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ROADWAY DESIGN IN SEASONAL FROST AREAS

T. C. Johnson, et al

Cold Regions Research and Engineering Laboratories
Hanover, New Hampshire

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T.C. Johnson, R.L. Berg, K.L. Carey and C.W. Kaplar

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NATIONAL ACADEMY OF SCIENCES
TRANSPORTATION RESEARCH BOARD
AND
CORPS OF ENGINEERS, U.S. ARMY
COLD REGIONS RESEARCH AND ENGINEERING LABORATORY
HANOVER, NEW HAMPSHIRE

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PREFACE

This report was prepared by Thaddeus C. Johnson, Civil Engineer, Dr. Richard L. Berg and Kevin L. Carey, Research Civil Engineers, and Chester W. Kaplar, former Research Civil Engineer, of the Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory. The report is published under DA Project 4A062112A894, *Engineering in Cold Environments,* Task 20, *Cold Regions Systems Analysis, Systems Engineering, Production and Assessment Studies for Military Installations,* Work Unit 002, *Analysis of the State of the Art in Cold Regions Technology;* and DA Project 4K078012AAM1, *Engineering Criteria for Design and Construction, Cold Regions,* Work Unit 025, *Pavement Design Criteria for Seasonal Frost.*

The report was reviewed, on the part of USA CRREL, by K.A. Linell, formerly Chief, Experimental Engineering Division; E.F. Lobacz, Chief, Construction Engineering Research Branch; and W.F. Quinn, Chief, Northern Engineering Research Branch. Principal review on the part of NAS-TRB was provided by an advisory panel, whose members were W.J. Dunphy, Research Engineer, Maine State Highway Commission; W.M. Haas, Professor of Civil Engineering, Michigan Technological University; G.W. McAlpin, formerly Chief Engineer, New York State Department of Transportation; T.F. McMahon, Leader, Pavement Systems Group, Structures and Applied Mechanics Division, Federal Highway Administration; and E. Penner, Head, Geotechnical Section, Division of Building Research, National Research Council of Canada. The authors wish to thank the reviewers for their constructive criticism.

In addition, the authors are especially grateful to T.L. Copas, Special Projects Engineer of NAS-TRB, and his colleagues of the TRB staff for constant support and assistance during the investigation and preparation of the report.

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This report was prepared as a joint project of USA CRREL and the National Academy of Sciences-Transportation Research Board. It was published by NAS-TRB as Synthesis of Highway Practice No. 26, the press run being extended to produce this CRREL Technical Report.

ROADWAY DESIGN IN SEASONAL FROST AREAS

RESEARCH SPONSORED JOINTLY BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS AND THE U.S. ARMY COLD REGIONS RESEARCH AND ENGINEERING LABORATORY IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
- PAVEMENT DESIGN
- CONSTRUCTION
- FOUNDATIONS (SOILS)
- SOIL SCIENCE

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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Notice

The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council, acting in behalf of the National Academy of Sciences. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

The members of the advisory committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the advisory committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the National Academy of Sciences, or the program sponsors.

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PREFACE

There exists a vast storehouse of information relating to nearly every subject of concern to highway administrators and engineers. Much of it resulted from research and much from successful application of the engineering ideas of men faced with problems in their day-to-day work. Because there has been a lack of systematic means for bringing such useful information together and making it available to the entire highway fraternity, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize the useful knowledge from all possible sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series attempts to report on the various practices without in fact making specific recommendations as would be found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available concerning those measures found to be the most successful in resolving specific problems. The extent to which they are utilized in this fashion will quite logically be tempered by the breadth of the user's knowledge in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of special interest and usefulness to design, materials, soils, and construction engineers concerned with the problems of identifying potential locations of pavement-damaging seasonal frost action, and of selecting and applying control measures to prevent detrimental effects on pavement performance. Attention is given to the structural design of pavements, and to the selection of earthworks and drainage facilities to provide acceptable pavement serviceability under seasonal frost conditions.

Administrators, engineers, and researchers are faced continually with many highway problems on which much information already exists either in documented form or in terms of undocumented experience and practice. Unfortunately, this information often is fragmented, scattered, and unevaluated. As a consequence, full information on what has been learned about a problem frequently is not assembled in seeking a solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem. In an effort to resolve this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of synthesizing and reporting on common highway problems—a synthesis being identified as a composition or combination of separate parts or elements so as to form a whole greater than the sum of the separate parts. Reports from this endeavor constitute an NCHRP report series that collects and assembles the various forms of information into single concise documents pertaining to specific highway problems or sets of closely related problems.

Although syntheses normally are accomplished entirely with support from highway sources, by agreement in this instance additional substantial support was provided by the U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL).

Highway engineers in seasonal frost areas have been aware for decades of the damaging effects that frost in the underlying support materials can have on pavement performance. Pavement heaving and cracking resulting from frost action, and thaw-induced breakups, are familiar problems. In the highway environment, the factors of climate, soil, water, pavement structure, and traffic are known to interact in freezing and thawing situations to the detriment of pavements. The exact nature of the physical processes that take place, the sensitivities of the several contributing factors, and the magnitude of the responses to freezing action are not well understood. Nevertheless, usable criteria exist and are reported in this synthesis for identifying frost susceptibility, and for selecting measures that will avoid harmful pavement reaction. These measures rarely involve total removal of troublesome conditions because of economic considerations.

This report of the Transportation Research Board describes and assesses the merit of current practice in roadway design in seasonal frost areas. Information is presented on the mechanisms of frost heaving and thaw weakening, the factors that contribute to frost problems, criteria for estimating frost susceptibility, surveying practices for locating areas of detrimental frost action, subgrade treatment and drainage to minimize the effects of frost action, and the structural design of pavements to accommodate the influences of frost action. Research needs in the area are identified.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information from many highway departments and agencies responsible for highway planning, design, construction, operations, and maintenance. A topic advisory panel of experts in the subject area was established to guide the researchers in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. Meanwhile, the search for better methods is a continuing process that should go on undiminished.

CONTENTS

ACKNOWLEDGMENTS

This synthesis was a joint effort of the Transportation Research Board (TRB) and the U. S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL). The synthesis effort was supervised by Paul E. Irick, Assistant Director for Special Projects, of the TRB. The Principal Investigators responsible for conduct of the synthesis were Thomas L. Copas and Herbert A. Pennock, Special Projects Engineers.

Special appreciation is expressed to the CRREL representatives who were responsible for the collection of data and preparation of the report: Thaddeus C. Johnson, Civil Engineer; Richard L. Berg, Civil Engineer; Kevin L. Carey, Civil Engineer; and Chester W. Kaplar, Civil Engineer.

Valuable assistance in the preparation of this synthesis was provided by the Topic Advisory Panel, consisting of Wilbur J. Dunphy, Jr., Research Engineer, Materials and Research Division, Maine State Highway Commission; Wilbur M. Haas, Professor of Civil Engineering, College of Engineering, Michigan Technological University; George W. McAlpin, Chief Engineer, New York State Department of Transportation; Thurmul F. McMahon, Leader, Pavement Systems Group, Structures and Applied Mechanics Division, Office of Research, Federal Highway Administration; and Edward Penner, Assistant Head, Geotechnical Section, Division of Building Research, National Research Council of Canada.

J. W. Guinnee, Engineer of Soils, Geology, and Foundations, and L. F. Spaine, Engineer of Design, both of the Transportation Research Board, assisted the Special Projects staff and the Topic Advisory Panel.

Information on current practice and ongoing research was provided by many Canadian and U. S. highway agencies. Their cooperation and assistance were most helpful.

ROADWAY DESIGN IN SEASONAL FROST AREAS

SUMMARY

Design, construction, and maintenance of roads in areas of seasonal frost entail special requirements in order to provide all-season service and long-term performance.

One effect of frost action on pavements is heaving caused by crystallization of ice within the large voids of soils containing fine particles. Three conditions must be present: (1) frost-susceptible soils, (2) freezing temperatures in the soil, and (3) a source of water. If these conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur, causing surface irregularities, roughness, and possibly cracking.

Another effect of frost action is thaw weakening. Bearing capacity may be substantially reduced during midwinter thawing periods and subsequent frost heaving is usually more severe. In the more southerly areas of the frost zone, several cycles of freeze and thaw may occur during a winter season and cause more damage than one longer period of freezing in a more northerly area. Spring thaws produce a loss of bearing capacity to well below summer and fall values, followed by a gradual recovery over a period of weeks or months.

Other effects of frost include cracking caused by thermal contraction, migration of large stones upward to the surface, and formation of ice in drainageways.

Factors contributing to frost problems include climate, frost-susceptible soils, and water. Air temperature, solar radiation received at the road surface, and wind are the climatic factors affecting depth of frost, number of freeze-thaw cycles, and duration of freeze and thaw periods. Many investigators seeking to calculate the depth of frost penetration use a freezing index, which expresses the cumulative effect of intensity and duration of subfreezing temperatures.

Frost-susceptible soils are only those that contain fine particles. Several methods of identifying these soils have been used since the 1930's. The most widely known rule-of-thumb for identification of a frost-susceptible soil is Casagrande's—more than 3 percent of grains smaller than 0.02 mm in nonuniform soils. Data from other work indicate that frost susceptibility is generally related to the amount of fine material, but that other factors have a significant influence. Several laboratory test methods have been devised, but they may still be too complicated and time consuming for wide acceptance as a design tool.

Water sources are either surface (infiltration through cracks and joints, or seepage through permeable pavements or adjacent unpaved areas) or subsurface (high groundwater table, capillary action, or lateral movement from external sources, such as through pervious strata).

Design of pavements in seasonal frost areas begins with a soils and materials survey. Methods used vary among agencies but generally include use of existing data, such as pedological and geological surveys; indirect assessment techniques, such as airphoto interpretation and remote sensing; and direct field methods, such as geophysical surveys and direct sampling.

The primary approach for minimizing the detrimental effects of frost action in

the subgrade beneath flexible pavements is either (1) removal of frost-susceptible soils to below the level of frost penetration and replacement with nonfrost-susceptible soils, or (2) accommodation of the frost action (heave and thaw-weakening) during the structural design process by eliminating discontinuities leading to differential heave, and by strengthening the pavement structure. The first approach is more desirable, but usually is limited to relatively short sections that are identified at the time of final grade check during construction operations. This approach is particularly effective in preventing differential frost heave.

Unlike subgrade materials, base and subbase course materials are usually processed to meet predetermined requirements and may be treated with a modifying or binding agent. In untreated courses, frost heave and thaw-weakening are generally controlled by restrictions of grain size and plasticity.

With regard to rigid pavements, differential frost heave and loss of supporting capacity during thaw are the most significant detrimental effects associated directly with frost action. As with flexible pavements, corrective measures include removal and replacement of frost-susceptible soils, use of a stronger pavement section that compensates for subgrade weakening during thaw, scarifying and blending the top 1 to 2 ft (0.3 to 0.6 m) of subgrade to interrupt undesirable stratification, and transition wedges at changes from cut to fill and over culverts.

Adequate surface and subsurface drainage is more essential in areas of seasonal frost than elsewhere, because water is responsible for the majority of the ill effects of low temperatures. Underdrains or interceptor drains are widely used to lower the water table, or the grade line may be raised to provide greater separation between water table and subgrade.

Other methods to reduce the effects of frost action include thermal barriers, moisture barriers, encapsulation, and soil stabilization. Thermal barriers have been used to reduce the effect of subfreezing air temperatures on frost-susceptible soils. Polystyrene boards have performed adequately, but a few agencies are concerned about the occasional occurrence of differential icing conditions.

Moisture barriers, such as sand layers or plastic film, generally have not been used in the United States, but some experimental tests have been done with soils encapsulated in asphalt or plastic membranes and these show favorable results in reducing frost heave.

Stabilization is widely used as a method of processing subgrade and base course materials to improve their performance under traffic and climatic conditions. Effective stabilization of a frost-susceptible material usually involves (1) eliminating the effects of soil fines by their removal or by immobilization, such as cementacious binding; or (2) reducing the quantity of water available at the freezing plane by blocking migration passages. The commonly used stabilizing additives include portland cement, bitumen, lime, and lime-fly ash.

Conditions such as variable subgrade soils and shallow bedrock are difficult to foresee during the design process, but often can be recognized and corrected during construction. Many potential frost problems can be reduced considerably if, based on experience and terrain investigations, the conditions and specific locations along the route where such problems are likely to occur are identified for correction during construction.

Selective grading is a construction technique for reducing frost action by placing the more highly frost-susceptible soils in the lower portions of embankments and the less frost-susceptible materials in the upper portion of the subgrade.

Cut sections frequently have been reported as sources of trouble because they alter the natural drainage and provide ample sources of water from adjoining higher ground. In some cases the problem can be eased by subsurface drainage or by undercutting and removal of the unstable materials.

One of the first requirements during construction should be inspection of the subgrade to verify the validity of design assumptions and to locate silt pockets, frost-susceptible materials, seepage, ground or capillary water, logs, stumps, boulders, and other nonuniform subgrade conditions.

Achievement of the highest possible density of subgrade and base courses is essential to good pavement performance. Construction operations frequently will expose potential frost-troublesome conditions previously unknown. Personnel assigned to construction control should be aware of the situations that require special treatment and field correction.

Spring load restrictions may be considered preventive maintenance. Many agencies in seasonal frost areas indicated that load restrictions are applied to older secondary roads during the thaw-weakening period to reduce pavement damage.

The two most commonly used procedures for repair of frost-damaged roads are (1) removal of frost-susceptible soil and replacement with nonfrost-susceptible material, and (2) installation of a subsurface drainage network.

There is a need for applied research to answer questions such as: What are the soil factors that determine the severity of ice segregation, frost heave, and thaw weakening? and: How can moisture migrations engendered by frost action be defined and forecast? Research is also needed to (1) develop and verify innovative procedures such as use of thermal barriers, encapsulation, and stabilization to more economically deal with frost action and reduce use of scarce high-quality materials, and (2) further advance mechanistic approaches to pavement design, including techniques for material characterization, assessment of environmental influences, and prediction of cumulative damage.

CHAPTER ONE

INTRODUCTION

Most roads are kept open to traffic in all seasons and in all weather, to serve the transportation needs of their users economically and conveniently. In the seasonal frost areas of the earth, defined (*1*) as areas in which significant freezing occurs during the winter season but without development of permafrost, the need for pavements serviceable throughout the year imposes special requirements of roadway design and maintenance. This synthesis addresses the problem that engages the attention of roadway designers as they select for a proposed road in a seasonal frost area a particular layered system, with appropriate earthworks and drainage facilities, that will provide optimum all-season serviceability and long-term performance at the lowest possible cost. It is a complex problem to which ideal economic solutions have not been found.

THE NATURE OF THE PROBLEM

Pavement Serviceability and Performance

The serviceability of a pavement refers to the level of service it provides to road users at any particular time. Deterioration caused by traffic and the passage of time causes the serviceability of a pavement to decrease continuously, although not necessarily at a constant rate. When the serviceability reaches some lower limiting level of user acceptance, the pavement is improved or reconstructed. Performance of a pavement may be measured in part as the rate of such deterioration, and the designer of a pavement seeks to optimize the performance within the constraints of the adopted strategy for investments in the road system.

A widely used measure of pavement serviceability is the present serviceability index (PSI) established by the AASHO Road Test (2). The principal parameter in the PSI is the longitudinal slope variance in the two wheelpaths; lesser terms are measures of cracking, patching, and either rutting, for flexible pavements, or spalling, for rigid pavements. The PSI accordingly is an expression of the degree to which the surface is smooth, intact, and undeformed. Frost acts to the detriment of all these desirable characteristics of the pavement surface.

Seasonal Variation in Serviceability

Any decrease in the PSI of a pavement is a reflection of increasing distress in the pavement, which may occur under a variety of modes (Table 1). Traffic-associated damage to the pavement is cumulative, and the rate of its accumulation can vary with changes in distribution and intensity of traffic, environmental loads, and material properties. Both environmental effects and material properties vary with seasonal temperature and moisture changes; in frost areas the dominant effect in moisture-sensitive materials occurs at temperatures near the freezing point of water, whereas the properties of materials containing a bituminous binder are highly dependent on temperature throughout the range in temperatures to which they are subjected.

Under most of the distress modes that are not associated with traffic, damage to pavements in frost areas occurs under the direct causative influence of low temperatures. Tensile stresses that build up when asphaltic pavements contract under falling temperatures, while in an embrittled condition, cause thermal cracks, or low-temperature contraction cracks. Moisture changes can occur independently of falling or rising temperatures, but to a large extent moisture migrations and detrimental changes in moisture content take place under the influence of both seasonal and short-term temperature gradients and cycles of freezing and thawing. Shrinkage of supporting layers, causing reflection cracking, is induced by falling temperatures as well as desiccation. Differential heave and subsequent settlement, caused by frost action, develop in direct response to temperatures falling to subfreezing levels, although certain other requisite conditions of soil and moisture also prevail.

The direct dependence of the rate of pavement deterioration on seasons is evidenced by Painter's (*4*) analysis of the AASHO Road Test data for asphaltic pavements, which showed (Fig. 1) that the deterioration rate was constant throughout the year, except during the spring thaw period, when it increased to much higher rates. The permanent loss in serviceability engendered during a brief period in the spring may equal or exceed the loss during the rest of the year.

Designing for Optimum Performance

Most pavement design equations in use in North America express required thickness and component layers of a pavement structure in relation to a certain level of performance. The relationship may be chosen solely from years of experience with roads in service and test roads (empirical approach) or from research results and analysis of the action of forces on the layered system (mechanistic approach). The equation parameters usually include traffic loads (magnitude and number), properties of construction materials, subgrade support values, and environmental coefficients.

TABLE 1

MODES OF DISTRESS IN PAVEMENTS[a]

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

[a] After Monismith (*3*).

Inasmuch as low temperatures and frost strongly affect all of these parameters except traffic loads, assessments of temperature regime and frost effects are an essential part of development of suitable designs for pavements in frost areas. Performance requirements the designs are required to satisfy involve control within tolerable limits of the rates of deterioration of the pavement under all the applicable modes of distress listed in Table 1. Designing for optimum performance requires the further steps of comparison of alternative designs having acceptable projected deterioration rates and selection of the pavement section that is projected to cost the least.

SCOPE OF SYNTHESIS

Any attempt to synthesize the practice of designing roadways in seasonal frost areas is an ambitious venture, because the special design processes directed to control or prevention of those modes of distress that are dependent on frost or low temperature are actually subsystems of the more generalized design system embracing all distress modes, which unfortunately has not yet been the subject of a Synthesis of Highway Practice. Accordingly, it is necessary to mention concepts and practices not unique to seasonal frost areas, although the emphasis is on those practices more directly related to the low-temperature environment.

The practices of agencies engaged in road design or establishment of design criteria for seasonal frost areas were surveyed by means of (a) a literature review, (b) questionnaires mailed to North American design agencies, and (c) visits to ten of the agencies for discussion of their practices. The literature review included not only articles on design practices but also articles on research results and concepts. The questionnaire sought information principally

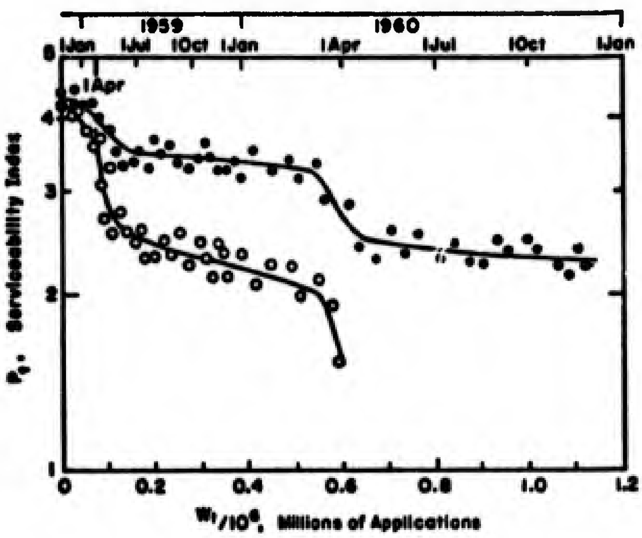


Figure 1. Typical performance data from two test sections, AASHO Road Test (4).

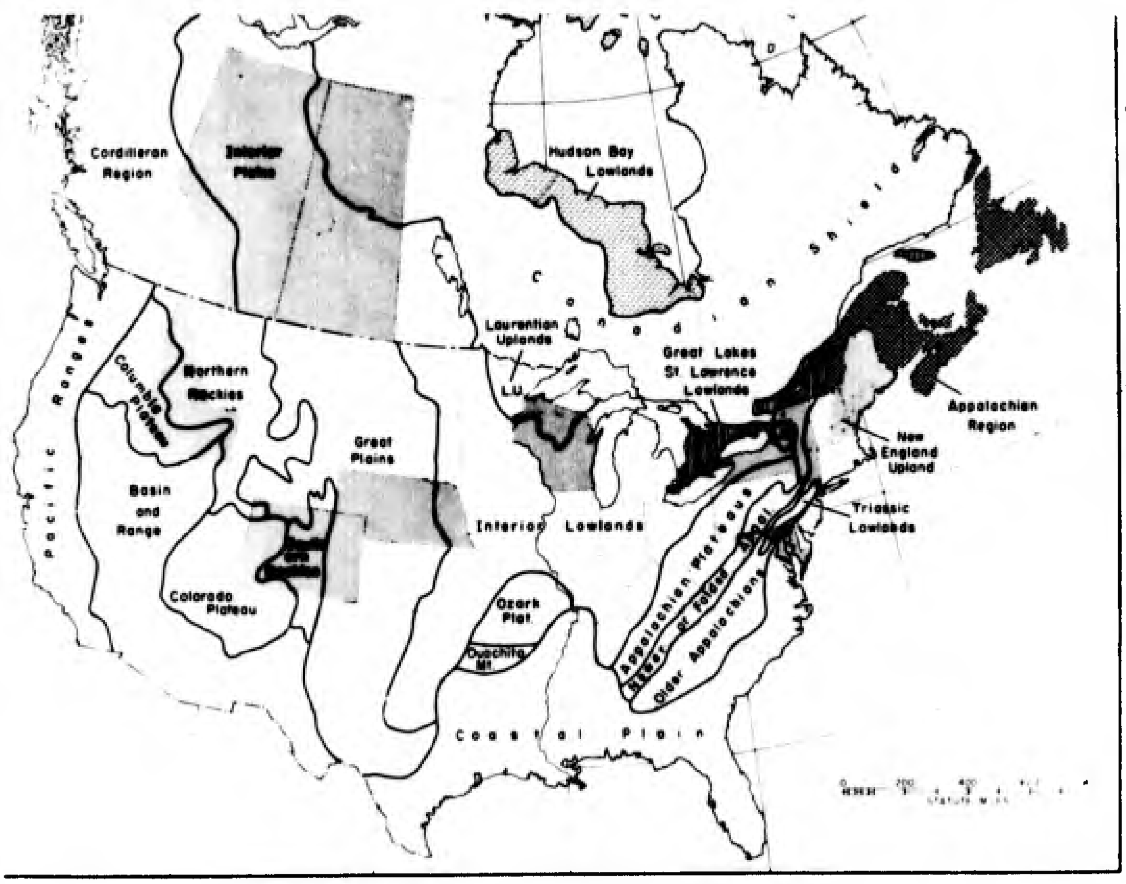


Figure 2. States and provinces visited.

on design practices, but also queried practices in maintenance and planning and topics of current and needed research. Questionnaires were sent to the departments of highways (or transportation) of all U.S. states and all Canadian provinces and to agencies of both federal governments, certain turnpike authorities, and producers' associations. After an initial perusal of the 62 replies the highway departments of eight U.S. states and two Canadian provinces were selected for more detailed study by means of one-day visits and discussions. The selection was made to include a sampling of North American conditions and practices, including geographic distribution and physiography (Fig. 2), freezing index, method of characterization of subgrade support, and flexible pavement design method. General summaries of the information obtained from the completed questionnaires are tabulated in Appendix A. The more extensive and detailed information obtained from the states and provinces that were visited is summarized in Appendix B. The information in Appendices A and B, together with research results and design practices gleaned from the extensive literature, is assessed and synthesized in this report.

CHAPTER TWO

FACTORS FUNDAMENTAL TO FROST ACTION

DESCRIPTION OF FROST EFFECTS

Heaving

Frost heaving of soil within or beneath a pavement is caused by crystallization of ice within the larger soil voids and usually a subsequent extension to form continuous ice lenses, layers, veins, or other ice masses. The growth of such distinct bodies of ice is termed ice segregation. A lens grows in thickness in the direction of heat transfer until water supply is depleted, as by formation of a new lens at a lower level, or until freezing conditions at the freezing interface will no longer support further crystallization. Numerous investigations have shown that ice segregation occurs only in soils containing fine particles. Such soils are said to be frost susceptible; clean sands and gravels are non-frost-susceptible soils. The degree of frost susceptibility is principally a function of the percentage of fine particles and, to a lesser degree, of particle shape, distribution of grain sizes, and mineral composition. Figure 3 depicts how ice lenses may be initiated and developed within soils.

Conditions Necessary for Ice Segregation

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

1. *Soil.* The soil must be frost susceptible.
2. *Temperature.* Freezing temperatures must penetrate the soil. In general, the thickness of a particular layer or lens of ice is inversely proportional to the rate of penetration of freezing temperature into the soil.
3. *Water.* A source of water must be available from an underlying groundwater table, infiltration or gravitational flow, an aquifer, or the water held within the voids of fine-grained soil.

Uniform Heaving

Uniform heaving results when soil and moisture conditions conducive to ice segregation exist under a pavement but are uniform in longitudinal and transverse directions, and freezing temperatures penetrate into the pavement structure at the same rate throughout the paved area. Under those idealized conditions the surface of the pavement is raised uniformly.

Uniform heave is not noticeable to a motorist even though the vertical displacement may amount to several inches, and has no effect on the serviceability of the pavement as long as the frozen and heaved condition lasts. Undesirable effects of heaves, whether uniform or irregular, may become evident during the spring, however, when thaw weakening and upward release of water into the base course may increase the rate of deterioration and thus adversely affect the pavement's performance.

It is seldom economically feasible to preclude heave, and the usual objective is to reduce its magnitude and make it more uniform. The idealized conditions producing uniform heave are difficult to achieve, owing to natural variations in subgrade soil and moisture conditions, and to discontinuities in the pavement profile such as culverts and changes from cut to fill. Design practices and construction techniques directed toward achieving greater uniformity are highly beneficial, and are treated in this report.

Differential Heaving

Differential heaving is indicated by the presence of surface irregularities, bumps, and general surface roughness in winter as a result of nonuniform conditions of soil and moisture availability. Distinctive cracking of the wearing surface may be caused by severe differential heave. Differential heaving may reduce traffic speeds, and in severe cases

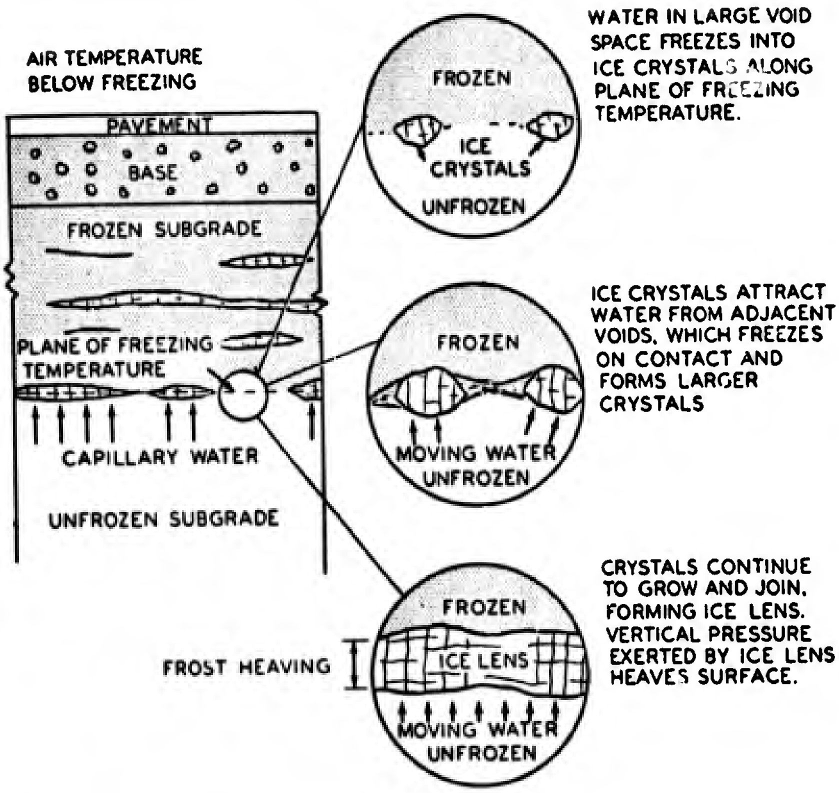


Figure 3. Formation of ice lenses at frost line (5).

may cause damage to vehicles and loss of vehicle control. Surface drainage may be impeded. Pavement roughness is reflected in a marked decrease in serviceability during the frozen condition; subsequent weakening during thaw causes accelerated deterioration and impaired pavement performance.

Conditions conducive to differential heave occur, for example, at locations where subgrades change from clean nonfrost-susceptible sands to silty frost-susceptible materials, at abrupt transitions from cut to fill with the groundwater close to the surface, or where excavation exposes water-bearing strata. Drains, culverts, or utility ducts placed in frost-susceptible subgrades under pavements frequently result in abrupt differential heaving due to different backfill material or compaction and to the changed thermal regime occasioned by the buried pipe left open to the atmosphere. Where the placing of such facilities under pavements cannot be avoided, special design and construction techniques are needed.

Development of Permanent Roughness

Permanent pavement roughness develops from the cumulative effects of traffic loads and from various nontraffic-load-associated causes (Table 1). Among the latter, differential frost heave causes a distortion of the surface that in its most severe form is temporary, lasting only until shortly after thawing. Nevertheless, some residual irregularity of the surface often remains as permanent roughness, which may gradually increase over the years. The effect is particularly striking in the case of upward migration of large stones or boulders left in the base course or subgrade within the frost depth (see "Other Frost Effects, Migration of Coarse Particles"). Permanent surface roughness caused by these boulder heaves becomes progressively worse over a period of years.

Permanent roughness that develops from the cumulative effects of traffic loads also is directly affected by frost action because the pronounced weakening of the subgrade during thawing of segregated ice leads to acceleration of rutting in flexible pavements and to cracking and faulting in rigid pavements.

Thawing

Temperature Distribution During Thawing

Periods of thawing are among the most critical phases in the annual cycle of environmental changes affecting pavements in seasonal frost areas. In the more southerly areas of the frost zone several cycles of freezing and thawing may occur during the course of a relatively mild winter, some of them causing complete elimination of frost from the ground. In the more northerly sections, brief thawing periods during more severe winters only partially thaw the top portions of pavement sections. Figure 4 shows both of these conditions in the ground temperature distributions measured at the AASHO Road Test site, located in Ottawa, Ill., with a moderate freezing temperature regime.

Such thawing cycles are in many cases very disruptive, depending on the rapidity of the thaw and the drainage capabilities of the pavement system. During thaw periods considerable melting of snow may occur, with melt water filling the ditches and infiltrating into the pavement from the shoulders and through surface cracks in the pavement itself. During thawing periods, bearing capacity may be severely reduced, and frost heaving frequently is more severe after midwinter thaw periods. In areas of deep frost penetration, the period of complete thawing of thicker pavement structures in the spring is usually the most damaging of the thaw periods because it affects the subgrade as well as subbase and base layers. The severity of the adverse effect on the supporting capacity of a given subgrade is largely dependent on the temperature distribution in the ground during the thawing period.

Thawing can proceed from the top downward, or from the bottom upward, or both. The manner of thawing depends on the pavement surface temperature. During a sudden spring thaw, melting will proceed almost entirely from the surface downward. This type of thawing leads to extremely adverse drainage conditions. The still-frozen soil beneath the thawed layer traps the water released by the melting ice lenses so that lateral and surface drainage are the only means of egress. In granular soil lateral drainage may be restricted by still-frozen shoulders resulting from the insulating effect of snow and/or different thermal conductivity and surface reflectivity characteristics. If air temperatures in the spring remain cool and frosty at night, upward conduction of heat stored in the ground from the previous summer and of heat from the interior of the earth will produce thawing, principally from the bottom upward. Such thawing permits soil moisture from melted ice lenses to drain downward while the material above it remains frozen.

Figure 5 shows temperatures measured during the freezing and thawing periods under a pavement with deep sand subbase in which no subgrade freezing occurred. The onset of higher temperatures in early March caused thawing of the upper layer of the subbase while a deeper layer remained frozen. Had the subbase been much thinner, this condition would have developed in the subgrade, with more detrimental consequences. In a more severe climate with deeper frost penetration even a relatively thick pavement structure would not preclude such a condition in the subgrade (Fig. 6).

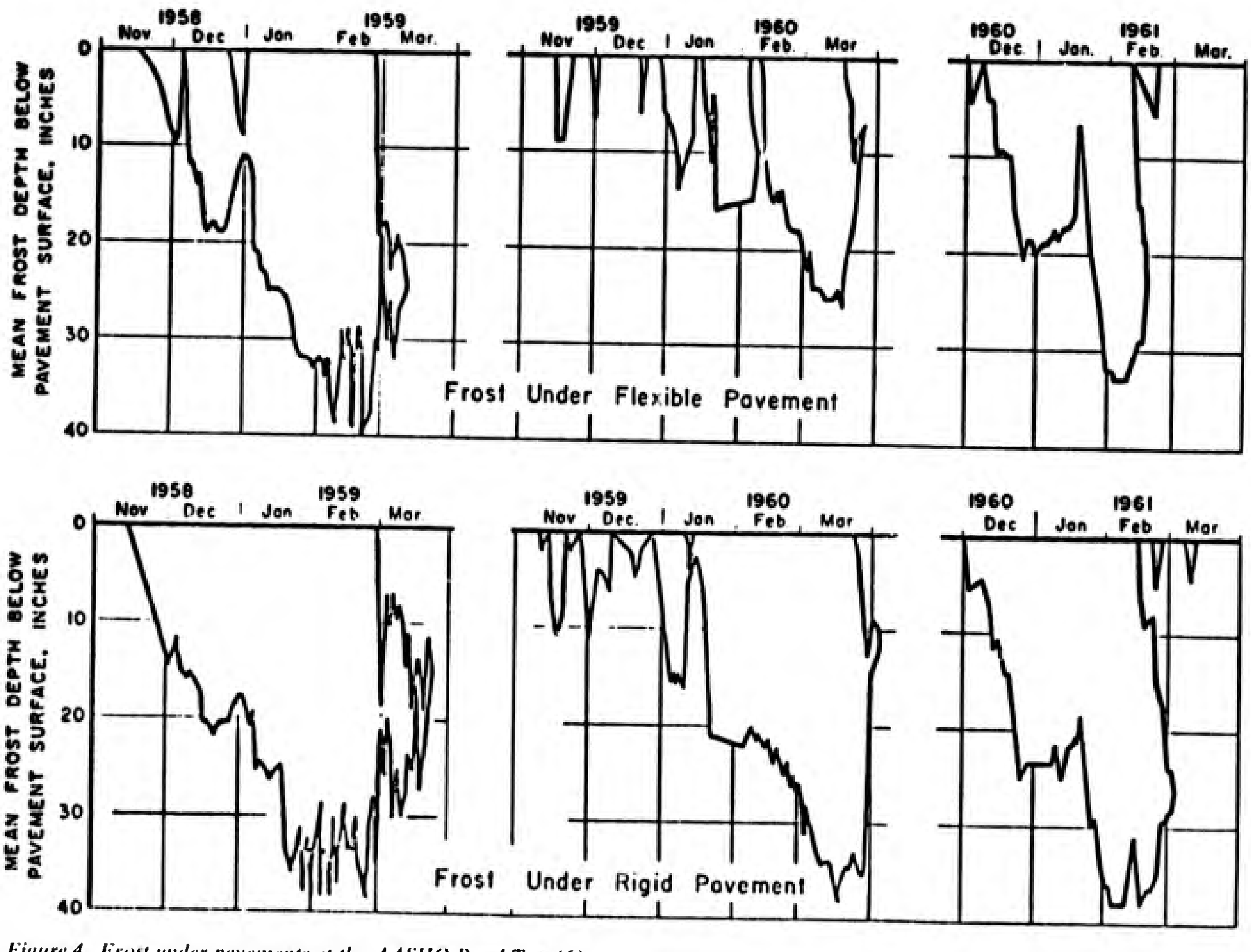


Figure 4. Frost under pavements at the AASHO Road Test (6).

The distribution of temperatures across a transverse section of a road is affected by the insulating effect of snow banked on the shoulders and by the lesser absorption of incoming radiation by the snow-covered area. The result is a somewhat greater rate of thaw under the central part of the roadway, particularly if the pavement is dark in color (Fig. 7).

Loss of Bearing Capacity

Loss of bearing capacity during spring thawing periods is a recognized effect of frost action that severely impairs the performance of pavements. The usual pattern of seasonal variation in subgrade support includes a sharp increase from normal summer/fall values during the period the subgrade remains frozen. Thawing produces an immediate decrease to levels well below the summer/fall values, followed by a gradual recovery over a period of several weeks or months (Fig. 8). The level to which the subgrade support rises while frozen is of little practical importance because it is much higher than necessary to preclude pavement damage originating in subgrade weakness. The assessment of thaw-weakened subgrade support is an essential part of the design process and is treated in an appropriate section of the report.

There are probably several distinct mechanisms of thaw weakening; Chamberlain (*11*) has reviewed and analyzed the literature and described three mechanisms suggested by research results. The first derives from the migration of moisture toward the freezing front during the development of ice segregation and from the substantial increase in volume (decrease in density) of the soil to accommodate the ice. Numerous field data (*12, 13*) and laboratory freezing tests (*14, 15*) reveal that a migration of moisture takes place toward the freezing zone in frost-susceptible soils to form ice lenses of various thickness as governed by the heat extraction rate and water availability. Ice lenses may form merely by redistribution of moisture within a narrow range of soil depth, causing saturation in an expanded state (decrease in density) at one level and a relative depletion of moisture from an adjoining area or level (with an increase in density). Thawing of segregated ice at a rate faster than the released moisture can escape into underdrains or into more pervious layers of the pavement system, or be reabsorbed into adjacent drier areas, accompanied by loading of the soil in its loosened state, results in generation of excess pore-water pressure, which causes a decrease in the load-carrying capacity. The inability of the melt water to escape is an essential condition of this process, a condition which derives from entrapment of the water above a still-frozen layer below (Fig. 7). At greater depths thawing proceeds from below (Fig. 6) at a slower rate, reducing pore-pressure buildup, and loss of supporting capacity is less significant.

The mechanism described in the preceding paragraph presupposes ice segregation and a heaved and loosened condition in that part of the soil containing the segregated ice. Yet it is known that certain clay soils that show no evidence of ice segregation, measurable moisture migration, or significant volume increase when frozen, undergo significant loss of supporting capacity during the thaw period. Chamberlain (*11*) states that Cook (*16*) and Titov (*17*) attributed such weakening to the formation of nodules or nuggets of soil particles, the surfaces of which appeared to have higher moisture contents than the interiors. They concluded that the strength reduction was due primarily to a reduction in the cohesion between the nuggets of clay particles. Chamberlain states further that Mikhailov and Bredyuk (*18*) attributed thaw weakening in clay not to ice segregation or the formation of nuggets but to conversion of bound water to free water by freeze-thaw action.

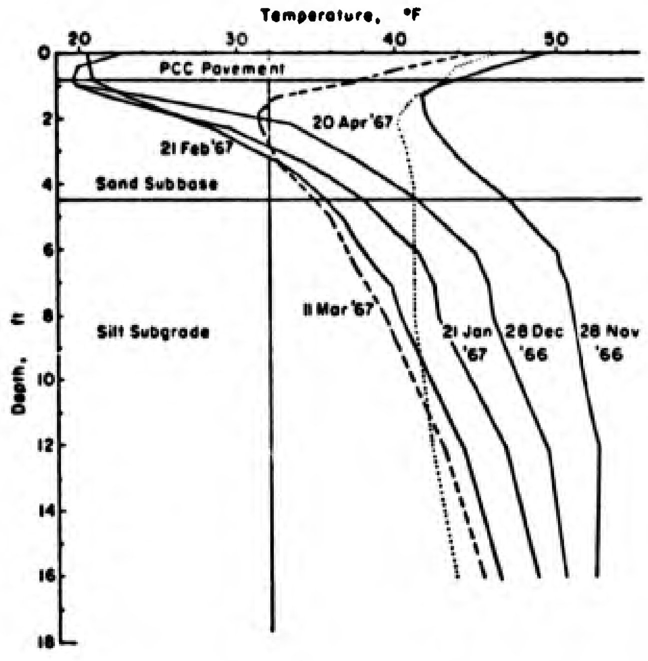


Figure 5. Temperature tautochrones, test pavement at Lebanon, N.H., Regional Airport (unpublished CRREL data).

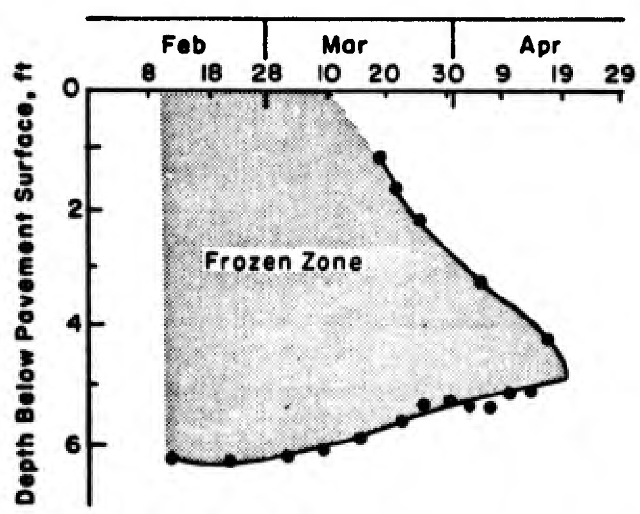


Figure 6. Limits of frost measured in 1957 near Minneapolis, Minn. (7).

