad" or "very bad." When the accumulation was rapid the road conditions that occurred in the spring were assessed as "good" or "very good." This is in agreement with the performance of secondary roads in Germany during thawing periods in relation to the degree-day curves as described by Mr. Kübler.

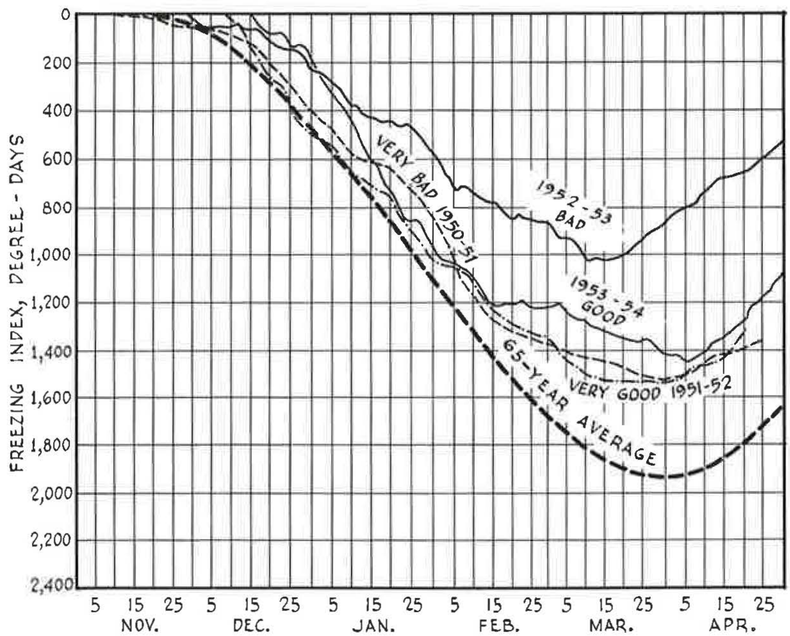

Figure 16. Freezing index at Ottawa, 1950-54.

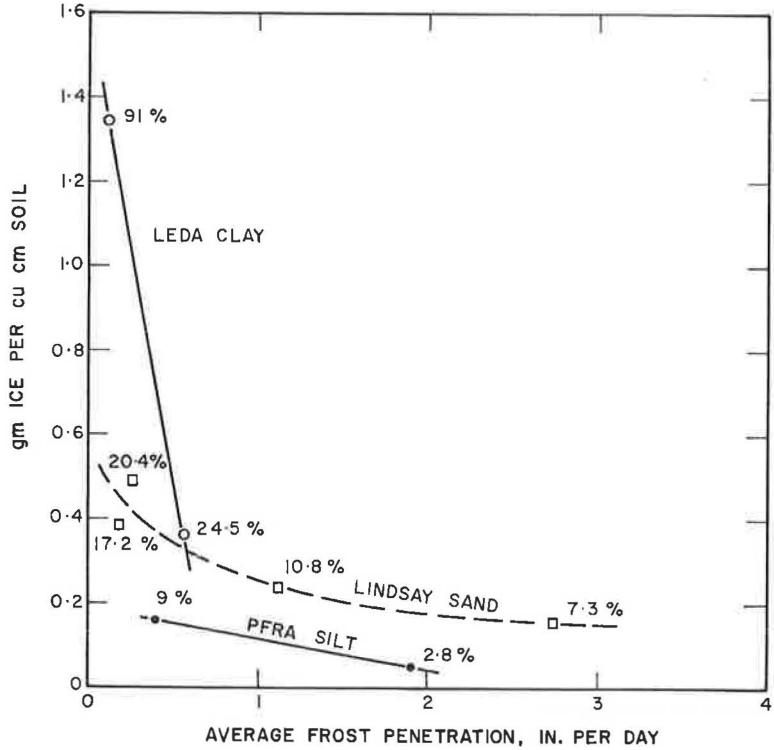

Figure 17. Average amount of ice accumulation per unit of original volume of soil vs frost-line penetration. (Values indicate percent moisture content weight added by the frost action processes; i.e., gr water/gr dry soil).