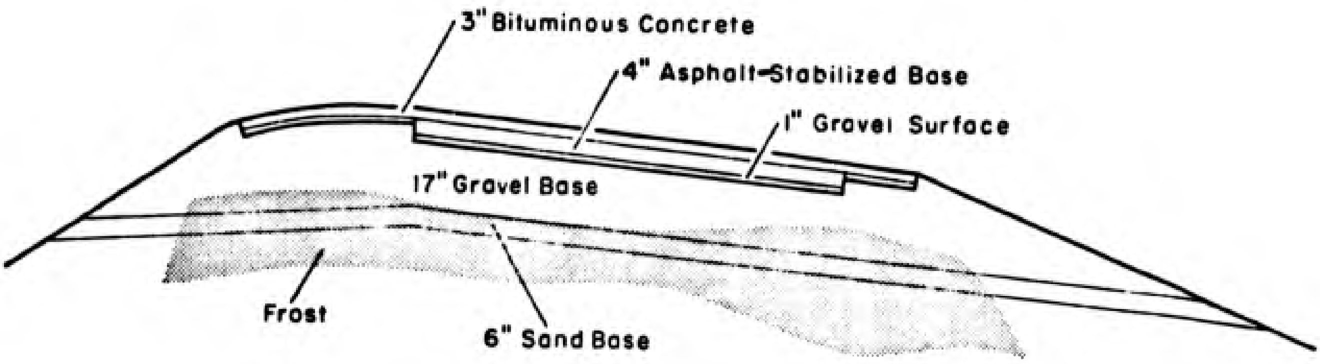
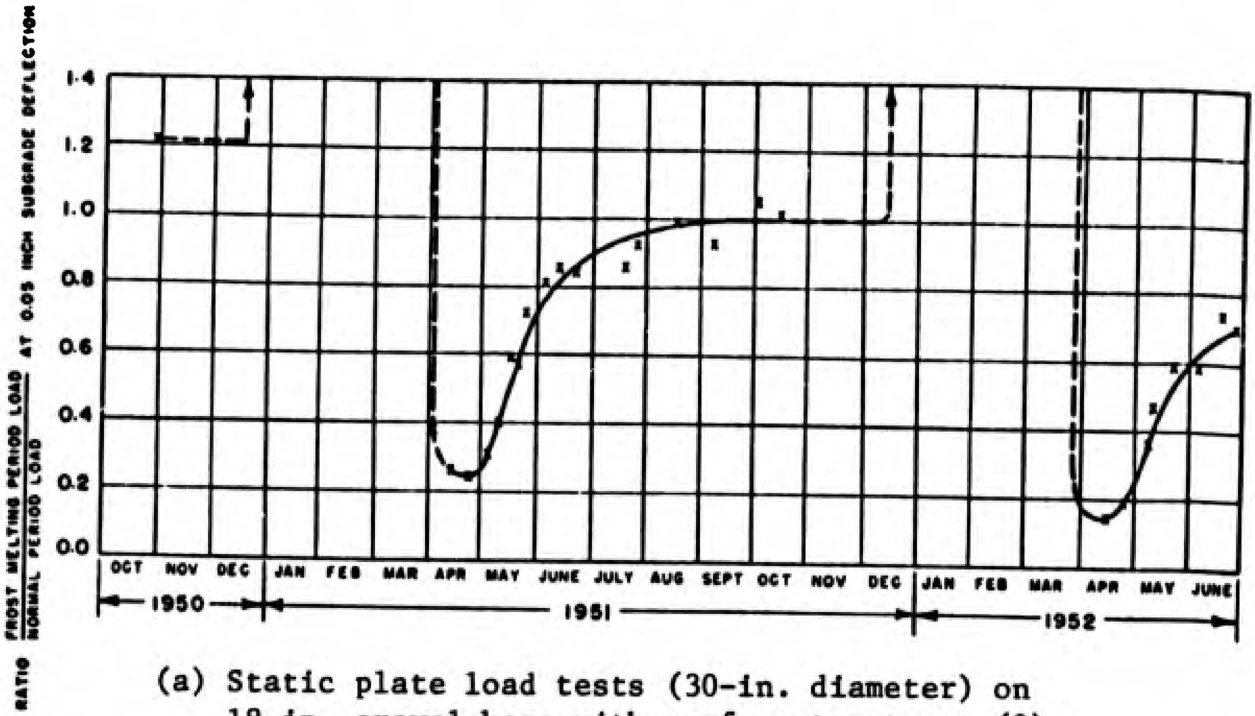
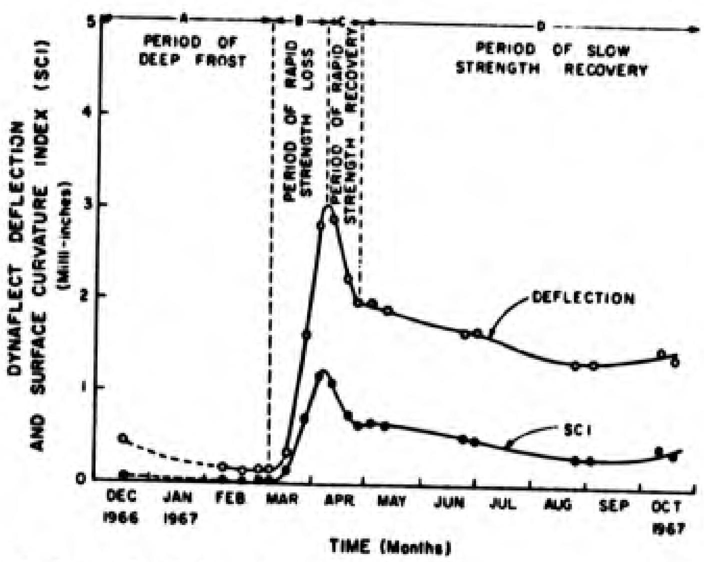


Figure 7. Frost remaining in partially thawed subbase and subgrade. (After Maine State Highway Commission, 8).



(a) Static plate load tests (30-in. diameter) on 18-in. gravel base with surface treatment (9).



(b) Tests with Dynaflect apparatus on 3-in. AC pavement over 12 in. of granular base and subbase (10)

Figure 8. Seasonal variation in bearing capacity.

Effect of Snow Melt

The effect of snow melt on the thawing of seasonal frost is minor. The melting of snow cover occurs almost entirely at its top surface and, until a melting snow cover becomes fully ripened (i.e., has reformed the snow grains into coarse ice crystals), very little melt-water can percolate to the soil/snow interface (*19*). Once ripening has occurred and melt-water is available at the soil surface, the water temperature is at or very near 32°F (0°C), and its heat content is negligible for overcoming the latent heat of the frozen soil. Thus the frozen soil, the melt-water, and the base of the snow cover are all nearly in thermal equilibrium with each other.

Where the snow cover has disappeared and seasonal frost has begun to thaw from the ground surface downward, the drainage of water from the thawed soil layer is restricted by the impermeability of the still-frozen soil beneath. In this case, melt-water from snowbanks remaining at the pavement edge, or from remaining nearby snow patches or drifts, as in wooded areas or shaded locations, may flow or percolate to the site and contribute to the saturation of the thawed soil layer. Thus the snow-melt, by adding directly to the moisture content of the surface soil layers at the shoulders and under the pavement, may lead to still greater reductions in bearing capacity. Also it may be noted that surface water exposed to solar radiation absorbs heat, which then may become available in the heat transfer processes of soil thawing.

Differential Thaw

If thawing does not proceed at the same rate over all parts of a paved area, nonuniform subsidence of the previously heaved surface results, causing transient pavement roughness and nonuniform subgrade support for the pavement. Studies of pavements (*20*), especially 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 actual freezing period itself. Differential thaw may be due to several factors, such as:

1. Differing thermal properties of adjacent sections of pavement, caused by nonuniformity of subsoil strata and soil conditions, including type of soil, water (ice) content, and density.
2. Nonuniform exposure to sun's rays and differing angle of incidence.
3. Shaded portions on pavement due to deep cuts, trees, overpasses, or buildings.
4. Proximity to surface and subsurface drainage.
5. Differing color of pavement. Studies show that lighter-colored pavement reflects more of the sun's rays than do dark surfaces.

Subsidence of Coarse Layers Into Fine-Grained Soils

One of the undesirable consequences of thawing of frost-susceptible subgrades is the subsidence of coarse, open-graded base or subbase materials into thaw-weakened silt and clay subgrade soils as the latter flow into the large pore spaces in the coarse material. It has been reported (*21*) that in numerous instances a nonfrost-susceptible base course has become frost-susceptible by impregnation of a silt subgrade upward into the base course. A further undesirable consequence is that the base course contaminated with fine subgrade soils occupies less volume than the two materials separately, resulting in irregular subsidence of the pavement surface. Both these effects can be avoided in some cases by selection of a base course containing more medium to fine sand. It is good practice to use a special filter layer at least 4 in. (100 mm) thick, or a subbase graded to insure proper filter action, to prevent movement of fine subgrade soils into the base course.

Loss of Stability of Slopes

Side slopes at cuts and fills display a great tendency to slough off and erode as a consequence of frost action (Fig. 9). The damage usually occurs during the thawing period when the soil exposed on the slope is weakened and saturated as a result of melting of ice lenses; the condition may be aggravated also by surface runoff and emergence of underground seepage. Some of the effects of surface runoff may be minimized by providing ditches near the edge of the cut or fill to intercept and divert surface drainage and avoid undue erosion.

Various insulating materials can be used on slopes to control sloughing caused by frost action. A layer of hay, bark, straw, leaves, wood chips, or mulch is effective as temporary protection (Fig. 10). Alternatively, for more permanent treatment a blanket of coarse material, gravel, or crushed rock 6 to 30 in. (150 to 760 mm) thick permits drainage and at the same time stabilizes the slope (*23*). Slopes may be flattened and their tops rounded to reduce sloughing. Newly seeded slopes may be protected with a hay mulch and light covering of bituminous material. Other means are pegging, wire meshing, or placement of a layer of cinders or granular materials.

Other Frost Effects

Pavement Cracking

Frost or low temperatures comprise the direct cause of two important types of pavement cracking. The first, with random orientation and spacing, is an advanced stage of the distortion mode of distress (Table 1), and is caused by differential frost heaving, occurring in both asphaltic concrete

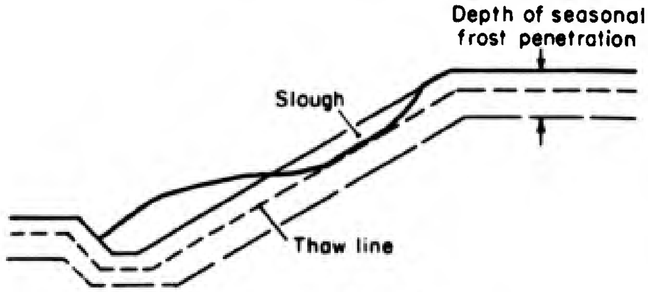
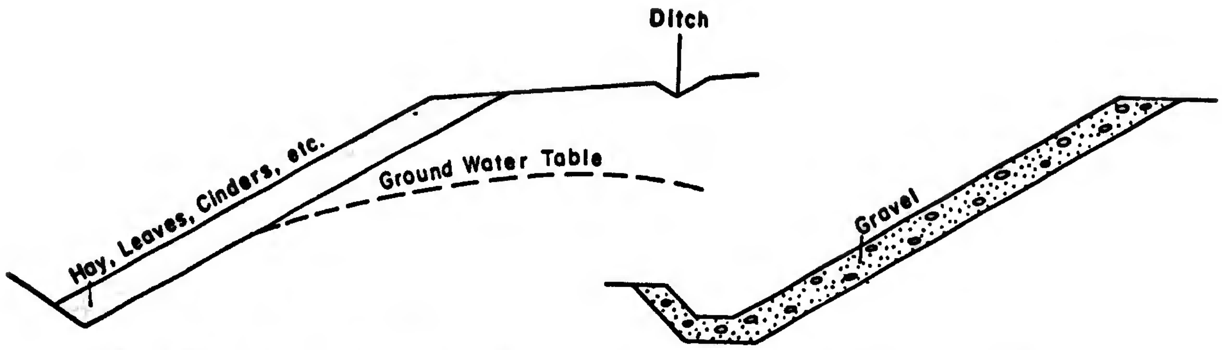


Figure 9. Thaw-sloughing of cut slope (22).



(a) Temporary treatment (5). (b) Permanent treatment (22).

Figure 10. Treatment of slopes to control sloughing due to frost action.

and portland cement concrete pavements. Flexible pavements, in particular those with relatively thin asphalt-bound layers, can tolerate some differential heave without the development of severe cracking; rigid pavements for obvious reasons have less tolerance of differential heave. Major roads of recent design and construction, with effective control of differential heaving through use of special techniques of subgrade preparation and/or subbase and base course layers of sufficient thickness, do not show this type of cracking. On older road systems and secondary roads with inadequate frost protection, underlain by frost-susceptible soils with adverse moisture conditions, cracking caused by frost heave is a major problem contributing to the accelerated deterioration of the pavement. Methods of minimizing frost heaves and resulting cracking, and methods of repair of heave-damaged pavements, are discussed elsewhere in this report.

Cracking of the second type results from thermal contraction, which induces stresses in the surfacing materials because they are partially restrained by friction along the interface with the supporting layer. Differential horizontal movements occur in both flexible and rigid pavements, but in the latter they are accommodated by joints or resisted and distributed by steel reinforcing. Therefore, these movements result in pavement distress only in flexible pavements.

Two broad classes of low-temperature contraction cracking in bituminous pavements are described by Fromm and Phang (*24*): (1) deep cracking penetrating through the pavement, base, and into the subgrade, and (2) shallow cracking affecting only the bituminous surfacing materials. In the first category are those wide transverse cracks that sometimes appear in pavements in very cold weather, penetrating the entire pavement structure and extending into the shoulders of the road; the cracks may be several inches wide and a number of feet in depth. This type of crack occurs when the entire pavement structure is frozen into a solid mass; the layered structure contracts as the temperature drops; its tensile strength is finally exceeded and a major crack develops. It may be unrelated to the type of materials in the pavement structure; in Alaska it is reported that gravel-surfaced roads also crack. This type of cracking may be associated more directly with regional cracking that appears in the frozen earth mantle during very low temperatures, crossing pavements and pastures indiscriminately, but it is usually more intense in areas cleared of snow. Pavement cracking of this kind is not of primary concern as it has been reported to occur only in certain localities and on a relatively wide spacing. In Alberta it is reported such cracks are found in some areas on a spacing of about 300 ft (90 m), as a subsystem of a total system of cracks that typically are spaced about 25 ft (7.6 m) apart.

The second category of low-temperature cracking is exemplified in the Alberta experience by the latter, more closely spaced cracks that appear only in the bituminous layer, occur much more extensively, and therefore present a more serious problem. Often beginning during the winter as hairline cracks, they slowly extend and widen with time. This form of cracking is common and has been reported in most Canadian provinces and in the northern United States (Fig. 11). Forty of the agencies surveyed reported the occurrence of low-temperature tranverse contraction cracks in asphaltic pavements, and most of them recognized such cracks as significantly affecting the serviceability of their roads. The ingress of water through these cracks tends to increase the rate of stripping, resulting in some cases in a depression at the crack brought about by raveling of the crack lip and pumping of the fine fraction of base material. During the winter months, when the entire roadbed is frozen and raised slightly above its normal summer level, deicing solution can enter these cracks and cause localized thawing of the base and a pavement depression adjacent to the crack. In other cases, water entering these cracks can result in formation of an ice lens below the crack, producing an upward lipping of the crack edges. Both of these effects result in rough riding qualities. Often, further secondary cracks are produced paralleling the major crack (Fig. 12).

In Wisconsin it was reported that asphalt-treated base (ATB) is now used under bituminous pavement instead of cement-treated base (CTB) because the latter was found to be highly susceptible to development of ridges at low-temperature transverse cracks, termed "frost tenting." In Saskatchewan similar distress, termed "transverse ridging," occurs most severely over highly plastic lacustrine clay subgrades; it has long been a problem on pavements with unbound bases, it has recently become apparent also with soil-cement bases, and it is very severe with ATB. The heave at transverse cracks varies up to 2 in. (50 mm) or more and affects the pavement about 2 to 3 ft (0.6 to 0.9 m) on each side of the crack. In Alberta similar phenomena, termed "crack-bumps," are especially severe on pavements with unbound bases over highly plastic clay subgrades, where as much as 3-in. (76 mm) heave above normal pavement grade has been reported; damage is slight on pavements with soil-cement bases, and problems are now appearing on some full-depth asphaltic pavements.

A recent study by Shahin and McCullough (*26*) indicates that a substantial portion of cracks that to date have been attributed to low-temperature shrinkage may actually be caused by thermal fatigue. The computer model developed as part of the study includes both low-temperature cracking and thermal-fatigue cracking and the computed data correlate closely with observed data at a few locations in the United States and Canada. Additional field data are necessary to validate the thermal-fatigue concept.

Thermal changes also are usually the cause of reflection cracks, characterized by upward propagation of cracks through an overlay pavement from cracks or joints in the former surface layer. Both daily and seasonal temperature fluctuations can give rise to reflection cracking, which is by no means unique to the seasonal frost areas. The problem of control or prevention of reflection cracks through asphaltic concrete overlays has not been adequately solved, and in the colder regions it is even more difficult because it becomes an integral problem with the low-temperature contraction cracking phenomenon. Design approaches to solutions for the latter are outlined elsewhere herein; techniques for minimizing reflection cracking are outside the present scope and were treated recently by the Highway Research Board (*27*).

Migration of Coarse Particles

As a result of freeze-thaw cycles over a period of years, large stones present within the frost zone in a subgrade or base course that is even slightly frost susceptible will eventually be heaved upward to the surface and will produce surface roughness and damage to both rigid and flexible pavements. Figure 13 shows a stone that has surfaced through a flexible pavement. The mechanism of the raising of a stone by frost action is shown in Figure 14, obtained by time-lapse photography from a laboratory experiment (*29*). When the soil containing the heaved stone thaws, some of the softened soil next to the cavity beneath the stone flows or tumbles into the cavity, preventing the stone from recovering entirely its previous position. In successive freeze-thaw cycles the process is repeated and in this way the stone rises steadily toward the surface.

Figure 11. Pavement in Ontario severely affected by low-temperature contraction cracks (25).

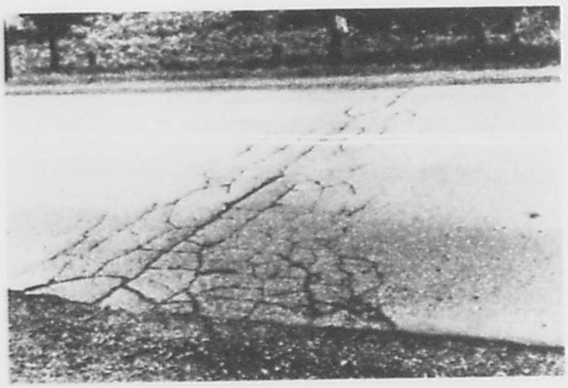


Figure 12. Example of multiple secondary crack (24).

Figure 13. Stone (approx. 9 in.) raised by frost action until it has broken through the pavement surface (28).

Icings in Drainage Structures and on Roadways

The term *icing* is applied to several different phenomena in seasonal frost areas, and these all differ in the way ice develops or accumulates. In the present context, however, an icing is a formation of ice on a surface or in a location where it presents an operational or maintenance problem. Typical examples are ice-filled culverts, ice buildups on the faces of rock-cuts, ice-filled ditches, and ice layers on pavements originating from surface water flow. (Direct icing of pavements by meteorological processes—i.e., glaze or freezing rain—is outside the scope of this synthesis.) Flooding of roadways and possible ice formation on pavements may occur where ice obstructs culverts or ditches; the origin of the flow may be snow melt, rainfall, or groundwater seepage.

In the warmer seasonal frost areas, many of the icings tend to form during periods of alternate freezing and thawing weather, especially when freezing and thawing occur rapidly, as in early spring when thawing in the daytime is followed by freezing at night. In addition, sites where water emerges as a spring or seepage may experience icings during much or all of the winter. In the colder seasonal frost areas (i.e., the upper Midwest, the northern Great Plains, the northern Rocky Mountains, most of Canada, and almost all of Alaska), the icing problems are often of long duration. Icings may grow in size throughout much of the winter season, reaching pavement elevations and requiring periodic

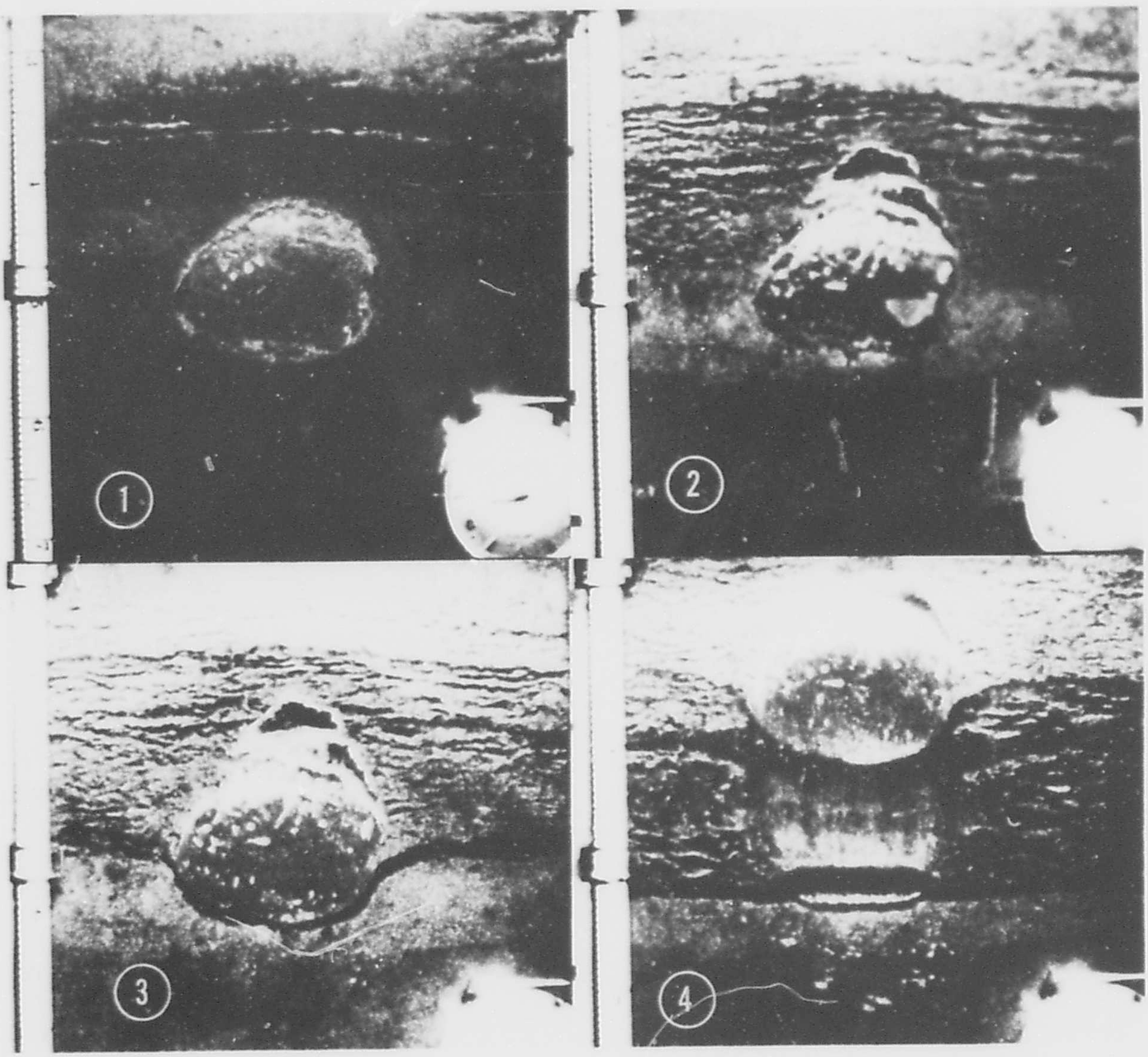


Figure 14. Time-lapse photos of a stone being raised through soil by frost action (29), showing: (1) frost line approaching the top of the stone; (2) (about 30 hr later) the soil above the stone has heaved, leaving a void; (3) (about 12 hr later) the stone is being lifted by the grip of the frozen soil around it, leaving a void under the stone; (4) (about 79 hr later) the stone has moved up a considerable distance and the cavity below has become slightly narrower and filled with water, which has frozen.

ice-removal operations. In spring, ice must be thawed from affected drainage structures (using steam, burners, etc.) to prevent flooding and possible washouts. In these northerly regions, especially in Alaska and Northern Canada, many icings are caused directly by the more severe low temperatures: the deep seasonal frost penetration may block normal paths of near-surface groundwater flow, or thickening river ice covers may obstruct normal under-ice stream flow. These obstructions can force the flow to reach the ground surface in certain places, and exposure of the water to the cold atmosphere begins to form an icing. If flow continues, it freezes upon exposure, and the icing grows layer by layer and spreads laterally as the topography permits (*30*).

Freeze-Thaw Effects on Surfacing Materials

Surfacing materials such as concrete suffer severe thermal stressing from alternate freezing and thawing, and if the resistance of the materials is deficient, deterioration may progress rapidly to the disintegration mode of distress (Table 1). This extensive topic, which pertains also to materials used in structures other than pavements, is beyond the scope of the present report.

Pavement Wear from Studded Tires

Despite the severity of the cold-regions problem of pavement wear by studded tires, which markedly affects the serviceability of pavements in seasonal frost areas, space limitations preclude its treatment in this report.

IDENTIFICATION OF FACTORS CONTRIBUTING TO FROST PROBLEMS

Climate

The climatic factors of air temperature, solar radiation received at the surface, wind, and precipitation are major parameters that affect the severity of frost effects in a given geographical area. The first three affect mainly the temperature regime in the pavement structure, including the important parameters of depth of frost penetration, number of freeze-thaw cycles, and duration of the freezing and thawing periods. Precipitation affects mainly the moisture regime but causes changes in the thermal properties of the soil and interacts with the other climatic variables determining ground temperatures as well.

Relationships between climate and frost in the ground have been studied in countless investigations. Many investigators endeavoring to calculate the depth of frost penetration found it convenient to make use of a freezing index, which expresses the cumulative effect of intensity and duration of subfreezing air temperatures. The air freezing index for a given year and site location can be calculated from average daily air temperature records, which should be obtained from a station situated close to the construction site. This is necessary because differences in elevation and topography, and nearness to centers of population or bodies of water (rivers, lakes, seacoast) and other sources of heat, are likely to cause considerable variations in the value of the freezing index over short distances. Such variations may be of sufficient magnitude to affect a pavement design based on depth of frost penetration, particularly in areas where the freezing index used in the calculations is less than about 100 degree-days.

Freezing Index

The number of degree-days for any one day is the algebraic difference between the average daily air temperature (in deg F) and 32°F. The average daily temperature is the average of the maximum and minimum temperatures for one day or the average of several temperature readings taken at equal time intervals, generally hourly, during one day. The degree-days are negative when the average daily temperature is below 32°F and positive when the average daily is above 32°F; Figure 15 shows an example of the summation of daily degree-days. The freezing index is the number of degree-days between the maximum and minimum points on a plot of cumulative degree-days of below-freezing temperature for one freezing season. A cumulative degree-days-time curve is obtained by plotting the cumulative summation of degree-days versus time, starting a few days prior to onset of continuous freezing temperatures and continuing until consistent average daily temperatures above freezing prevail (Fig. 16). The freezing index, expressed in Fahrenheit degree-days in accordance with North American practice, may be converted to Celsius degree-hours, as conventionally employed in northern European countries, by multiplying the Fahrenheit degree-days by 13.33.

In the mathematical formulas developed for predicting the penetration of freezing temperature into the ground for the average year, the mean air freezing index is used. The mean air freezing index should be based on air-temperature records for 10 or more years, if available, and may be computed from mean daily temperatures for a given day for several years of record. Mean monthly temperatures may also be used for computation of the mean freezing index.

When freezing indexes are used in designing pavements, selection of data corresponding to years that are colder than

Day	1	2	3	4	5	6	7	8	9	10
Average temperature (°F)	36	34	32	30	32	33	30	28	29	30
Degree-days	+4	+2	0	-2	0	+1	-2	-4	-3	-2
Cumulative degree-days	+4	+6	+6	+4	+4	+5	+3	-1	-4	-6

Figure 15. Example of determination of cumulative degree-days, using typical data for 10 days (31).

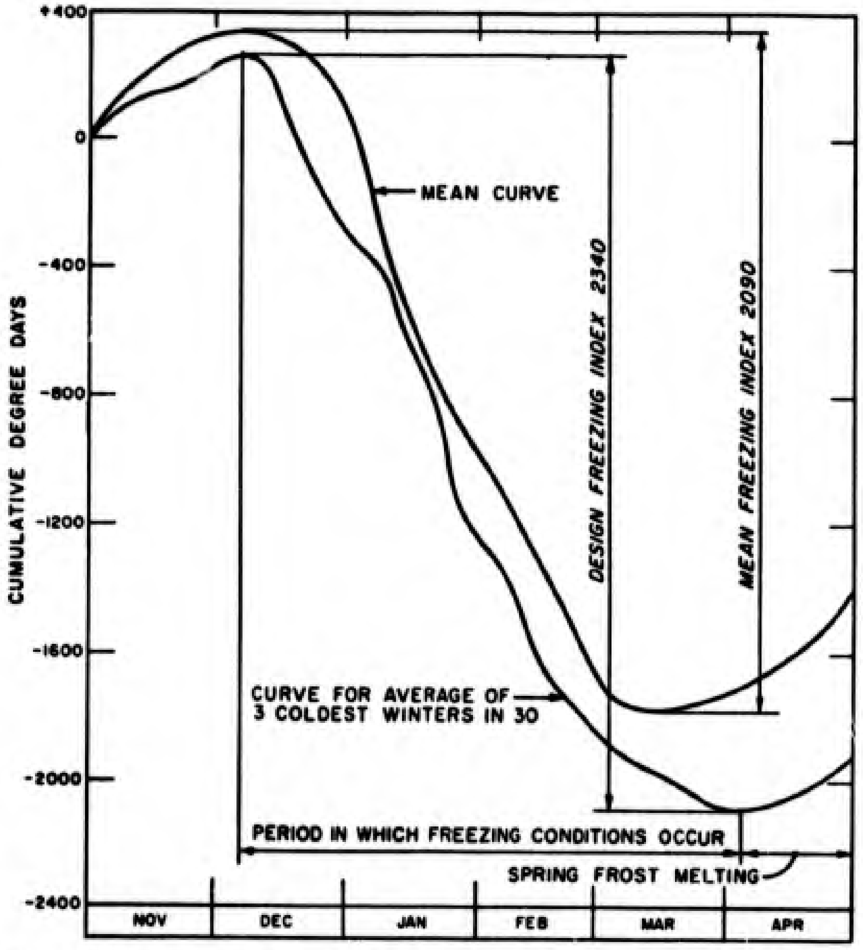


Figure 16. Determination of freezing index (32).

normal provides the means of guarding against damage to the pavements by these more severe temperature conditions. The Corps of Engineers calculates the freezing index for the three coldest winters in the latest 30 years of record, if available, or the coldest winter in the latest 10-year period; when calculated on this basis, it is termed the "design freezing index" (*32*). The relationship between the mean and design freezing indexes is shown in Figures 16 and 17.

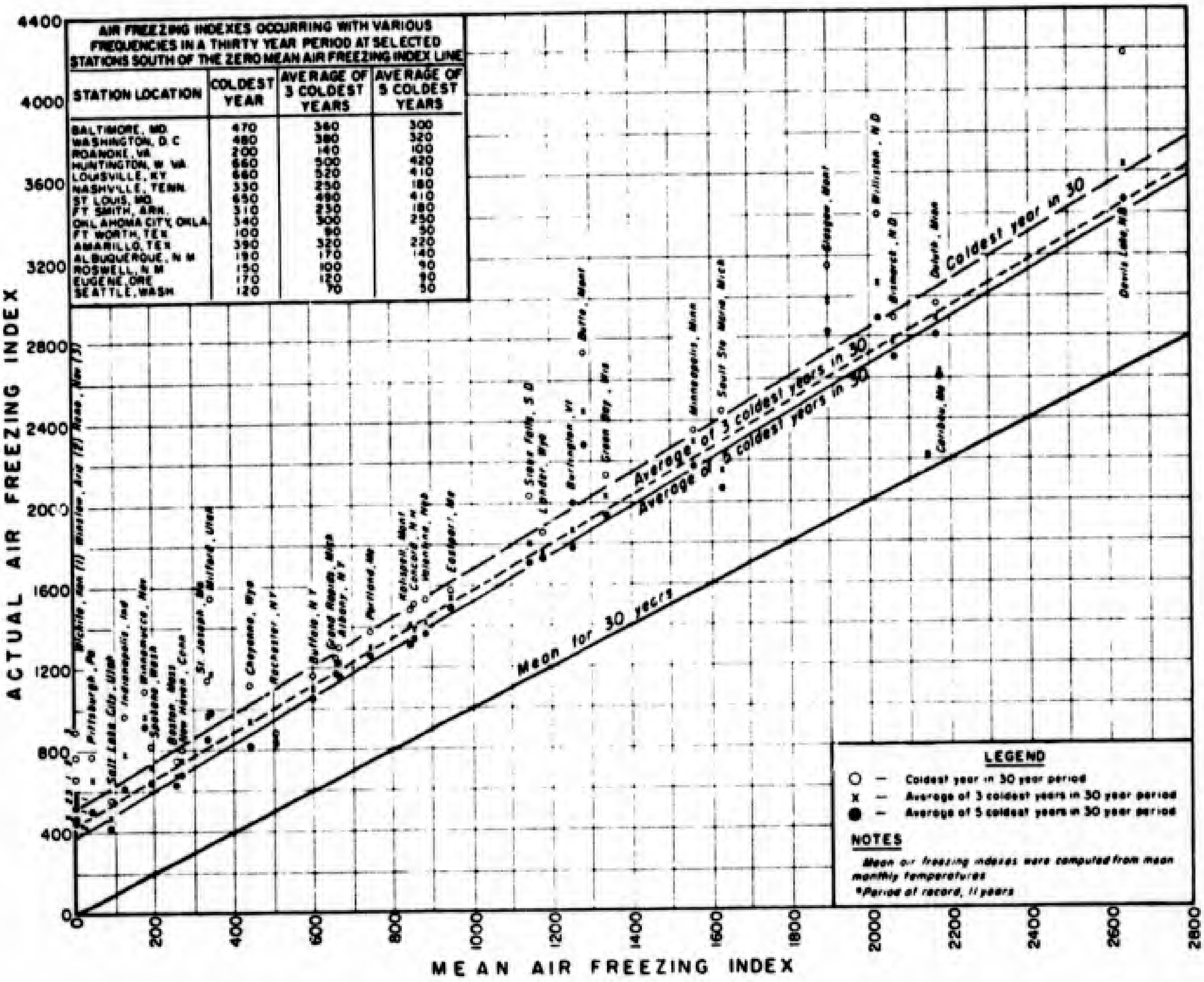
Air Versus Surface Freezing Index

The freezing index may be calculated based on either air temperature or pavement surface temperature. Because air temperature data are much more readily available than pavement surface temperature data, the air freezing index is most commonly used, although it involves the added complication of accounting for the difference between air and pavement surface temperatures. The index determined for air temperatures at approximately 4.5 ft (1.4 m) above the ground is commonly designated as the "air freezing index," whereas that determined for temperatures immediately below a surface is known as the "surface freezing index." Pavement surface temperatures and air temperatures are interdependent in a complex relationship that involves heat transfer at the air-ground interface and is affected by evaporation and condensation of moisture, snow and ice melt, and, principally, direct and diffuse solar radiation, long-wave radiation between pavement and the sky, and convections (*33*).

A number of investigators have attempted to establish correction factors for predicting freezing temperatures by comparing calculated and observed depths of frost penetration or by comparing freezing indexes based on measured air temperatures and pavement surface temperatures. This empirical correction factor was defined as the ratio of pavement surface freezing index to the air freezing index and referred to (*34*) as the "*n*-factor." The *n*-factor has been found to be strongly dependent on the type of pavement surfacing materials and somewhat less on location, indicating a need for a more realistic approach through theoretical and experimental considerations of the controlling variables. Studies in this area have been reviewed and summarized by Thompson (*35*). The *n*-factor is used in frost depth prediction formulas, mentioned in subsequent paragraphs.

Sources of Climatological Data

Summaries of daily and mean monthly air temperature records for all stations that report to the United States



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	80
SEATTLE, WASH	120	70	50

Figure 17. Relationship between mean air freezing index and freezing indexes during colder years for 30 consecutive years (32).

National Weather Service are available at the various National Weather Service Centers. Usually, one of these centers is located in each state. In Canada, weather records are available from the Meteorological Branch of the Department of Transport. A "Local Climatological Data" summary is published annually by the National Weather Service for each of the approximately 350 first-order stations. If official data are unavailable or inadequate, other reliable data from nearer to the site should be used.

Measurement and Prediction of Frost Penetration

Frost penetration into soil depends not only on the intensity and duration of subfreezing air temperatures, but also on the surface and other conditions as previously outlined, plus the type of soil and its density and moisture content. Frost penetration is considerably greater in paved areas underlain by granular soils with low moisture content than in fine-grained turf-covered shoulder areas with high water content. High moisture content retards frost penetration due to its high latent heat (80 calories per gram) released upon freezing.

It is well known that a snow cover acts as an insulating layer on the ground. Depending on the density and depth of the snow, the time of its occurrence, the climatic conditions, and the soil conditions, a snow cover may retard, terminate, or prevent seasonal frost penetration into the ground. Although the upper layers of the ground may freeze at the beginning of the winter before a heavy snow cover develops, it is often found that the ground is entirely unfrozen beneath the snow late in the winter. This wintertime thawing of seasonal frost results from the normal upward flow of heat that had been added to the ground during the previous summer, acting in combination with the insulating effect of the snow cover; the seasonal frost actually thaws from the bottom upward. Where snow is absent, as on windswept surfaces or plowed pavements, the temperature at the ground surface is lower than under the snow cover. Thus, the subsurface temperature gradient in these areas is shifted toward lower temperatures. The high rate of heat loss to the atmosphere from these bare surfaces causes the freezing level to maintain its position or move downward, even though the geothermal heat flow below the seasonal frost level may be substantially the same in bare and snow-covered areas (*19, 36*).

Depth of freezing can be measured by several methods, such as coring, test pits, temperature sensors, and frost tubes. The frost tube is perhaps the simplest and most economical (*37*). Frequently, locally available data from utility companies, town officials, and contractors may provide some idea of maximum depths of frost penetration. Information from these sources is usually poorly documented or not at all, and may be grossly unreliable. A number of empirical relationships and formulas have been developed over the years; some of these methods, which are still used in some areas (*35*), utilize either air or pavement surface freezing index, but take no account for other key variables such as soil type and moisture content.

Computational methods that account for the principal governing parameters have been developed and are reasonably reliable. The Stefan equation is the simplest of the frost penetration equations. It was first published in 1890 (*38*). In its application to soils, the Stefan equation is based on the hypothesis that the latent heat of soil moisture is the only heat that needs to be conducted to or from a point that is in the process of thawing or freezing. A common form for uniform soil conditions is

$$X = \sqrt{\frac{48\,K\,F}{L}} \tag{1}$$

in which

K = thermal conductivity of the soil layer;
F = freezing index;
L = latent heat of fusion; and
X = depth of frost penetration.

Because it did not consider the heat capacity of the soil, the Stefan equation usually gave frost penetrations that were too large. The modified Berggren equation (*39*)

$$X = \lambda\sqrt{\frac{48\,K\,nF}{L}} \tag{2}$$

corrects the Stefan equation by means of the term λ, a coefficient that takes into consideration the effect of temperature changes in the soil mass. Either equation can include the air freezing index, F, or the surface freezing index, nF.

The modified Berggren equation has been adapted to layered systems by using weighted values of heat capacity and latent heat within the estimated depths of frost penetration. Figures 18 and 19 may be used to estimate values of frost penetration beneath areas kept free of snow and ice, and also turf areas with and without snow cover. They have been computed for an assumed 12-in. (300 mm) thick rigid pavement, using the modified Berggren formula and correction factors derived from field experience under different conditions. The curves yield maximum depths to which the 32°F (0°C) temperature will penetrate from the pavement surface under the total winter freezing index value, into homogeneous materials for the indicated density and moisture content. Variations due to use of other pavement types and of rigid pavements less than 12 in. thick may be neglected. Where individual analysis is desired or where unusual conditions make special computations desirable, the modified Berggren formula may be applied (see notes on Fig. 19). Neither Eq. 2 nor the curves in Figures 18 and 19 are applicable for determining transient penetration depths under partial freezing index values. Methods of estimating frost depths beneath surfaces other than those indicated, and for soils of varying density and water content, are discussed by Aldrich and Paynter (*39*).

Criteria for Identifying Frost-Susceptible Soil

Most studies have shown that a soil is susceptible to frost action only if it contains fine particles. Early investigators found that soils free of fines, comprising only particles retained on the 200-mesh sieve, did not develop significant ice segregation (*41, 42*). It has been observed that other soil properties—such as over-all grain-size distribution (texture), grain shape, mineral composition, and plasticity characteristics—are contributory to varying degrees. The packing of the soil particles also has important effect because it involves variations in particle arrangement and density of the soil mass and, thus, affects interparticle void dimensions, soil permeability, and capillary forces. The following are some of the principal methods that have come into use during the past 40 years for identification of potential frost susceptibility based on these properties.

Beskow's Criteria

Freezing studies by Beskow (*43*) on fine-soil fractions showed that heaving of specimens subjected to freezing with water freely available was first discernible in the soil fraction containing particle sizes ranging from 0.1 to 0.05 mm (200-mesh size = 0.074 mm). Heaving increased with decrease in particle sizes until the trend was reversed beginning with the soil fraction containing particles ranging from 0.0016 to 0.0005 mm in diameter (Fig. 20). Beskow concluded that for uniformly graded soil ". . . above an average diameter of 0.1 mm practically no [ice] stratification . . . occurs; for more rapid freezing the size is reduced to 0.05 mm." Beskow demonstrated the effect of other important soil parameters, such as grain-size distribution, uniformity of grain sizes, capillarity, packing (density), permeability, suction, and effect of restraining pressure, and concluded that for practical purposes grain size and capillarity are the basic properties for evaluating potential frost behavior. He recognized the significance of inter-particle void space and its relation to moisture transport and surface energy characteristics of a soil mass and developed a moisture tension test to measure the capillary potential of a soil. Beskow's classic work and the concepts he developed remain essentially unchallenged today.

Casagrande Criterion

As a result of extensive laboratory and field studies of New Hampshire soils, A. Casagrande (*44*) proposed the following widely known rule-of-thumb criterion for identifying potentially frost-susceptible soils:

> Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in nonuniform soils containing more than 3 per-

cent of grains smaller than 0.02 mm, and in very uniform soils containing more than 10 percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1 percent of grains smaller than 0.02 mm, even if the groundwater level was as high as the frost line.

Casagrande does not mention directly the coefficient of uniformity (C_u) in the reference cited (*44*) nor in a later paper (*45*); Riis (*46*), however, makes reference to an earlier Casagrande classification that differentiates between nonuniform soils having $C_u(d_{60}/d_{10}) > 5$ and uniform soils with $C_u < 5$. Application of the Casagrande criterion requires a hydrometer test of a soil suspension to determine the distribution of particles passing the No. 200 sieve and to compute the percentage of particles finer than 0.02 mm.

Corps of Engineers Frost Design Classification

The Corps of Engineers frost design classification system was developed in the late 1940's to make use of the Casagrande criterion regarding frost susceptibility and to account for the reduced stability of the various types of frost-susceptible soils during the thaw-weakened period.

Results of traffic tests and field CBR and plate bearing tests conducted at a number of airfields throughout the northern United States during the spring thaw (*47*) were the basis for the system and for the frost condition pavement design curves. In this system frost-susceptible soils (with 3 percent or more, by weight, finer than 0.02 mm) are classified into one of the four groups F1, F2, F3, and F4, for frost design purposes (termed in this report FS groups). Soil types are listed in Table 2 in approximate order of in-

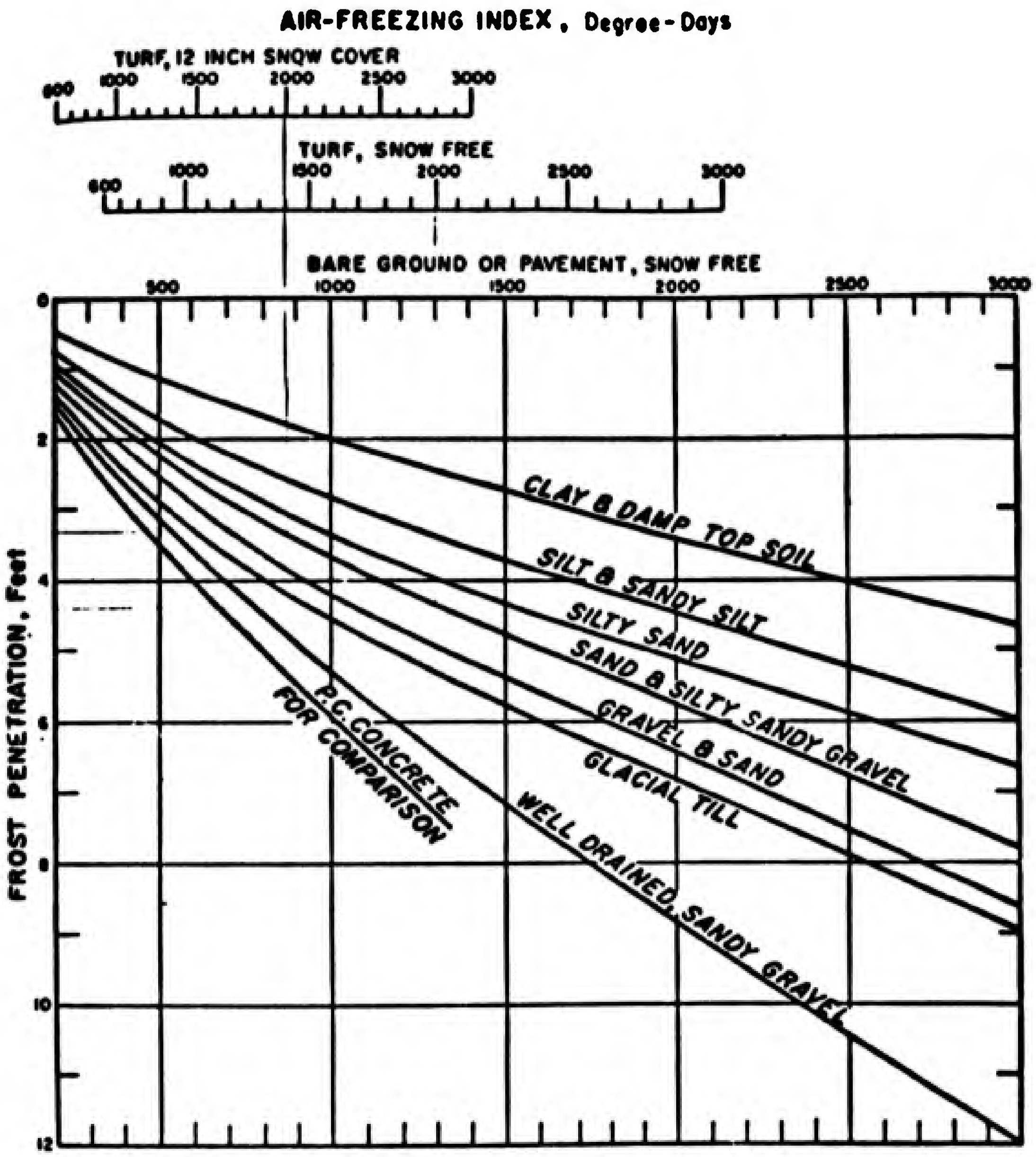
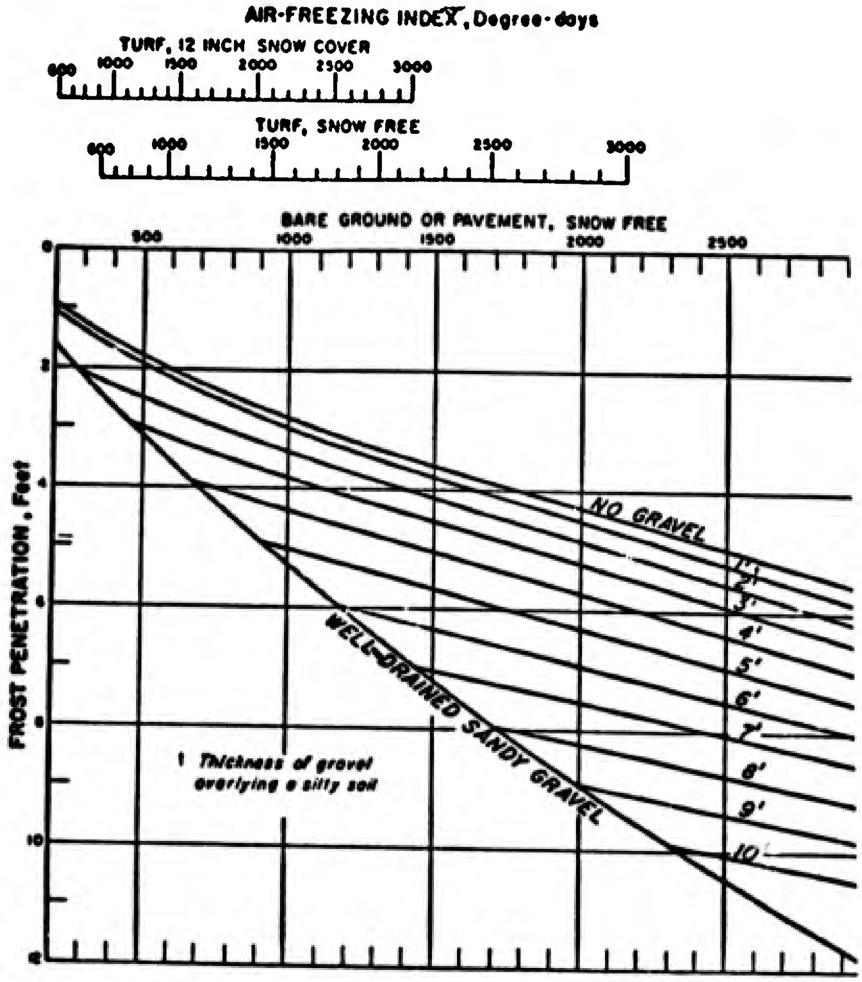


Figure 18. Relationship between air freezing index, surface cover, and frost penetration into homogeneous soils (40).



NOTES:

1. Frost penetration depths are based on Modified Berggren Equation and computation procedures outlined in the following reports:
   a. "Analytical Studies of Freezing and Thawing of Soils," H.P. Aldrich and H.M. Paynter, Arctic Construction and Frost Effects Laboratory, Corps of Engineers, U.S. Army, June 1958.
   b. "Frost Penetration in Multilayer Soil ?rofiles," MIT Dept. of Civil and Sanitary Engineering, Soil Engineering Division, Arctic Construction and Frost Effects Laboratory, Corps of Engineers, U.S. Army, June 1957.
2. It was assumed in computations that all of soil moisture freezes when soil is cooled below 32°F.
3. Frost penetration depths shown are measured from pavement surface. Depths for 12" PCC pavements are good approximations for bituminous pavements over 6 to 9 in. high quality base. For a given locality, depths may be computed with the Modified Berggren Equation if necessary data are available.

Figure 19. Relationship between air-freezing index, surface cover, and frost penetration into a granular soil overlying a fine-grained soil (40).

creasing susceptibility to frost heaving and/or weakening as a result of frost melting, although the order of listing subgroups under Groups F3 and F4 does not necessarily indicate the order of susceptibility to frost heaving of these subgroups. The basis for distinction between the F1 and F2 groups is that F1 material may be expected to show higher bearing capacity than F2 material during thaw, even though both may have experienced equal ice segregation. The F3 and F4 soils, grouped together for reduced strength design, show the greatest weakening during thaw.

Interparticle Void Sizes

Jackson and Chalmers (*48*) have proposed that work energy for heaving and attracting water into ice lenses derives from supercooling of the thin liquid films separating the ic-

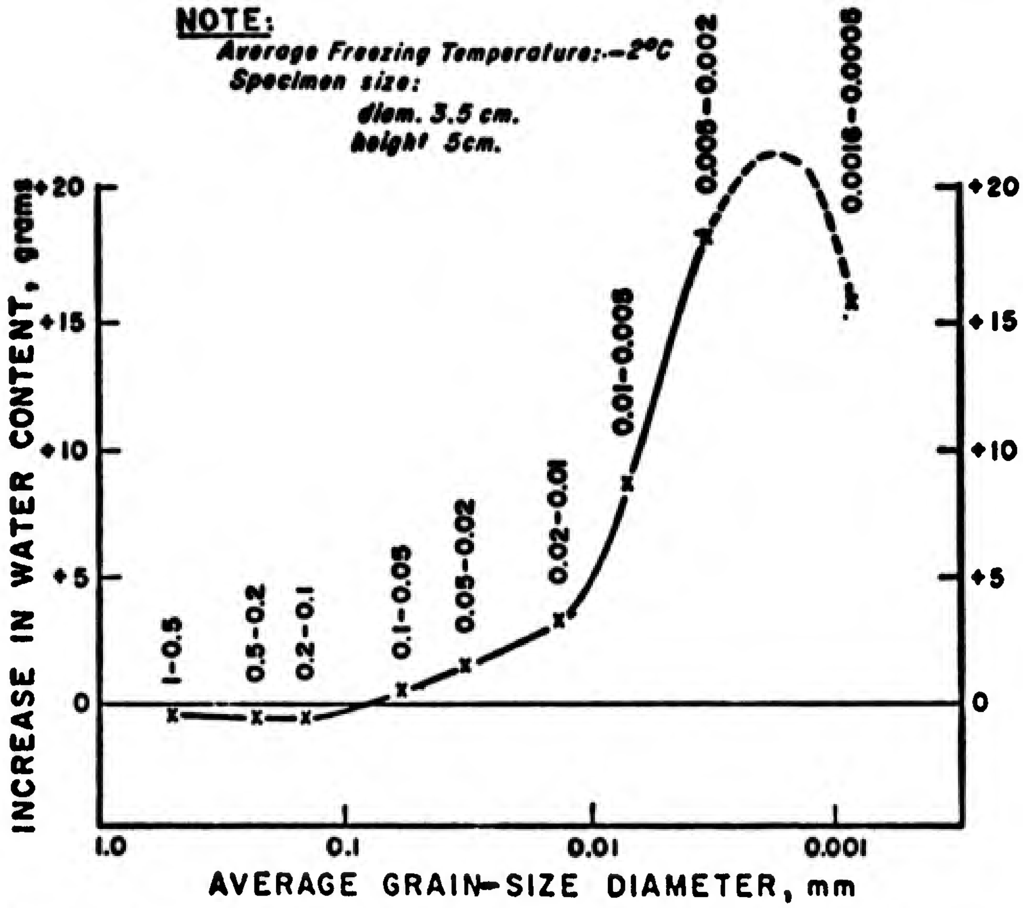


Figure 20. Relation between moisture increase caused by freezing and average grain-size diameter of pure soil fractions (43).

TABLE 2

CORPS OF ENGINEERS FROST DESIGN SOIL CLASSIFICATION AND USCS EQUIVALENT GROUPING [a]

FROST GROUP	SOIL TYPE	PERCENTAGE FINER THAN 0.02 MM, BY WEIGHT	TYPICAL SOIL TYPES UNDER UNIFIED SOIL CLASSIFICATION SYSTEM
F1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	(a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F3	(a) Gravelly soils	>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, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

[a] From Corps of Engineers (*20*).

lens from the soil particle. The free energy, equal to the difference, ΔT, between the undercooled temperature and the normal freezing temperature of water, is released upon freezing of film water and is available to do work. According to this theory, the free energy available for work is greater as supercooling increases with decreasing film thickness. The theory further stipulates that ice in lenses cannot propagate through inter-particle channels (capillaries) unless the equilibrium freezing temperature of capillary moisture as affected by channel size (thickness) is exceeded.

From an extension of these theoretical considerations, Chalmers and Jackson (*49*) subsequently introduced an expression relating pressure and free energy, and thus related effective void size radius and maximum heave pressure. The validity of the pressure-void size relationship was also demonstrated by model studies (*50, 51*).

The interconnecting channels and interparticle void dimensions are a function of soil particle sizes, shapes, grain-size distribution, packing arrangement, and unit density. If the soil particles are of such a shape and size distribution that they pack well between each other, the voids and interconnecting channels will be smaller than for uniformly sized particles of the same average linear dimensions. If the range of particle sizes present is very large and the smaller particles tightly fill the spaces between the larger sizes, the voids and channels formed will have characteristics based on the smallest particles present. This may account for observations that well-graded clayey gravelly soils with only a relatively small percentage of fines may be severely frost susceptible (*52, 15*).

Townsend and Csathy (*53*) propose using the void size distribution in a soil mass as an indicator of potential frost susceptibility. Its measurement requires determination of the height of capillary rise of water in a soil column after equilibrium has been established. From the distribution of moisture content with depth in a column of soil a void size distribution curve is calculated. The calculated void size distributions of 39 soils were correlated with observations of frost heaving of Ontario roads, from which Townsend and Csathy proposed the following criteria to identify frost-susceptible soils:

$$\text{if} \quad P_u = \frac{P_{90}}{P_{70}} < 6, \text{ the soil is FS}$$

$$\text{if} \quad P_u = \frac{P_{90}}{P_{70}} > 6, \text{ the soil is NFS}$$

in which P_{90} is the effective pore diameter larger than 90 percent of all pores (by volume) and P_{70} is the effective pore diameter larger than 70 percent of all pores (by volume).

Criteria Based on Heaving Pressure Developed During Freezing

Laboratory and field experience have shown that heaving pressures are developed in frost-susceptible soils during freezing in a condition of confinement or restraint. The pressures are of sufficient magnitude to crack pavements, distort retaining walls, and lift foundation footings and columns. The phenomenon of the apparently simultaneous existence of heave pressure in the soil/ice structure and soil suction (subpressure in the void water at the freezing front) seems contradictory to many engineers and laymen. Nevertheless, it can be explained by the fact that for an ice lens to grow, and thus exert heave pressure, it is necessary that a thin quasi-liquid film separate the soil particles from the ice; the water in the film is absorbed to the soil particle, and its properties are affected by the attractive forces between the water molecules and the soil surface. As water molecules in the film freeze, new water molecules are attracted from below to replace those lost. This creates a suction, or reduced pressure, immediately below the quasi-liquid film.

Laboratory studies and field observations and tests (*43, 54, 14, 55*) show that the heaving rate and total heave of frost-susceptible soils may be reduced by surcharge loads. Linell and Kaplar found that a modest load of 6 psi (41 kPa) (equivalent to 6 ft—1.8 m—of overburden) reduced heave rate 80 percent in laboratory tests (see Fig. 21). Aitken (*55*) demonstrated the validity of the laboratory results in field pavement test sections. If sufficient weight of nonfrost-susceptible base course or subbase could be placed over a frost-susceptible subgrade in which a given freezing pressure is known to develop, heaving could be completely eliminated, although in practice this is not feasible because of the great thickness of base and subbase course that would be required. The principle of heave control by surcharge loading is used by one of the airfield pavement design methods of the Department of the Army (*56*). Williams (*57*) proposes this principle as a basis for utilization of the more frost-susceptible soils at greater depth, where their heaving propensity may be counterbalanced by the weight of better-quality soil above. To achieve such a layered system by a rational design, Williams proposes that heave pressures be determined for the soils

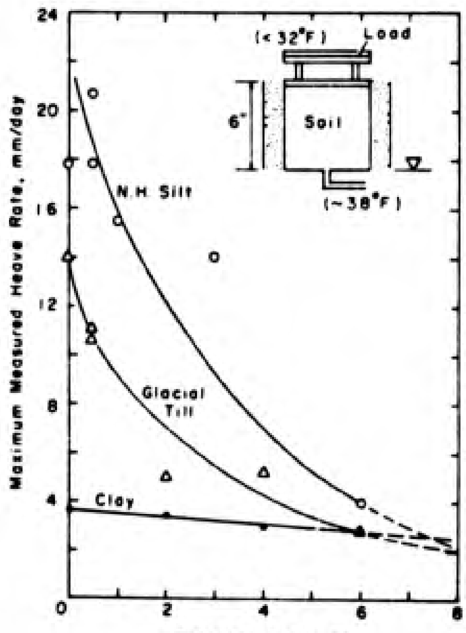


Figure 21. Experimental heave rate vs external surcharge load for several typical soils (58).

either by a laboratory freezing test or by computation from results of an air intrusion test and application of interfacial energy equations available from capillary theory.

Hoekstra, et al. (*59*), measured heaving pressures on several soils in the laboratory and showed that the heave pressure is governed by pore size. They suggested that if heaving pressures were measured in many more soils, frost-susceptibility criteria based on heaving pressure might be developed. Wissa and Martin (*60*) have attempted to devise tentative guidelines for heave pressure tests and their interpretation but recognize that field validation is required.

Use of Soil Classification System as Basis for Frost-Susceptibility Evaluation

In highway engineering the Unified Soil Classification System (USCS) and the AASHTO System (including Group Index) are the soil classification systems most widely used in the United States and Canada. In some cases the Group Index alone is used, and various other textural systems also are used. One of the conclusions drawn from results of CBR tests during spring thaw on 30 New Jersey soils (*61*) was that there is little correlation between the AASHTO classification and the reduction in bearing capacity, although generally under equivalent conditions granular soils retain the highest bearing capacities. This conclusion also can be applied to the other general soil classification systems, inasmuch as none is expressly a frost-susceptibility classification and none incorporates assessment of the various soil factors that show adequate correlation with frost susceptibility. Local or regional experience may indicate a reasonable relationship between frost susceptibility of certain soils and their corresponding classification under one of these systems. In some states (Appendix B) special adjustments have been made to the classification system adopted for use to improve its consistency with locally observed relative severity of frost effects. An extension to the USCS is available (*62*) for classification of frozen soils; it includes a method of describing the ice content that can be a useful indication of frost susceptibility.

Methods of Direct Assessment

In the early 1950's a laboratory freezing test was developed by the Corps of Engineers (*14, 52*), principally to determine the suitability of silty granular materials whose gradation, compared with the existing Casagrande criterion, caused uncertainty regarding their suitability for use as base and subbase in airfield pavements. The freezing test was also intended to serve as a basis for identification of frost-susceptible soils and for classification according to their relative degree of frost susceptibility. The test consists of unidirectional laboratory freezing from the top downward, at a relatively slow rate, of cylindrical soil specimens, 6 in. (150 mm) high and 6 in. in diameter, in an open-system test where free water is available at the base of the specimen. Specimen containers are insulated on the sides, slightly tapered inside (wider at the top), and lubricated to reduce wall friction. The heave rate is measured under a freezing penetration rate of ¼ to ½ in. (6 to 13 mm) per day. A minimum surcharge pressure of 0.5 psi (3.4 kPa) is applied to each specimen to simulate the overburden pressure of minimum thickness of 6 in. (150 mm) of pavement and base.

The freezing test thus subjects the soil to a severe combination of conditions conducive to frost heaving. The unlimited source of water corresponds to an extremely pervious aquifer only 6 in. (150 mm) or less below the freezing plane and it results in virtually the maximum possible rate of ice segregation and heave that the soil could exhibit under natural field conditions. The results do not quantitatively represent the actual magnitude of heave to be anticipated in the field. They provide, however, a satisfactory relative measure of potential frost behavior. The test has been used by the Corps of Engineers for more than 20 years for design and construction of military airfield pavements and roads. The following scale of frost susceptibility based on the average measured rate of heave in laboratory tests has been adopted:

AVERAGE RATE OF HEAVE (MM/DAY)	FROST SUSCEPTIBILITY CLASSIFICATION
0.0-0.5	Negligible
0.5-1.0	Very low
1.0-2.0	Low
2.0-4.0	Medium
4.0-8.0	High
>8.0	Very high

Figure 22 shows composite envelopes developed from plots of average heave rate versus the percentage of particles finer than 0.02 mm for all standard laboratory freezing tests made by the Corps of Engineers from 1950 to 1970. The F1 to F4 frost design classification groups were developed prior to the availability of the laboratory freezing test results shown in Figure 21, but are shown thereon to illustrate their relationship to laboratory heave rate results and the USCS classification. The data that form the basis for the envelopes in Figure 22 (*52*) reveal that there is no sharp dividing line between frost-susceptible and nonfrost-susceptible soils, nor a unique well-defined relationship with respect to percentage of particles finer than 0.02 mm. The data confirm indications of earlier investigators (*43*) that factors other than grain size influence frost behavior. The various factors and their significance have been analyzed by Linell and Kaplar (*14*) and Penner (*15*).

Based on an analysis of the results of the freezing tests and on field observations, the Corps of Engineers (*32*) concluded that the potential intensity of ice segregation in a soil is dependent to a large degree on its void sizes but 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 1½ to 3 percent finer by weight than the 0.02-mm size should be considered as possibly frost susceptible, and should be subjected to a standard laboratory frost-susceptibility test to evaluate actual behavior during freezing. Uniform sandy soils may have as much

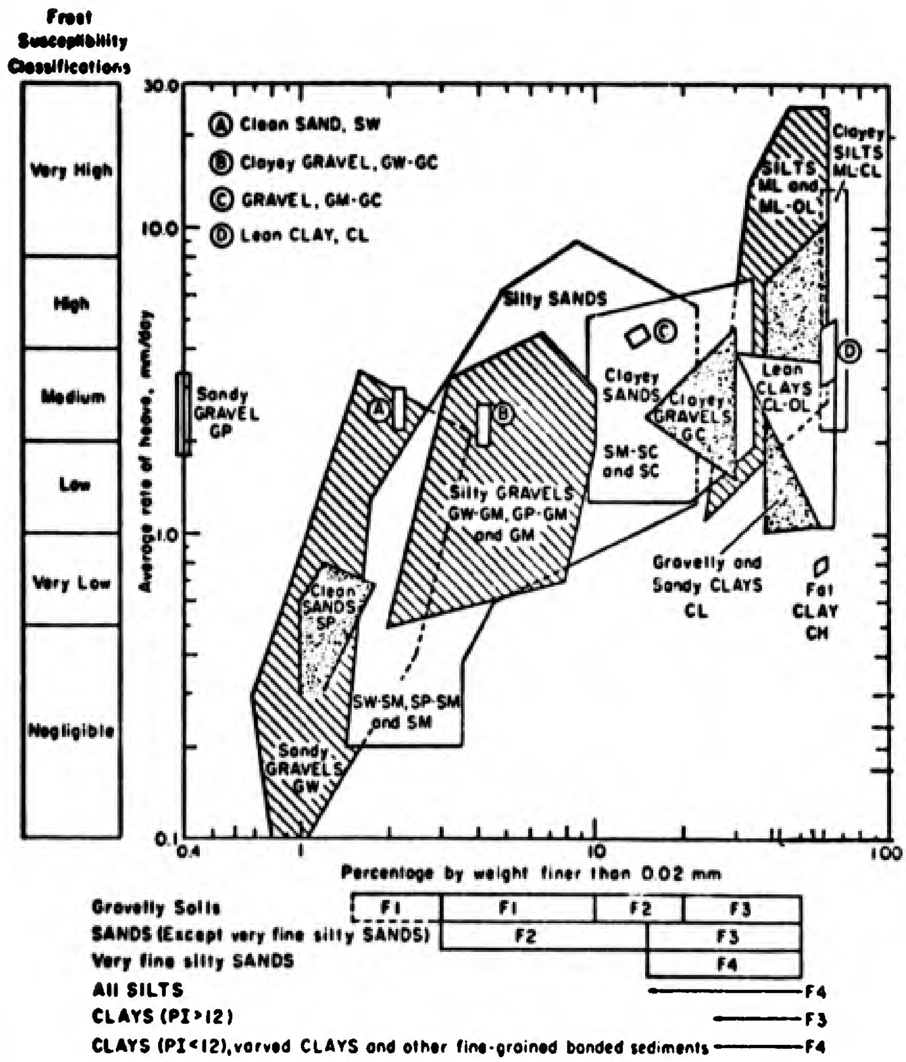


Figure 22. Summary envelope of all standard laboratory freezing tests by Corps of Engineers, 1950-1970 (52).

as 10 percent of their 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.

The British Road Research Laboratory (*63*) also adopted a freezing test, which is performed on compacted cylindrical specimens (4 in.—100 mm—in diameter and 6 in.—150 mm—high) that are frozen unidirectionally from the top downward. The specimens are insulated with dry sand and are frozen at a constant air temperature of −17°C (+1.4°F), maintained for 10 days under open-system conditions—i.e., with free water available at +4°C (39.2°F). Heave measurements are taken daily. From field experience obtained during severe winters in Great Britain, it was concluded that soils that heave in the laboratory 0.5 in. (13 mm) or less in 10 days are satisfactory and those heaving greater than that amount are considered very frost susceptible.

Recently Developed Approaches for Evaluation of Frost-Susceptibility of Soils

A principal objection to the freezing tests described in the foregoing is the length of time required for their completion (10 to 24 days after start of freezing). Several investigators have studied variations to the test procedures, as well as totally different approaches for more rapid evaluation.

Kaplar (*64*) published experimental data demonstrating that useful and reliable comparative results can be obtained by an accelerated freezing test procedure in a matter of two days after start of freezing. The method uses a constant freezing temperature rather than a decremental change of temperature. A similar procedure using a Peltier battery-type freezing cell has been developed by researchers at the University of New Hampshire (*65*) and is being evaluated by the New Hampshire Department of Public Works and Highways for use as a design tool. Wissa and Martin (*66*) also have recently developed more rapid methods of identifying soil frost susceptibility. One of the procedures, developed for the Pennsylvania Department of Transportation, utilizes as the chief criterion a measure of permeability as a function of degree of saturation. The other procedure, developed for the Massachusetts Department of Transportation, is described earlier herein in relation to heaving pressure. Both of these methods are currently under evaluation and correlation with field experience in their respective states.

Criteria Based on Stability Characteristics During Thawing and Recovery

Most of the foregoing criteria for assessment of frost susceptibility derive principally from considerations of potential frost heave caused by ice segregation. Excepting the Corps of Engineers FS groups, which are primarily an index of susceptibility to thaw weakening, and only approximately of heave potential, loss of supporting capacity during thaw is not the primary basis for the criteria. Yet, as noted earlier, in many regions frost heave is not a serious problem whereas thaw-weakening is a prime factor in pavement design. Some soils exhibit thaw-weakening to a degree roughly proportionate to their heave; on the other hand, clays, because of their low permeability, heave much less than silts but may be severely weakened during thaw and may also be slower in regaining the normal stability. In recognition that thaw-weakening is an essential design parameter, the Corps of Engineers FS groups are derived from traffic testing during the spring thaw period. Laboratory tests of stability during thaw, which can be performed more expeditiously, could also serve as a basis for identifying frost-susceptible soils. Jessberger and Carbee (*67*) have suggested that the reduction in laboratory CBR value caused by frost action can be used directly as a scale of classification of frost susceptibility. They ran CBR tests on samples of 17 soils that had been frozen and thawed for several cycles in an open-system test. They found large reductions in CBR values, caused by freezing and thawing, in many of the specimens tested. They concluded that CBR tests immediately after thawing, on specimens that had been subjected to freeze-thaw cycles, can be used to modify the established frost-susceptibility criteria based on the fraction finer than 0.02 mm.

Other studies of post-thaw strength based on unconfined compression, triaxial compression, and direct shear tests (*68, 69, 11, 70, 71*) have not been directed toward development of frost-susceptibility criteria but indicate increasing interest and research activity in this important topic.

Comments and Concluding Remarks Regarding Frost-Susceptibility Criteria

The general meaning of the term frost action in soils alludes to both heaving and thaw-weakening effects, both of which stem in part from ice segregation. However, thaw-weakening is not necessarily directly proportional to heaving because field experience shows that thaw-weakened but well-drained sandy gravelly materials recover bearing strength quite rapidly, whereas clayey soils may show little heave but recover stability very slowly over a prolonged period of time. These manifestations point out the need for separate classifications of frost susceptibility, one based on heave susceptibility and the other on thaw-weakening. Future emphasis of research is expected to be directed toward developing separate criteria for each condition.

Assessment of the relative worth of the various frost-susceptibility criteria requires an examination of the performance of many pavements and comparison with the performance that could be inferred from the frost susceptibility of the soils according to the various criteria. Yet the soil factor is only one of the key parameters that determine the performance of a pavement, and quantitative analysis is exceedingly difficult. Nonetheless, further field validation studies appear essential, because even the criteria having the widest use, based on the percentage finer than 0.02 mm after Casagrande or the Corps of Engineers, have as their principal validation the good performance of pavements, showing that frost-susceptible soils are being identified and excluded or effectively counteracted. Additional evidence is needed to determine whether acceptable soils are being excluded. The same limitation on existing field validation studies is applicable to direct assessment by freezing tests. Direct assessment of frost susceptibility by freezing tests is less likely to result in rejection of soils that would perform satisfactorily, but further field validation still is essential because at present it is not possible to predict the frost heaving that will develop under field conditions.

The various proposals suggesting the use of heave pressure tests, permeability as a function of degree of saturation, suction measurements, void-size distribution determination, and rapid freeze tests for identifying potential frost-susceptible soils have not been correlated with field behavior or have received extremely limited field validation. Some of the proposed techniques also require more expensive, sophisticated equipment and highly trained technicians, and represent an increase in complexity over the relatively simple procedures of sieve analysis and hydrometer analysis, which themselves have failed to be adopted by many jurisdictions because they are considered to be too difficult and time consuming.

Thus, it appears that all of the test techniques available today and under development may still be too complicated and time consuming for wide acceptance by highway departments as design and quality-control tools. A great need exists for simple and rapid techniques for frost-susceptibility evaluation of soils—techniques that are adaptable to a wide range of soil gradations and that can accommodate testing of variables such as thermal regime, surcharge load, density, moisture content and availability, and additives, including, if possible, simple comparisons that can be made quickly in a field laboratory during construction. It is doubtful that the conflicting needs of simplicity and adequate appraisal of numerous variables can be met in a single test technique. Accordingly, solution of the problem of characterizing the susceptibility of soils to frost action probably will require also the acceptance by highway engineers of more complex testing and analytical procedures.

Sources of Water

Sources of water contributing to frost problems in pavements can be separated into the two broad categories of surface water and subsurface water (Fig. 23). Surface water enters the pavement primarily by infiltration through surface cracks and joints, and through adjacent unpaved surfaces, during periods of rain and melting snow and ice. Saltwater solutions may enter the pavement systems during periods of melting. Many pavements are not entirely impermeable to moisture, and moisture intrusion may also occur through the pavement itself during rainstorms and periods of melting snow and ice. The volume of moisture transmitted by this mechanism is related to the permeability of the pavement and the duration of the ponding of moisture on the pavement surface. Cedergren (*72*) outlined methods of estimating the permeability of pavements.

There are three primary sources of subsurface water. The first, and generally most important to the design of roads, is from the groundwater table. If the groundwater table is near the surface, it is normally considered that an unlimited supply of water may be available to the freezing front; hence, the impact of a high groundwater table on road design generally is to dictate that the grade line be established high enough to be well above its direct influence. Although the normal watertable of a specific area may be too deep to comprise a ready source of water for ice segregation at the freezing front, perched watertables may exist on some intervals of a roadway due to topographic and/or subsoil conditions. The second source comprises moisture held in the voids of the soil or drawn upward from a watertable by capillary forces. Moisture from this source may adversely affect a pavement even though the grade line is elevated and subgrade drains are provided. The third source is moisture that moves laterally beneath a pavement from an external source; in some cases, such as a road located in a sidehill cut that intersects pervious strata, a significant volume of moisture may enter the pavement profile by this mechanism.

Thompson (*35*) and Dempsey and Thompson (*124*) provide thorough discussions of the engineering aspects of moisture flow in pavement systems. Aitchison (*73*) and the Organization for Economic Cooperation and Development (*74*) present results from several recent research studies on moisture and its movement in pavement systems. Prior to these publications, the most important publication on this subject was *Highway Research Board Special Report 40* (*75*), which summarized the state of the art at that time.

Jumikis (*76*) studied three modes of moisture transfer in soils: vapor diffusion, film transfer, and capillarity. Figure 24 shows schematically the various moisture transfer modes and relative magnitudes of each at various porosities of a soil. The importance of each mode is influenced primarily by the soil type, degree of saturation, and density of the soil. The three modes of moisture transfer may be present simultaneously with hydraulic and gravity potentials beneath various segments of a pavement. Hoekstra (*77*) reported that moisture movement may occur in some soils after freezing. His studies indicated that the rate of moisture migration in the frozen soil is temperature dependent, and he postulated that it is also dependent on the gradation of the soil. Mass transfer in frozen soils is considered to be primarily by film transfer along the surfaces of the soil particles.

It is evident from the number of modes of moisture transfer, the number of potentials inducing moisture transfer, the number of soil properties influencing moisture transfer, and the interaction of all of these properties that movement of soil moisture is a very complex phenomenon.

Owing to the complexity of moisture movement in soils, the phenomenon is not clearly understood and methods of predicting moisture flow coupled with heat flow have only recently been emphasized. Thompson (*35*) summarizes the state of the art as follows:

> The prediction of the moisture conditions in a pavement system for a given time, climate, and topographical location is a complicated matter. The multitude of the variables involved and uncertain boundary conditions tend to limit the conclusiveness of both the field measurements and the rational or theoretical type of approach. Although there are deficiencies in the present knowledge of moisture movement and moisture equilibria in pavement systems, substantial understanding of moisture conditions in soils has been gained from research in hydrology, agriculture, and soil science. It is evident that moisture research contributions made by other fields will aid in the study of moisture in pavement systems.

Interaction of Climate, Soil, and Water

The interaction of environmental parameters and the freezing soil system is very complex and not thoroughly understood. The three conditions of climate, soil, and water listed in the first section of this chapter and discussed in the preceding paragraphs must be present simultaneously for detrimental ice segregation to result from frost penetration into the soil. For given conditions of heat flux at the air-pavement interface, determined by the climatic conditions at a specific site, the thermal properties of the materials within the pavement system and the subgrade control the rate and depth of penetration of freezing temperatures. The thermal properties also have significant influence on the rate and magnitude of frost heaving. The most important soil thermal properties are:

1. The *volumetric heat capacity*, defined as the quantity of heat required to change the temperature of a unit volume of soil by one degree.
2. The *volumetric latent heat of fusion*, which is the quantity of heat required to freeze (or melt) the water in a unit volume of soil without changing the temperature.
3. The *coefficient of thermal conductivity*, which is the quantity of heat flow in a unit time through a unit area of a soil caused by a unit thermal gradient.

Each of the thermal properties is influenced by the type of soil, its density, and its moisture content, as well as other parameters that generally are of secondary importance. Of the three thermal properties, the volumetric latent heat of fusion and the thermal conductivity are most important and generally have opposing influences on the depth and rate of frost penetration. For example, an increase in soil moisture content tends to increase the volumetric latent heat of

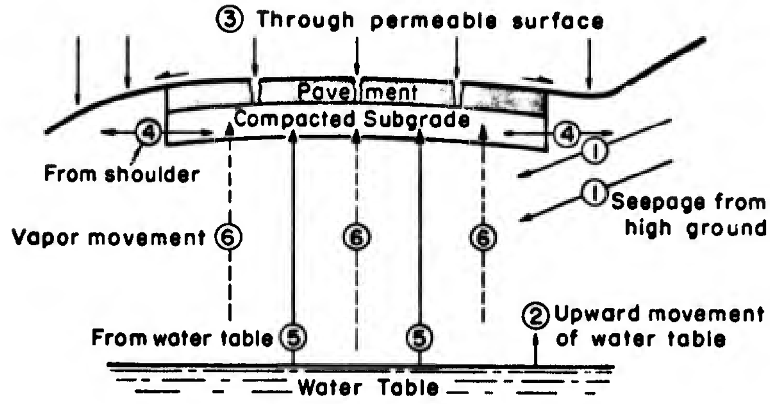


Figure 23. Sources of water contributing to frost problems in pavements (63).

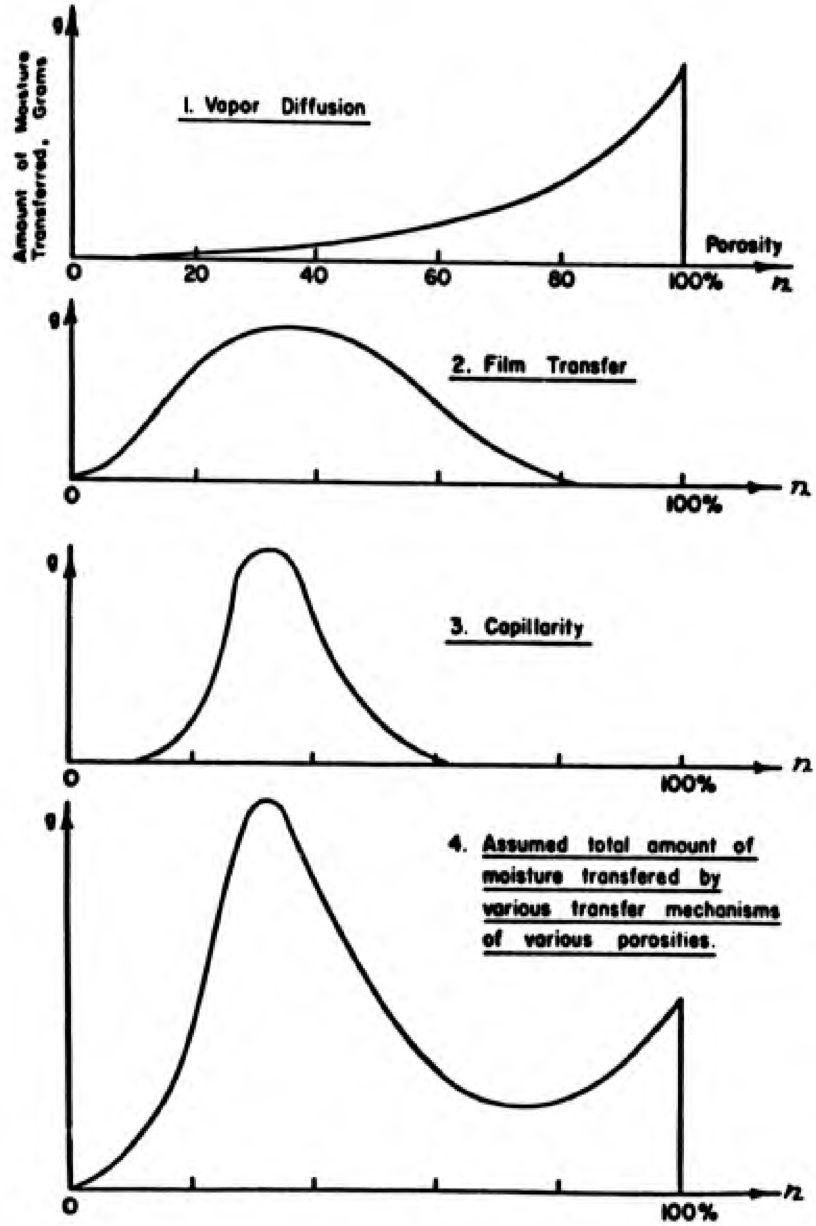


Figure 24. Various moisture transfer mechanisms and their assumed effects on the amounts of moisture transferred (76).

fusion, which reduces the frost penetration. However, the increase in moisture content also normally increases the coefficient of thermal conductivity, which tends to increase the depth of frost penetration. Because the increase in volumetric latent heat of fusion is normally greater than the increase in thermal conductivity, frost penetration tends to advance more slowly due to an increase in moisture at the freezing front. In many soils significant redistribution of moisture occurs as the frost front advances; thus, the soil moisture is not in static equilibrium during freezing, but rather is a very dynamic factor influencing frost penetration. This is one of the factors contributing to the complexity of the process, and one of the reasons why the movement of moisture in freezing soils is difficult to express quantitatively.