This kind of behavior can be anticipated from the laboratory determined results given in Figure 17 (Fig. 10, 54), which shows that the amount of ice accumulated per unit volume of soil during the freezing period varies inversely as the frost penetration rate.

It follows from Figure 17 that if the upper layers of a road are frost susceptible and if freezing occurs slowly (low rate of degree-day accumulation in fall) the most favorable conditions for ice accumulation near the surface are achieved. During the thawing period the melt water is released within the soil, causing a loss in the bearing strength where it is most required to carry the traffic satisfactorily. It may also be pointed out in this connection that the bearing strength of frost-susceptible soils is extremely sensitive to changes in moisture content. Alternatively, better road conditions would be expected in the spring if the beginning of the winter period was characterized by a rapid degree-day accumulation rate.

The writer is most anxious to learn about the success that has been experienced in using degree-day curves for traffic control during thawing periods.

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GEORG KÜBLER, Closure—Professor Winterkorn's discussion and kind endorsement of the guiding principles of this investigation and the criterion developed are appreciated. The author is in full agreement with him on the point that in the consideration of specific cases, as well as of the entire frost damage problem, it is not permissible to disregard the pertinent physico-chemical properties, including the thermal characteristics of soil-water systems to which he has made reference. As a matter of fact, the work in the Federal Republic has been guided by this conviction for quite some time and there is still the opinion that much remains to be done along this line. For this reason, the paper emphasizes that its scope was to predict the average degree of frost danger to be expected in a large climatic region rather than to be concerned with smaller areas and with specific cases. The publications cited by Professor Winterkorn are well known in Germany, especially those authored by him, which have been thoroughly studied not only with regard to the frost problem, but also in connection with work on soil stabilization.

The suggestion that one should try to extend the range of applicability of this criterion into areas of different climates and soil-water regimes is welcomed. Unfortunately, the author is not in a position to do so, because he does not have at his command the large amount of climatic and other pertinent data required for this purpose, including statistics on frost damage over a significant number of years in all the countries in which freezing temperatures occur. It is believed, however, that in climatic regions in which marked frost damage occurs there is enough annual precipitation to provide a relatively high level of water saturation. It would give the author great pleasure, though, if this paper and the criterion developed herein could serve as a starting point for its suggested extension to other climatic regions. This opportunity is used to point out again that the criterion does not hold for rigid pavements, such as portland cement concrete slabs, in which case the major damage is done by the heaving during the freezing of the soil and not through loss in bearing power during thawing.

The paper by Crawford and Boyd (15) had been discussed already, pointing out that, if viewed from the standpoint of this concept, the climatic data of these research workers are in accord with the observed frost damage. The author is grateful to Mr. Penner for having taken the trouble to establish the cumulative temperature curves for two additional years, which also confirm the proposed criterion.

His contribution is particularly welcome because it permits extension of the range of validity of the criterion to include the climatic conditions of Canada, thus confirming the statement made in connection with Prof. Dr. Winterkorn's discussion. It is also gratifying that the laboratory investigations reported by Mr. Penner (54) are in agreement with the fundamental considerations of the present study.

Symposium Summary

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•IN SUMMARIZING Part II of this symposium, it is desirable to quote from the foreword of Part I: Basic Considerations:

> The objective of this symposium was to review, summarize and report the state of knowledge concerning the design of highway pavement in frost areas, treating individually the factors of temperatures, water, types of soil and materials, and the freezing mechanisms of soil-water systems; and relating these factors to the design problems associated with pavement surfaces, bases, subbases and subgrades, topography, highway cross-section and grade line, subsurface drainage, climate, and traffic weight and volume.
>
> The purpose of the symposium was to provide for the practicing engineer, who is now confronted with the greatest highway program in history, a digest of current scientific knowledge that he may use as a guide in the solution of highway design problems in frost areas; and to provide for the research engineer and scientist information concerning practical highway design and construction problems in frost areas for use as a guide in experimental and investigational programs.

Part I of the symposium dealt with basic considerations; namely, water, temperature, soil, and the freezing mechanism of soil-water systems. Part II of the symposium was concerned with design considerations.