The conditions most conducive to pavement distress due to frost action typically occur in the late winter when the frost front is advancing very slowly and frequent diurnal and longer-term freeze-thaw cycles are experienced. Because it is near the end of the freezing season, near-maximum differential heaving will have occurred and pavement roughness and distress are more apparent than during earlier portions of the winter. Often, a slowly advancing frost front causes the greatest magnitude of frost heaving because moisture below the freezing front has the maximum time to move upward. Laboratory tests (*58, 78, 79*) indicate that as the rate of frost penetration increases the heave rate of a particular soil increases to a maximum, and then decreases at higher freezing rates (Fig. 25). At very high rates of advance of the freezing front, mobilization of water is apparently not possible and the frost heave rate corresponds to the volume change associated with changing the void water to ice.

A common problem in seasonal frost areas is that of restriction of drainage by frozen soil. As thaw progresses, the moisture that may have accumulated in the base, subbase, and subgrade is released. If it could drain downward and/or laterally, little loss of strength would result. However, during thawing periods vertical and lateral drainage may be blocked by frozen material. Consolidation of the frost-loosened subgrade is thus hindered, and the base and subbase materials also may become weakened by excess moisture and pore-water pressure, leading to accelerated pavement distress. Figures 5 and 7 illustrate the possibility of drainage blockage due to frozen layers. Figure 7 shows that thaw may progress more rapidly near the center of a roadway, creating a ponding effect. The more rapid thawing near the center of the roadway is caused, in part, by the insulating effect of the snowbanks that accumulate on and near the shoulders.

To date most methods used to estimate seasonal frost depths have been closed-form solutions that could be solved with a slide rule or a desk calculator. The Stefan and modified Berggren equations are typical examples of these techniques. By necessity, simplifying assumptions and boundary conditions were used in developing these solutions. Although they were developed from rather simplified and idealized models, these procedures have been very helpful in providing estimates of frost depths for design purposes.

The widespread availability of high-speed, large-capacity digital computers has provided a mechanism for applying the basic heat and mass transfer laws to frost penetration and frost-heaving problems. Currently the restraint on the accuracy of predictions using numerical solutions is the state-of-the-art knowledge of the physical processes involved. The individual processes of heat and mass flux are relatively well understood, but when they are coupled, as in a soil-water system undergoing freezing temperatures, many of the basic physical relationships become complex and are currently not adequately understood. Research studies are under way to provide better relationships for these parameters. When using these relationships to correlate with field observations, the soil, moisture, and climatic conditions must also be considered.

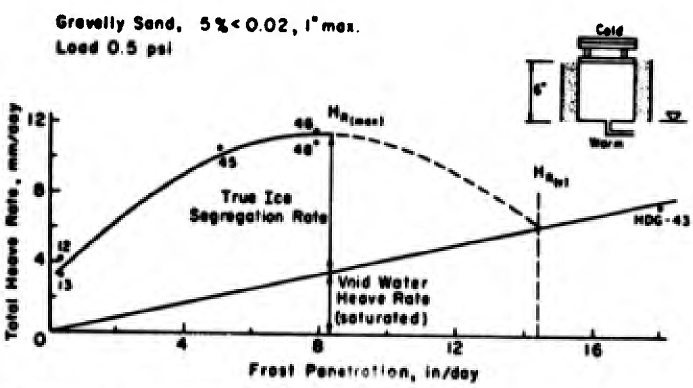


Figure 25. Heave rate vs rate of frost penetration (58).

CHAPTER THREE

THE PROCESS OF DESIGN

SURVEYS OF SOILS AND MATERIALS

The soil factors that contribute to frost problems have been discussed in Chapter Two. This chapter, then, discusses the design of pavement structures and the soils and materials parameters used in design. Between the knowledge of which soil factors are important in roadway design for seasonal frost areas, and the use of these soil parameter values in the process of design, there lies the task of gathering information about the soils and materials that exist in the field. For seasonal frost areas, this task includes information acquisition on soil moisture availability, groundwater conditions, frost depth, frost heave and, where possible, pavement performance experience. This range of concerns constitutes the subject of surveys of soils and materials. Many texts, manuals, and handbooks are available that provide more complete treatment of surveys of soils and materials; a concise rendering of the topic by McAlpin and Hofmann (*80*) may be consulted.

It first should be stated that the practice of carefully surveying soils and materials in the field, determining by laboratory tests the engineering properties of the samples of all soils strata that will be exposed in the subgrade, and then designing pavement structures uniquely suited to the soils on which they will be placed, may be a logical but impractical procedure under certain circumstances. Where soil characteristics vary with great spatial frequency, as in some glaciated terrains, it is virtually impossible for a survey to characterize accurately and at acceptable cost the distribution and limits of the various soil types and the characteristics of each. Even if this were practical, the design and construction costs of the resulting roadway, with pavement sections that change every few feet or tens of feet along the alignment, would be unacceptable. In these special circumstances it may be more economical, and may lead to much more serviceable highways, to establish designs for sections of road perhaps a mile (1.6 km) or more in length, based on the least-favorable soil types encountered in each section (*81*).

Types of Surveys

Although all surveys of soils have the objective of gaining information about materials and conditions as they exist in the field, there are wide variations in the immediate purposes for which the data are required. Thus, there is a wide range in the degree of detail a survey may have. Corresponding to the varying degrees of detail required, there are differing approaches for gathering the data. It is convenient to represent surveys as being of three separate types: *reconnaissance*, *preliminary*, and *final*, although these categories really encompass a continuum from the most generalized to the most detailed collections of data on soils, materials, soil moisture, groundwater conditions, and seasonal frost phenomena.

The ideal example of a reconnaissance survey is that which provides soils information for the initial route selection of a highway through virgin territory. In such a case, only generalized information is needed, so that the locations of suitable soils and materials can be distinguished from areas lacking these or having clearly undesirable ground or moisture conditions. Then, in combination with the other constraints acting upon route selection, this information can contribute to the initial choice for the highway location. More commonly, a reconnaissance survey is made to arrive at initial cost estimates for highway construction where the routing is already largely determined by other factors, or where major reconstruction or rerouting is planned to upgade an existing highway. In general, the purposes of a reconnaissance survey are to outline where the major engineering problems will arise, what types of designs will be required, and where the required materials will be found. A reconnaissance survey is not expected to provide the actual soils or materials parameters that are to be used in the design process.

A good share of the input data for the preliminary design phase comes from the preliminary survey of soils and materials. Unlike the reconnaissance survey, which is an areal or regional survey, the preliminary survey focuses on the soils, materials, moisture conditions, and expected response to seasonal frost along the preliminary alignment of the highway. Greater definition of the soil types is made, and the extent of particular problem areas is more closely established. The preliminary survey usually includes direct sampling of subsurface materials, but only to the extent of providing one or a few representative samples of the major soils or materials that are encountered along the preliminary line, and making a preliminary assessment of the frost penetration depths. As the preliminary design progresses, gaps in subsurface information are identified, and these gaps either are filled by acquiring further information from the field or are earmarked for special attention during the later survey and design phases. It is in the preliminary survey and design phase that final judgments are made concerning possible relocations to avoid particular problem areas, and that the final alignment becomes established.

The final survey of soils and materials seeks to provide all of the information that contributes to the final design. The detail of the final survey is great enough to allow determination of all the materials characteristics and parameters upon which the pavement design is based, as well as the governing groundwater conditions, soil moisture availability, frost penetration, and frost heave. This does not mean, however, that all parts of the line are surveyed in equal detail. Instead, the portions of the line having poorer or variable soil or water conditions receive greater definition than long stretches of homogeneous favorable materials with a deep watertable. Although the final survey is, in many respects, a more detailed version of the preliminary survey, there is a significant philosophical difference. Because the final objective is a satisfactorily operational road, the final survey does not come to a discrete termination but remains open-ended throughout the design and construction effort. Thus, as the final design progresses additional field samples may be needed from critical locations. In addition, as construction activities bring the line up to grade verification is made that the soils and moisture conditions encountered are as expected, and unexpected conditions are recognized and sampled to provide a basis for possible design changes.

Methods of Soil-Type Determination

Many different methods are used to acquire soils and materials information. These methods vary greatly in their accuracy and detail; thus their applicability depends on the type of survey being considered. It is useful to group the various methods into the following scheme: (1) methods that are largely reliant on preexisting data (e.g., pedological information, geological information); (2) methods that are primarily indirect assessments of soils and materials (e.g., airphoto interpretation, remote sensing techniques); and (3) methods that involve direct field contact with the surveyed area (e.g., geophysical methods, direct sampling).

Pedological information, in the form of agricultural soils maps, is available for much of the United States and the populated portions of Canada. Although these maps are prepared from studies having purposes quite different from highway engineering, they do provide information that is relevant to the soils explorations engineer: soil composition, texture, variations with depth, soil moisture, depth to groundwater, drainage characteristics, etc. This information is not presented in traditional engineering terminology, but several highway design agencies have made significant advances in using pedological classifications as a means of inferring the engineering properties of soils. For example, Michigan has made use of pedological information for more than 50 years, and has extensively developed the techniques of applying this information in highway engineering practice. Similarly, Wisconsin has done much work in correlating pedological soil series with major series units according to engineering properties, drainage, topographic and geologic features, pedological classification units, and engineering uses. As another example, Nebraska uses published pedological soil survey maps for programing purposes and for design purposes. The soils maps form the basis for planning and conducting the soils and materials survey, and they significantly reduce the field effort for locating deposits of materials for highway construction. Nebraska also employs soils maps to contribute to estimating drainage and runoff characteristics for drainage structure design.

It should be noted that older pedological soils maps are less accurate and less useful than those completed within the last 10 to 15 years. This is because soil surveying practice has evolved a great deal since it began a half century ago, expanding to serve the needs of not only agriculture but also other fields where soils information is important.

Geological information shares some similarity with pedological information. Both are most commonly portrayed in map form, usually with accompanying reports, and generally both are prepared for nonengineering purposes. Maps of surficial geology (sometimes termed "glacial geology" where glaciation has occurred) are the most valuable to the soils engineer, because they deal with the unconsolidated material lying above bedrock. Thus the surficial geologic map is a valuable tool in locating deposits of sand, gravel, and embankment material, as well as defining the materials that will form the roadway subgrade. Maps of bedrock geology have a different application than surficial geologic maps in highway engineering. Unless the area of interest is largely made up of residual soils (that is, soils that are not transported but are formed in place by weathering of the parent rock material), a bedrock geologic map will not provide information about the unconsolidated materials with which the engineer usually works. But in mountainous or hilly terrain, where rock excavation is commonly involved in highway construction, bedrock geologic maps are essential as a first step in defining the rock-removal procedures that will be required (drilling and blasting, ripping, etc.) In addition, knowledge of the bedrock types is required to assess the possibilities of using rock as fill, and the suitability and cost of crushing rock for use as granular material. Some rock types may yield highly frost-susceptible mixtures because excessive fines are produced during crushing and processing. Both surficial and bedrock geologic maps are used to identify potential geologic hazards (landslides, areas of subsidence, sinkholes in karstic limestone terrain, areas of excessively erodible soils, etc.)

Both pedological information and geological information are most suited for use in reconnaissance surveys of soils and materials, because of their areal coverage.

Chief among the methods that are indirect assessments of soils and materials is the technique of *airphoto interpretation*. By means of vertical aerial photos, especially overlapping photos that can be viewed stereoscopically to give a three-dimensional model of the earth's surface, one can see in proper perspective and arrangement all of the natural and man-made features that make up the earth's surface. For their proper utilization the photointerpreter must be thoroughly familiar with pedology and geology, while retaining the basic engineering approach to the acquisition and utilization of soils and materials information for highway engineering purposes. Through the photointerpreter's ability to recognize and discriminate among various landforms, drainage patterns, erosional aspects, photo tones and textures, vegetation types, special features, and cultural features, plus his familiarity with local conditions, he is able to make logical inferences about subsurface materials, probable soil moisture conditions, and groundwater distribution, and to map these according to the requirements of the project development stage.

The greatest degree of reliability results from a combination of interpretation and field study for verification of the mapping. This latter phase is known as ground-truth acquisition. Probably no other method for determining soil type or moisture relationships depends so heavily on the cognitive skills of the persons involved than does airphoto interpretation. Consequently, many highway design agencies have not been able to acquire the personnel needed to make airphoto interpretation a standard procedure that can be called upon when required. This is the more unfortunate because aerial photographs are commonly acquired specifically for highway design purposes, but are used only for the photogrammetric production of topographic maps. However, several agencies do successfully employ photointerpretation for soil survey purposes. In such cases, airphotos usually are applied in the reconnaissance and preliminary survey stages.

It should be emphasized that not only soil mapping is possible with photointerpretation; airphotos also provide an efficient means to assess general soil moisture and drainage conditions, and to identify potential geologic hazards. These tasks are commonly done more effectively with airphotos than with pedologic or geologic maps. The Soil Conservation Service has prepared an Agricultural Handbook (*82*) which serves as an introduction to soil mapping via photointerpretation.

A principal example of the use of airphoto interpretation is the statewide series of engineering soils maps prepared for New Jersey during the period from 1947 to 1955 (*83, 84*). This effort relied primarily on photointerpretation, but made extensive use of agricultural soils maps, geologic maps, land-use maps, other published data, field examination, soil sampling, and laboratory testing. The resultant product comprises 20 separate county reports, plus two over-all documents describing the work and discussing its potential applications.

Beyond airphoto interpretation, *other remote sensing techniques* that may be applicable to soil-type determination include multispectral imaging, thermal infrared scanning, side-looking airborne radar (SLAR), airborne resistivity, and microwave radiometry. Applications of these various techniques are in the developmental stage; none can be said to be a part of highway engineering practice at present. However, several recent and ongoing studies indicate that under certain conditions these approaches may facilitate soil mapping, soil moisture determinations, qualitative estimates of depth to groundwater, and detection of certain anomalies such as subsurface voids. Recent publications (*85, 86*) provide an insight into the efforts now being made to provide soils engineers with advanced remote-sensing tools. Computer processing of the data, digitizing, density-slicing, electro-optical techniques, etc., promise to automate portions of the interpretive process, but the judgment of the interpreter remains the fundamental ingredient in remote-sensing technology.

Among the direct field methods that may be applied to the survey of soils and materials, *geophysical methods* offer the possibilities of rapidly inferring soil types and depths to interfaces between soil types, watertables, or bedrock surfaces. The principal methods are seismic refraction and earth resistivity techniques; other techniques using acoustic or electromagnetic energy have been proposed and are under development and testing, but are not yet included in highway engineering practice.

In seismic refraction, the dispersal in the ground of shock waves created by small detonations or mechanical impacts is monitored by detectors placed in a line along the ground surface. The data acquired are analyzed to yield values for the velocities of seismic energy in the earth materials, and the depths at which changes in velocity occur, indicating changes in materials or moisture contents. The analyzed data are interpreted to yield conclusions concerning the soil types making up the various layers and the thicknesses of the layers. Supplemental direct sampling of the earth materials enhances the reliability of the inferences. Seismic refraction works only where the seismic energy velocity increases with depth. Thus, seismic refraction cannot be used to determine the thickness of the seasonal frost layer, because the seismic energy velocity in frozen ground is higher than in the unfrozen ground beneath.

Earth resistivity techniques rely on the differences of various earth materials to conduct an electrical current. An electrical current is imposed in the ground through two electrodes, and the potential drop between two other electrodes is measured, all electrodes being arranged in a straight line. Varying the spacing of the electrodes yields data from which the depths where resistivity changes occur can be inferred. Shifting the entire electrode array allows determination of horizontal variations in the resistivity of the soils. Earth resistivity is much more reliant on direct sampling for accurate interpretation than is seismic refraction; samples must be obtained and measured for their electrical properties, because these properties are variable from place to place and from time to time for a given soil type, largely due to the variability in moisture content. In theory, resistivity techniques could be used to determine the thickness of the seasonal frost layer. But because this layer is relatively thin, a close electrode spacing would be required and the technique would become impractical over a significant area or length of route.

Certainly the most reliable approach to surveys of soils and materials is by *direct sampling*. However, direct sampling requires careful extrapolation, and, as stated previously, certain ground conditions limit the value of direct sampling for highway design purposes. The techniques of direct sampling offer generally unambiguous determinations of soil type, moisture contents, and groundwater depths. Frost depths may be determined by direct sampling, but such measurements may not be directly indicative of frost depths beneath the proposed pavement section. Direct sampling techniques are the only ones that provide the basis for laboratory determinations of the discrete parameters essential to the process of design. At the same time, direct sampling is more expensive and time consuming than the other methods described, thus explaining the use of the other methods as supportive of direct sampling programs.

Because the various procedures of direct sampling are assumed to be well-known and are thoroughly discussed in other publications, they are only mentioned here for informational purposes. Included are probing (which yields no samples), auger borings by either hand or mechanical means, or drive sample borings (which yield samples disturbed to varying degrees, depending on the equipment used), tube sample borings or core borings (which provide undisturbed samples), and test pit excavations (which permit large undisturbed samples to be acquired, plus a visual inspection of the entire soil profile).

Measurement of frost penetration depths has been discussed in Chapter Two under "Climate." Actual frost heave may be measured by reference to stable benchmarks. Where pavements already exist, their performance experience may be documented. In seasonal frost areas, all these activities may be construed as being a part of surveys of soils and materials.

TRAFFIC ANALYSIS

For pavement design, traffic analysis consists of determining the axle loads and the number of applications of each expected in the analysis period, and in organizing the data from a mixed traffic stream to serve as input parameters to the equation used for structural design of the pavement. Collection of traffic data and its projection to account for growth during the analysis period is the same for seasonal frost areas as for areas not experiencing frost. For pavements on which special spring load restrictions will be imposed, the seasonal reduction in axle loads must be accounted for in their design. Otherwise, organization of the data is the same in seasonal frost areas as elsewhere; the two principal approaches employed are described briefly in the following, because these approaches influence the design process.

Analysis of Equivalent Loads

In the analysis-of-equivalent-loads method the procedure is to convert the stream of varying axle loads to a common denominator and express the projected traffic as the sum of the projected numbers of applications of each of the converted loads. This method is used by AASHTO (*87*), The Asphalt Institute (*88*), the Corps of Engineers (*89*), and many states and other agencies. The common denominator may be an equivalent 18-kip (8,200 kg) single-axle load; an equivalent wheel load of 9 kips (4,100 kg), 5 kips (2,300 kg), etc.; or some other suitable wheel or axle loading. Conversion of each axle load to the selected equivalent axle load is accomplished by means of traffic equivalence factors. The various agencies have differing scales of such factors, and in some cases the factors are dependent on the particular pavement structure that is chosen.

Analysis of Individual Load Classifications

In the analysis-of-individual-load-classifications method the traffic data are grouped conveniently by magnitude of axle or wheel loads. Under the design procedure of the Portland Cement Association (*90*), single-axle loadings are grouped in ranges of 1,000 to 3,000 lb (450 to 1400 kg) and tandem-axle loadings are similarly grouped, with the number of applications of loads in each group being estimated during the period of analysis. Various means are employed to account for the growth of traffic in each group. To design the pavement one calculates the stress under an application of an average load in each group, and then calculates the fatigue damage that occurs under the projected number of applications of all the loads within each group. Thus, there is no need to make use of a common denominator to convert the various axle loads to some selected equivalent loading; instead, the various loadings are analyzed separately.

This method can also be applied in the design of flexible pavements. Bergan and Monismith (*91*) have shown how a mixed traffic stream can be handled in a fatigue analysis of a flexible pavement system. The traffic loadings were classified in convenient load groups, similar to those of the Portland Cement Association, and from traffic counts the number of applications in each group was determined. The fatigue damage for each group was then analyzed separately, as a percentage of the total fatigue life.

It appears that in any design system that accounts for repetitions of loading, either for the analysis of fatigue or pavement distortion from traffic loads, separate analysis of the effects of each group of traffic loadings is the preferred method, and probably will be more widely used in the future.

GEOMETRIC DESIGN

Frost action in soils, and the severe winter weather conditions that prevail in frost areas, have had some degree of influence on certain aspects of the geometric design adopted by the various responsible agencies. Little information on this subject was obtained from the survey, however, because it was considered to be of marginal interest. Nevertheless it appears that some of the effects on geometric design may be grouped as follows:

1. In frost areas special consideration must be given to *surface and subsurface drainage*. Examples: In frost areas the cross section would be affected by the need to provide for drainage of the base and subbase and to carry the ditch bottoms at grades lower than the subgrade, and the grade line by the need to maintain at least a certain minimum height above the seasonal high groundwater table. General aspects of drainage are outlined in a succeeding paragraph.

2. For adequate *pavement bearing capacity* the grade line may have to be established at not less than a certain minimum height above a highly frost-susceptible soil stratum.

3. *Pavement slipperiness* caused by ice or snow necessitates lower maximum longitudinal and transverse grades and also affects the horizontal and vertical curvature adopted to afford the advisable minimum stopping distance.

4. The cross section conventionally adopted for low-volume roads may have *shoulder widths* that are inadequate for storage of banked snow plowed from the pavement and may have to be increased; also the grade line may need to be raised to minimize snow drifting on the pavement and to facilitate plowing.

5. The *side slopes in cuts and fills* are affected by frost melting and may have to be made less steep to insure year-round stability.

Some of these topics are covered elsewhere in this report; others are not mentioned further but are adequately covered in readily available highway engineering reference books.

FLEXIBLE PAVEMENT DESIGN

The flexible pavement design process is very complex. The foremost distress modes caused by traffic loading (Table 1) are fatigue failure (evidenced by alligator cracking in the wheelpaths) and wheelpath rutting, both caused by repeated loading; and plastic flow, creep, or shear failure caused by a few excessive loads. Pavement distress of the types is of equal importance in seasonal frost areas and elsewhere. It has been found (*92, 93*) that a significant indicator of distress by wheelpath cracking is the maximum tangential strain at the bottom of the asphalt-stabilized layer. The capacity of a pavement to resist rutting by consolidation under repeated loads or by excessive plastic flow or creep is related to the vertical compressive strain in the top of the subgrade (*92*) and to the radial stress at the bottom of an unbound granular base course (*94*). A sensitivity analysis (*95*) showed that both the vertical subgrade strain and the fatigue life itself are strongly influenced by the stress-strain properties of subgrade, base course, and asphaltic concrete pavement. Accordingly, an essential phase of the design process is the characterization of the materials that will be used within the layered structure. Most pavement design methods in current use are essentially empirical. To characterize subgrade soils and unbound base materials, they make use of (a) measures of resistance to penetration by a loaded piston, (b) resistance to lateral deformation when loaded axially, or (c) index properties; to characterize bituminous mixtures, they make use of (a) resistance to plastic flow, (b) resistance to deformation, or (c) compressive strength.

The nontraffic-associated distress modes that are of special concern in the cold regions are contraction cracks caused by low temperatures (influenced by the stiffness-temperature relationship for the asphalt-bound layers), and distortion from differential heave caused by frost action and from subsequent reconsolidation (influenced mainly by the properties of subgrade and unbound base materials). Other modes of distress that are not associated with traffic—caused by moisture changes, shrinkage (causing reflection cracking), differential heave caused by swelling clays, and differential settlement caused by consolidation in the subgrade—may be important in seasonal frost areas or elsewhere. Pavement distress of these nontraffic-associated modes is influenced most strongly by the properties of one or more of the materials comprising the layered system, and by environmental conditions.

Accordingly, the first topic considered under this heading is the characterization of the various materials under current design practices of the agencies surveyed. This is followed by the methods of treatment of environmental factors, and by the types of pavement response and performance models (design equations) used to determine the pavement sections that will resist the traffic loads. Special design provisions used to control frost heave, low-temperature cracking, and disintegration are summarized. Finally, comments are presented regarding methods of treating the cumulative aspects of damage to flexible pavements caused by traffic loads.

Characterization of Materials

The response of materials within a flexible pavement system under wheel loads has the two components of (a) strength, the limiting or failure condition such as fracture or slip, and (b) deformability, the stress-strain-time response before failure is reached (Table 3). Failure can occur in fracture due to excessive tensile loads induced by traffic or thermal effects, in fracture due to repetitively applied tensile loads less than the ultimate tensile strength, or in shear. Tests to characterize the pertinent material properties would measure tensile strength, fatigue strength, and shear strength. The stress-strain-time response under loads short of failure includes recoverable and nonrecoverable (permanent) deformations. The principal means for characterizing such response is through use of a modulus defined as stress-strain, and Poisson's ratio. The modulus is highly dependent on levels of stress, loading time, temperature, loading and freeze-thaw history, moisture conditions, or all of these, depending on the type of material. Consequently, tests used to define the modulus need to simulate adequately the real-world conditions (*96*).

Although moisture and temperature affect directly the response of some types of materials to traffic loads, and consequently must be included in the real-world conditions being simulated by tests, both of these environmental factors can also cause pavement distress not attributable to traffic, including cracking associated with thermal and moisture changes and distortion caused by differential heave or settlement. Accordingly, it is quite essential not only that the expected range and cycles of these environmental factors be superimposed on the stress states and other conditions of tests for strength and deformability (Table 3), but also that tests be made to characterize the volume changes in pavement materials induced by environmental changes superimposed over the stress states that are expected to prevail in the pavement.

For development of a mechanistic (sometimes termed "rational") method of pavement design, characterization of materials by the fundamental approaches described in the foregoing is essential. Both the mechanistic approach to pavement design and fundamental methods of characterizing materials are now almost exclusively within the domain of research, whereas the practice of roadway design follows criteria developed essentially empirically. For use in these empirical procedures, the characterization of materials is by arbitrary test procedures that have been found suitable and practical for production testing in the design agencies' laboratories. Because fundamental properties are not measured, the adequacy and validity of current methods of characterizing materials cannot be judged independently of other criteria and procedures, but only by the performance of the pavements whose design is the product of a particular combination of materials characterization, assessment of environmental factors, and application of a design equation. This is but one reason for the current interest in characterization of materials by approaches more fundamental than the empirical procedures now in use.

Subgrade Materials

Table 3 gives the properties of materials believed to be significant to the performance of flexible pavements in seasonal frost areas. Those applicable to subgrades include shear strength, stress-strain modulus, Poisson's ratio, and volume change parameters. A strength parameter similar to fatigue strength may be applicable also; both strength and resistance to deformation of clays are lower under repeated loads than single loads (*97, 98*). In most cases the dependence of the deformability on load applications has

TABLE 3

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

PROPERTIES THAT SHOULD BE CHARACTERIZED	IMPORTANT FOR (AREA OF DESIGN)	NEED TO CHARACTERIZE DEPENDENCE ON TEMPERATURE, FROST, OR MOISTURE? [b]
Strength:		
Tensile (bound layers only)	Fracture under large load Slippage under braking load Thermal cracking	Temperature
Fatigue (bound layers only)	Fracture under repeated loading	Temperature
Shear	Plastic flow or shear under single or few excessive loads	Frost, moisture, temperature
Resistance to deformation:		
Modulus (stress-strain) Poisson's ratio	Analysis of stresses and strains for application to fracture and distortion modes of distress	Asphalt-bound: temperature Clean granular unbound base: moisture Clean granular cohesionless soil: none Silty and clayey soils: frost, moisture, temperature
Constancy of volume:		
Susceptibility to frost heave	Distortion by differential heave	Frost, moisture
Expansive properties	Distortion by differential heave	Moisture, frost
Underconsolidation	Distortion by differential settlement	Moisture, frost (consolidation during thaw)

[a] After Deacon (*96*).
[b] Dependence on other parameters, though highly significant, is beyond the scope of the present synthesis.

greater significance in pavement design because the stress levels in subgrades under well-designed pavements are below those at which failure under repeated loads occurs (*99*). One of the more significant research studies of the characterization of the stiffness of subgrade soils affected by freezing and thawing was reported by Bergan and Monismith (*100*). It was found that the resilient modulus (stress/recoverable strain) measured on undisturbed samples of a clay subgrade taken in the spring and tested fully thawed could be reasonably duplicated by tests on recompacted samples only if the samples were subjected to at least two cycles of freezing and thawing. Quinn et al. (*70*) also reported measurements of resilient deformation in clay developed under repeated loading starting with the sample completely frozen and then progressing through the thawing phase and continuing after it was fully thawed. The methods in use by design agencies to characterize subgrades (Table 4) are, almost without exception, either indirect measures of strength or measures or estimates of index properties, of practical suitability for the empirical design procedures in current use. Excepting the Saskatchewan Department of Highways and Transportation, which is starting to use measurements of resilient modulus determined in a triaxial test after three freeze-thaw cycles, and the Kansas Highway Commission, which uses a modulus determined in a triaxial test, none of the agencies surveyed characterizes the strength or resistance to deformation of subgrades by any of the more fundamental properties listed in Table 3.

R-value and CBR are widely used, and their advocates have achieved many successes with pavements dimensioned in accordance with CBR or *R*-value criteria. The tests themselves have the advantages of simplicity and low cost; continued use of the corresponding empirical design procedures has the advantage that both are proven by many years of experience. Accordingly, one may ask why so many of the leading North American flexible pavement engineers, from both research and design agencies, urge the adoption of more fundamental properties to characterize subgrades, usually linear elastic or viscoelastic parameters (*101*). Doubtless there are many reasons, but a principal objection to empirical bases for characterizing materials is that use of the data is limited to a particular design system for particular combinations of environmental and traffic conditions and for particular traditional types of pavement materials. To depart from the narrow bounds of the experiences from which a system was developed, for example to develop criteria for a previously unaccustomed type of base course, would require a long-term investigation to broaden the data base, by means of experimental loads, traffic testing, and other research. A second principal weakness of the widely used empirical bases for characterizing subgrades is the failure of such tests as *R*-value and CBR, as conventionally performed, to account for the dependence of the pertinent properties on frost. CBR tests are usually run on specimens that had been soaked four days, and *R*-value determined on relatively loosely compacted specimens tha'

TABLE 4

SUBGRADE CHARACTERIZATION IN CURRENT PRACTICE [a]

PARAMETER	ELEVEN AGENCIES SURVEYED IN DETAIL	OTHER 29 AGENCIES REPORTING FROST CONSIDERED IN DESIGN
General soil classification:		
AASHTO	1	Not determined
AASHTO and FS [b] group	1	
AASHTO and group index	2	
USCS	2	
USCS and FS [b] group	1	
USCS and group index	1	
FS [b] group	1	
Group index	1	
Textural	1	
Frost-susceptibility classification or criteria [c]:		
GSD,[d] 0.02-mm size after Casagrande or Corps of Engineers	7	7
Percentage passing No. 200 sieve	–	5
GSD, other	–	2
General soil classification	–	5
Soil classification and silt content	–	3
Visual identification	–	1
Judgment	1	–
None	3	5
No information	–	1
Strength, support value, or other soil index serving as basis for pavement design:		
R-value	1	9
R-value and soil classification	–	1
R-value and expansion pressure	1	–
CBR	1	6
CBR and FS [b] group	2	–
CBR and soil classification	1	2
CBR, group index, and post-thaw resilient modulus	1	–
CBR, *R*-value, or bearing value	–	1
Group index	2	–
BB [e] deflection	–	1
BB [e] deflection and soil classification	1	–
FS group	–	1
General soil classification	–	3
Triaxial modulus	–	1
Support value and soil classification	–	1
Many different parameters depending on geographic area	–	1
None	1	2

[a] From information summarized in Appendixes A and B, which also include keys to abbreviations.
[b] Frost-susceptibility classification of Corps of Engineers.
[c] Susceptibility to thaw-weakening and/or frost heave.
[d] Grain-size distribution.
[e] Benkelman beam.

had been compressed to the point of exuding moisture from them. These conditions are probably more severe than the conditions prevailing during part of the year in a pavement subgrade. But they are also much less severe than the frost melting conditions, in which a subgrade previously expanded and loosened by ice segregation is saturated with melt water. Jessberger and Carbee (*67*) showed that the CBR value determined conventionally on soaked samples is much higher than CBR values determined for the same soils after one cycle of freezing and thawing. Conventional CBR and *R*-value accordingly represent some intermediate de-[illegible]e of severity of environmental conditions and it is probably this averaging of seasonal effects that has made them serve quite well as bases for empirical pavement design methods. Although these tests do not account for thaw-weakening, it is possible to factor the transient weakened condition of the subgrade into the design equation by other means; the Minnesota Department of Highways, for example, characterizes subgrade support by *R*-value and soil classification, which are input parameters for a mathematical design model based on spring deflections measured by the Benkelman beam. Moulton and Schaub (*102*) developed a design procedure for the frost conditions of West Virginia that accounts for the reduction in *R*-value during thaw. The reduction of R-value was studied in relation to plate bearing and CBR tests and was expressed as a function of the normal R-value, the frost classification, and the water content in the saturated state (Table 5).

Other means of characterizing subgrade support (Table 4), such as soil classification and group index, rely on correlations, developed over years of experience, between subgrade classification and pavement performance. Most of the classification systems are not oriented in any special way toward the behavior of subgrades under the action of freezing and thawing; in these cases the loss of support in spring cannot be accounted for, except in a general way through additional pavement thickness based on experience. The Ontario Department of Transportation and Communications uses as soil support parameter merely a textural classification; however, the design equation is based on limiting the spring deflections to certain levels proven by experience to assure good performance. The Wisconsin Division of Highways uses group index to characterize soil support, but the conventional group-index numerical scale has been modified to downgrade the more frost-susceptible soils found in that state.

Only three agencies use a special frost-susceptibility classification, the FS group, as inputs to the design equation with the function of characterizing subgrade support. The FS groups were developed to classify the relative supporting capacities of different soils during thaw, and are determined principally from grain size and Atterberg limits. They have been correlated approximately (Fig. 22) with classification criteria developed from heave rates measured in laboratory freezing tests (*32*). Their value as parameters for selection of combined thickness of pavement and base has been proven in extensive use by the Corps of Engineers. Nevertheless, their use as sole index of soil support could be subject to criticism in the sense that whereas CBR and R-value fail to account for the most critical part of the annual cycle of subgrade supporting capacity, validation of the FS groups was based on performance of test pavements under unusually intensive traffic during the spring melting period, without consideration of the pavement deterioration that would occur at different rates during other seasons. Clearly, there is a need for an evaluation of the changing levels of supporting capacity throughout its complete annual cycle.

TABLE 5

LOSS OF SUBGRADE SUPPORT AS A FUNCTION OF DESIGN STABILOMETER VALUE, SATURATED MOISTURE CONTENT, AND FROST CLASSIFICATION [a]

CORPS OF ENGRS. FROST CLASS.	NORMAL SAT. MOIST. CONTENT,[b] w_s	VALUE OF $(R_N\text{-}R_F)$ FOR R_N OF										
		5	10	15	20	25	30	35	40	45	50	55
F-1	0.05	0.5	0.9	1.4	1.9	2.3	2.8	3.2	3.7	4.2	4.6	5.1
	0.10	0.5	1.0	1.5	2.1	2.6	3.1	3.6	4.1	4.6	5.1	5.6
	0.15	0.6	1.1	1.7	2.3	2.8	3.4	3.9	4.5	5.0	5.6	6.2
	0.20	0.6	1.2	1.8	2.5	3.1	3.7	4.3	4.9	5.5	6.1	6.8
	0.25	0.7	1.3	2.0	2.7	3.3	4.0	4.7	5.3	6.0	6.8	7.3
	0.30	0.7	1.4	2.2	2.9	3.6	4.3	5.0	5.7	6.4	7.2	7.9
	0.35	0.8	1.5	2.3	3.0	3.8	4.6	5.3	6.1	6.8	7.6	8.3
	0.40	0.8	1.6	2.4	3.3	4.1	4.9	5.7	6.5	7.3	8.1	8.9
F-2	0.05	0.9	1.8	2.6	3.5	4.4	5.3	6.2	7.0	7.9	8.8	9.7
	0.10	1.0	2.0	3.0	4.0	4.9	5.8	6.8	7.8	8.8	9.7	10.7
	0.15	1.1	2.1	3.2	4.3	5.3	6.4	7.5	8.5	9.6	10.7	11.7
	0.20	1.2	2.3	3.5	4.6	5.8	6.9	8.1	9.2	10.4	11.5	12.7
	0.25	1.2	2.5	3.7	5.0	6.2	7.4	8.7	9.9	11.2	12.4	14.6
	0.30	1.3	2.7	4.0	5.3	6.6	8.0	9.3	10.6	12.0	13.3	14.6
	0.35	1.4	2.8	4.2	5.6	7.1	8.5	9.9	11.3	12.7	14.1	15.5
	0.40	1.5	3.0	4.5	6.0	7.5	9.0	10.5	12.0	13.4	14.9	16.4
F-3	0.05	1.6	3.2	4.8	6.4	8.0	9.6	11.2	12.8	14.4	16.0	17.6
	0.10	1.8	3.5	5.3	7.0	8.8	10.6	12.3	14.0	15.8	17.6	19.3
	0.15	2.0	4.0	5.9	7.8	9.8	11.7	13.7	15.6	17.6	19.5	21.5
	0.20	2.0	4.1	6.1	8.2	10.2	12.3	14.3	16.3	18.4	20.4	22.5
	0.25	2.2	4.3	6.5	8.6	10.8	13.0	15.1	17.2	19.4	21.5	23.6
	0.30	2.3	4.6	7.0	9.2	11.5	13.8	16.1	18.4	20.7	23.0	25.3
	0.35	2.4	4.9	7.3	9.7	12.1	14.5	17.0	19.4	21.8	24.2	26.7
	0.40	2.5	5.1	7.6	10.2	12.7	15.2	17.8	20.3	22.9	25.4	27.9
F-4	0.05	2.7	5.4	8.1	10.8	13.5	16.2	18.9	21.6	24.2	26.9	29.6
	0.10	3.0	5.8	8.7	11.6	14.5	17.4	20.3	23.2	26.1	29.0	31.9
	0.15	3.1	6.2	9.2	12.3	15.4	18.5	21.6	24.7	27.7	30.8	34.0
	0.20	3.3	6.5	9.8	13.0	16.3	19.5	22.8	26.0	29.3	32.5	35.8
	0.25	3.4	6.8	10.2	13.6	17.0	20.4	23.9	27.2	30.6	34.0	37.4
	0.30	3.6	7.1	10.6	14.2	17.7	21.3	24.8	28.4	32.0	35.5	39.0
	0.35	3.7	7.4	11.0	14.7	18.4	22.0	25.7	29.4	33.0	36.7	40.4
	0.40	3.8	7.6	11.4	15.2	19.0	22.7	26.5	30.3	34.1	37.9	41.7

[a] After Moulton and Schaub (*102*).
[b] Decimal form.

The highway agencies of the Provinces of Alberta and British Columbia assess the supporting capacity of the subgrade by rebound deflections measured with the Benkelman beam. Deflections may be measured on the subgrade, for design of the layered structure, or on a partially constructed or completed pavement, in which case the deflections comprise the response of the layered system, including the subgrade, to loads. The Canadian Good Roads Association analyzed a large number of deflection measurements made throughout Canada (*103*). Deflections were measured throughout the spring, summer, and fall (Fig. 26), but the peak spring deflections comprise the critical input parameters to the RTAC design procedure except for roads on which spring axle loads will be restricted, in which case the average fall deflections govern the design. A major advantage of this method of characterizing subgrade support is that the seasonal variation in support is measured, and to some extent is factored into the design procedure. The fact that the measured deflection is a system response can be advantageous, because it gives insight into the seasonal variation of stiffness of the other layers and into the effect of the superposition of layers of different stiffness. It may also be disadvantageous, because the measured deflections, and the criteria for their use in design of conventional-type layered systems, are not easily interpreted for use in designing pavements with radically different materials or layered structures, or for pavements for greatly different loading conditions.

The dimensioning of pavement structures in three of the agencies surveyed is not dependent on subgrade soil properties; no characterization of soil support is made. Standard sections have been established as suitable for the conditions prevailing within each of the respective states, and one of these is selected in relation to projected traffic. Subgrade properties are a factor in judging the depth of undercutting and replacement with better materials; nevertheless, the question remaining for consideration is whether standard sections that perform adequately over weak subgrades are not overdesigned and unnecessarily expensive for other locations where subgrade support is stronger.

Turning to the characterization of susceptibility of subgrade soils to frost heave, Table 3 indicates that the frost-susceptibility classifications or criteria most widely used are based on the percentage finer than 0.02 mm, the percentage passing the No. 200 sieve, and soil classification. Eight of the agencies surveyed have no criteria for identifying or classifying frost-susceptible subgrade soils. The research studies mentioned previously have shown that the percentage finer than 0.02 mm is a reasonably reliable indicator of the relative severity of frost heaving that would be expected in various soils under given conditions of moisture and freezing temperature regime. Measurement of the fraction finer than 0.02 mm necessitates a hydrometer analysis or other test based on Stokes' law for the velocity of a spherical particle falling through a fluid medium, and consequently is subject to the complications and inaccuracies inherent in those procedures. On the other hand, soil classification including grain-size analysis by sieves down to the 200-mesh size is not necessarily indicative of the fraction finer than 0.02 mm, and it can be questioned whether the percentage passing the No. 200 sieve can serve as a reliable indicator of frost action except within particular soil deposits laid down under unchanging geologic conditions. The relationship of even the 0.02-mm fraction to observed frost heaves experienced in pavements is not yet well documented. Many measurements of heave have been made (*104, 105*) on pavements constructed on subgrades for which the fraction finer than 0.02 mm was known and on which laboratory freezing tests had been performed, but it was found impossible to isolate the soil factor as a determinant of frost heave from other variables of moisture and temperature conditions. Further research is needed to relate the fraction finer than 0.02 mm to measured pavement frost heaves; pending such further investigation, the 0.02-mm fraction still appears to be the most reliable indicator of susceptibility to frost heave, even though its principal utility is comparison of the relative frost susceptibility of different soils. As a basis for identification and classification of frost-susceptible soils, it can indicate the relative degrees of protection required over various subgrades to prevent damaging pavement distortion by frost heave.