The reporting technique which was adopted for Part II of the symposium resulted in a wealth of practical information and a great amount of detailed information regarding design procedures. From an over-all point of view, it can be said that there is substantial agreement in the general approach to the problem, yet there is by no means a standardized design technique. The reports indicated that several agencies used experimental techniques for extending the results of observations on roads under service to the problem of designing new roads. As would be expected when reporting on a problem which covers such a wide geographic area, there were a variety of approaches. There were various points of emphasis and various definitions of what constituted a frost-susceptible soil. Some of these differences may be reconciled on the basis of differences in climatic factors and other environmental conditions, whereas others may be reconciled after a consideration of the predominant types of soils in any given area or lack of suitable granular materials.

One of the questions that this symposium should attempt to answer is "What basic knowledge of frost action do we have that we are not using in design practices?" Part of this summary will be directed towards an attempt to answer this question.

Some years ago, the frost problem was considered to be primarily that of frost heaving, especially differential heaving. More recently, greater recognition has been given to the problem of loss of stability of the pavement upon thawing. Some fairly elaborate field studies have been made for the purpose of quantitatively evaluating the damaging effects of frost action on roadways, or conversely, determining the necessary limitations of loadings to minimize or eliminate the damage to roadways during the

Paper sponsored by Committee on Frost Heave and Frost Action in Soils.

thaw period (19). Still more recently, there has been a gradual increase in awareness of another problem resulting from frost action effects; namely, residual roughness. Recent designs have, in many cases, eliminated or minimized differential heaving and loss of stability upon thawing. However, the interest of the traveling public in high-speed transportation, regardless of season, has focused attention on residual roughness of pavements following one or more seasons of frost action.

SUMMARY OF DESIGN PROCEDURES

To design successfully, to minimize or eliminate differential heaving and the loss of stability upon thawing, and to minimize the effects of residual roughness, it is necessary to recognize the three requirements for detrimental frost action and the mechanism by which these three factors are combined to produce undesirable pavement conditions. It is not the purpose to review the findings of Part I of this symposium, but simply to restate these factors and to comment on them from the design point of view.

Temperature

The most general statement that could be made about temperatures is that for detrimental frost action to occur, there must be a sufficient duration of sub-freezing temperatures. This has been expressed quantitatively in terms of the freezing index or accumulation of degree-days of freezing temperatures. Some fairly good correlations have been established between frost phenomena and the design problems, especially concerning the anticipated depth of frost penetration which then becomes a guide to establishing the necessary depth of treatment.

Some agencies replace objectionable material with non-frost-susceptible soils to a depth equal to the depth of frost penetration. Other agencies do not attempt complete elimination of frost penetration into frost-susceptible soil, but provide for partial protection by covering frost-susceptible soils with a thickness of selected material equal to some arbitrary fraction of the frost penetration depth, such as one-half or three-quarters.

Some agencies are using freezing index data effectively, whereas others are reluctant to use this approach. It must be recognized that over the length of a given highway there may be considerable variations in the severity of freezing. Therefore the use of the freezing index approach becomes more difficult than in the case of a specific site such as an airfield.

During symposium discussions, the point has been made that although soil and moisture conditions can be modified to some extent, little can be done about the temperatures which occur in a given location. It is true that the temperature pattern is a phenomenon of nature, and more directly, a phenomenon that is not readily controlled by man. However, it is possible to control temperatures to a considerable extent if a different point of view is taken than that related to surface or air temperatures. For example, by placing non-frost-susceptible materials over frost-susceptible soils, the initial portion of the degree-day curve is neutralized. Alternately, this may be thought of as reducing the effective freezing index relative to the frost-susceptible soil.

Another approach that has been used in delaying the penetration of freezing into a frost-susceptible subgrade is that of the Corps of Engineers (5). This approach takes into consideration the quantity of heat which is stored in the water of a moist subbase. The higher the water content, the greater is the quantity of heat which must be removed from a given thickness of subbase before it will freeze, thus retarding the advance of freezing temperatures. Of course, the subbase must not exhibit detrimental frost action at this water content; therefore the moisture must be closely controlled.

Frost-Susceptible Soil Texture

There is essential agreement on the more general concepts of what constitutes a frost-susceptible soil. For example, silty soils are generally considered to be the poorest soils, whereas very clean sands and gravel give no trouble. Also, the so-called "dirty" sands and gravels often are troublesome. However, there are variations in the definition of frost-susceptible soils when a closer definition is attempted. Various criteria are used, such as the percent finer than the number 200 sieve, the percent finer than 0.02 millimeters, and the percent of clay sizes. For example, in some of the western states a non-frost-susceptible soil may have as much as 25 percent finer than the number 200 sieve (4). This high percentage of fines would be considered intolerable in the eastern and midwest areas. This difference points out the fact that frost susceptibility as it relates to actual damage to the pavement must be considered from more standpoints than the texture of the soil alone. In the case of the western states, it appears that the relative lack of moisture is an important consideration in determining the damage to be expected as a result of frost action.

Although there is difference of opinion as to what constitutes a frost-susceptible soil on the basis of texture, texture is used as the primary criterion for frost susceptibility in practically all cases. That is, the extent of frost damage anticipated is based on some measure of the texture, including the use of the Atterberg limits. A notable exception is that of British Columbia, where design is based primarily on the results of observed deflections under loading on selected observation sections of roadways in actual service (6). Although this procedure has not found general acceptance elsewhere, it is nevertheless an interesting approach and in many respects represents a more direct approach than that based on soil texture alone.

Another question that is raised and is discussed in several of the papers is whether frost susceptibility should be determined on the basis of frost heaving or on the basis of actual pavement damage. It seems to be implicit in many of the discussions that frost susceptibility is based on the relative amount of frost heaving. However, it has also been observed that some states are modifying this approach to account for the actual damage to the pavement, regardless of the amount of heaving.

Although not a direct contribution to this symposium, the studies of Csathy and Townsend (11) in Canada are of considerable interest in any discussion on frost susceptibility. These investigators concerned themselves with the distribution of pore sizes as a basis for the frost-susceptibility criteria. Because frost effects are concerned with the relationship of the moisture in the soil pores to the frost phenomena, this approach has much to commend itself.

Moisture in Soil

It is difficult to discuss soil moisture independently of soil texture because of the close relationship of moisture to texture. Generally, those soils which naturally hold relatively high contents of water will exhibit more severe frost action. For a given soil, however, differences in the environment as they affect the natural water content are significant. The previously mentioned criterion of 25 percent finer than the number 200 sieve, as used in some of the western states, is a clear example of the importance of over-all environmental effects.

Many approaches are used in an attempt to reduce the moisture content, and therefore to reduce the extent of detrimental frost action. One approach is to place the grade line as high as practical thereby increasing the distance from the pavement surface to the free water table. Another approach is to attempt to lower the free water table with subdrainage structures incorporated into the highway. The success of this design depends on the availability of a suitable outlet for the drainage facilities. In many cases, this requirement imposes a severe limitation on the effectiveness of this approach.

Another aspect of drainage concerns the dissipation of the excess moisture which is released by the subgrade soil when it thaws with the coming of spring. Because most of the thawing is from the surface downward rather than from the bottom of the frost layer upward, the problem is how to drain effectively this excess water. A number of design agencies have given a great deal of thought to the design of the base courses and subbases to serve the function of a filter. The fundamental approach is to have the base and subbase sufficiently permeable so as to permit drainage of the excess moisture from the subgrade as rapidly as it is formed by the thawing. Some of the states (4) mention the use of the generally accepted criteria for the design of filters. This approach is covered extensively in the paper from the Corps of Engineers (5).

The general concept of using bases and subbases as filters, although good, does have some limitations. Most of the discussions suggest that the moisture should first move upward from the subgrade into the base and/or subbase, from which it is then distributed laterally to the edges of the highway structures. If the base and subbase are sufficiently permeable and are sufficiently thick, this should result in an effective design. It is important that the designer recognize the possibility that the base and subbase may be weakened somewhat by the development of pore pressures. However, if the base and subbase are made sufficiently thick, this should not be a serious problem. What is of concern is whether or not the filter has a free outlet at the edges. Practically, this may be a severe limitation of the effectiveness of the filter principle because the outlet to the filter may be blocked by snow remaining in the ditches or by frozen soil remaining under the shoulders. Probably more attention needs to be given to this feature to insure effectiveness of transverse drainage.