Base-Course Materials

The essential properties of base and subbase courses affecting the design and performance of flexible pavements are (Table 3) strength, resistance to deformation, and constancy of volume. Deacon (*96*) showed that the key strength parameter for unbound granular base is shear strength, whereas for asphalt-treated bases (ATB) tensile and fatigue strength are the critical parameters. For cement-treated bases (CTB), shear strength, flexural

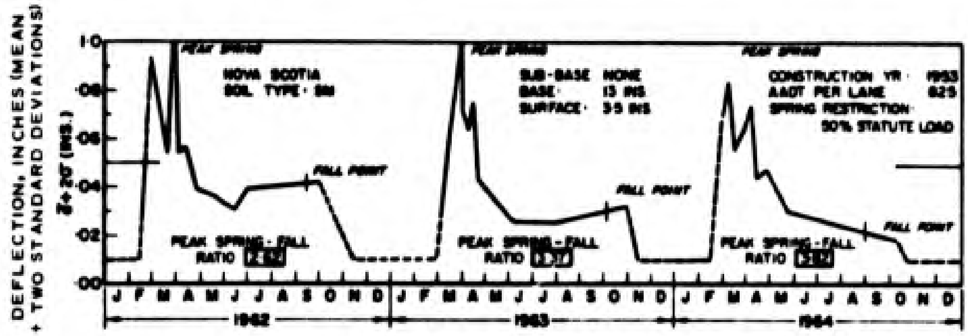


Figure 26. Example of seasonal variation in rebound deflection under 18,000-lb single-axle load (103).

strength, and fatigue strength are the most important strength parameters (*106*). The modulus of resilient deformation (resilient modulus) is a useful characterization of the resistance to deformation of the unbound granular base and ATB. The resilient modulus of unbound base is highly dependent on the confining pressure and moisture content (*107*), and the modulus of ATB varies principally with type of bitumen, temperature, and rate of loading (*108*). Strength and modulus of elasticity of CTB are affected by mix variables and curing conditions.

Because the strength and resilient modulus of unbound granular base are key determinants of the performance of pavements that incorporate such material, the designer seeks to optimize these properties by increasing the confining pressure. The effective confining pressure during application of a wheel load depends on the thickness of the overlying layers and the pressure induced by the wheel load itself, reduced by the porewater pressure, if any, in the unbound granular base. At degrees of saturation above about 75 to 80 percent, pore pressure buildup in unbound granular base under repeated loads has been shown by several investigators to severely reduce its resilient modulus (*109, 110, 111, 112*). Johnson (*113*) has reviewed these results and concluded that special drainage layers as advocated by Lovering and Cedergren (*114*) are necessary to control pore pressures in unbound base materials in frost areas, because even a very low percentage of fines markedly reduces the coefficient of permeability (Fig. 27). In characterizing unbound granular base the effect of a high degree of saturation must be accounted for, allowing the designer the alternatives of draining the base effectively or strengthening the layered pavement structure to compensate for the severe weakening of the base when it becomes saturated.

Unlike subgrade materials, which usually are not modified and consequently must be tested to determine their properties for use in the design process, base course materials usually are mechanically processed, and often are chemically treated, to meet certain predetermined specification requirements for which certain properties and/or performance are assumed to apply. Volume changes in unbound base courses caused by frost heave, and weakening during spring thaws, accordingly are restricted in current practice by establishing and enforcing certain requirements of grain size and plasticity (Table 6). Sieve analysis is the most widespread basis reported for restricting frost susceptibility, supplemented in many cases by Atterberg limits. As an indicator of frost susceptibility, the fraction passing the No. 200 and coarser sieves suffers from the same limitations as mentioned previously for subgrade materials. Subgrade soils are much more variable, however, and probably a better correlation could be established for base-course materials between that fraction and the finer fraction that governs frost susceptibility. It is also interesting to note that the maximum permissible percentage passing the No. 200 sieve, as specified by the agencies surveyed (Table 7), ranges from 5 to 15 percent, irrespective of the existence of further specification controls on the 0.02-mm fraction. With bases containing up to 15 percent fines being used, there can be little doubt that some of those bases are detrimentally affected by frost action.

The parameters characterizing the relative strength or stiffness of base and subbase courses, used in current practice as inputs to the design equations, were determined only for the 11 agencies surveyed in greater detail. Pertinent information from the other 29 agencies that reported consideration of frost in their flexible pavement designs is limited to responses to the question asking whether the total thickness of the pavement structure varies with the type and properties of base courses (see Appendixes A and B). The input parameters can be inferred from this information (Table 8), although the types of tests that are performed to determine some of the parameters are not known. HRB (*101*) has summarized the tests and criteria for mix design of bituminous-bound base used by the state highway departments of the United States. It is clear that the characterization of base-course materials in current practice is indirect and suitable only for design procedures that are based on experience with accustomed types of pavement

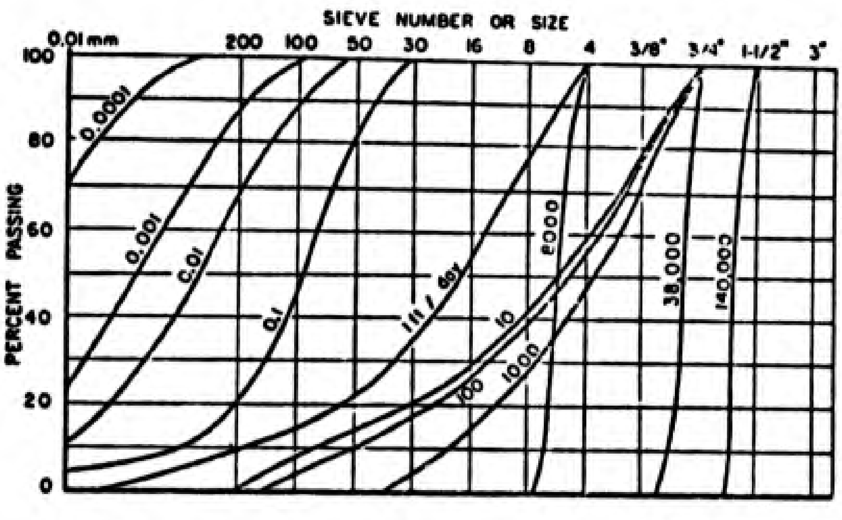


Figure 27. Gradation curves and associated permeabilities, in feet per day (114).

TABLE 6

MEANS CURRENTLY USED TO RESTRICT FROST SUSCEPTIBILITY OF UNBOUND GRANULAR BASE AND SUBBASE MATERIAL [a]

TEST	ELEVEN AGENCIES SURVEYED IN DETAIL	OTHER 29 AGENCIES REPORTING FROST CONSIDERED IN DESIGN
Grain-size distribution (GSD) by sieve analysis including No. 200 sieve	1	13
GSD plus Atterberg limits	1	6
GSD plus sand equivalent	1	1
GSD plus use of only crushed and processed materials	1	—
GSD plus $NaSO_4$ soundness	—	1
GSD plus abrasion and soundness	—	1
Percentage finer than 0.02 mm	2	1
0.02-mm fraction plus freezing test	2	—
None	2	6
None, because use only bound base	1	—

[a] From information summarized in Appendixes A and B.

TABLE 7

MAXIMUM PERCENTAGE PASSING NO. 200 SIEVE ALLOWED IN UNBOUND BASE AND SUBBASE

MAXIMUM PERCENTAGE ALLOWED	7 AGENCIES THAT SPECIFY ALSO 0.02-MM FRACTION OR SAND EQUIVALENT	33 AGENCIES THAT DO NOT SPECIFY 0.02-MM FRACTION OR SAND EQUIVALENT
5–8	3	4
10	2	7
12–15	—	5
No information	2	17

TABLE 8

CHARACTERIZATION OF BASE AND SUBBASE COURSES USED AS INPUTS TO DESIGN EQUATIONS FOR FLEXIBLE PAVEMENTS

PARAMETER	ELEVEN AGENCIES SURVEYED IN DETAIL	OTHER 29 AGENCIES REPORTING FROST CONSIDERED IN DESIGN
Layer coefficients (AASHTO method)	5	13
Substitution ratios	2	1
Gravel equivalencies	—	4
Crushed base aggregate equivalencies	—	2
CBR	1	—
Triaxial modulus (ATB only)	—	1
Stiffness, as function of layer thickness and subgrade strength	1	—
Benkelman beam deflection	1	1
None (standard sections)	1	2
No information	—	5

structures under given environmental conditions. Under other, unaccustomed, conditions the dependence of the resilient modulus of unbound base on effective confining pressure (a function of depth in the pavement profile and degree of saturation) and of asphalt-treated base on temperature could lead to unsatisfactory performance of pavements designed by empirical methods.

Asphaltic Surfacing Mixtures

The properties of asphaltic surfacing mixtures that need to be characterized (Table 3) are tensile strength, fatigue strength, and stiffness. None is unique: instead, a strong dependence on loading conditions, mixture variables, and temperature has been shown (*96*). Meaningful characterization of these properties is of critical importance to the design of pavements for adequate performance under particular conditions, but such characterization is also very complex. With one exception the current practices in characterizing asphaltic surfacing mixtures have been excluded from the survey reported herein. The excluded topic is as important in frost areas as elsewhere; the high pavement temperatures that occur even in seasonal frost areas are an important cause of summer pavement distress, owing to the consequent severe reduction in strength and stiffness of asphaltic mixtures. The exclusion is justified only because the solutions to such problems reside essentially in the technology of warm climates, and because of space limitations affecting the scope of this report. The exception mentioned is the current practices in designing pavements to minimize low-temperature cracking. Haas et al. (*115*) summarized the investigations made in Canada of the serious problem of low-temperature transverse cracking of asphaltic pavements, which showed that the properties of the asphalt comprise the principal factor affecting the severity of such cracking in pavements in a given locality, and that the most satisfactory parameter to characterize the low-temperature response of asphalts and mixes is stiffness modulus. According to the definition of van der Poel (*116*),

$$S(t, T) = \frac{\sigma}{\epsilon} \tag{3}$$

in which

$S(t, T)$ = stiffness modulus of the material for a particular time, t, and temperature, T;
σ = stress, at t and T; and
ϵ = strain, at t and T.

Stiffness modulus may be determined by direct testing or by indirect estimation. The relationships indicated by direct testing are well summarized by Haas et al. (*115*), who also outlined the estimation of stiffness modulus of the bitumen by the method of van der Poel (*116*), and the estimation of stiffness of the mix by an equation presented by Heukelom and Klomp (*117*). Hajek and Haas (*118*) refer to experiments showing that indirect methods of estimating stiffness are satisfactory, and finding that the method of McLeod (*119*) is the best indirect method. Hajek and Haas found it advantageous to use stiffness modulus of the bitumen as the basis for predicting the frequency of low-temperature cracks. The stiffness modulus is calculated for a loading time of 20,000 sec and a temperature equal to the minimum ambient temperature anticipated. Using only the penetration of the asphalt cement at 77°F (25°C) and its kinematic viscosity at 275°F (135°C), the penetration index is determined (Fig. 28) and used to determine a base temperature corresponding to the temperature at the softening point in the ring and ball test (Fig. 29). The difference between the base temperature and the minimum ambient temperature chosen for design is entered into the nomograph of van der Poel (Fig. 30) to determine the stiffness modulus of the bitumen.

According to the approach outlined in the preceding paragraph, only the penetration and viscosity are necessary to characterize the stiffness modulus of the bitumen for selection of asphaltic bitumens for mixtures that will resist low-temperature cracking. The specifications for viscosity-graded asphalts were developed (*121*) to include both these properties: together they define the temperature-susceptibility of the asphalt, those asphalts whose viscosity (275°F, 135°C) is high for a given penetration grade or whose penetration (77°F, 25°C) is high for a given viscosity grade (Fig. 31) being less affected by temperature change and less susceptible to cracking. Tests at temperatures other than 275°F and 77°F may also be used to define temperature-susceptibility. Viscosity determinations at 140°F (60°C) are widely used, although the validity of tests at that temperature is controversial (*122*). Also, it is widely recognized that control of low-temperature cracking ideally ought to be exercised through tests of the bitumen at low temperatures, rather than extrapolation from tests at moderate and high temperatures; unfortunately, simple tests at low temperatures with acceptable accuracy have not been developed as yet.

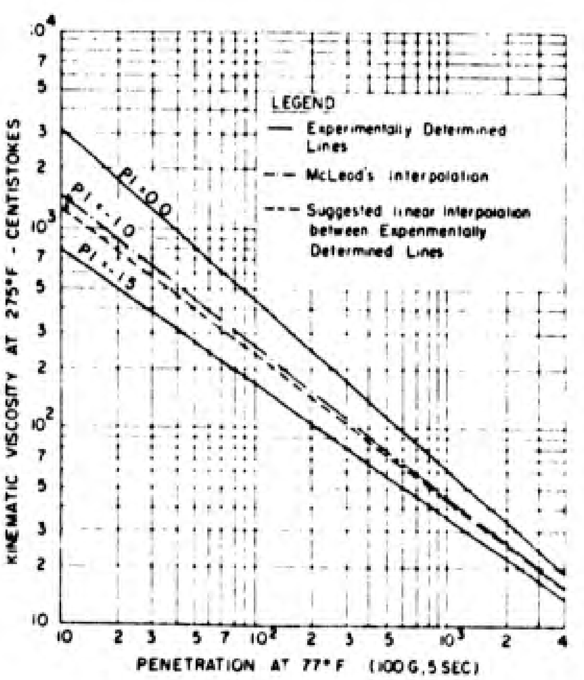


Figure 28. Modification by Hajek and Haas (118) of McLeod's graph for estimation of penetration index.

Environmental Factors

It has been shown in the preceding section that the properties of materials used in pavement systems in seasonal frost areas are dependent on the environmental variables of moisture and temperature, including the change from frozen to thawed state and the critical transitional state that develops during the thawing process. Adequate characterization of materials includes definition of the effects of each of these variables. Application of the material parameters in the design process requires a forecast of the moisture and temperature (including frost) conditions that will exist in the proposed pavement system and the diurnal, seasonal, and long-term variations in those parameters. The methods available for making such forecasts are well summarized by Thompson (*35*) and Dempsey and Thompson (*124*), and are commented upon in the following paragraphs, which include also a summary of current practices. In most cases, materials characterization in practice does not include definition of variation in properties across the broad range of moisture and temperatures to which the materials will be subjected. Instead, design of pavements in much of the current practice is based on the most critical condition or some intermediate condition adjusted by a regional or climatic factor (*123*).

Temperatures

Temperatures (including frost) in the various layers are influenced by many climatic and intrinsic factors (*35*), including:

1. Temperature factors, such as air temperature, short-wave solar radiation received at the paved surface, long-wave radiation emitted by the paved surface, and wind.
2. Hydrologic factors, such as precipitation, evaporation, and condensation.
3. Geographical location factors, such as elevation, latitude, degree of exposure, and proximity to bodies of water.
4. Intrinsic factors governed by the properties of the soil, of which the most important are thermal conductivity, heat capacity, and latent heat of fusion.

The methods of predicting temperatures in pavement systems have been developed in relation to studies either of frost action or of the temperature-dependence of the properties of asphalt-stabilized layers. The depth of frost penetration can be predicted by empirical and theoretical methods. Empirical methods make frost penetration predictions based on direct measurement of frost depth in relation to air temperature, air freezing index, soil type, or moisture content, or combinations of these factors. Theoretical methods are based on fundamental heat-transfer equations, from which formulas and charts have been developed for routine

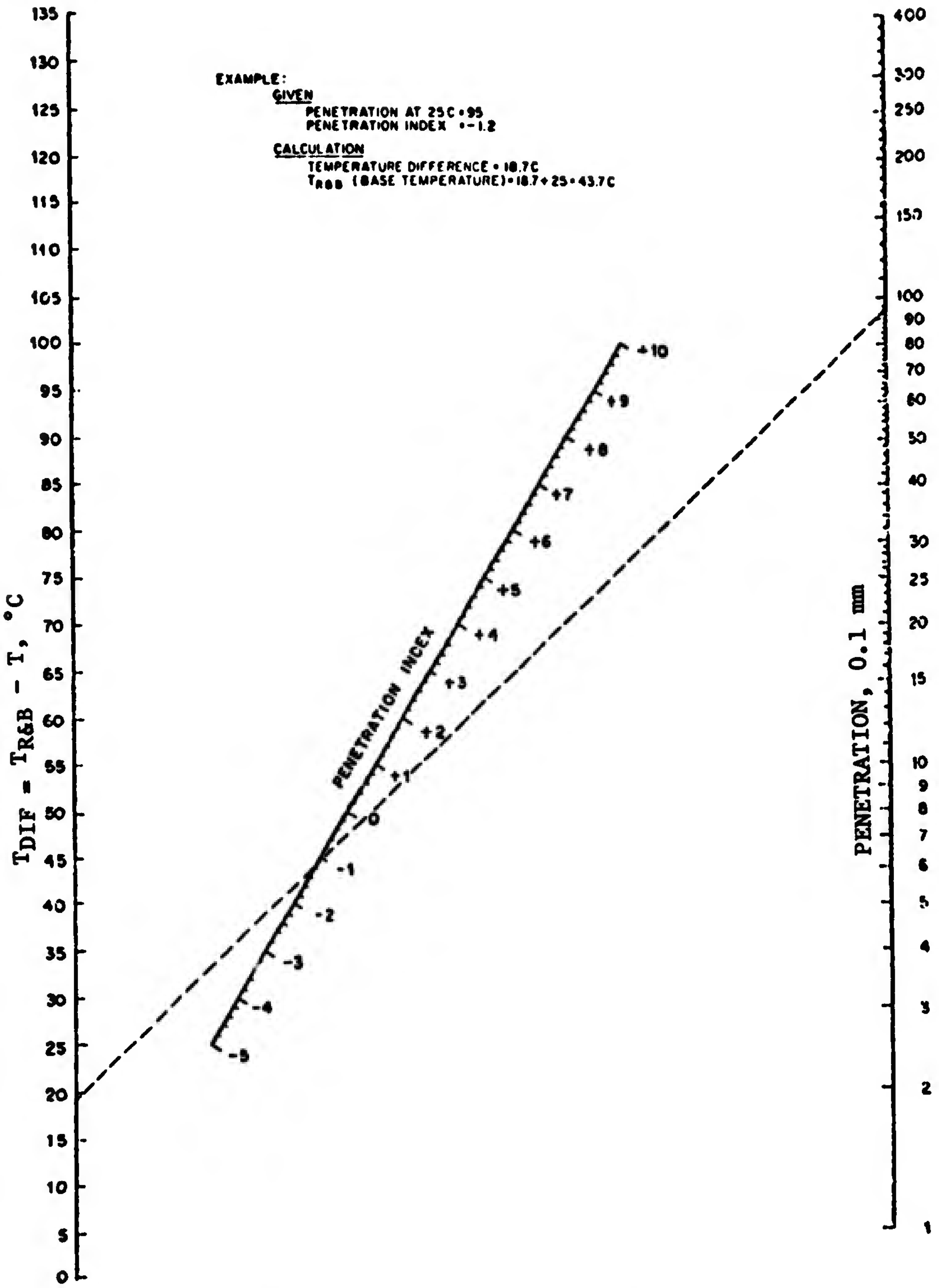


Figure 29. Nomograph of Pfeiffer and Van Doormaal (120) for determination of penetration index.

use in design. The Stefan and modified Berggren formulas have been widely used in predicting the frost depth. Depth of frost penetration can also be predicted from heat-transfer theory, for which computer programs have been developed (*124*, from *35*) that include as input parameters the climatic and intrinsic factors previously listed. Berg (*125*) summarizes the numerical methods that have been applied to heat-transfer problems in soil/water systems, for which 20 computer programs are currently available.

These same heat-transfer models can be used also to predict temperatures throughout the layered structure of the pavement, as a function of time and space. However, their use in practice has not been generalized as yet, and in those cases where temperature predictions are made at all they

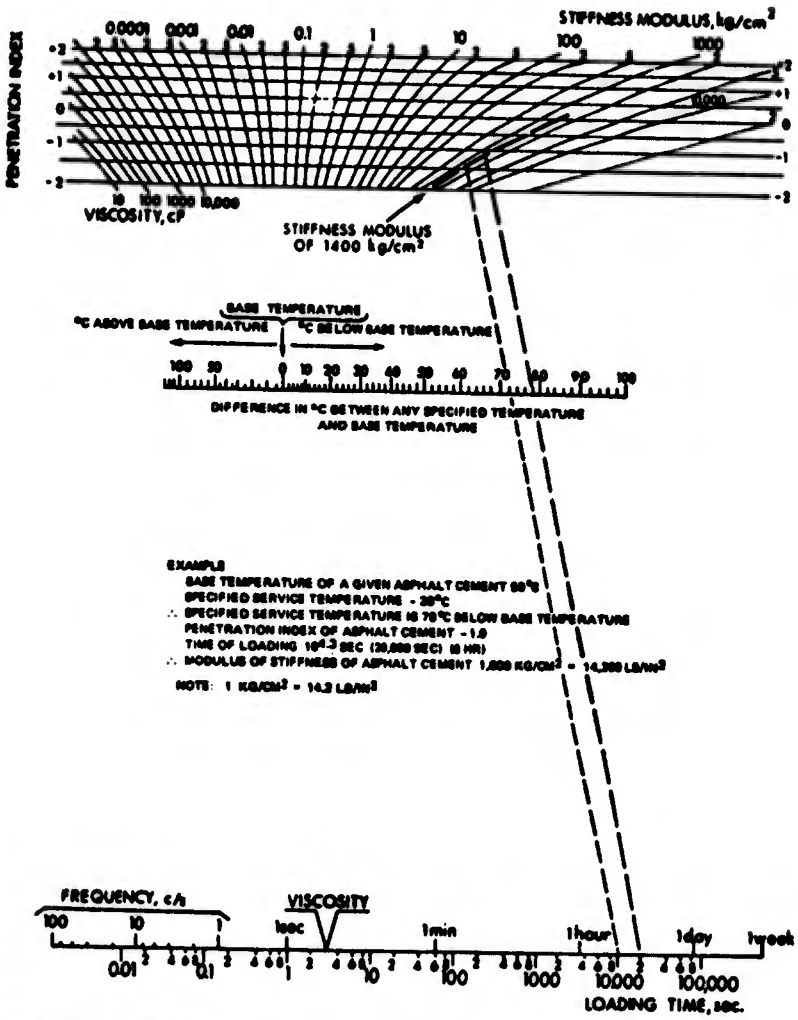


Figure 30. Modification of Heukelom and Klomp's version of van der Poel's nomograph for determining modulus of stiffness of asphalt cements (after McLeod, 119).

are usually based on correlation studies between pavement temperatures and air temperature data recorded at nearby weather stations (*126, 127, 128*).

Moisture

Moisture in the layered structure of a pavement, like temperature, varies as a function of time and space, under a variety of mechanisms (see Chapter Two, "Identification of Factors Contributing to Frost Problems; Sources of Water"). Soil water may be classified (*35, 124*) as groundwater, gravitational water, and held water. The flow of groundwater usually is not of major concern to the performance of a well-designed road, which would include appropriate drainage facilities to negate its adverse effects, or would have the grade line set high enough to be above its influence. Gravitational water, which moves toward the water table under the influence of gravity, also is of secondary importance to that of water held in the pores after gravitational flow has ceased, or which is drawn upward into the pores by surface tension and absorptive forces, whose effects create what is termed "soil suction." Soil suction is influenced by moisture content, soil structure, vegetation, and climate. The effect of each is summarized by Thompson (*35*), who concludes that "climate is the dominant factor influencing the space-moisture conditions in pavement systems." Thompson cites the work of Croney and Coleman (*129*) and Gurr et al. (*130*) relating to moisture transfer in the vapor phase under temperature gradients, and the work of Hoekstra (*131*), which indicated significant transfer of moisture in the liquid phase toward a freezing front. Thompson also has summarized the available empirical and theoretical methods for predicting moisture movement and moisture equilibria in pavement systems, and concluded that although some of the empirical methods have been used successfully for assessing soil moisture and groundwater levels beneath pavement surfaces, "it is not presently possible to accurately predict field moisture conditions in pavement systems as a function of time and space."

Assessment of Environmental Variables in Practice

In view of Thompson's conclusion, it should not be surprising that empirical factors to account for environmental conditions are so widely used (Table 9). Of the 21 agencies reporting that frost is considered in design and that environmental factors are used, a majority use the regional factor of the AASHO (*87*) design procedure. This is a factor included in the design equation to make it applicable for design of pavements in areas with climatic and environmental conditions different from those at the AASHO Road Test site. Van Til et al. (*132*) summarized the use of regional factors by the state highway departments, and also the variables considered in each state in determining the values of the regional factors. Moisture, temperature, and depth of frost penetration are among the variables that have been used, but it is clear that the regional factor as currently used is essentially a judgmental appraisal of environmental conditions. Van Til et al. also found that many agencies employing a regional factor use it only to make a general adjustment of the pavement design equation presented in the AASHO Interim Guide (*87*), according to the difference in severity of average environmental conditions within the particular state compared with those prevailing at the site of the AASHO Road Test. A smaller number of states have established a scale of intrastate regional factors to account for variation in the severity of environmental conditions within their boundaries. Under the AASHO flexible pavement design equation the regional factor is actually a multiplier on traffic, and its effect on structural number (proportional to pavement thickness) is modest. A typical range in regional factor of 1.0 to 3.0, for example, signifies variation in structural number of only about 16 percent. In most cases this variation would signify a difference of 1 to 2 in. (25 to 50 mm) of asphaltic concrete, or up to 6 in. (150 mm) of gravel. Differences of this magnitude possibly can adjust adequately for differences in rainfall, temperature (excluding frost), drainage, etc., but further environmental factors or adjustments in design are necessary to account for the more damaging effects of frost heave and thaw-weakening. The AASHO Interim Guide recognizes that regional factors do not account for serious frost conditions, and leaves any additional thickness needed for frost protection to be determined by local experience.

In some cases local experience is reflected in pavement designs by adjustment of thicknesses in certain locations, or over certain types of soil; in other cases freezing index or frost penetration is used as a factor that impacts directly upon design thickness, either in lieu of, or supplementing, the adjustment accomplished by the regional factor. Also, many agencies make an assessment of environmental influences either in lieu of, or in addition to, the regional factor by observations of the depth to the groundwater table (*133*) or the uniformity of subgrade conditions (suggesting little differential frost heave), or simply by the subjective judgment of the designing soils engineer.

Selection of pavement structure based on the designer's perception of the most critical environmental conditions is exemplified by the practice of observation of the highest level attained by groundwater during the year, the highest freezing index experienced in a certain period of years, the highest static deflection observed on the subgrade or on nearby roads during the year, etc. Clearly, definition of the most critical levels reached by a time- and space-dependent variable is an essential element of the design process, be-

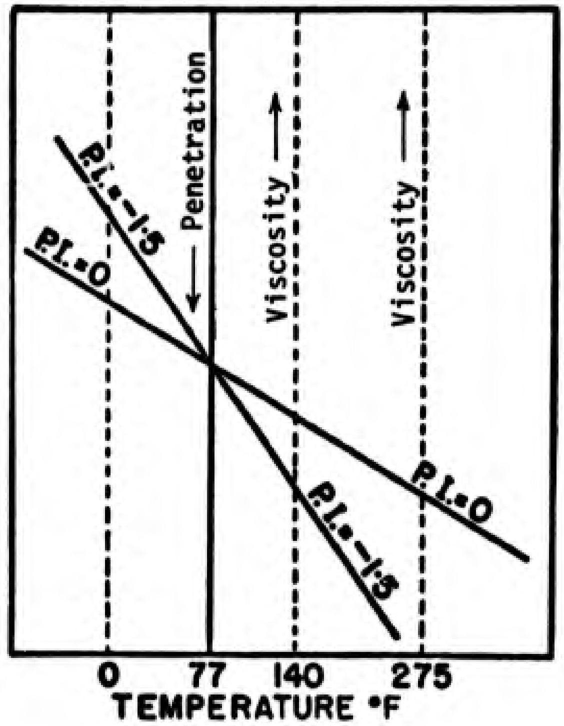


Figure 31. General relationships between viscosity, penetration, temperature, and penetration indices for asphalt cements (after McLeod, 25).

cause failure under some modes can be caused by a severe condition that is reached only once. Nevertheless, with automatic data processing designers now can make use of large masses of data, and it is within their reach to base designs on the entire spectrum of values that each variable may assume. This subject is mentioned further in the next section under the heading "Cumulative Damage."

As a general conclusion to this section, it appears that assessment of environmental influences is one of the weakest subsystems in the process of design of flexible pavements as currently practiced in seasonal frost areas. To a large

TABLE 9

USE OF ENVIRONMENTAL FACTORS IN CURRENT PRACTICE

ENVIRONMENTAL FACTOR	11 AGENCIES SURVEYED IN DETAIL	OTHER 29 AGENCIES REPORTING FROST CONSIDERED IN DESIGN
Regional or environmental factor used in design	7	14
Freezing index or frost depth also used as design parameter	2	4
Regional or environmental factor not used in design	4	15
Freezing index or frost depth used as design parameter	1	5

extent the assessment is made by judgment rather than by evaluation of measurable parameters. Temperature variables (including frost) can be modeled as a function of time and space by predictive techniques currently available. Moisture conditions cannot be accurately predicted, but useful empirical means of assessing soil moisture and groundwater are available.

Pavement Design Equations

The purpose of pavement design equations is to provide a framework for the systematic analysis of layered pavement structures to formulate the response to anticipated loads and environmental effects. Pavement design is but one essential part, or subsystem, of a pavement management system that embraces the diverse factors of cost, convenience, and safety of pavements throughout their life cycle. The design process must evaluate and account for all the specific modes of distress (Table 1) to ensure a given level of serviceability and a given rate of deterioration.

Fatigue

HRB and FHWA (*123*), in summarizing research needs, concluded that well-developed and documented means exist for predicting the stress, strain, and deflection states within and at the surface of the pavement structure. The principal traffic-load-associated modes of distress, cracking under repeated loading (fatigue) and rutting under repeated loading and single excessive loads, can be analyzed by the linear theories of elasticity and viscoelasticity. Monismith (*3*) suggests use of elastic-layer theory for structural analysis of distress caused by fatigue; computer programs CHEV 5L (Chevron Research Company), BISAR (Shell Oil Company), and ELSYM (University of California) are available for this purpose. Bergan and Monismith (*91*) used a modification of the fatigue formulation of Kasianchuk (*127*) to simulate fatigue in a study of a four-lane divided highway near Regina, Saskatchewan. The fatigue failure predicted by the analytical procedure agreed well with the date on which the failure actually occurred. This study is an outstanding example of application of seasonal variations in material properties to the analysis of the cumulative effects of traffic leading to fatigue failure of a road in a seasonal frost area.

Permanent Deformation

The state of the art in the analysis of permanent deformation (rutting) under repeated loads is less advanced than the existing fatigue formulations. The stress-strain conditions in each layer must be analyzed to ensure that the contribution of each layer to surface rutting will be minimal. For the subgrade, an elastic-layer analysis can be used to compute the vertical compressive strain at top of subgrade, for comparison with the limiting criteria of Shell (*134*). The limiting criteria of The Asphalt Institute (*128*) were developed for airfield pavements, but the principles can be applied to roads as well. For unbound base/subbase courses, Brown and Pell (*94*) suggest as design criteria that the horizontal tensile stress be limited to 0.5 times the vertical stress plus the horizontal overburden pressure. Barksdale (*135*) computes the principal stresses within the unbound materials under wheel loads by linear or nonlinear elastic theory, and uses repeated-load triaxial tests to ascertain the corresponding plastic strains, which are summed to give rut depths. Field trials are needed to verify these approaches to design of unbound layers. For asphalt-bound materials, two possible approaches still in the domain of research are viscoelastic analysis, with materials being described in terms of creep compliance functions, (*136*) and an elastic analysis suggested by Heukelom and Klomp (*137*). Neither of these approaches is readily available as a design method, and instead of predicting plastic strain in asphalt-bound layers the designer's alternative approach is to minimize the contribution of asphalt-bound layers to surface rutting by following current standards for selection of bitumen of suitable viscosity, mix design, and adequate compaction.

Frost Heave

The principal nontraffic-associated modes of distress that depend on cold regions environmental variables are pavement distortion and cracking caused by frost heave and subsequent thaw-consolidation, disintegration of stabilized layers caused by freeze-thaw degradation, and pavement cracking caused by low temperatures. There is no mechanistic model available, even in the domain of research, for prediction of frost heave or the consequent transient or permanent pavement roughness. The Corps of Engineers (*20*) has established design criteria for runway overrun pavements that express the thickness required over frost-susceptible subgrades to restrict differential heave within certain predetermined limits. These criteria, which have been used with apparent success on airfield pavements for high-speed aircraft, are based on limited field test data from airfield pavements and from pavement test sections. There are insufficient data to verify their applicability to the generally thinner sections used on road pavements, on which greater differential heaves usually are tolerable. Experience has shown that heave at a point on the pavement surface depends on the temperature regime, the properties and moisture conditions of the layer or layers in which ice segregation occurs, and the thickness, stiffness, and weight of materials overlying the layer in question. A pressing need exists for research, including field measurements on pavements, for development of suitable probabilistic models for prediction of frost heave in terms of these parameters. Even more useful would be a model to express the effects of ice segregation directly in terms of differential heave, or pavement roughness. A statistical approach offers the best prospects for success in solving this difficult problem; however, the natural variability of subgrade details will always leave some uncertainty, except to the extent that the materials within the depth of freezing can be thoroughly processed for uniformity.

Base-Course Drainage

Effective drainage of unbound granular bases, to restrict transient pore-pressure buildup under rapidly applied wheel loads, improves pavement performance under the three distress modes mentioned in the preceding paragraphs—fatigue cracking, permanent deformation (rutting), and frost heave. Elsewhere in this report general drainage practices are summarized, including subsurface drainage to intercept inflows from springs and remove high groundwater. Special drainage facilities are needed, however, to remove excess water released by melting subsurface ice masses and water entering the pavement substructure by infiltration through pavement joints and cracks and through the pavement itself. Substantial economies in the layered pavement structure may be possible if pore pressures in unbound granular base or subgrade materials directly beneath the asphalt-bound layers may be precluded or relieved.

Long-standing theoretical and experimental analyses by Casagrande and Shannon (*138*) form the basis for criteria adopted by the Corps of Engineers (56) for design of subsurface drainage facilities for airfields. The spacing between perforated pipe drains under large paved areas for airfields is calculated to achieve 50 percent drainage of a saturated base course in a period not to exceed 10 days. No infiltration is assumed during that period, and flow of water from the base toward the drain is nonsteady, decreasing at a diminishing rate.

Among the most significant developments related to subsurface drainage for highways is the work of Cedergren and his colleagues (*114, 139, 140*). Instead of assuming extended periods of no infiltration, these investigators calculated the drainage conditions necessary to remove water from the base at a rate that would keep pace with the rate of infiltration of surface water through the pavement. Lateral flow towards a pipe drain through a base course containing even a small percentage of fines will not achieve this objective. According to their calculations, special open-graded drainage layers are needed, below or above the base course, to allow vertical flow within the less pervious base along a short seepage path to the drainage layer. FHWA (*141*) adopted one of the details of the arrangements of drainage layers recommended by Cedergren and colleagues (Fig. 32), which includes an asphalt-treated (1½ to 2 percent bitumen) open-graded aggregate (graded from about ¾ in. to the No. 4 sieve) below the base course.

The arrangement shown in Figure 32 should be effective when the base and subgrade are unfrozen, but will not drain the base during midwinter partial thaws or during the early part of the spring thaw, when the upper part of the base itself would be thawed but drainage to the underlying layer would be blocked by the still-frozen condition at that lower level. Ring (*142*) recognized this problem and advocated a drainage layer placed high within the pavement system. Johnson (*113*) concluded that the open-graded drainage layer will have to be placed above the graded aggregate base, where it will thaw and become functional as a drain before the base itself thaws and requires pore-pressure relief. The essential function of the drainage layer in this case will be pore-pressure relief; the work of Seed et al. (*111*) with drained triaxial tests on saturated gravel, shows that the presence of water filling the voids of a gravel specimen may not detrimentally affect the resilient modulus as long as pore-pressure buildup is prevented. Lovering and Cedergren mentioned the concept of placement of the drainage layer directly below the dense-graded asphalt surface course, and Cedergren/KOA (*140*) advocated an open-graded drainage layer of nearly uniform grain sizes, bound with about 2 percent asphalt if necessary for stability (Fig. 33). The sound logic of this concept, particularly for frost areas, deserves the attention of designers and researchers concerned with roads in frost areas.

Current Practice in Dimensioning Flexible Pavements

In current design practice only empirical methods are used to dimension pavements to guard against excessive frost heave. Also, the available methods for analysis of traffic-load-associated distress modes are little used (Table 10). Nine of the eleven agencies whose practices were surveyed in more detail use an empirical procedure to dimension the pavement structures as a function of traffic, subgrade support, and frost conditions; one uses standard designs depending only on traffic; and one uses elastic layered system theory.

The most widely used design method, among these agencies, is that given in the AASHO interim guides. This method is not a frost design method, but, although relying on partial integration of environmental variables into the design by means of the regional factor, essentially leaves any additional thickness needed for frost protection to be determined by local experience. The effect of local experience can be seen in Table 10. For example, Maryland uses 12 in. (300 mm) of granular cap over frost-susceptible subgrades, Maine uses an additional 24 in. (600 mm) of granular material in the north, and New Hampshire uses

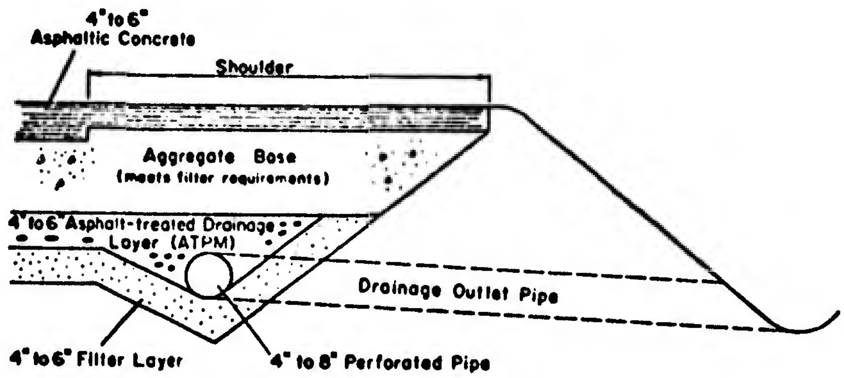


Figure 32. Two-layer drainage blanket for flexible pavement system (141).

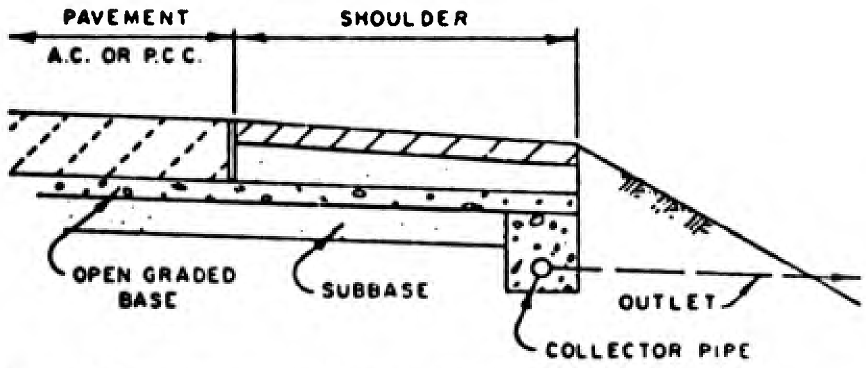


Figure 33. Drainage system for frost areas (140).

a nonfrost-susceptible subbase from 0 to 24 in. thick, depending on the recommendation of the soils engineer. Advocates of the New York standard pavement sections, with selection of the appropriate one based only on traffic, justify this system on the basis that "it works," even under the great variations in terrain and environmental influences that are found in New York State.

The Corps of Engineers' method includes the two alternative approaches summarized in Table 10. The method generally applied for road design in deep seasonal frost areas is based on traffic tests on airfield pavements during the frost-melting period. The reduced strength of the subgrade determines the design thickness required for various levels of traffic, in a manner similar to a CBR-type relationship; in fact, equivalent CBR values can be inferred for each FS group. The method also permits application of local experience, if it indicates a need for additional thickness to provide satisfactory performance.

Both Canadian provinces included in the tabulation use interesting approaches. Alberta's procedure using the deflection measured in the spring on each layer as the basis for design of succeeding layers is, like the other methods based on AASHO, Corps of Engineers, standard thickness design, etc., performance proven. Saskatchewan uses the Shell Oil procedure based on layered system theory; currently the resilient modulus of the subgrade is being modeled as 1,500 × CBR, but the laboratory test for direct measurement of resilient modulus is being integrated into the system.

The methods of dimensioning flexible pavements were not queried of the other 29 agencies that reported frost is considered in design of their pavements. Other information provided in the questionnaires indicates that empirical design methods are universally applied. The design equations used by a number of these states are described in the technical literature. Minnesota's method (*143, 144*) is an excellent example of the adaptation of the AASHO Road Test results to design of pavements over moderately to highly frost-susceptible soils in a region of deep seasonal frost. It, like the CGRA procedure (*103*), is based on spring deflections, but the Minnesota procedure uses allowable deflections that vary with traffic and thickness of surface course. In several states the empirical criteria include a requirement that the total thickness of the pavement structure be not less than a prescribed fraction of the frost depth (Table 11). The diverse practices of the various agencies regarding undercutting of frost-susceptible soils in cut sections also are an essential part of their respective thickness design practices, because the undercuts in many cases are backfilled with granular soil, which then comprises structurally superior layers within the subgrade that are additional to the layered pavement structure dimensioned according to the respective empirical design method. These undercutting practices, summarized in Appendixes A and B, are influenced in some states and provinces by environmental variables.

The empirical procedures used to determine the design thickness of flexible pavement structures include, in most of the agencies surveyed, an evaluation of the structural advantage of bound (treated with asphalt, portland cement, or other cementing agent) as compared with unbound, base courses. The usual method is by means of layer coefficients, substitution ratios, or gravel equivalencies (Table 8 and Appendixes A and B). A notable exception is the criteria of the Corps of Engineers, which currently assign no additional structural value to bound bases. Recent research findings have confirmed the structural advantage of bound bases, however, and recent designs by the Corps of Engineers have recognized these findings; modification of current criteria is anticipated. Bound bases in frost areas are particularly suitable for pavement structures whose design is not governed by the frost heave distress mode, but rather by traffic-load-associated distress. Examples are pavements over clay subgrade soils that are subject to thaw-weakening but do not heave severely, and over any type subgrade if insulation is used to prevent or restrict subgrade freezing.