Another factor that is frequently overlooked is that in many cases the longitudinal slope may exceed the transverse slope. In this case, the drainage water will be more likely to flow in the longitudinal direction rather than in the transverse direction, with the result that water may be accumulated to an intolerable degree at the low points in the highway profile in spite of satisfactory lateral drainage. If the lateral drainage were blocked, this problem would be accentuated. As a result of the longitudinal drainage, serious damage may occur at the low points or sags in the highway profile even though the design otherwise might be quite adequate.

From the foregoing, it can be seen that the application of the filter concept probably needs a more elaborate analysis than is obtained by considering only the typical cross-section. It may be necessary to increase the thickness of the base and subbase in the vicinity of the sag in the profile in order to develop sufficient hydraulic capacity for longitudinal flow through the filter.

ARE CURRENT DESIGNS UTILIZING AVAILABLE BASIC KNOWLEDGE?

As previously stated, one of the questions that this symposium should attempt to answer is: "What basic knowledge of frost action do we have that we are not using in design practices?" In general, it appears that in fact, design concepts are making use of nearly all of the basic information which is presently available and which is sufficiently developed to be used in design. This is especially true if one considers the many variables in the problem and the wide varieties of environmental conditions and availability of materials in various parts of this country, in Canada, and in Europe. An attempt will be made to answer this question somewhat more explicitly by considering in turn the mechanism of frost action and the three factors: moisture, temperature, and soils.

Mechanism of Soil Freezing

In recent years, and in particular in Part I of this symposium (14), a definite point was made regarding the importance of the unsaturated permeability of soils and the relationship between the moisture tension (or suction) and the movement of moisture. The design methods currently being used do not directly utilize these concepts. However, it must be recognized that these concepts are utilized indirectly in current design practices. Examples are the reduction of the unsaturated permeability by the use of selected coarse materials and the practice of elevating the grade line above the surrounding terrain to increase the moisture suction and thereby decrease the unsaturated permeability. Also in the same category is the lowering of the water table by subsurface drainage structures.

Another factor considered in the mechanism of soil freezing and the formation of ice lenses is the effect of the overburden pressure in reducing the magnitude of frost heaving. Again this is not utilized directly, but the effect is introduced automatically by the practice of placing a considerable thickness of non-frost-susceptible materials over the frost-susceptible subgrade.

Moisture

In the control of moisture it is generally accepted that the obvious approach is to provide better drainage. This is stated in the preceeding papers in Part II of the symposium, as well as in the part dealing with basic considerations. It is recognized that there are limitations on the effectiveness of drainage both from the standpoint of availability of suitable outfalls and the well-documented observation that many of the soils which are troublesome from a frost point of view do not drain by gravity to any great extent. It would appear that current design practices are making good use of all that is known about drainage.

Other methods for the control of moisture have been suggested by research, but these are not generally considered practical at least for conditions in the United States. One approach is to provide cut-off layers using either porous material such as sand to interrupt the capillary rise of the soil, or a layer of clay which would effectively serve as a moisture barrier (7). This has been tried in practice and is generally not considered feasible by those agencies within the United States, although this approach is used in Europe to some extent. Another variation of this scheme is to provide a membrane of film plastic to cut off the flow of moisture to the freezing zone. Not enough experience has been gained to know how this method will work out. Also, there are a number of practical problems associated with such an approach, such as a loss of effectiveness of the barrier if it becomes punctured.

Still another category of moisture control concerns the use of chemical additives to modify the wetting of the soil particles and water. This appears to be good on the basis of laboratory studies, but questions remain as to the relative permanency of the treatment and there is a serious question on the relative economy. One statement has been made to the effect that if dirty gravels are available in substantial supply it will probably be less expensive to wash the fines out of the gravel rather than to attempt treatment by chemical means (17). As time goes on and sources of suitable non-frost-susceptible materials are depleted and as water supplies for washing become increasingly more limited, this situation may change.

Temperature

Research has shown that the penetration of freezing temperatures into the soil can be predicted reasonably well on the basis of the freezing index which is developed from the degree-day curve. There remain, however, some questions as to the relationship between the freezing index and the soil characteristics which should be resolved (16). In spite of this, a fairly reliable estimate of the penetration depth can be made by this approach.

This approach is not universally adopted by the states and provinces. Instead, this approach is used mostly by the Corps of Engineers and other agencies whose operations extend over entire continents. It is possible that if more adequate temperature data were available for the determination of the freezing index in a greater variety of locations, this approach might gain more acceptance, especially as the supply of non-frost-susceptible materials becomes progressively more critical.

Definition of Frost-Susceptible Soils

Research has defined fairly well just what is meant by frost-susceptible soil when the frost susceptibility is judged on the basis of frost heaving. Most notable in this area is the work by the Corps of Engineers on the basis of laboratory freezing studies (17, 5). This information is generally available and should prove quite helpful. The extent to which this information is reflected in the design of highways by other agencies is not very clear, but it seems that it has influenced the decisions of several highway departments in establishing the frost-susceptibility criteria which they use even though they have not adopted the system in its entirety. It should be pointed out that the laboratory test utilized by the Corps of Engineers for this determination relates to the worst possible conditions that would likely occur in a highway.

One observation has been made which probably bears repeating, and that is that highway engineers could make more use of pedological soils data for the design of highways.

At present, this information is actively used by only a few states. Although other organizations do recognize the importance of it, the direct use of this source of information is not specifically in evidence. The importance of pedological data is that they reflect to a considerable extent the natural drainage condition of the soil and, therefore, probably would give valuable information regarding the relative frost hazard.

Chemical Additives

Chemical additives are treated separately here although they relate both to the control of moisture and to the modification of soils. Some additives are considered to have the effect of altering the particle size composition of the soil. These operate by lumping the finer particles together into collections of larger particles so that the net effect is an increase in the average size of the particles. Other chemicals tend to change the relationship between the surface of the soil particle and the included water in the soil. By reducing the wettability of the soil particles the transmission of moisture is reduced and as a consequence heaving and other detrimental effects of frost action are reduced.

It is presently considered that chemicals are too expensive for treating the soil. Also, certain chemicals are considered to be unpredictable, are non-uniform in their results and give only questionable permanence. However, there are certain chemicals which behave quite well as additives. Although it may be true that widespread use of chemical additives is not as yet practical or economical, it seems likely that in the near furture they may become increasingly important. This prediction is based on the known fact that the supply of suitably graded coarse material is running out and that it will be necessary to use materials which are now considered marginal. It is also to be expected that the chemical industry will continue to work on the problem and probably will, in the course of time, produce some form of chemical treatment which is reasonable in cost, easy to handle under construction operations, and is sufficiently permanent to find widespread use.

SUGGESTIONS FOR FURTHER RESEARCH

If it is true that designers are using all available knowledge of frost action, it follows that further research on frost action is needed if designs are to be improved. Some of the specific problems which warrant further study as suggested by this symposium are presented in the following paragraphs.

Use of Freezing Index Data

In general, highway departments have not used freezing index data very extensively. This may be partly due to the fact that there can be significant variations in the freezing index over a given length of highway. In the mountain states, freezing conditions change rapidly within a short horizontal distance as the altitude increases. Even in some areas of the midwest it can be visualized that the freezing index would change within a few miles for any given road in certain localities.

It is believed that freezing index data could be extended to cover these variations by considering differences in the degree of exposure to cooling, which would depend on topographic position, the relationship to the prevailing winds, and solar exposure. By taking these factors into account the freezing index obtained from a study of temperatures at regular weather stations could be corrected to develop the freezing index appropriate to any given remote point.

Another aspect is the relationship between the freezing index and the penetration of frost into different kinds of soils. The data reported in the literature indicate a considerable variation of penetration depth for a given freezing index (16). Apparently this variation is due to differences in soil and moisture conditions. It would be helpful if these relationships could be defined to the point that universal predictions of frost penetration could be made from the freezing index. If this concept could also be extended to frost heaving, it would be that much more valuable.