Because none of the agencies surveyed customarily employs pavement structures thick enough to prevent subgrade freezing, frost heaves occur generally in roads constructed throughout the northern tier of states and the Canadian provinces. A number of the agencies consider that the effect of frost heave on the serviceability of their pavements is minor or nil (Appendix B). This stems in part from the type of subgrade soils that prevail, for example, in the Great Plains, where moderately to highly plastic clays occur widely without inclusions or alternating layers of material that are more susceptible to heave. Volume changes upon freezing of such clays in uniform deposits, and particularly where favored by low rainfall regime, not only are slight to moderate but also tend to be

TABLE 10

METHODS USED BY ELEVEN SELECTED AGENCIES FOR DIMENSIONING OF FLEXIBLE PAVEMENTS

STATE OR PROVINCE	ESSENCE OF FLEXIBLE PAVEMENT DESIGN METHOD, BY WHICH THICKNESS FOR GIVEN TRAFFIC AND SERVICEABILITY CONDITIONS IS DETERMINED AS A FUNCTION OF SUBGRADE SUPPORT AND FROST CONDITIONS
Alberta	Emphasis is on achieving uniformity of subgrade conditions. Use stage construction. First construct subgrade and 2"–3" gravel. Second measure BB deflection, which determines base design. Construct base plus 2" "oil-bound" (temporary cold-mix pavement with MC asphalt). Third (1–4 years later) measure BB deflection, which determines surface course design. Construct AC surface course.
Colorado	AASHO interim guides, using R-value and regional factor. Design adjusted during construction after measuring R-value of subgrade in completed cuts and fills.
Idaho	Thickness (in gravel equivalent) determined from R-value of subgrade and traffic index. Climatic factors are direct multipliers on gravel equivalent. Adjusted gravel equivalent converted to standard thicknesses of AC surface and ATB or CTB, to determine required thickness additional base of unbound gravel.
Maryland	Design equation of AASHO interim guides used to determine thickness of AC plus equivalent DGA. Substitution ratios used to convert DGA to other types of bound and unbound bases. Frost conditions enter design equation only through regional factor and FS classification of subgrade. FS subgrades are reinforced with 12" granular cap (assigning it a CBR of 7) or stabilized with cement.
Maine	AASHO interim guides used for Interstates, but checked also against thickness requirement from C of E design curve for 20-k wheel load in relation to FS group. Usual thickness for HD roads about 32", but I-95 north of Bangor has additional 24" gravel. Thickness I and S roads determined principally from C of E curve related to FS groups.
Nebraska	Usual design is "full-depth AC," whose thickness is determined as the AC surface course thickness from AASHO interim guides, plus black base to achieve the structural number required by the interim guides. Frost conditions do not enter design equation. Full-depth used even on heavy clays, but only if treated with lime.
New Hampshire	Prefer provide NFS material to full depth of frost but actually get some subgrade freezing because Interstates have 5½"–10" AC plus 48" NFS, and other highways less. Thickness AC and granular base standardized for particular values of AADT, but thickness NFS subbase varies from 0 to 24" on recommendation of soils engineer.
New York	Standard designs depending only on class of highway and DHV, whose thicknesses were determined by evaluation of performance of pavements under similar terrain and environmental conditions. Emphasis is on achieving uniformity of subgrade conditions.
Saskatchewan	Shell Oil procedure, which utilizes layered system theory. Stiffness value used for AC is a function of properties of bitumen and temperature. Stiffness of unbound bases varies with thickness and with strength of subgrade. Stiffness of subgrade taken as 1500 × CBR, after adjustment of CBR downward by 2 percent in cuts, downward severely in lake basin sediments, and upward by 2 percent where drainage is excellent.
Wisconsin	AASHO interim guides.
USA Corps of Engineers	Thickness required above subbase, and above base, determined by empirical design curves, from their respective CBR values. Total thickness of NFS material required above subgrade determined as the lesser of: (1) thickness required to limit frost penetration into subgrade, which depends principally on DFI; or (2) thickness required over thaw-weakened subgrade to maintain structural adequacy of pavement, which depends on FS group. In latter case, additional thickness may be required to reduce heave.

uniform and contribute little to pavement roughness. Some agencies take special precautions to avoid abrupt discontinuities in the subgrade at culverts by using sloping transitions in the depth of granular backfill (Table 12 and Fig. 34). Similar granular wedges are also provided by some agencies at transitions from cut to fill (Fig. 35).

In soils that undergo serious ice segregation various techniques of subgrade preparation are employed to minimize differential frost heave. These include removal of isolated deposits of the most heave-susceptible soils, scarifying and blending the top 1 to 2 ft (0.3 to 0.6 m) of the subgrade to break up undesirable stratification and distribute uni-

TABLE 11

MINIMUM PAVEMENT STRUCTURE THICKNESSES IN RELATION TO FROST DEPTH

STATE	MINIMUM PAVEMENT STRUCTURE THICKNESS DESIGN PRACTICE [a]
Connecticut	⅔ to ¾ depth of frost in coldest year in 5 years.
Nevada	½ estimated frost depth.
New Hampshire	Prefer 100% of frost depth, but not always feasible.
Oregon	½ maxmium frost depth.
Pennsylvania	⅓ to ¾ depth of frost in coldest year in 10 years.
Washington	½ recorded frost depth.

[a] In most cases the base and subbase must be NFS (see Appendixes A and B).

formly pockets of silt or other heave-susceptible material, removal of boulders from the subgrade within the depth of frost penetration, special drainage facilities in wet areas, cleaning of soil-filled joints of rock in cuts and re-filling with better material, and fragmentation of rock in cuts to break up stratification and distribute heave-susceptible materials present in joints and seams.

Low-Temperature Cracking

The second principal nontraffic-associated mode of distress that depends on cold regions environmental variables is transverse pavement cracking caused by contraction at low temperatures. A number of analytical approaches for solution of this problem are available. Christison and Anderson (*145*) analyzed test roads in Alberta and Manitoba and predicted their susceptibility to low-temperature cracking. Thermally induced stresses in the asphaltic pavement were calculated by applying an assumed coefficient of thermal contraction in formulas applicable to an elastic beam, a viscoelastic beam, and a viscoelastic slab; the computed stresses were compared to tensile strengths of the asphaltic mixture determined by tensile splitting tests.

Hajek and Haas (*118*) developed a mathematical model for predicting the cracking index (number of transverse cracks per 500-ft (150 m) section of two-lane highway) by means of a regression analysis of observed cracking indices and other pertinent data from 32 sites on roads in Ontario and the Ste. Anne Test Road. The equation expresses cracking index as a function of stiffness of the asphalt cement, total thickness of the asphaltic concrete layers, age of the asphaltic concrete layers, type of subgrade, and minimum winter temperature selected for design. Fromm and Phang (*146*) developed a simplified chart for selection of suitable asphalt cement for different temperature regimes. Their relationship between design temperature, penetration, and penetration index is based on findings from the Ste. Anne Test Road that a pavement should still be crack-free until the stiffness modulus of the asphalt cement has reached 20,000 psi (140 MPa). Based also on the Ste. Anne Test Road and observations of other roads in Canada, McLeod (*25*) concluded that cracking will not occur when the modulus of stiffness of the asphaltic concrete is less than 1,000,000 psi (7,000 MPa), measured at a loading time of 20,000 sec (5.55 hr), and developed

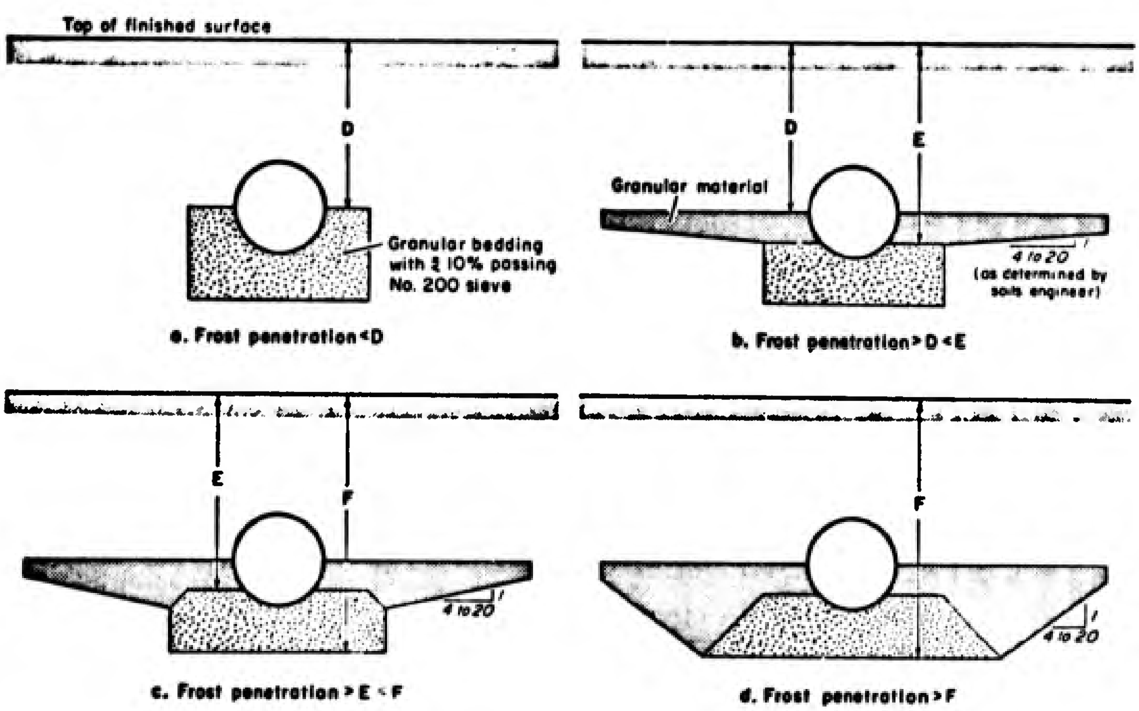


Figure 34. Treatment of centerline culverts in plastic soils (Minnesota).

charts to select grades of asphalt cement to avoid low-temperature cracking (Fig. 36).

As stated previously, 40 of the agencies surveyed reported the occurrence of low-temperature transverse contraction cracks in their asphaltic pavements, and most of them recognized such cracks as significantly affecting the serviceability of their roads. The seriousness of the problem has been recognized only within the past 10 to 15 years, however, and it is probably for this reason that so many agencies are taking no special action in selecting asphalts to minimize low-temperature cracking (Table 13). Sixteen of the 40 agencies are taking no precautions toward that end. The effectiveness of the practices of many of the other agencies also is questionable, insofar as specifications require a measure of stiffness (viscosity or penetration) at only one temperature, thus admitting asphalts of widely differing temperature susceptibilities. Penetration grading under ASTM D 946 requires conformance with a specified range of penetration at 77°F (25°C), and unless the 12 agencies using penetration-graded asphalts have supplemented the ASTM standard with viscosity or penetration at another temperature the performance of their pavements at both high and low temperatures could be adversely affected. Viscosity grading is used by 6 agencies, and most of these adhere to the limits set by AASHO M226, which offers the important advantage of including requirements of penetration at 77°F and viscosity at higher temperature (two temperatures in fact, at 140°F—60°C—and 275°F—135°C). Viscosity grading per se, however, does not control low-temperature cracking, particularly when the limiting penetration and viscosities were developed to include nearly all asphalts produced in the United States (*21*) and may not be high enough to control temperature susceptibility sufficiently to reduce low-temperature cracking (*147*). The asphalt selection practices that appear most effective are those of four Canadian provinces, where high-viscosity asphalt cement is specified or the asphalt is selected to be within a limiting stiffness at the design low temperature. Softer grades of asphalt cement (to 300-400 pen.) also are used, as well as viscous grades of slow-curing road oils.

Shahin and McCullough (*26*) have viewed thermally-induced cracks differently from most other investigators referenced in this section. Results from Shahin and McCullough's computer simulation, which predicted cracking by thermal fatigue and thermal shrinkage, agreed well with observed results from the Ontario Test Roads and the Ste. Anne Test Road in Canada. Additional quantitative data are necessary to more completely validate this procedure.

Freeze-Thaw Effects in Stabilized Layers

The third principal nontraffic-associated mode of distress that depends on cold regions environmental variables is disintegration of stabilized layers caused by freeze-thaw degradation. The state of the art in the analysis and control of such effects is summarized in a succeeding section of this report.

Cumulative Damage

The combined destructive action of traffic and climatic or environmental loads causes distress mechanisms to develop within the layered pavement structure and to propagate and produce distress under the modes of cracking, distortion, and disintegration. In most cases the damage takes place under repeated applications of either wheel loads or cycles of freezing and thawing, each repetition causing damage which, in its cumulative effects, eventually leads to a level of distress such that the pavement is no longer serviceable and must be rehabilitated. The rate of damage accumulation is highly dependent on seasonal environmental changes.

It has been shown, for example, that the rate of consumption of the fatigue life of a pavement is strongly influenced by the stress-strain properties of the asphaltic concrete and the subgrade soil (*95*), both of which change radically from one season to the next. Traffic-load-associated deformation also increases with increasing repetitions of wheel loads, and the effects of a given number of repetitions are more severe when the temperature of the asphaltic concrete is high and when the subgrade is softened by thawing conditions. Freeze-thaw degradation of stabilized layers clearly is progressive as the number of freezing cycles increases. Low-temperature cracking becomes more pronounced as the age of the bitumen increases, and probably also with an increasing number of diurnal and longer-term cycles of sharply falling temperatures. Even distortion caused by frost heave, often considered a transient manifestation of distress that produces roughness only in winter, sometimes leaves residual irregularities after thawing is complete, which become more severe after several years of seasonal freezing. This cumulative effect is particularly pronounced in the case of boulder heaves, which are described in Chapter Two.

In design practice the cumulative nature of damage to pavements, leading to manifestations of distress, is recognized and treated in varying degrees of effectiveness and in various ways. Those agencies whose design procedures account for low-temperature cracking, distortion caused by frost heave, and freeze-thaw degradation of stabilized layers seek to employ materials or layered structures that will prevent or minimize the initiation of distress of these types, rather than to limit the accumulated damage under a selected number of years or seasonal cycles to a certain predetermined level in consonance with the desired performance. Currently, no better alternative approach is available. The cumulative nature of damage caused by traffic loads is recognized and treated in current design practice by the dependence of the pavement structure thickness on the number of load repetitions selected for the design of each project. Such dependence is common to the design practices of all the agencies surveyed, excepting those that employ the CGRA procedure. The equations expressing the dependence have generally been developed through experience showing that under the environmental conditions prevailing in the particular state, a given pavement structure reaches a terminal limiting level of serviceability after a

TABLE 12

USE OF TRANSITIONS AND OTHER MEANS TO REDUCE FROST ACTION AT CULVERTS AND STRUCTURES[a]

STATE, PROVINCE, OR AGENCY	USE NFS BACKFILL ADJACENT TO CULVERTS?	USE TRANSITIONS OF DEPTH OF NFS MATERIAL ADJACENT TO CULVERTS?	ANY OTHER SPECIAL TREATMENTS TO MINIMIZE FROST ACTION AT STRUCTURES?
Alaska	Yes. <12% passing No. 200 sieve and PI < 6.	Yes. 3:1 cut slope for trench filled with bedding material.	Sleeves for piles to prevent frost jacking.
Alberta	No. Strive to prevent discontinuity in subgrade performance.	No, require ≥2 feet cover of common fill between top of pipe and subgrade.	Piles driven deep to resist frost.
British Columbia	Occasionally	Occasionally	NI
California	Pervious backfill used for all structures.	No	No
Colorado	No	No	No
Connecticut	Yes, pervious backfill for better compaction.	No	No
Idaho	No, always backfilled with soil.	No	Some pervious backfill used for structures at high embankments.
Illinois	No	No	No
Indiana	Yes, granular fill, < 10% passing No. 200.	No	No
Iowa	No	No	Granular fill behind bridge abutments, utility ducts below frost.
Kansas	No	No	No
Maine	Not done except by error in emergency repairs.	Sometimes. 20:1 slopes.	Backfill with excavated soil. Use insulation above or below shallow culverts.
Maryland	No	No	No
Massachusetts	No	No	Gravel backfill for bridge footings and walls.
Michigan	Yes, loss by washing not to exceed 7%.	Yes	Compacted granular backfill for footings. Minimum cover 3 feet over culverts, 4 to 5 feet over substructure units.
Minnesota	Use granular material with < 15% passing No. 200.	Yes, of granular backfill. See Fig.	Granular fill for bedding and for bridge approaches.
Montana	No	No	No
Nebraska	No	No	No
Nevada	No	No	Membranes to protect decks from deicing chemicals.
New Hampshire	Above and beside culverts, use sand and gravel with < 15% of minus No. 4 passing No. 200.	Only if cover <4 feet.	Grade set to provide cover over culverts ≥4 feet.
New Mexico	Use A-2-4 or better.	No	No
New York	Yes, with <10% passing No. 200, and <70% passing No. 40.	Yes, if regional soils engineer so requests.	Synthetic insulation for shallow pipes (experimental). Granular backfill for all structures.
North Dakota	Use slightly cohesive granular material.	Yes, in designated locations.	No
Nova Scotia	Yes, free-draining granular.	No	No
Ohio	No	No	No
Ontario	Yes, granular backfill if frost below top of pipe.	Yes	Native soil backfill for warm pipes such as sewers; transition of granular fill at bridge approach.
Oregon	Yes, free-draining material.	No	No
Pennsylvania	Yes, equivalent to subbase.	No	Nonfrost-susceptible pervious backfill in critical areas.
Quebec	Yes, <10% passing No. 200, and <3% passing No. 270.	Yes	High strength required for concrete exposed to deicing chemicals and to freeze-thaw.
Saskatchewan	NI	NI	NI
Utah	Yes	No	No
Vermont	Yes, granular backfill for structures and culverts.	Yes	Polystyrene insulation beneath box culverts on frost-susceptible soils.
Washington	Yes, free-draining material	No	No
West Virginia	Yes, granular material.	No	Underdrains for structures.
Wisconsin	Yes, granular backfill.	Yes	Special drainage, polystyrene insulation.

TABLE 12 (*Continued*)

STATE, PROVINCE, OR AGENCY	USE NFS BACKFILL ADJACENT TO CULVERTS?	USE TRANSITIONS OF DEPTH OF NFS MATERIAL ADJACENT TO CULVERTS?	ANY OTHER SPECIAL TREATMENTS TO MINIMIZE FROST ACTION AT STRUCTURES?
Wyoming	Sometimes use pervious backfill.	No	No
USDA-Forest Service	Yes	No	No
USDOD-Corps of Engineers	Yes	Yes	Culverts and foundations below frost depth, also special drainage.
USDT-Federal Highway Admin., Region 15	Yes, free-draining material that will not be intruded by local soil.	No	No
Massachusetts Turnpike Authority	Yes, clean sand or gravel to top of slope.	No	Minimum cover over culverts 3 feet.
Portland Cement Association	NI	Yes, we recommend this.	NI

[a] NI = no information.

certain number of traffic-load repetitions. The seasonally varying rate of damage accumulation in most cases has not been explicitly treated; instead, the total accumulation over a period of several years has been averaged. In other cases, such as the Corps of Engineers method and the Minnesota method, the design is based on the critical spring thaw condition, during which the rate of damage accumulation reaches a maximum.

Methods are available, however, for design of pavements based on the annual accumulation of damage that occurs at widely varying rates depending on temperature and freezing cycles. Moulton and Schaub (*102*) expressed the duration of the spring period of reduced subgrade support as a function of frost penetration, pavement thickness, and frost-susceptibility classification, and assumed the occurrence, immediately following construction, of a period of reduced

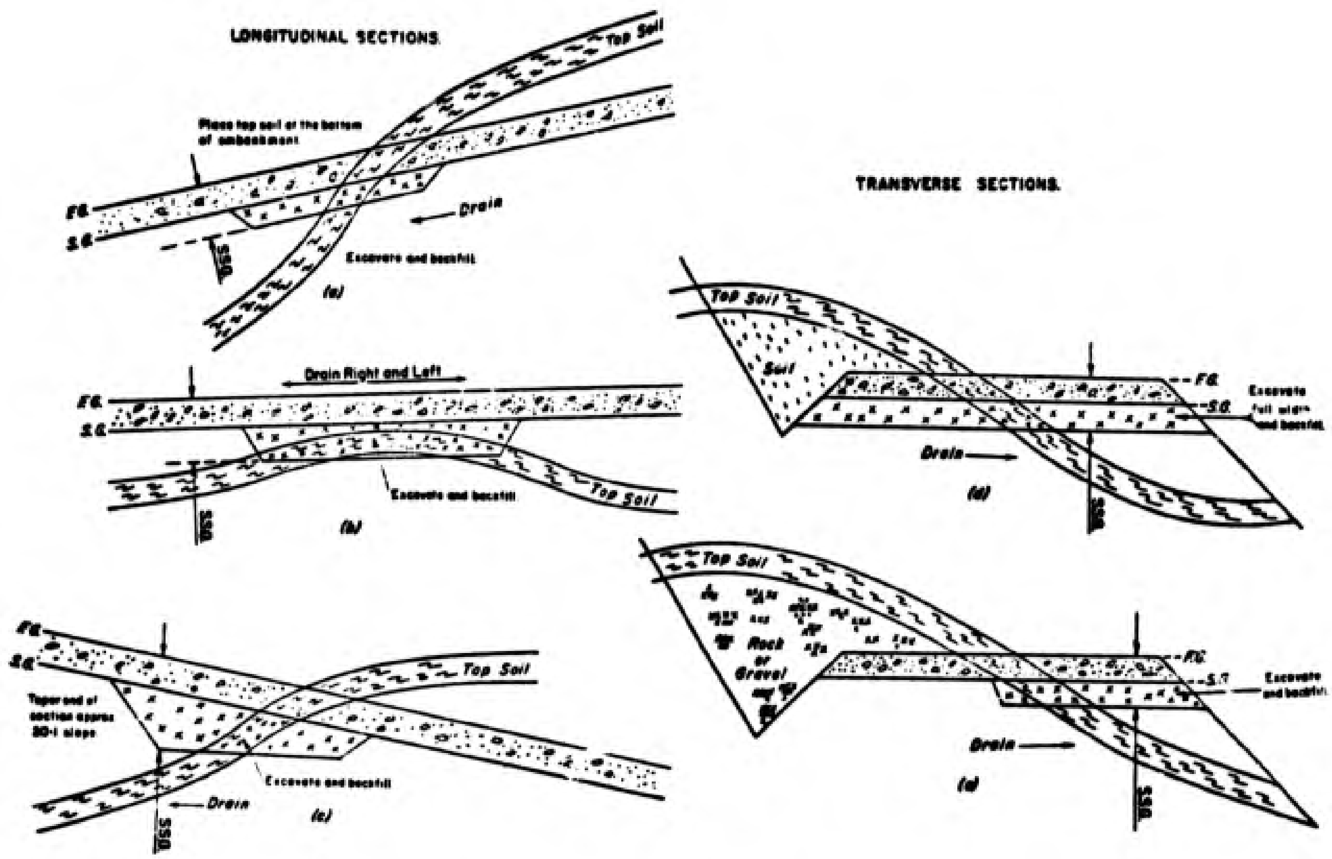


Figure 35. Granular wedges at transitions from cut to fill (Idaho).

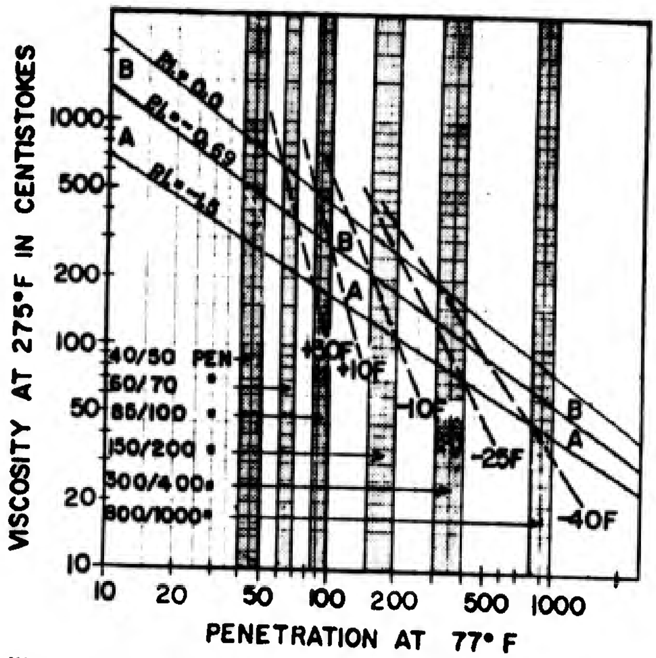


Figure 36. Guide for selecting grades of asphalt cement to avoid low-temperature transverse pavement cracking (25).

support equal to the sum of the annual weakened periods that occur throughout the design life. During the remainder of the design life, normal subgrade support was assumed to prevail, and separate increments of pavement thickness were calculated to carry the traffic projected during each period; the sum of the two incremental thicknesses gave the total thickness required. Marek and Dempsey (*148*) showed how a heat-transfer model may be combined with an elastic layered system analysis to compute stresses and deflections throughout the entire spectrum of seasonal variation in material properties that occurs in a 12-month period. This is an essential analytical process without which no analysis of cumulative damage could be wholly plausible. As the authors made many simplifying assumptions regarding seasonal variations in material response, the results are not definitive; their principal value is in the illusration of the concept and a procedure for use of heat-transfer models as an adjunct to stress analysis. An analysis of accumulated fatigue damage on a highway in Saskatchewan has been reported by Bergan and Monismith (*91*). The stiffness of the asphaltic concrete in the critical spring, summer, and fall months of April through October was determined in relation to pavement temperatures calculated for an average day in each month. Subgrade moduli were calculated from deflection measurements during the same period. Daily fatigue was accumulated during six 4-hour intervals, and summed for each year since the road was opened to traffic; the accumulated fatigue damage was used successfully to predict the year of pavement failure.

RIGID PAVEMENT DESIGN

Rigid pavements also are layered structural systems. But, unlike flexible pavements, they consist usually of not more than three layers—the concrete slab, sometimes a base, and the subgrade. Rigid pavements evidence distress (Table 1) in the form of cracking of the concrete slabs under the combined action of repeated traffic loading (fatigue) and changes in temperature and moisture, and distortion (which may lead to cracking) either by differential heave and settlement, pumping, curling caused by differential temperature and moisture conditions, or by faulting at joints and cracks. Distress in the form of abrasion by traffic and disintegration of the concrete is excluded from the scope of this report.

Of the various causes of cracking and distortion, only differential frost heave and thaw settlement are unique to the frost areas. The load-associated-cracking mode of distress also is materially affected by seasonal freezing and thawing—affected favorably when stresses in the slab caused by traffic loads are reduced owing to the frozen condition of the supporting medium, and adversely when thaw-weakening and differential thaw consolidation in supporting materials cause higher stresses to develop in the concrete slab. The net result of these contrary seasonal effects is a matter of controversy, as is seen in the following brief outline of the procedures currently available for dimensioning concrete pavements to resist traffic and environmental effects.

TABLE 13

SELECTION OF ASPHALTS TO CONTROL LOW-TEMPERATURE CONTRACTION CRACKING

ASPHALT SELECTION	11 AGENCIES SURVEYED IN DETAIL [a]	OTHER 30 AGENCIES REPORTING OCCURRENCE OF LOW-TEMPERATURE TRANSVERSE CRACKS IN ASPHALTIC PAVEMENTS
Penetration grading, to 150 pen.	4	5
Penetration grading, to 300 pen.	1	2
Viscosity grading	—	2
Viscosity grading, and use of softer grades, to AC-5	1	1
Viscosity grading, and use of high-float emulsion	2	—
High-viscosity asphalt cement in soft grades to 300–400 pen.	—	1
High-viscosity asphalt cement, and viscous grades of SC liquid asphalts	2	—
Asphalts selected to have stiffness of 30,000 psi at design temp.	—	1
No precautions taken	—	16
No information	—	2

[a] One agency reported that low-temperature cracking is not a problem.

Methods of Stress Analysis

Tensile and flexural stresses causing cracking in rigid pavement slabs are of three principal types:

1. *Warping stresses*, caused as the weight of the slab resists the tendency of the slab to curl under temperature and moisture gradients from top to bottom.

2. *Longitudinal restraint (friction) stresses*, caused as either the friction between the slab and the supporting medium or a physical tie to longer lengths of pavement, anchors, or structures restrains shrinkage of the concrete as its temperature and moisture content fall.

3. *Load stresses*, caused as the slab resists wheel loads.

In addition to the cracking caused by these stresses, cracking or crushing can also be caused by compressive longitudinal restraint producing lifting and buckling of the slab, commonly called "blow up" effects.

These stresses can be calculated by methods summarized in any textbook or reference book on pavement design (*149, 150*, for example). In usual practice, warping and friction stresses are relieved by dividing the pavement into discrete rectangular elements and providing properly designed joints between those elements, or are counteracted by steel reinforcement of the slab to control the ensuing cracking. The computed wheel load stress may also be increased by a factor to account for the residual effects from temperature and moisture gradients that are not neutralized by joints and steel reinforcement. The thickness of the slab is determined by calculating the flexural stresses in the concrete caused by wheel loads and comparing them with the modulus of rupture (flexural strength) determined for the selected concrete mix. The cumulative effects of repeated wheel loads, causing fatigue in the concrete, also are taken into account in the stress analyses.

Stresses in the slab caused by wheel loads can be analyzed by theories of continuously supported elastic slabs, theories of layered systems, or finite element models. A comprehensive review of these methods by Barenberg (*151*) includes an enlightening analysis and comparison of the critical assumptions, advantages and disadvantages, and potential of each approach for future development and application to pavement systems. The methods currently in use for analysis of rigid pavements are of the first type (*152, 153*), whose application is limited to two-layer systems in which the top layer, the concrete slab, is much stiffer than the bottom layer, the subgrade. (When a very stiff base is introduced between the slab and the subgrade, a modified theory is more applicable.) In the Westergaard theory the assumption is made that the deflection at any point on the subgrade surface is directly proportional to the vertical stress applied at that point. The constant of proportionality, k, is termed the modulus of subgrade reaction, expressed in pounds per square inch per inch, or pounds per cubic inch. A principal concern in designing pavements in frost areas to resist applications of wheel loads is the effect of seasonal variation of subgrade modulus on the performance of concrete pavements.

Three of the methods in current use (Table 14) for dimensioning concrete pavement structures are those of AASHO (*87*), Portland Cement Association (*90*), and Corps of Engineers (*154, 20*). Key assumptions and approaches inherent in the three methods are compiled in Table 15; they are believed to be self-explanatory, and only certain aspects of subgrade support and of use of bases is enlarged upon.

TABLE 14

ANALYTICAL METHODS FOR DESIGN OF RIGID PAVEMENTS IN CURRENT USE BY ELEVEN AGENCIES [a]

ANALYTICAL METHOD	NO. OF AGENCIES
AASHO	4
PCA	1
Corps of Engineers	1
None; standard designs	2
None; rigid pavements not used	3

[a] See Appendix B.

Subgrade Support

Most of the agencies surveyed stated (Table 16) that the slab thickness selected by their designers varies with the modulus of subgrade reaction, k. Values of k determined in summer and fall are used in the design procedures of AASHO and PCA; by the former although recognizing that further research is needed to enable the design procedure to account for environmental conditions differing from those of the AASHO Road Test site, and by the latter in the professed belief that the periods of reduced subgrade support have very little effect on concrete pavements. PCA, in justifying that opinion, takes the position that concrete slabs reduce soil pressures to safe limits by distributing loads over large areas and that the accelerated fatigue effects during periods of reduced subgrade strength are offset by reduced fatigue during the period that the subgrade is frozen.

Vertical pressures on subgrades certainly are lower under rigid pavements than under flexible pavements; Barenberg (*157*) estimated that maximum stresses transmitted to the subgrade through flexible pavements under highway vehicles range from about 5 to 30 psi (34 to 210 kPa), whereas the corresponding range under rigid pavements is about 1 to 5 psi (7 to 34 kPa). This relationship does not necessarily mean, however, that seasonal changes in subgrade stiffness are relatively less significant for rigid pavements. A possible contrary perception of the lower subgrade pressures under rigid pavements is that if it can be assumed that both flexible and rigid pavements are dimensioned to have the least thickness the subgrade response will permit, the lower subgrade pressures under rigid pavements merely signify that to ensure acceptable performance this type of pavement must be dimensioned to transmit much lower pressures to the subgrade. Thus, simple consideration of the magnitude of subgrade stresses does not seem to provide a definitive answer to this question. Measurements of pavement performance in terms of varying rates of damage accumulation throughout the year probably are better indicators of the significance of seasonal changes in subgrade k-value, but such measurements are extremely rare. At the AASHO Road Test (*6*), a seasonal weighting function was

TABLE 15

COMPARISON OF ASSUMPTIONS AND APPROACHES FOR PAVEMENT DESIGN IN SEASONAL FROST AREAS BY THREE CURRENT RIGID PAVEMENT ANALYTICAL METHODS

APPROACH	AASHO [a]	PORTLAND CEMENT ASSOCIATION [b]
Calculation of wheel load stresses		
Equations of	Westergaard (as modified by Spangler)	Westergaard
Definition of failure	From performance model	Not explicitly defined. Design criteria adjusted by experience on in-service pavements.
Stress calculated at	Corner of slab	Transverse joint edge
Factors applied to calculated stresses		
Impact factor	Not applied, because stress analysis used only to account for conditions differing from those at AASHO Road Test, such as properties of subgrade and concrete.	—
Load factor		Interstate class, 1.2; other, 1.0–1.1
Design factor		—
Warping and friction stresses treated by	Design criteria account only for such stresses that developed in pavements at AASHO Road Test	Control by proper joint design, and/or steel reinforcement
Load repetitions treated by	Relationship derived from AASHO Road Test, expressing serviceability in terms of load applications	Fatigue analysis based on calculated stress ratios,[e] accumulating damage under each axle-load group
Determination of concrete strength		
Test for flexural strength	Modulus of rupture (M–R)	M–R
Age of specimens when tested	28 days	28 days [f]
Factor applied to test value of M-R for use in design charts	0.75	1.00
Subgrade support		
Parameter	Modulus of subgrade reaction (k)	k
Determined by:		
Nonfrost areas	Plate test or estimate	Plate test or estimate
Frost areas	Plate test in summer or fall, or estimate	Plate test in summer or fall, or estimate
Increased as function of subbase thickness and type?	Yes, both	Yes, both
Seasonal variation accounted for?	No, except that performance equation accounts for whatever seasonal variation occurred during AASHO Road Test.	No. Summer/fall value used as reasonable mean value.
Base required by design criteria		
Required for protection against pumping or frost action?	Pumping and frost action	Pumping
Types recommended:		
Unbound	Dense-graded, open-graded	Dense-graded, open-graded
Bound	CTB, ATB, lime-treated	CTB
Thickness recommended	4 in. min. Additional thickness for frost protection determined by local experience.	4–6 in.

[a] AASHO 1972, Highway Research Board 1962 (*87, 6*).
[b] Portland Cement Association 1966, Portland Cement Association 1971 (*90, 155*).
[c] Department of the Army 1969, Department of the Army 1965, Hutchinson 1966, Ohio River Division Laboratories 1961 (*154, 20, 156, 89*).
[d] More advanced cracking is foreseen in thickness criteria for subgrades with $k \geq 300$ psi/in.
[e] Load stress divided by modulus of rupture.
[f] Under certain conditions use of 90-day strength is advocated.

developed for the performance analyses of flexible pavements; but for rigid pavements all observed distress was related to pumping, seasonal variations in damage rate were less pronounced, and no seasonal weighting function was developed. Although this result of the AASHO Road Test supports the position taken by the Portland Cement Association, it was shown by the Corps of Engineers' Frost Effects Laboratory (*158, 159*)* that the spring reduction in subgrade reaction in plate bearing tests is very large, and examination of design criteria of the Portland Cement Association (*90*) shows that its effect on fatigue life also is

* Many other research investigations of loss in bearing capacity during thaw also have been conducted. For a comprehensive review of the literature, see Cumberledge (*160*).

CORPS OF ENGINEERS [c]
Westergaard First crack in 50% of slabs [d]
Free edge or joint without load transfer device
1.25 Stress reduced by 33% to account for load transfer 1.30
Partially included in design factor, and partially controlled by proper joint design
Empirical relationship, from traffic tests on airfield pavements, between load repetitions and a thickness parameter. Design factor accounts for fatigue.
M-R 28 days 1.00
k
Plate test or estimate Chart of k as function of FS group of subgrade, and thickness of subbase Thickness only
No. Thaw-weakened value used for design.
Pumping and frost action; must satisfy drainage criteria.
Open-graded
—
Pumping: 4 in. min. Frost action: varies; usually min. thickness equal to thickness of slab, unless soil and groundwater conditions favorable.

very large. It can be shown that in accordance with the latter criteria the total damage accumulation (consumption of fatigue life) during the period of subgrade weakening, as determined by Sayman, would in some cases far exceed the total damage accumulation for an entire year computed by means of the design criteria by assuming the summer/fall k-value applied as a reasonable mean throughout the year.

TABLE 16

DETERMINATION AND USE OF MODULUS OF SUBGRADE REACTION, k, IN DESIGN OF RIGID PAVEMENTS

ITEM	11 AGENCIES SURVEYED IN DETAIL [a]	OTHER 29 AGENCIES REPORTING FROST CONSIDERED IN DESIGN [b]
Use of rigid pavements:		
Used	8	22
Not used	3	7
Does design thickness of slab vary with k-value?		
Yes	7	14
No	1	7
No information	—	1
How is k-value determined?		
Correlated with CBR, R-value, or triaxial test data	4	7
Correlated with soil classification data	1	1
Inferred from FS group	1	—
Estimated	1	—
Plate test or estimate	—	1
No information	—	5
Is k-value increased depending on thickness and/or type of selected subbase?		
Yes	5	10
No	2	4

[a] See Appendix B.
[b] See Appendix A.

The Corps of Engineers, in current design criteria for rigid pavements in seasonal frost areas (*20*), makes use of sharply reduced k-values, which were developed from extensive traffic tests on airfield pavements during frost melting periods. Values of reduced k-value divided by normal summer k-value corrected for saturation depend upon the frost-susceptibility classification (FS group) of the subgrade soil, and with the thinnest permissible bases (4 in., 100 mm) generally are between about 0.1 and 0.5. With thicker bases the reduction is considerably less. The thickness of the concrete slab is determined as if the reduced k-value were applicable throughout the year.

The practice of the Corps of Engineers of applying the reduced k-value in the design of pavements for service throughout the year appears to be excessively conservative, although the assumption by the Portland Cement Association that the summer/fall value is a reasonable mean annual value seems to be decidedly unconservative. On the other hand, experience indicates satisfactory performance of pavements designed by both these methods. The AASHO method, leaving to local experience the determination of necessary frost protection, could lend itself to either underestimation or exaggeration of the seasonal effects. To complicate matters further, the diversity of special factors applied to stresses and strengths, and other differing practices of the three agencies (Table 15), makes it virtually impossible to compare the design methods solely on the basis of method of appraisal of seasonal variation of k-value.

One is led to conclude that the state of the art in assessing subgrade support is quite rudimentary on at least two counts: First, excepting the design k-values used by the Corps of Engineers that are based on traffic tests, the subgrade modulus is conventionally measured under a single application of a static load on a plate, or estimated from the results of other static and nonrepetitive load tests, even though the dynamic effects of traffic loads, and their many repetitions, are known to cause a different subgrade response. Second, the subgrade modulus is conventionally assessed at only one season, even though its seasonal variation has a marked effect on the rate of accumulation of fatigue damage.

Use of Base

Bases are used beneath rigid pavements for many reasons, including: (a) to prevent pumping; (b) to control volume change in expansive clay subgrades; (c) to reduce frost heave; (d) to minimize loss of subgrade support during spring thaw; (e) to increase the modulus of subgrade reaction; (f) to provide a more stable working platform for construction equipment; (g) to replace soft or highly compressible subgrade soils; and (h) to provide drainage. The prevention of pumping is a primary and essential requirement and most of the agencies surveyed recognize its importance (Table 17). Pumping is a problem that must be addressed in designing any rigid pavements whose supporting materials may at some time have high moisture contents. Consequently, prevention of pumping through use of a suitable base course is especially critical in pavements whose subgrades will experience freezing and thawing, which in most instances cause a sharp increase in moisture content during the spring. Criteria are available, from the three design procedures already referred to, for selecting either granular unbound base or one of several types of treated bases for the purpose of affording protection against pumping.

Among the various functions of bases, only those related to frost heave and loss of subgrade support during thaw are considered further herein, as the others are important also in regions that do not experience frost. The use of granular base or subbase as a means of draining a flexible pavement substructure (see earlier sections on "Base-Course Materials" and "Base-Course Drainage") in large part is applicable to rigid pavements as well. Both detrimental heave and thaw-weakening can be controlled and largely prevented by use of thick bases. Criteria of the Corps of Engineers (*56*) that were developed from research and experience with airfield pavements can be used to determine the combined thickness of pavement and nonfrost-susceptible base that would be necessary either to prevent subgrade freezing or to severely limit the depth of frost penetration into the subgrade. These criteria were developed to restrict total and differential heave to amounts acceptable for high-speed airfield runways. In areas of high freezing index, the thickness required to fulfill these criteria is too great to be economically feasible for roads. Furthermore, the high degree of control of frost-induced pavement roughness afforded by these airfield pavement criteria is not necessary for roads, which under Corps of Engineers' criteria generally are designed by the alternative approach of ensuring only that the pavement can resist the imposed wheel loads during the critical spring period of reduced subgrade support (*133*). Frost heave in such cases can be excessive, however, and additional base thickness is sometimes used to keep pavement heave and consequent cracking within tolerable limits (Table 18). A more generalized practice of the agencies surveyed is to control differential heave by special techniques of subgrade preparation and tapered transition wedges. These techniques are similar to those described for flexible pavements (see earlier section on "Current Practice in Dimensioning Flexible Pavements") and include undercutting of heave-susceptible soils and replacement with selected material, removal of boulders, scarifying and blending the top 1 to 2 ft (0.3 to 0.6 m) of subgrade to interrupt undesirable stratification, cleaning of soil-filled joints of rock in cuts and refilling with better material, in-place fragmentation of rock in cut sections by blasting, and transition wedges of granular soils at centerline culverts and at changes from cut to fill either in the subgrade profile or cross section.

There is general agreement that base course reinforces the subgrade; the three design methods summarized in Table 15 recognize an increasing k-value with increasing thickness of base, and 15 of the 21 agencies that use k-values in design apply a scale of increases of that parameter related to increases in base thickness (Table 16). As the fatigue damage to a concrete slab appears to depend heavily on the degree of spring weakening of the subgrade, it follows that use of base course of adequate thickness should be an effective means of reducing the incremental slab thickness that, in the absence of a base, would be necessary for satisfactory performance in fatigue during the weakened period. The strengthening effect of untreated and treated base course is shown by the criteria of the Portland Cement Association (*90*) reproduced in Figure 37. The values given refer to average summer/fall conditions and do not include any reduction during the spring thaw. Corps of Engineers' practice in selection of the thaw-weakened k-value of a given subgrade soil, as a function of the thickness of granular nonfrost-susceptible base, is shown in Figure 38.

Use of Rigid Pavements and Selection of Design Thickness in Practice

Although the extent of use of different types of pavements was not a principal topic of inquiry in the survey of practices, it is worthy of note that 10 of the 40 agencies reporting that frost is considered in their road design do not use rigid pavements. The reasons for this policy were not queried, but it is known that among the various factors are local or regional conditions affecting construction costs and concern with the effects of differential frost heaving. On the other hand, Appendix B shows that several agencies have made only limited use of flexible pavements, particularly on Interstate and other heavy-duty roads, where pavements in those states have been almost entirely rigid. These use patterns appear to have developed in response partly to performance and cost experience, partly to traditional preferences and prejudices of the agencies concerned, and partly to the particular established sources of financing for construction and maintenance activities.

It has been noted that in the design practices of several agencies no measurement or estimate of subgrade support is made and slab thicknesses are selected without regard to subgrade properties, whose reasonable uniformity and adequacy are achieved through various techniques of subgrade preparation. In those cases, the slab thickness depends only on a projection of the number of repetitions and weight of axle loadings. Nevertheless, it appears that even those agencies making use of k-values in their design procedures have adopted slab thicknesses, and in many cases bases as well, that are essentially standardized. Only a slight dependence, even on traffic, can be discerned from the pattern of use of particular slab thicknesses. The limited data on layered pavement structures summarized in Appendix B tend to confirm the extensive survey of practice reported by the Portland Cement Association (*161*), which shows that most concrete pavements for highways in the United States and Canada are 8 or 9 in. (200 or 230 mm) thick, with a few as thick as 10 in. (250 mm) and fewer still that are less than 8 in. These results, which apply across a wide range of temperature and moisture conditions, and to a wide range of traffic loadings and repetitions, strongly suggest an element of overdesign in many existing pavements where environmental conditions and traffic are less severe; they also lay emphasis on a need for adoption of procedures to optimize the slab thickness as a function of these critical parameters. This need can be met in part by making use of a cumulative damage approach similar to the fatigue analysis of the Portland Cement Association (*90*), but with the important distinction of use of seasonal mean k-values from measurements in spring, summer/fall, and winter rather than values assumed to represent annual means. Further developments, to assess more accurately the effects of bases, stabilized or cemented materials in multiple layers, composite pavements comprising several stages of overlays, dynamic effects of moving loads, etc., must await evolution, from the domain of research to that of design practice, of the layered system and finite element models, which have greater potential for these purposes than the elastic slab models currently in use.

ROADWAY DRAINAGE PRACTICE

As is well known, adequate surface and subsurface drainage is a fundamental objective of the pavement design process. Adequate drainage is even more essential in areas of seasonal frost than elsewhere, because water is responsible for the majority of the ill effects of low temperatures. Virtually all highway design agencies in areas subject to seasonal frost employ drainage features in design that are particularly suited to minimizing frost problems. It is not intended here to give a synthesis of general drainage practice, but rather to detail the aspects of drainage that are used or adjusted to achieve satisfactory drainage in the seasonal frost environment.

All agencies were queried as to whether the water-table elevation with respect to frost-susceptible soil influences roadway design. Only California, New Mexico, New York, and Pennsylvania indicated that it did not. However, it is probable that their practices do not disregard this consideration, but accommodate it in fashions similar to Colorado and Utah (which indicate that drainage design is influenced) or Kansas (where standard sections include deep ditches). All the other respondents offered remarks to indicate the way in which the water table with respect to frost-susceptible soil influences their designs. Some states or provinces have several approaches to the problem.

The use of underdrains or interceptor drains is the most widely employed technique (22 of the respondents), with the objective of lowering the water table. Alternatively,

TABLE 17

USE OF BASE UNDER RIGID PAVEMENTS

ITEM	11 AGENCIES SURVEYED IN DETAIL	OTHER 29 AGENCIES REPORTING FROST CONSIDERED IN DESIGN
Rigid pavement not used	3	7
Type of base used:		
Granular	4	9
CTB	1	6
Granular or CTB	1	3
Granular, CTB, or ATB	1	3
Granular or ATB	1	—
ATB or CTB	—	1
Base is intended to:		
Control pumping	4	10
Reduce frost heave	1	1
Neither	—	1
Both	3	8
No information	—	2
How base thickness is determined:		
Standard thickness	4	13
Varies with subgrade classification or strength	—	3
For a certain total thickness in relation to frost depth	1	5
Leveling course	—	1
As necessary to achieve required strength	2	—
As necessary to support construction traffic	1	—

TABLE 18

USE OF ADDITIONAL BASE UNDER RIGID PAVEMENTS FOR FROST PROTECTION BY ELEVEN AGENCIES SURVEYED IN DETAIL

ITEM	NO. OF AGENCIES
Rigid pavement not used	3
Additional base used for frost protection?	
Yes	3
No	5

Table 1. Effect of Untreated Subbase on *k* Values, pci

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

Table 2. Design *k* Values for Cement-Treated Subbases

(Subgrade *k* value—approx. 100 pci)

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

Figure 37. Effect of base on modulus of subgrade reaction, k, *according to design criteria of Portland Cement Association* (90).

raising the grade to provide greater separation between the water table and the subgrade is a technique used by 12 agencies. Some of these respondents have as a criterion a minimum specified height that the final grade shall be above the water table: Massachusetts, 7 ft (2.1 m); Michigan and Minnesota, 5 ft (1.5 m); Saskatchewan, 8 to 12 ft (2.4 to 3.7 m). Nebraska's criterion is that the subgrade shall be 3 to 4 ft (0.9 to 1.2 m) above the water table in granular materials, and 7 ft (2.1 m) above the water table in cohesive soils.

The removal of frost-susceptible soils and backfilling with acceptable materials can also be done where the water table and frost-susceptible soils lie in an unfavorable relationship. This practice was reported by four states.

Alterations in details of the typical section are also made. Alberta, Idaho, Illinois, Maine, Minnesota, Nebraska, Quebec, Wisconsin, and Wyoming use deepened ditches. Alberta places ditches 1 to 2 ft (0.3 to 0.6 m) lower than normal on frost-susceptible soils; in Quebec the ditch depths are designed to be 0.8 times the depth of frost penetration. Thickening of the base is practiced by Alaska, Illinois, and Oregon. In Alaska, a high water table dictates an increase of 0.5 to 1.5 ft (0.2 to 0.5 m) in the total thickness of the pavement structure. Oregon's practice, wherever a high water table is encountered, is to increase the base thickness to something more than their standard of one-half the depth of frost penetration. Nevada and Washington use deeper typical sections; the former's decision criterion is capillary wetting of the subgrade, and the latter's is that the water table lies within 5 ft (1.5 m) of the subgrade.

Other respondents have given comments that do not indicate how their designs are influenced by the relationship between the water table and frost-susceptible soil, but rather provide criteria that determine only whether their designs shall be influenced. Delaware indicates concern "only if this condition exists relatively close to the subgrade." Iowa considers the condition a determinant in design if the water table is found within frost-susceptible soil with sand pockets below. Montana does not consider a soil to be frost-susceptible unless the water table is present. Vermont states that material classified A-4 under the AASHO system, if dry, is generally not replaced at the subgrade level. In West Virginia it is assumed that water has an influence if the silt content is greater than 30 percent in silty clays or clayey silts. Wyoming's designs are affected only if water can be anticipated from below, within the vertical limits of frost penetration. In Ohio, frost-susceptible soils are specially treated in nonfill sections, but the treatment is unspecified.

The questionnaire sought to determine if different drainage features are used in areas subject to frost problems compared with nonfrost areas. Owing to the wording of the question, many states replied in the negative because their entire areas are subject to frost problems. Nonetheless, several states responded affirmatively and gave comments (some of which duplicate or amplify the information already given in the foregoing). In Alaska, slopes have been planted with willow and alder, so that evapo-transpiration will remove water from the soil. Alaska also has a practice of placing 2 ft (0.6 m) of gravel as an underlay beneath culverts on frost-susceptible soil. Connecticut's underdrains are 6-in (150 mm) pipe, placed 5 ft (1.5 m) below the pavement on the high side of the road. As the severity of frost problems increases in Illinois, the number of underdrains used increases, and ditches are placed deeper. A similar comment comes from Minnesota, with the added notations of deeper subgrade corrections and the use of granular drainage material. In Massachusetts, underdrains in cut sections and bleeders in embankment sections are standard throughout the state. Montana indicates that culvert sizings vary with respect to frost problems, and it is understood that this is done in response to icing difficulties. Designers in Nevada attempt to maintain ditch bottoms within the subgrade at all locations, and no special approaches apply to frost areas in particular. Saskatchewan uses drainage tiles in cuts or slopes to intercept water from broad aquifers. Vermont covers wet slopes in frost-susceptible soil with gravel or rock. Wisconsin's treatment of ditches includes not only deepening but also widening; that state also uses a full-width base where the improvement of drainage is required.

Finally, the respondents were asked if ditch bottoms are carried at a lower level than the subgrade in frost areas, and if so, how much lower. Only California and Colorado indicated that this is not done. But among the design agencies that do carry ditches lower than the subgrade, there is a broad range in the elevation difference chosen (Table 19). Three states gave their standard ditch depths as referenced to final pavement grade: Delaware, 2 ft (0.6 m); Iowa, 5 ft (1.5 m); and North Dakota, 4.9 ft (1.5 m). The District of Columbia, Indiana, Maine, Quebec, and Saskatchewan indicated that ditch bottoms are always placed below the subgrade, and Texas indicated that this is generally done, but no depths were stated. However, as noted

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.

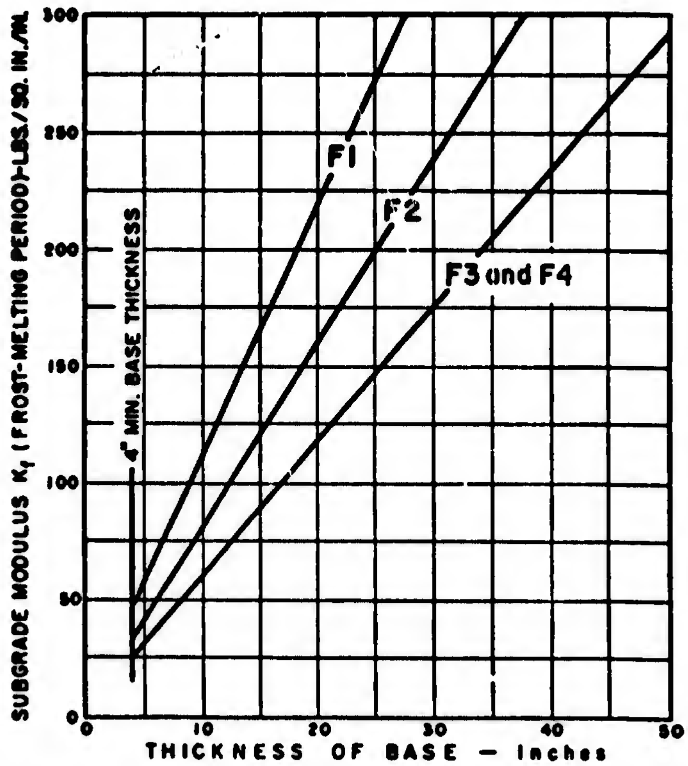


Figure 38. Effect of granular base on modulus of subgrade reaction during spring thaw, according to criteria of Corps of Engineers (133).

earlier, in Quebec the design ditch depth is 80 percent of the frost-penetration depth. In New Hampshire, ditches are level with the subgrade, or lower if possible. Massachusetts also places ditches lower than the subgrade, but not below the depth of any nonfrost-susceptible material that has been used to replace frost-susceptible soil. In Ohio, the ditches are not always placed below the subgrade level; rather, the bottom elevation is dependent on the pavement depth.

It is somewhat surprising that there was little or no mention by the respondents of certain items such as base-course drainage, base-course permeability criteria, filter courses, filter criteria, the effect of width of base in relation to that of the pavement, paved shoulders, or particular culvert and underdrain location practices. These items can affect drainage performance vis-a-vis the seasonal frost environment. For example, paved shoulders are widely used for a variety of purposes. However, one benefit of paved shoulders is the limitation of water entry into the base, subbase, and subgrade. For a second example, the Corps of Engineers specifies the general criterion that base courses shall be capable of 50 percent drainage in 10 days (see section on "Flexible Pavement Design, Base-Course Drainage"). A third example is that in Alaska and parts of Canada one component of the selection practice for culvert locations is to anticipate where extra culvert capacity will be needed when normally designed culverts will be obstructed by icing. This leads to more numerous cross drainage on valley sides, larger-than-normal culvert sizes, and the use of "stacked" culverts (i.e., culverts located well above the base of a fill, generally above and to one side of the normal culvert). Stacked culverts can carry the flow that runs on the surface of an icing in the spring, such that a washout is prevented during the critical period while the normal culvert is blocked by an icing; oversize culverts and more numerous culverts serve a similar purpose. Other approaches applicable to icings, which may be considered in the design process, are discussed by Carey (*30*).

TABLE 19

ELEVATION DIFFERENCES FOR DITCHES LOWER THAN SUBGRADE IN FROST AREAS, BY RESPONDING AGENCIES

ELEV. DIFF. (FT)	(M)	AGENCIES
0.5	0.15	Minnesota (minimum); Michigan (minimum); Vermont (normally); Washington
0.7	0.2	Wisconsin (minimum)
1.0	0.3	Idaho (minimum); Michigan (preferred); Nebraska (minimum); U.S. Forest Service
2.5	0.8	Illinois
3.0	0.9	Oregon (as much as); Wyoming (standard, but may be as deep as frost-penetration depth)
3.5	1.1	Alberta (standard, but 4 to 6 ft—1.2 to 1.8 m—deep in frost-susceptible soil)
Variable		Alaska; Brit. Columbia; Connecticut; Kansas; Maryland; Mass. Turnpike Auth.; Montana; New Mexico; New York; Nova Scotia (2 to 3 ft—0.6 to 0.9 m); Ontario (1 to 5 ft—0.3 to 1.5 m); Tennessee (1 to 2 ft—0.3 to 0.6 m); Utah

OTHER FACTORS

Differential Frost Action at Culverts, Utility Ducts, Drains, and Structures

A culvert, duct, drain, or other structure crossing the pavement comprises a discontinuity in a roadway foundation in terms of its strength, its deformation under load, and its behavior under conditions of seasonal frost. Consequently, the design of any such structure must tend toward minimizing the effects of the discontinuity as well as providing for structural integrity and functional serviceability. Insofar as behavior under frost conditions is concerned, a variety of approaches is used by highway design agencies to avoid or reduce any ill effects these discontinuities may impose (see Table 12).

The most common technique employs nonfrost-susceptible granular material as backfill around structures generally, and especially around culverts. Although most respondents simply indicated that granular, pervious, free-draining material is used, some agencies provided their gradation limits, as follows:

Alaska	Maximum of 12% passing No. 200, PI $\leq$ 6
Delaware	Maximum of 25% passing No. 200
Indiana	Maximum of 10% passing No. 200
Minnesota	Maximum of 15% passing No. 200
New Hampshire	Of the minus No. 4 fraction, maximum of 15% passing No. 200
New York	Maximum of 70% passing No. 40, and maximum of 10% passing No. 200
Quebec	Maximum of 10% passing No. 200, and maximum of 3% passing No. 270

Idaho notes that granular backfill is used only on certain structures supporting high embankments. In Iowa, granular backfill is used only behind bridge abutments, where it extends from the full abutment depth upward on a 1:1 slope out from the structure. The practice in Massachusetts is to use gravel as backfill, placed a minimum of 12 in. outside of bridge footings, walls, etc. Vermont indicates that granular backfill is applied to culverts and structures, and in the latter case the backfill that is placed adjacent to structural elements is as much as 4 ft (1.2 m) in width. The criterion in Pennsylvania is that structure backfill shall be equivalent to subbase material. In North Dakota there is a preference for using slightly cohesive granular material. Granular material is used at structures in Ontario, and also around culverts if the frost penetration depth is below the top of the pipe. However, for warm pipes, such as sewers or certain utility ducts, Ontario uses a native soil backfill. In Maine there is a strong belief in maintaining soil uniformity by using the native soil as backfill. The philosophy in Alberta is somewhat similar, with the aim being to prevent discontinuities in final performance.

The occurrence of abrupt differential heaving at culverts can be minimized by a transition or gradual increase in depth of nonfrost-susceptible material on each side of a culvert. Only 12 questionnaire respondents indicated that this technique is used. Of these, Alaska uses a 3:1 cut slope for the transition, Maine uses 20:1 transition slopes, and New Hampshire applies the practice only to culverts having less than 4 ft (1.2 m) of cover. The Portland Cement Association also endorsed this design approach.

The placing of structure footings below seasonal frost is probably a much more common design practice than the questionnaire responses would indicate. Only eight agencies mentioned this approach; of these, Wyoming noted that both the top and bottom surfaces of spread footings are placed below the frost penetration depth.

Similarly, a minimum-cover requirement for drains, culverts, and ducts is believed to be a more widespread criterion than could be inferred from the questionnaire responses. Only five agencies indicated a minimum cover requirement: Alberta (2 ft—0.6 m—or more between top of culvert and subgrade), Iowa (for utility ducts only, kept below the frost penetration depth), Massachusetts Turnpike Authority (3 ft—0.9 m), Michigan (3 ft, with the note that substructure units receive 4 to 5 ft—1.2 to 1.5 m—of cover, depending on soil textures), and New Hampshire (4 ft).

The insulation of culverts, utility ducts, and drains is finding increasing use, but the practice is not yet widespread. In Alaska, in situations where other constraints may make the deep placement of water and sewer lines impractical, these facilities are insulated. On very shallow drainage facilities in Maine it is standard practice to apply insulating material as a layer either above or below the conduit. In Michigan, only utility ducts receive insulation, where this is indicated by the specific site conditions. Wisconsin sometimes employs polystyrene insulation layers above drainage facilities to reduce frost penetration. New York has conducted experiments with insulating shallow culvert pipes; rigid polystyrene and foamed-in-place polyurethane have been used, but conclusive results of these experiments are not yet available. On frost-susceptible soils in Vermont, polystyrene insulation has been used beneath reinforced concrete box culverts to prevent or minimize frost heaving.

A variety of other techniques was reported to be employed in connection with structures in areas subject to frost effects. In both Alaska and Alberta, piles are driven deep to prevent heaving and frost-jacking. Additionally, pile sleeves may be used in Alaska to prevent adfreezing of heaving soils to the pile surfaces. In West Virginia it is standard practice to provide underdrains for structures, with the objective of reducing seasonal frost problems.

Thermal Barriers

Two principal types of thermal barriers are used in highway construction. The first type consists of materials with large heat storage capacities. Normally these are materials with high moisture content, resulting in large volumetric latent heats of fusion of the moisture. They have been used primarily in the Scandinavian countries; various types of "heat sink" materials have been used, but tree bark has been the most successful. Materials of this type have not been used to a significant extent in highway practice in seasonal frost areas of the United States or Canada.

The second type of thermal barriers consists of materials with low thermal conductivity, generally termed "thermal insulators." Generally, two basic types of insulators are considered: (1) relatively rigid materials that exhibit brittle fracture, and (2) materials that behave plastically, for which the strength is normally determined at some arbitrary strain. Materials of the former type include lightweight portland cement concrete, lightweight asphaltic concrete, cellular glass, and foamed sulfur. Materials in the latter category are molded and extruded polystyrene boards and foamed-in-place polyurethane. Certain composites of these materials also are available, such as portland cement concrete that makes use of polystyrene beads as aggregate. For this report, however, these composites would be rigid materials.

Of the states and provinces that responded to the questionnaire, 19 have used insulating materials in roadways (Table 20). Nearly all of the states have reported on the performance of their test sections, either in publications of the Highway Research Board or in state research reports. AASHO-ARBA (*162*) also summarized performance of most of these sections. From Table 20 it is apparent that all states and provinces with sufficient data indicated that the insulated roadways have performed adequately, except that three states indicated reservations regarding the problem of surface icing. Most agencies have used the manufacturers' recommendations on the thickness of insulation used and the depth of burial of the insulating materials. Extruded polystyrene boards have been used by all of the 19 states and provinces; 4 have also used other types of insulation.

Eleven agencies have observed differential icing conditions between insulated and uninsulated pavements. Seven of these indicated that differential icing conditions are to some extent an obstacle to more widespread use of insulating layers. A wide range in severity of freezing temperatures is represented in this group, with freezing indexes from less than 500 to more than 4,000 degree-days. Areas with low freezing indexes may sustain near-freezing temperatures for longer periods, thus increasing the possibilities of differential icing between insulated and uninsulated pavements. It is believed that the hazard of differential icing will be reduced with construction of longer segments of insulated roadway. The New York State Department of Transportation is currently engaged in a study to investigate the causes and occurrences of differential icing on pavements.

Starting in 1973 the sole manufacturer of extruded polystyrene began requiring a "release and hold harmless agreement" as a condition for selling their insulating material for use in most highway construction applications. The stated reason for this requirement was the occurrence of differential icing between insulated and uninsulated pavements. To date, most states have chosen not to sign the agreement. As a result, little insulation was installed in 1973 and use of insulation in highway applications may decline in the near future.

As indicated in Table 20, most states still use the manufacturers' recommendations concerning the necessary thickness of the insulation. AASHO-ARBA (*162*) recommended that "thicknesses should equal ½ inch (13 mm) per 500 degree-days plus ½ inch." The recommendation was based on experiences by the various agencies, and thicknesses obtained by this rule of thumb were indicated to be adequate to prevent frost penetration into the underlying soil. Analytical methods are also available to estimate the required thickness of insulation for a specific project. Numerical methods offer the most sophisticated models and are becoming increasingly more readily available (*125*). For the next few years, closed-form procedures appear to be most suited for estimating thickness requirements for insulation; two suitable procedures are described in the following.

The first, an analytical technique developed by Lachenbruch (*163*), can be used to estimate insulation thickness required to prevent frost penetration beneath the insulating layer. In using this technique, the surface freezing index and mean annual temperature are used to determine the sinusoidal surface temperature variation. The technique estimates damping of this sinusoidal temperature variation with depth, and the amount of damping is dependent on the thermal properties of the soil. In using this method the latent heat of fusion of the moisture in the base-course material above the insulating layer is not considered; therefore, the method slightly overestimates the required thickness of insulation. In Lachenbruch's model (Fig. 39), the variables are v_o, the difference between the mean annual soil temperature and 32°F, and A, the surface temperature amplitude. The value of A is determined from (*40*)

$$|F| = \frac{365}{\pi}\left[\sqrt{A^2 - v_o^2} - v_o \cos^{-1}\frac{v_o}{A}\right] \qquad (4)$$

in which F is the surface freezing index in °F-days. This equation can be solved readily by trial and error on a desk calculator or by iteration on a digital computer. To illustrate the use of Lachenbruch's model, the required insulation thicknesses for various air freezing indexes have been calculated for the pavement profile shown in Figure 40. The results of the calculation using Lachenbruch's tech-

TABLE 20

STATES OR PROVINCES REPORTING USE OF INSULATION IN DESIGN OR MAINTENANCE [a]

STATE OR PROVINCE	BASIS FOR USE OF INSULATION	THICKNESS OF INSULATION DETERMINED BY: CRITERIA OR RECOMMENDATIONS OF MANUFACTURER	THICKNESS OF INSULATION DETERMINED BY: OTHER	INSULATING LAYERS STILL CONSIDERED EXPERIMENTAL?	DIFFERENTIAL ICING BETWEEN INSULATED AND UNINSULATED PAVEMENTS: OBSERVED?	DIFFERENTIAL ICING BETWEEN INSULATED AND UNINSULATED PAVEMENTS: SERIOUS OBSTACLE TO BROADER USE?	ARE YOUR INSULATED PAVEMENTS SATISFACTORY?
Alaska	Economics		●	No	Yes	No, except on curves.	Yes
Colorado	Research	●		Yes	Occasionally	No	Yes
Idaho	Research	●		Yes	No		Yes
Illinois	Research	●		Yes	No		Yes
Indiana	Research		●	Yes	Like bridge decks.	Yes	Yes [b]
Iowa	Economics	●		No	5-10 times per year.	Yes	Yes
Maine	Economics	●		No	One section during winter rain.	Must be considered.	Yes
Michigan	Research	●		No	No	Believe icing will not occur if cover > 25 inches.	Yes
Minnesota	Economics		●	No	Frequent enough to cause concern.	Yes	Yes [b]
New Hampshire	Research	●		Yes	Surface frozen all winter.	—	Yes
New York	Economics	●		No	Infrequently	Yes	Yes
North Dakota	Research	●		No	No	—	Yes
Ontario	Economics		●	No	Infrequently	No	Yes
Pennsylvania	Economics		●	Yes	Project not completed.	—	—
Quebec	Economics		●	Yes	No	—	Yes
Saskatchewan	Repair	●		No	One area only.	—	Yes
Vermont	Economics	●		Yes	—	Insufficient experience to make a judgment.	Yes
Wisconsin	Economics	●		No	Seldom	Yes, especially on curves.	Yes [b]
Wyoming	Economics	●		Yes	—	Too soon to tell.	—

[a] No reply received from Arizona, Manitoba, New Brunswick, Newfoundland, New Jersey, Rhode Island, Saskatchewan, South Dakota.
[b] Except icing.

nique are given by Curve A in Figure 40, showing reasonably good agreement with the AASHO-ARBA recommendation below freezing indexes of about 1,000 degree-days, but rapid divergence at high freezing indexes. Accordingly, the AASHO-ARBA linear relationship appears suitable for use only in moderate frost areas.

Assuming that extruded polystyrene boards are used, the Lachenbruch curve of Figure 40 should be adequate to estimate the amount of insulation required to prevent subgrade freezing at any given location, provided the pavement profile is similar to that shown. Thicker bases or bases with higher moisture content will cause larger discrepancies between actual and theoretical requirements for insulation, owing to the neglected latent heat of fusion. A second approach, the modified Berggren equation (Eq. 2), accounts for the heat of fusion of the base course, but cannot be used to determine the thickness of insulation required to totally prevent freezing of the subgrade. If 1 ft (0.3 m) of frost penetration is allowed into the subgrade beneath the same pavement section shown in Figure 40, the thickness of insulation indicated by this approach is given by Curve C. Curves D and E were presented by Refsdal (*164*). The Norwegian design, Curve D, corresponds closely with the Lachenbruch curve for complete protection; i.e., allowing no frost penetration beneath the insulation. The thickness design used in Finland corresponds to the AASHO-ARBA line. Based on Curve C, it is expected that slightly more than 1 ft of frost penetration is allowed beneath the insulating layer in Finland.

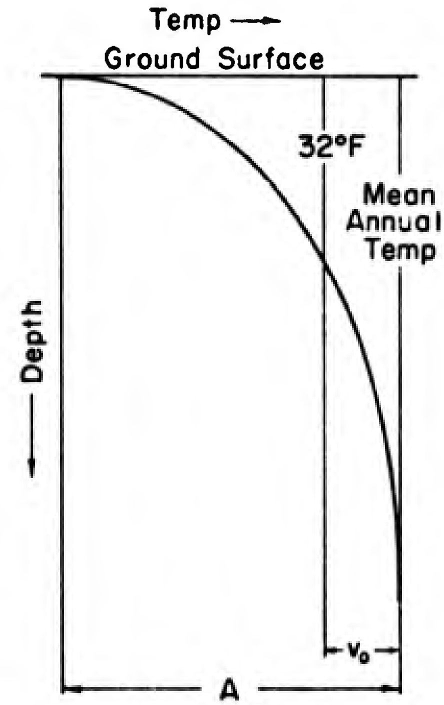


Figure 39. Temperature variables in Lachenbruch's model for determination of damping of sinusoidal surface temperature variations (163).

Moisture Barriers

A dearth of recent U.S. and Canadian literature on the subject of moisture barriers apparently reflects the feeling that such barriers will not solve the problem of preventing migration of significant volumes of subsurface water to the freezing front. Krebs and Walker (*165*) state that the use of a capillary cutoff below the freezing zone but above the water table has a sound theoretical basis, but caution that an impervious layer used for this purpose may cause infiltration water from the surface to become trapped, thereby forming a perched water table above the cutoff material and possibly within the freezing zone. Fear that a perched water table may be formed or that the moisture barrier may not be permanently effective probably contributes to the scant use of such layers.

In summarizing methods of controlling subsurface moisture in pavement systems, Haas (*166*) states:

> 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 cutoff 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. 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

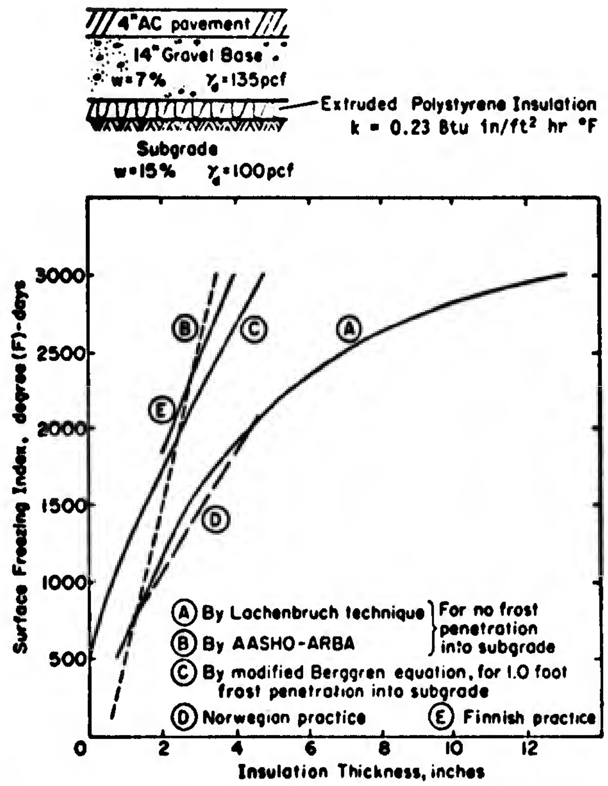


Figure 40. Insulation required under indicated pavement section for various freezing indexes.

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.

Yoder (*149*) suggests that clean sand or gravel can be used below the frost zone as a capillary cutoff material. Armstrong and Csathy (*167*) reported that some Canadian provinces used a granular or a clay capillary cutoff layer where that treatment was more economical than raising the subgrade surface elevation to conform with some established minimum height above the groundwater table. Taivainen (*168*) stated that nonfrost-susceptible sand had been used as a capillary cutoff in Finland, and Rengmark (*169*) stated that sand and asphalt-impregnated felt had been successfully used in Sweden. When granular material is used as a capillary cutoff, the layer must be placed above the bottom of the ditches so that it will not become saturated. Rengmark stated that solifluction from the slopes into the diches has occasionally caused water percolation into the cutoff layer; therefore in Sweden the cut-off layer is now used only on light-duty unsurfaced roads.

Encapsulation

It has long been recognized that fine-grained cohesive soils exhibit the desirable properties of high strength and modulus of deformation when well compacted at water contents below the optimum at which maximum density is obtained. It has not been possible to rely on the maintenance of these desirable properties, however, because various mechanisms of moisture transfer lead to a gradual increase in the moisture contents of the base course and subgrade beneath a pavement. The concept of complete encapsulation of soils to preserve the moisture content at its initial low level is not new, and information is available from both laboratory testing and field demonstration projects.

In 1930, an asphalt membrane was used on a test section of road in Bavaria (*170*). Benson (*171*) discussed encapsulation of soils with sprayed-in-place asphalt membranes. Harris (*172*) reported on 10 to 14 years of experience with envelope membranes in Texas to stabilize medium to highly plastic clay soils. The Department of Highways, Province of Alberta, completely encapsulated a gravel base course within an asphalt membrane (*173*). Results of a comprehensive research study of membrane-encapsulated system were presented by Bell and Yoder (*174*). They discussed, in quantitative terms, the likelihood of moisture movement associated with freezing of encapsulated soil. From coldroom studies of frost action in soils, ACFEL (*175*) concluded that in controlled closed-system freezing of a soil sample 6 in. (150 mm) in height, limiting the initial percent saturation of compacted soil specimens to about 70 percent reduced heave substantially and also reduced moisture gain in the top inch of the sample.

Peyton et al. (*176*) reported a laboratory study on the encapsulation, within either polyethylene or vinyl plastic sheets, of the silt (loess) soil common to interior Alaska. Soils were subjected to closed-system freezing tests, and heave, moisture migration, and post-thaw CBR were measured. The use of membrane-encapsulated soil layers (MESL) in construction of expedient airfields also has been studied at the Waterways Experiment Station (WES). Several test sections were subjected to accelerated traffic testing (*177*). Burns et al. (*178*) reported a MESL containing highly compacted lean clay (CL), was successfully used to support test traffic of a 12-wheel assembly load of 75,000 lb. Traffic tests showed that the MESL performed better than granular materials in layers having the same thickness and subjected to the same loading. Techniques developed in these investigations were applied, in mid-1972, in the construction of an 0.5-mile access road at Fort Hood, Tex., in which MESL serves as base course under a thin bituminous pavement. WES currently is continuing to explore the validity of previous findings in regard to thickness, design, and cost advantages of pavements that include MESL base course (*179*).

Quinn et al. (*70*) reported the results of closed-system freezing tests at the Cold Regions Research and Engineering Laboratory (CRREL) on a moderately plastic clay, a sandy silt, and a lean clay that were proposed for possible encapsulation projects. The test specimens were compacted at water contents ranging from slightly above to well below the optimum for compaction. The results for the moderately plastic clay were very favorable; in a single cycle of closed-system freeze-thaw, the soil showed little or no propensity to heave, much of the moisture remained unfrozen, moisture migration toward the freezing front was insignificant, thaw-weakening was moderate, and strength remained particularly high if the soil was initially molded to high density.

The results for the silt and lean clay, however, showed substantially less severe frost effects than would be expected in open-system freezing; nevertheless, significant moisture migration and thaw-weakening were observed. CRREL is continuing its work in this area and has constructed field test sections in frost areas with MESL base courses of both the lean clay and the sandy silt to further evaluate this promising technique. Practical problems that may comprise limitations on its application include the possible need to limit its use to encapsulated soil that can be located in a borrow pit at water contents somewhat below optimum, and the special care needed in field construction practices to prevent puncturing of the membrane.

Soil Stabilization

Soil stabilization as applied to roadway design and construction may be defined as a method of processing soil in order to render it suitable for base course, subbase, or improved subgrade under the prevailing traffic and climatic conditions. Due to lack of abrasion resistance, stabilized materials are not generally used as surface or wearing course. The objective of soil stabilization in seasonal frost areas may be twofold: (1) to produce a firmly stabilized or cemented mass that is capable of withstanding the stresses imposed on it by traffic loads in severe climatic conditions including frost action, without excessive deformation, and which will remain sound and resist degradation after repeated cycles of freezing and thawing; or (2) to modify or stabilize a frost-susceptible but otherwise satisfactory soil, enabling it to resist frost heaving and thaw-weakening, but not necessarily producing a cemented or firmly stabilized material of high stiffness or strength.

Alternative Approaches

Effective stabilization of a frost-susceptible soil against frost heaving requires modification of one or more of three intrinsic soil properties, by: (1) eliminating the effects of soil fines, by mechanical removal or by immobilization by means of physico-chemical means, such as cementacious bonding; (2) effectively reducing the quantity of soil moisture available for migration to the freezing plane, as by essentially blocking off all migratory passages; or (3) altering the freezing point of the soil moisture. Where large quantities of silty sand or silty gravel are involved and the hauling distance is not too great, mechanical removal of the fines by washing may be less costly and more permanent than chemical treatment.

Cementing agents and chemical and other commercial additives provide approaches by which frost-susceptible soils may be modified to eliminate or reduce heaving and thaw-weakening. Cementing agents include portland cement, bitumen, lime, and lime-flyash. Most of the successful soil-stabilizing chemicals belong in one of three categories—synthetic resins, trace waterproofing chemicals, and organic stabilizers (*180, 181, 182*). Some of the approaches are discussed in the following (*180*).

Plug Soil Voids.—If water cannot migrate, ice lenses cannot grow. The cementing agents mentioned in the foregoing comprise one type of void fillers, and are among the more effective additives. Many commercial void fillers (such as sodium metasilicate) are available but are expensive. The Corps of Engineers (*2, 13*) experimented successfully with Bunker C oil, Tar RT-2, and combinations of Bunker C oil and tar. A serious disadvantage that limits use of these methods for soil stabilization is that the amounts of the additives needed can approach the percentages used in pavement surface courses, making a particular method economically impractical. Also, many of the bituminous additives require special mixing and curing for best results, making field treatment slow and expensive.

Cement Soil Particles.—The cementing of soil particles not only effectively removes fine particles as individual entities but also acts to plug capillary passages. Portland cement, bitumen, lime, and lime-fly ash are quite effective for this purpose.

Alter Characteristics of Void Fluid.—Salt may be added to lower the freezing point of soil water. Lowering the freezing point may preclude freezing or reduce the number of freeze-thaw cycles under a given set of temperature conditions, but does not affect the soil's propensity for heave if and when freezing occurs. The main disadvantage to the use of soluble salts for void-water treatment is that they are removed by leaching. It has been suggested (*70*) that this problem might be avoided by complete encapsulation of the salt-treated soil within a plastic membrane.

Alter Soil Properties by Aggregation of Fines.—A frost-susceptible soil, such as a silty gravel base course, can be made nonfrost-susceptible by eliminating the troublesome fines. The most effective method is to remove the fines by washing, but the effect of fines also may be reduced by additives that cause small particles to "aggregate" to form larger units. Aggregating chemicals are expensive and limited laboratory results show the effectiveness to be unpredictable.

Alter Properties by Dispersion of Fines.—Treatments that can increase the interparticle repulsion in the soil fines tend to disperse the soil aggregates. Particles that do not stick together can be manipulated into a more orderly and denser structure, whose concomitant and advantageous effect is a lower permeability. The effectiveness of trace quantities of chemical polyphosphate dispersants in achieving this purpose by altering soil properties has been discussed in reports by Lambe et al. (*180*) and Croney and Jacobs (*183*). Available data (ACFEL unpublished), however, indicate gradual diminution of effectiveness. Recent advances in encapsulation techniques would make dispersants more attractive. Calcium lignosulfonate, a lignin derivative that is a major by-product of the papermaking industry, also was found to be effective in reducing frost heave in laboratory tests. Lignin is extracted from the wood by treatment with sulfide chemicals, and the acid thus obtained is treated with lime to produce calcium lignosulfonate, sometimes called "lignin sulfonate" or "lignin." Lignosulfonates are water-soluble polymers with high and variable molecular weight; in roads they act as a binder, cementing particles together and reducing dusting (*184*). During rain, lignosulfonates disperse the clays in the soil, causing them to swell and thus plugging the soil voids to reduce water penetration. The lowered permeability caused by clay dispersion is believed to be a factor in the reduction of frost susceptibility of lignin-treated roads. This treatment usually has to be repeated every few years because the lignin is gradually leached out.

Alter Characteristics of the Surfaces of Soil Particles.—With proper additives, mineral surfaces can be made hydrophobic. A soil waterproofed in this manner cannot be "wetted" and should have little or no absorbed moisture in the soil voids. Conversely, coating soils with additives that have highly polar groups exposed to the soil moisture can increase the amount of moisture absorbed and thus reduce the permeability of fine-grained soils enough to make them nonfrost-susceptible. The need to dry the soil and the high cost of waterproofers preclude use of this method even though waterproofers can be extremely effective in reducing frost effects.

The addition of certain chemicals to a soil has been found to have a marked effect in reducing water absorption (*183*). This may be due to modification of the suction characteristics of the soil or from a reduction in its permeability, although the latter seems the more likely mechanism. Croney and Jacobs' investigations of various chemicals indicated that Vinsol resin was effective but costly.

Mechanical Stabilization.—The importance of gradation control, adequate drainage, and compaction in contributing to soil stabilization cannot be overemphasized. Many base materials and subgrade soils that appear to be unsatisfactory may satisfactorily fulfill their intended function if the percentage of fines can be reduced, drainage facilities are provided, and the materials are well compacted. Should it be determined that chemical or physico-chemical soil stabilization is necessary, adequate drainage and proper compaction are essential to the proper construction and functioning of stabilized soil mixtures. Several possible courses of action that do not involve additives may be taken to improve the stability of the pavement substructure and enhance its resistance to frost action, as follows:

1. Drainage may be improved and soil may be compacted to a greater density.
2. Undesirable soil may be removed and replaced with better-quality material.
3. The grade line may be raised by placing better-quality materials over the weaker soil.
4. The water table may be lowered.
5. Frost-susceptible base course soils may be improved by washing out fines.
6. Frost-susceptible base-course soils may be blended with coarser-grained soil to reduce the percentage of fines.
7. A frost-susceptible soil may be completely encapsulated within a plastic membrane to curtail the availability of moisture and thus reduce frost effects.

Durability of Cemented Soil Mixtures

Many of the soil treatments outlined in the foregoing improve the properties of the soil and enhance its resistance to frost effects, but do not produce a firmly cemented mass. In those cases, the effectiveness of the stabilization treatment in reducing frost effects can be determined by the same methods used to determine frost susceptibility of untreated soils, with the emphasis principally on measurement of frost heave in laboratory freezing tests. The main factor on which the permanence of the treatment depends is the ability of the treated soil to resist leaching of the additive under the action of flowing or percolating water. With cemented or partially cemented soils, the mode of distress under repeated cycles of freezing and thawing is disintegration. In its early stages it is characterized by a gradual degradation in strength and soundness, sometimes accompanied by volume increase; later this leads to complete disintegration and reversion to an unbound and friable soil.