Increased Use of Pedological Soils Data

Some mention has been made in the discussion of use of pedological soils information. The application of this information has been largely based on a considerable length of experience with the system and with the soils and the observed behavior of highways. It is believed that if the logical relationship between this classification system and those factors governing the performance of highways could be established, the pedological system could be greatly extended. The pedological system is attractive because of the detailed mapping that has already been accomplished and the wealth of information that is already available.

Pedological soils data indicated the drainage characteristics of soils, their texture, and the relative natural water content for the undisturbed soil profile. When the soils are disturbed the situation is more complicated, of course, but nevertheless some useful correlations should be possible. Another problem that should be worked out is the effect of the boundary zone between adjacent series as it affects frost action. Adjacent series are often significantly different in their drainage characteristics so that they indicate a location of potential difficulty. Even within a single series, there may be significant differences at the boundaries between adjacent slope phases of the same soil series. Finally, it should be possible to use pedological soils data to determine the relative susceptibility to improvement of the soils by drainage.

Variations of Damage with Various Seasons

The paper by Kübler (9) reported some interesting observations in considerable detail. It emphasized or confirmed the general observations of highway engineers that the damage due to the frost is often more severe following a moderate winter than a winter of intense cold. It would appear that this subject could be explored further and eventually developed into such a form that it could be useful in predicting damage to pavements, or alternately, it could be used to estimate the load reduction necessary for that class of roads which for economic reasons could not be designed to be completely free of detrimental frost effects. It would be necessary to consider, in addition to the variation between seasons, different soil conditions and various moisture environments.

Moisture Tension Studies

Some research has already been conducted on the moisture tension developed during freezing and its relationship to frost action phenomena. It is clear that further work should be done in this area, again for the purpose of extending the knowledge to the stage where practical design decisions could be made. The importance of the relationship of moisture tension to the unsaturated permeability has been clearly demonstrated. Further research, including the correlation of unsaturated permeability to soil texture, is needed to make this concept useful.

Migration of Coarse Particles

The movement of coarse particles upward to the surface of the soil or through a pavement has been observed for some time. It is only recently, however, that any quantitative research has been accomplished in this area (13). It is believed that further studies should be made as a basis for establishing proper specifications governing large particles in a frost-susceptible subgrade. Furthermore, it is possible that such studies may lead to improvement in specifications and materials for base courses and subbase courses. The significance of particle movement is that it can lead to localized rough spots in the pavement because of the movement of large particles upward from the subgrade. It may lead to decreased density and to a change in the gradation of the material in the base and subbase.

Loss of Stability During Spring Thaw

The problem of loss of stability during and following the spring thaw is well recognized and a number of studies have been made to evaluate the significance of it. However, comparatively little has been done to study the fundamental nature of this problem. This phenomenon should be given intensive study to aid in designing pavements that will resist this loss of stability in frost-susceptible subgrades.

Design of the Base as a Filter

The reports in the present symposium have outlined the use of the filter approach to controlling loss of stability. One of the limitations that has been pointed out, however, is the possibility of the filter being blocked by retarded thawing of the shoulder zone of the highway. Effective methods of control need to be developed. Another problem is that of insuring that the filters drain to the side even when ice blocking is not a problem. It is believed that the filter could be made thicker near the shoulders in order to facilitate the drainage of the filter.

Thermal Insulation Layers

The use of the insulation layers has been proposed and has been used with some success in Europe, although this approach has not received general acceptance within the United States. One of the difficulties seems to be the provision of an insulation material that would have sufficient strength to withstand vehicle wheel loads. If insulation layers are to have an advantage over other materials, they must prevent freezing of the frost-susceptible subgrade. In areas where there is little or no granular material available, insulating materials may be more attractive. In either circumstance, the problem is to find a material that is a good thermal insulator, that is not subject to damage by moisture, that has adequate structural strength, and which is not excessive in cost.

Use of Nuclear By-Products

One of the problems of the nuclear age is the disposal of radioactive waste which involves, among other things, considerable quantities of heat. It has been suggested that it might be possible to utilize the radioactive materials to prevent frost-susceptible subgrades from freezing, thus controlling frost action. The feasibility of this will need to be established from the standpoint of highway economics, as well as from the standpoint of providing adequate protection against radiation damage or injury.

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