Durability of cemented soil mixtures is a topic far too extensive to treat adequately in the present synthesis, and no attempt is made to do so; only the most essential information is given and several references are cited as sources for detailed treatment.

Lime Stabilization.—Lime-pozzolanic treatment of soil to improve its strength is one of the oldest techniques used for road construction, dating back to highways built by the Romans. In the United States lime stabilization has been used extensively in some states, and both lime and lime-pozzolanic treatments are gaining use elsewhere in the country.

Considerable experience is now available in the United States on use and effectiveness of lime and lime-fly ash stabilization of subgrade soils. Only fine-grained soils can be effectively stabilized with lime. Thompson (*185*) has distinguished between nonreactive soils, in which the addition of lime improves plasticity, workability and expansive properties but does not greatly increase the strength, and reactive soils, in which the same beneficial changes occur and also substantial strength gain accrues from the cementing action associated with the pozzolanic reaction. Lime has been found most effective with clay soils containing montmorillonite, illite, and kaolinite. Case histories have been reported (Wisconsin, Nebraska, Iowa, Oklahoma) in the literature where performance of lime-treated frost-susceptible subgrades has been satisfactory under freezing conditions. On the other hand, tests at CRREL have shown that lime treatment of certain clay soils can convert a material that shows negligible to moderate frost heave into one that is highly susceptible to frost heaving, acquiring characteristics more typically associated with silts. It is believed that at least part of this adverse effect was caused by insufficient curing prior to freezing; Thompson (*106*) has emphasized the critical importance of adequate curing. Lime-fly ash stabilization is applicable to a broader range in soil types because its cementing action is less dependent on fines contained within the soil.

Thompson (*186*) in a comprehensive state-of-the-art report on soil stabilization, concluded that the major durability consideration for lime-soil mixtures is resistance to cyclic freezing and thawing. Freeze-thaw damage is characterized by volume increase and strength loss, which are interrelated. Thompson found that initial unconfined compressive strength, prior to freeze-thaw cycling, is a valid measure of freeze-thaw resistance, and proposed an approach to mixture design that includes compressive strength requirements (Table 21).

Portland Cement Stabilization.—Portland cement is widely used for stabilizing medium-textured sandy and granular materials to improve the engineering properties of strength and stiffness. Use of soil-cement stabilization has increased greatly since it was first introduced in the United States about 1935. Most coarse-grained and many fine-grained soils (both silts and clays) have been treated successfully, but soils containing organic matter or deleterious chemicals have been found unsuitable for soil-cement treatment (*63*). Increasing the cement content increases the quality of the mixture. At low cement contents the product is generally termed "cement-modified soil," which is a soil with improved properties such as plasticity, expansive characteristics, and frost susceptibility. At higher cement contents the end-product is soil-cement, defined by the Portland Cement Association as a "hardened soil-cement structural material." The design of soil-cement mixtures for use in seasonal frost areas is usually based on the requirements of resistance to disintegration or excessive degradation caused by cyclic freezing and thawing. AASHO and ASTM have standardized freeze-thaw tests applicable to soil-cement mixtures; both are closed-system tests. One of the requirements is that the maximum volume of the specimens at any time during 12 cycles of the freeze-thaw test shall not exceed the volume at the time of molding by more than 2 percent. Evaluation of the deterioration of the cemented mixture caused by freeze-thaw cycles is made by observing surface softening and loss of strength. The Portland Cement Association (*188*) concluded from its research on durability that generally a soil-cement mixture having a compressive strength of 300 psi (2,070 kPa) or more at 7 days, and that is increasing, will pass the freeze-thaw test.

Although procedures for assessing freeze-thaw resistance of soil-cement are established and widely accepted, Thompson (*186*) pointed out that formal procedures for determining the necessary cement contents for cement-modified soil in frost areas have not been established. He advocated

TABLE 21

TENTATIVE LIME-SOIL MIXTURE COMPRESSIVE STRENGTH REQUIREMENTS[a]

ANTICIPATED USE	RESIDUAL STRENGTH REQUIREMENT[b] (PSI)	STRENGTH REQUIREMENTS (PSI) FOR VARIOUS ANTICIPATED SERVICE CONDITIONS[c]				
		EXTENDED (8-DAY) SOAKING	CYCLIC FREEZE-THAW			
			3 CYCLES[d,e]	7 CYCLES[d,e]	7 CYCLES[d,f]	10 CYCLES[d,e]
Modified subgrade	20	50	50	90	50	120
Subbase:						
Rigid pavement	20	50	50	90	50	120
Flexible pavement:						
Cover thickness[g] 10 in.	30	60	60	100	60	130
8 in.	40	70	70	110	75	140
5 in.	60	90	90	130	100	160
Base	100[h]	130	130	170	150	200

[a] Source: Ref. (*187*).
[b] Minimum anticipated strength following first-winter exposure.
[c] Strength required at termination of field curing (following construction) to provide adequate residual strength.
[d] Number of freeze-thaw cycles expected in the lime-soil layer during the first winter of service.
[e] Freeze-thaw strength loss based on 10 psi per cycle.
[f] Freeze-thaw strength loss based on previously established regression equation.
[g] Total pavement thickness overlying the subbase. The requirements are based on the Boussinesq stress distribution. Rigid pavement requirements apply if cemented materials are used as base courses.
[h] Flexural strength should be considered in thickness design.

determination of the minimum tolerable strength and laboratory verification that the residual strength of the stabilized soil at the end of the first winter will not be less than the minimum required (Fig. 41).

Bituminous Stabilization.—Bituminous-stabilized materials are generally used for base and subbase construction. In many parts of the country well-graded coarse aggregate is scarce. However, these same regions may have an abundance of sandy and silty soils or other materials that can be readily stabilized with bituminous materials. The bituminous materials most commonly used in stabilization practice are asphalt cements, cutbacks, emulsions, and road oils, and various formulations of tars. The multiplicity of naturally occurring materials and bitumen types indicates a wide versatility possible with bituminous stabilization. Use of bitumen as a stabilizing agent produces different effects, depending on the soil, and may be divided into three principal groups, as follows:

1. Sand-bitumen. Produces strength in cohesionless soils, such as clean sands, by acting as binder or cementing agent.
2. Soil-bitumen. Stabilizes the moisture content of cohesive fine-grained soils.
3. Sand-gravel-bitumen. Provides cohesive strength and waterproofs pit-run gravelly soils with inherent frictional strength.

The mechanism of bituminous stabilization is primarily mechanical, and improved stability is due to cementing and/or waterproofing characteristics. Important properties of a bitumen-stabilized mixture include strength, modulus of deformation, and moisture absorption. Both strength and modulus of deformation are highly dependent on temperature and time of loading, and procedures for testing and determination of the amount of bituminous material required must account for both these variables. Thompson (*186*) has summarized the various laboratory procedures, empirical equations, and suggested criteria for selection of bitumen content. The durability of bitumen-stabilized mixtures generally can be assessed by measurement of their water-absorption characteristics. Herrin (*189*) suggests that water absorption should be less than 7 percent after 7 days immersion in water. A primary determinant of water absorption is percent air voids in the mixture, and both the air voids and the water absorption generally decrease with increasing bitumen content. With some soils, however, increasing the bitumen content also decreases the strength, and a thorough series of laboratory tests is essential in such cases.

Full-Depth Bituminous Pavements

Full-depth asphalt pavements as defined by The Asphalt Institute (*88*) are "pavements in which asphalt mixtures are employed for all courses above the subgrade or improved subgrade." The design procedure promulgated by The Asphalt Institute for full-depth pavements refers to pavement structures built entirely of dense-graded asphaltic concrete. Information obtained in this survey indicates, however, that a number of states and agencies construct asphaltic concrete surface courses over other types of asphaltic mixtures, herein termed collectively asphalt-treated base (ATB), and consider the pavements to be full-depth asphaltic pavements.

Application of the concept of full-depth asphaltic pavements in frost areas represents a substantial departure from two traditional pavement design practices that have been followed by some agencies. By definition, full-depth asphaltic pavements contain no layers of pervious granular materials, which in the past were widely held to be essential, especially in frost areas, for drainage of the subgrade and base course. Full-depth pavements, especially those built entirely of dense-graded asphaltic concrete, also represent a new concept for designers of flexible pavements who have held that the need for strength, stiffness, or stability

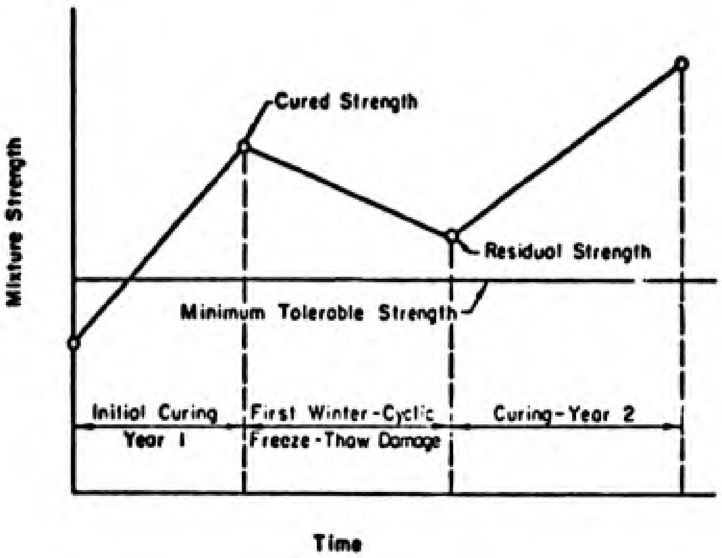


Figure 41. The residual strength concept (186).

in a particular layer of the pavement structure decreases as the depth of the layer below the surface increases. These two departures from traditional views of flexible pavement structures have left full-depth asphaltic pavements for frost areas still a controversial concept. The eleven agencies from which more detailed information was obtained include a broad range in opinions and practices regarding full-depth asphaltic pavements—from rejection of the concept to full adoption of full-depth structures for all flexible pavements (Table 22). Of the 40 agencies that reported frost action is considered in their pavement design, 20 have constructed full-depth asphaltic pavements (Table 23). Total thickness ranges from 40 to 75 percent of the thickness of pavements of conventional design that include unbound granular layers. Full-depth pavements have been constructed by some states and provinces on all classes of roadway; others have confined their test or demonstration sections to roads of heavy-duty or intermediate classifications; still others have used them only on intermediate or secondary roads.

TABLE 22

USE OF FULL-DEPTH ASPHALTIC PAVEMENTS BY ELEVEN AGENCIES

STATE OR PROVINCE	HAVE YOU USED FULL-DEPTH ASPHALTIC CONCRETE PAVEMENTS?
Alberta	Yes, have used 5 years. Have about 300-400 miles on primary roads and 300-400 miles on secondaries.
Colorado	Yes, about 50 miles, on primary and secondary roads. Choice based strictly on cost, except not used on Interstates, where always require unbound subbase.
Idaho	No, but expect to use over good subgrades.
Maryland	Use thick AC pavements extensively, but always use 4 in. crushed stone unless subgrade is gravel.
Maine	No, but now using greater thickness of asphaltic material.
Nebraska	Yes. Present design for intermediate and secondary roads is full-depth.
New Hampshire	No. Concerned about possible greater tendency for low-temperature contraction cracks, and reluctant to place AC directly on FS subgrade.
New York	No. This concept is objectionable because it violates the principle that strength needs decrease with depth.
Saskatchewan	Yes. About 200 miles, from 7½ to 9½ in.
Wisconsin	Yes, two projects of 6 miles each. One is 9¼ in., the other 12 in.
USA Corps of Eng.	No, except experimental pavements.

In most cases the thicknesses of full-depth asphaltic pavements are being determined by the AASHO interim guides or some other procedure that makes use of layer coefficients or equivalence factors to relate the thickness requirements for layers of different types of materials. Nebraska explained that they use the AASHO Interim Guide method to design a conventional pavement structure, and then add to the asphaltic pavement required in the conventional structure one-half the thickness of the granular base in additional asphaltic concrete. Some states indicated that economics was the final factor in their decision to construct full-depth pavements; as more full-depth sections are installed, and if their performance proves satisfactory, economics will undoubtedly be the final consideration in using larger segments of full-depth asphaltic pavement. Economic comparisons with pavements of other types will be strongly affected by the availability of asphalt.

Nine of the 20 agencies that have used full-depth pavements indicated that they had performed adequately; however, several added the phrase "to date," indicating that the evaluation of their performance is continuing. Five states indicated that they had inadequate data at this time to properly evaluate their full-depth pavements.

Alberta indicated that frost heaving has not been noticeably reduced by the use of full-depth pavement, but owing to their greater stiffness (compared to pavements with gravel base), full-depth pavements may more successfully bridge a subgrade that has been weakened during the thaw. Maryland uses thick asphaltic pavements, but always uses 4 in. (100 mm) of crushed stone on the subgrade.

TABLE 23

USE OF FULL-DEPTH ASPHALTIC PAVEMENTS BY THE 40 AGENCIES THAT REPORTED FROST IS CONSIDERED IN THEIR PAVEMENT DESIGN

STATE, PROVINCE, OR OTHER AGENCY	HAVE YOU USED FULL-DEPTH PAVEMENTS?	THICKNESS (% OF THICKNESS OF CONVENTIONAL PAVEMENTS)	CLASS OF ROADS ON WHICH FULL-DEPTH PAVEMENTS USED	ARE FULL-DEPTH PAVEMENTS CONSIDERED SUCCESSFUL?
Alberta	Yes	—	—	Yes
British Columbia	Yes	—	—	—
Colorado	Yes	60	Primary, secondary	Yes
Illinois	Yes	—	Secondary	—
Indiana	Yes	50–70	All	Insuff. experience
Iowa	Yes	—	Primary, secondary	Yes
Kansas	Yes	—	All	—
Michigan	Yes	50	Widening and intersection projects	Insuff. experience
Minnesota	Yes	55–65	All	Insuff. experience
Montana	Yes	—	—	—
Nebraska	Yes	75	Intermediate and secondary	Insuff. experience
Ohio	Yes	50	Intermediate and secondary	Yes
Ontario	Yes	50	Heavy duty	—
Pennsylvania	Yes	—	Secondary	Insuff. experience
Saskatchewan	Yes	>40	Arterial and collector	Yes
Washington	Yes	—	—	—
Wisconsin	Yes	60	Heavy and intermediate	Yes
Wyoming	Yes	40–60	All	Yes
Asphalt Institute	Recomm.	$\leqq 50$	All	Yes
U.S. Forest Service	Yes	$\leqq 50$	<400 ADT	Yes
20 other agencies	No	—	—	—

CHAPTER FOUR

CONSTRUCTION AND MAINTENANCE

CONSTRUCTION

Need for Uniformity and Gradual Transitions

Differential frost heaving during the winter may produce abrupt changes in pavement elevation at culvert and pipeline crossings, bridge abutments, catch basins, and wherever trenches have been cut. Differential heaves of several inches are not uncommon; they not only are damaging to vehicles but also may cause loss of vehicle control. Differential heaves cause pavement cracking and opening of pavement joints; the attendant ingress of water worsens the heave problem, leading to pavement disintegration and high maintenance costs.

Differential heaving frequently occurs in cut-to-fill transition zones, as a consequence of an abrupt change in subgrade soil type and of varying groundwater conditions. It may occur within a continuous cut or fill because the subgrade soil varies along the alignment as lenticular inclusions or differing strata are exposed in grading operations, or owing to the presence of shallow bedrock that may have frost-susceptible soil filling joints and seams and may have an irregular or undulating surface that impedes drainage and serves as irregular reservoirs for water that can feed growing ice lenses. The presence of large stones and boulders in the base course or subgrade also comprises a serious threat of irregular surface heaves, owing to their tendency to migrate upward toward the surface.

The locations of man-made discontinuities, such as culverts and trenches, can be anticipated during design stages. Other conditions, such as variable soils or shallow bedrock, are more difficult to foresee unless extremely detailed subsurface explorations are conducted for design, but often can be recognized during construction. The most common reason for abrupt elevation change at culvert locations is the practice of backfilling trenches with a more select material, usually coarser-grained and more pervious, than the material in the roadway adjacent to the trench. Abrupt changes in elevation can be avoided by providing gradual transitions of suitable material in these troublesome areas. Similarly, variable subgrade conditions can be treated by mixing, blending, and selective grading to guard against pronounced nonuniformity of frost heave and subgrade support.

Grading Operations

A roadway must be constructed within prescribed limits of grade and alignment. Grading or earthwork operations, consisting of excavating in high areas and filling in low areas, constitute a major phase of road construction, and require the greatest effort in terms of manpower and equipment.

Selective Grading

Many potential frost problems can be reduced considerably if, based on experience and terrain investigations, the conditions and specific locations along the route where such problems are likely to occur are recognized and treated. One approach to treatment of such problem areas is by means of various special grading practices that, collectively, have been termed "selective grading." Materials obtained from required roadway or borrow excavations are selected and segregated for specific uses in embankment construction and subgrade preparation. The more highly frost-susceptible soils are placed in the lower portions of embankments and less-susceptible soils are crosshauled to form the upper portion of the subgrade. Undesirable soils also are used in the outermost parts of embankments rather than beneath the roadbed. Crosshauling and mixing also are used at cut-fill transitions to correct abrupt changes in soil type (*155*).

Cut sections frequently have been reported as sources of trouble, and in saturated and poorly drained areas it is good practice to set the grade line high enough to minimize the number of cuts. Cuts alter the natural drainage, with adverse consequences including bearing-capacity failures and more severe frost heaves because of ample water supply from adjoining higher ground. In some cases where elevation of the grade line is not feasible, the problem can be eased by subsurface drainage, or it may be necessary to undercut and remove the wet and unstable soil and replace it with selected material.

Blending, Mixing, and Boulder Removal

In seasonal frost areas reasonably smooth pavements may be achieved over frost-heaving soils by providing base courses and subgrades of uniform texture and consistency throughout the roadway. This approach has long been recognized by the states, provinces, and federal agencies, a number of which place great emphasis on the need for achieving subgrade uniformity.

One of the first requirements during construction and prior to preparation of subgrade should be inspection of the subgrade to verify the validity of subgrade design assumptions, including an assessment of the texture of the soil and its uniformity in vertical and horizontal directions. In cuts, inspection should be made for frost-susceptible materials, seepage, ground or capillary water, logs, stumps, boulders, rock outcrops, and other conditions that may result in nonuniform subgrade conditions. Treatment will include removal of logs, stumps, and boulders; interception and drainage of seepage water; undercutting and replacement of unsuitable materials; and scarifying, mixing, and blending of the subgrade soils to achieve maximum uniformity.

The maximum size of stone permitted in road construction varies in different states and provinces, and also with position in the road profile and whether crushed gravel or crushed ledge is used. New Hampshire excludes 3-in. stones from the top 6 in. of sand and gravel base. Boulders larger than ½ cu yd are removed from within 4 ft of the grade line. New Brunswick excludes boulders larger than 8 in. from the top 24 in. of subgrade; Manitoba allows no stones over 4 in. in diameter within the top 12 in.; and Newfoundland requires all borrow be less than 6 in. in size. Ontario requires that all boulders be removed from the top 24 in. of the subgrade, or to a depth of 48 in. below the top of granular base, whichever is greater (Fig. 42).

Removal of boulders from the subgrade can be accomplished by excavating the top layer, scalping the boulders from it, and relaying it; or, during processing of the subgrade, by in-place scarifying, mixing, and blending. Where soils vary widely or frequently in texture, mixing them is effective in preventing differential frost heave. With modern construction equipment, the mixing of nonuniform soils to form a uniform subgrade is often more economical than importing select materials from borrow pits.

Silt Pockets

The soil profile has a profound influence on frost action, as has been widely experienced and reported in the literature. The most important effect of the soil profile is the influence that may be exerted by highly variable soil stratification or irregular pockets of extremely frost-susceptible soil. Hofmann (*81*) stated that in New York State 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, and topography of such deposits, as well as the presence of silt layers and lenses in them, produce great problems in differential heave. Cuts through folded rock formations produce differential heave problems of similar magnitude. In some cases stratification results from the manner of deposition; in other cases as a result of weathering and modification by percolating water.

Pockets, layers, or lenses of silt or other highly frost-susceptible soil not only may themselves contain considerable moisture, but concavities in the top surfaces of the lenses of fine-grained soil in contact with more pervious soil will act as basins to hold water that may supply moisture to potentially frost-susceptible soil above it, or to growing ice lenses within the silt lenses. Such lenses should either be removed or be scarified and thoroughly blended with the surrounding, more pervious, material. Silt lenses or strata within less frost-susceptible clay layers should be treated similarly. If the silt pocket is removed, it is usual practice to replace it either with nonfrost-susceptible soil, providing gradual transitions to adjacent slightly frost-susceptible soil, or with soil similar to the adjacent soil (Fig. 43). In Alberta, where fissured silt is present it is undercut up to 3 ft

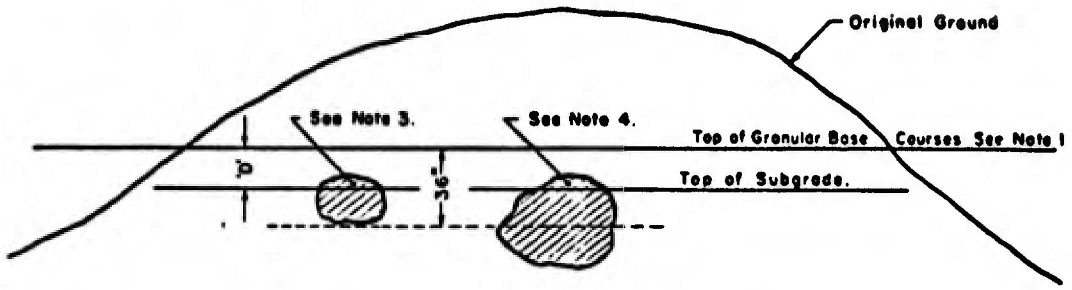


Figure 42. Boulder treatment (Ontario).

and the excavated material is replaced with less frost-susceptible material. Saskatchewan, however, uses undercut material after thorough mixing to achieve uniformity.

Subgrade Preparation in Rock Cuts

The irregular surface in rock cuts forms basins and catchment areas for gravitational water, which can cause adverse frost action in the base course. Also, bedrock often contains frost-susceptible soil fillings in seams, cracks, and joints. Aside from these two sources of problems, rock cuts also comprise a serious discontinuity in subgrade support conditions compared with the adjacent fills. To minimize adverse effects, current practice includes treatment of rock cuts by two alternative procedures, both intended to ease the abrupt change in subgrade conditions: (1) undercutting of the rock subgrade and replacement with material similar to the adjacent fills, providing tapered transitions of increasing depth at each extremity of the cut; and (2) in-place fragmentation of rock subgrades with blast holes 3 to 6 ft deep throughout the subgrade (Fig. 44). As an alternative to the latter method of fragmentation, New York sometimes requires drilling and blasting only along a toe-of-slope trench at the higher side of the cut. In addition to fragmentation by blasting, shale rock in subgrades is sometimes ripped and recompacted. In rock cuts a leveling course of asphaltic concrete or portland cement concrete may also be placed over the rock and then followed by a normal pavement section.

Transitions (Cut-to-Fill and Culverts)

Drains, culverts, or utility ducts placed under pavements on frost-susceptible subgrades are frequently the cause of abrupt differential heaving. Where such facilities are required they should be put in place before the base course, rather than trenching through it later, so the base-course material can be carried across them without interruption to obtain uniformity of pavement support. Otherwise, a marked discontinuity in uniformity and compaction of the base course would result, and some mixing of frost-susceptible subgrade with the base course would be inevitable, owing to difficulties of separating the two materials during excavation of the trench.

Two schools of thought regarding backfill material for culverts across the roadway centerline are evidenced by the design practices described in Chapter Three under "Other Factors, Differential Frost Action at Culverts, Utility Ducts, Drains, and Structures": (1) nonfrost-susceptible backfill, to minimize frost heave at the culvert; and (2) native soil backfill, to make heave at the culvert equal to that in adjacent sections of the roadway. Neither of these is completely successful in preventing a differential depression or heave at the culvert under all climatic conditions, because the prevalence of atmospheric temperatures within the culvert induces a thermal regime in the ground surrounding the culvert that differs from that elsewhere under the roadway. All-around insulation of the culvert may be helpful, but generally, in the interest of avoiding pavement rough-

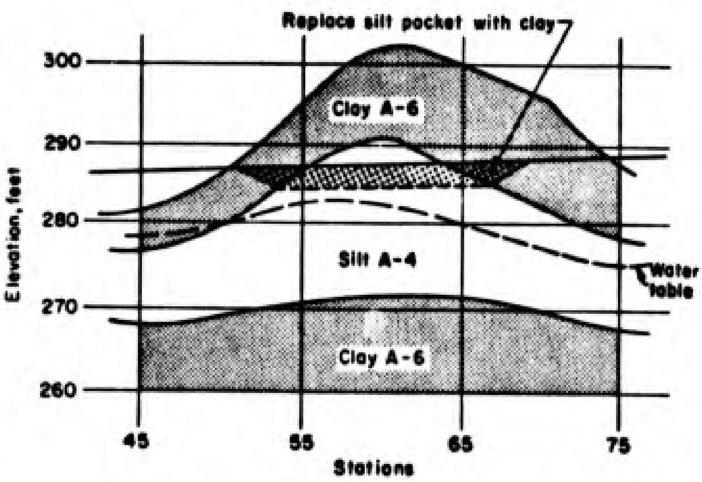


Figure 43. Frost-susceptible silt pocket replaced with soil similar to surrounding material (155).

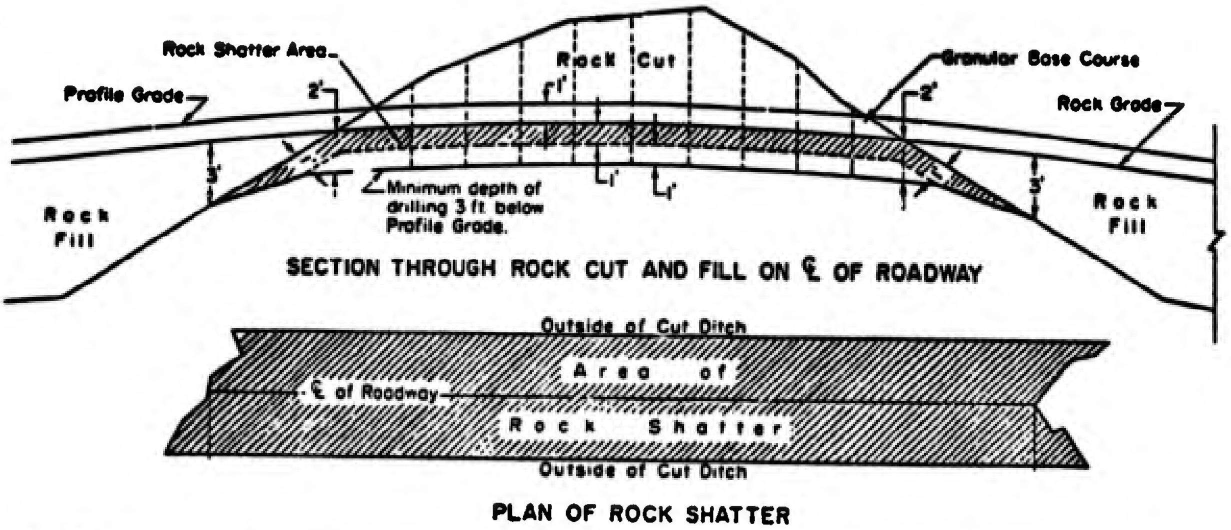


Figure 44. Rock subgrade fragmentation (Ontario).

ness over the culvert, it is good practice to provide gradual transitions in the depth of backfill material, sloping gently upward from the full depth of the trench to intersect the normal top-of-subgrade some distance away. Some of the current practices in design of transitions are described in the section mentioned.

It is also good practice to provide gradual transitions at other discontinuities of subgrade support, such as cut-fill intersections (which may occur in a transverse or longitudinal section), junctions of sharply differing types of subgrade soils, and lines of intersection of rock and earth subgrade. In the first two cases, mixing of the two types of soil is a feasible and effective practice, but an alternative practice, which also is effective for the rock/earth intersection, is to construct gradual transitions between the two intersecting materials. These transitions may be made of (a) embankment material (Fig. 45); (b) granular, nonfrost-susceptible material, sometimes called granular wedges (Fig. 35); or (c) combinations of earth fill, rock fill, and granular fill (Fig. 46).

Hauling, Placing, and Spreading of Materials

Placing and spreading of material on a prepared subgrade may begin at the point nearest the source or at the point farthest from the source. The advantage of working from a location nearest the source is that the hauling vehicles can be routed over the spread material to assist in compacting the base and avoid cutting up the subgrade. Advantages of working from the point farthest from the source are that hauling equipment will further compact the subgrade, reveal any weak spots so that they can be corrected promptly, and interfere less with the movement of spreading and compacting equipment. However, when hauling vehicles are not desired on the subgrade and placing begins at the point farthest from the source, hauling vehicles may be routed over adjacent finished working strips and the material spread transversely at the point of deposit. There is danger in hauling over finished base course that silt, clay, and soil fines will be carried by the truck tires and dropped over a clean base course, thus contaminating it with fines and making it frost-susceptible. Excessive compactive effort also can

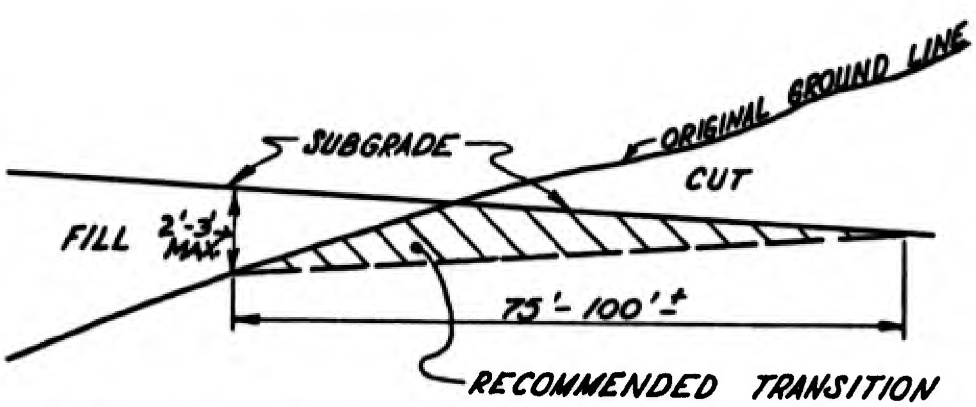


Figure 45. Detail of tapered transition used where embankment material differs from natural subgrade in cut (Maine).

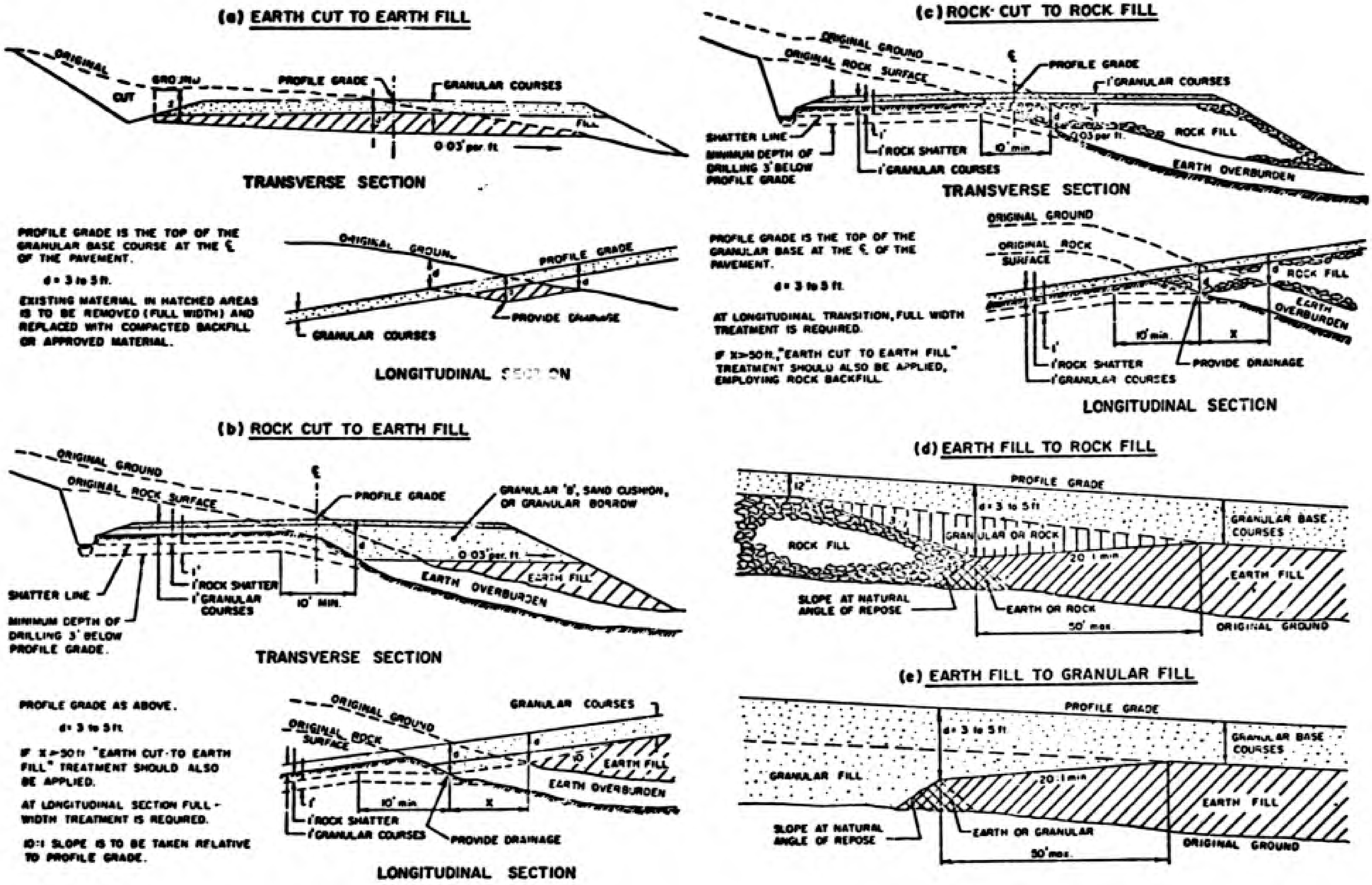


Figure 46. Transition details (Ontario).

degrade the particles in clean base courses (*190*). Base courses laden with fines lose their stability more rapidly with increase in water content and are susceptible to frost heaving and pumping. Hauling operations that tend to deposit fines on a clean base course should be rerouted to eliminate the problem. It also may be necessary to scrape off the top inch or so of compacted material and replace it with clean material.

Moisture-Density Control

Achievement of the highest possible density of subgrade and base course is essential to good road stability. High density of the base course provides resistance to heaving, increases resistance to moisture infiltration, and provides higher strength and stiffness modulus. Determination of density requirements, preparation of corresponding specifications, and construction control should be based on compaction tests. For large construction projects it may be highly desirable to investigate the compaction characteristics of the soil by means of a field test section, using prototype equipment. The range of moisture contents for effective compaction in silty soils and silts may be very narrow, and close moisture control is necessary for optimum results. Equipment and methods used must be adjusted on each job to suit the characteristics of the base material, because thorough compaction is highly important in developing maximum stability.

Rolling should not be attempted when subgrade has been softened by rain. In such cases it may be necessary to scarify and dry out the soil before continuing with compaction operations. Compaction difficulties and pavement problems can be avoided by limiting stone size in the base course and subgrade. Some specifications require that maximum stone size be no larger than two-thirds the thickness of the layer being compacted.

Inspection and Control of Work

Subgrade and Base Course

Field control of highway pavement construction in areas of seasonal freezing must specifically consider conditions and materials that will result in detrimental frost action. 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. Construction operations quite frequently expose potential frost-troublesome conditions not previously revealed by even the most thorough subsurface exploration program conducted during the design phase. Personnel assigned to field construction control should be made aware of their responsibility to recognize situations that require special treatment and field correction.

Visual inspection, although important, is not sufficient for control of the construction of subgrades, subbase and base courses, particularly those that contain considerable fine material. Depending on the type of material, necessary control tests include determinations of gradation, mixture proportions if blended, plasticity characteristics, moisture content, field density, layer thickness, and strength or stability values. Construction operations must be adjusted if requirements are not being met.

Subgrade Preparation.—Selective grading, cross-hauling, mixing, undercutting, and other practices that lead to better uniformity of the subgrade cannot be achieved merely by including requirements in the specifications, but only through the active collaboration of construction technical and management personnel and a corps of trained inspectors. Otherwise, with the exception of very undesirable materials such as humus or peat, soils usually will be taken as found and used to whatever advantage can be developed. Although it is desirable to separate soils from different horizons of natural formations and place those with the best properties in the most critical portions of embankments or at subgrade elevation, considerable mixing is unavoidable, and constant inspection and supervisory efforts are essential to achieve the most effective utilization of the available materials. The construction inspection personnel should check the validity of the design assumptions, and if pockets of unexpected frost-susceptible material or wet subgrade conditions are revealed, remedial measures should be initiated. Gradation tests should be performed on any questionable materials encountered during grading operations, and pockets of frost-susceptible soils in an otherwise nonfrost-susceptible subgrade should be removed and replaced with materials of the same type as the surrounding soils, or, if this is impractical, transition zones between areas of lower and higher frost susceptibility should be provided. Alternatively, the subgrade should be thoroughly scarified and processed to blend pockets of frost-susceptible soils with the surrounding soils. Clean granular soils should be employed in situations where frost action will affect the construction.

At the transition between cut and fill sections the topsoil and humus should be completely removed to the depth of frost penetration, even though specifications may not require general stripping in most fill areas. Special attention should be given to wet areas in the subgrade, and special drainage measures should be installed as required, particularly in providing 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. Excavations in rock should be made so that positive transverse drainage is provided and no pockets are left on the rock surface that will permit ponding water within the maximum depth of freezing. The irregular groundwater availability created by such conditions may result in markedly irregular heaving under freezing conditions. Rock subgrade fragmentation should be considered, or it may be necessary to fill drainage pockets with lean concrete or asphalt mix. 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 and replaced with nonfrost-susceptible material. If this is impractical, it may be necessary to remove the upper part of the rock subgrade and replace it with earth fill.

Base-Course Construction.—Where the available base-course materials are nonfrost-susceptible, the base-course construction control should follow normal practice. In instances where the base-course materials selected for use have a significant percentage of fines, frequent gradation checks should be made to ensure that the materials meet specification requirements, including gradation limits such as the 0.02-mm size, for control of frost susceptibility. Where borrow pits are variable, selection in the pit may be necessary in order to obtain suitable materials; on jobs involving large volumes of base course, inspection should be made at the pit, because usually it is more feasible to reject unsuitable material at the source. Complete surface stripping of pits should be enforced to prevent mixing of topsoil or other detrimental fine soil particles or lumps in clean base material. The gradation of the base-course materials after compaction should be determined at the start of the job, and checked frequently to see if excessive fines are being manufactured in the base under the abrasion and pounding of compaction equipment.

When pit material is variable, thorough mixing can be done at the pit by stockpiling and mixing in windrows, and by spreading the material in thin lifts on the subgrade to insure uniformity. To avoid mixing base-course materials with frost-susceptible subgrades during and after construction, the subgrade should be properly graded and compacted prior to placement of base course, and protected if necessary by a filter layer over the subgrade to prevent movement of subgrade fines into the subbase or base under traffic. 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. Soft or weak areas may be recognized both by visual examination of the materials and by observation of their action under compaction equipment, particularly when the materials are wet. The first layer of base or subbase may have to be thicker than normal to support hauling equipment and avoid mixing of soft subgrade materials into the granular layers. Fine-grained silty material should not be used as a construction expedient to choke open-graded base courses, because this action could make the base susceptible to both frost heave and thaw-weakening.

Drainage Details and Transitions

Settlement and depressions at drainage structures can be avoided by careful placing and compacting of the backfill in these areas. Extra care must be exercised in seeing that a pipe or culvert is uniformly bedded on a compacted select material free from stones and cobbles. The same care must be exercised in hand placing and hand compacting around the pipe until it is covered. Pushing in fill with a bulldozer is unsatisfactory and inevitably results in subsequent consolidation. Once the pipe or culvert section is covered with fill and hand compacted, additional layers may be compacted with field equipment with due care at these locations. The same concern and care applies to fill being placed around any structure, such as a bridge abutment, a retaining wall, or a sewer manhole. To minimize transient pavement roughness caused by differential frost heave at drainage structures, it is essential that careful consideration be given to selection of the backfill material and that gradual transitions in its depth be provided, as outlined previously.

Thermal and Moisture Barriers

Many states and provinces have recently experimented with or are designing roads with thermal insulating layers of thermoplastic foam in the form of panels or sprayed-in-place polyurethane, to prevent or reduce frost penetration into the subgrade. Use of thermal insulation has been demonstrated to be effective for this purpose. The long-term performance and effectiveness of such an installation is dependent on the workmanship and good field practice during installation. Subgrade soils should be graded smooth and crowned to the same slopes as the finished grade. If necessary a 1- or 2-in. under-bedding of clean sand should be applied to provide a uniform and smooth foundation. Insulation boards should be placed snugly together with staggered joints. Generally it is good practice to prohibit traffic of equipment over the insulation prior to placing at least 6 to 8 in. (150 to 200 mm) of base course by end-dumping and spreading with a small dozer. Heavy equipment and trafficking should be avoided until the second base-course layer is placed. With certain high-strength formulations of foamed plastic, traffic can be permitted directly on the insulation. To avoid abrupt changes in heave at termination of insulated sections, it is necessary to provide a transition zone between the insulated and conventional sections. If several layers of insulation are used, this can be accomplished to a reasonable degree by gradually tapering off the insulating layers.

Moisture barriers (either capillary cutoffs or complete encapsulation) require the strictest inspection and control. Principal problems are punctures, tears, imperfect sealing of joints, and incomplete sealing of pores of some materials that require impregnation with bitumen. Soil in contact with the membranes usually must be limited to sand or fine-grained soil free from gravel or sharp stones. Depending on the type of membrane, traffic directly on it may have to be prohibited.

Construction During Cold Weather

Embankment Construction

Compared with other categories of work within the construction industry, horizontal earthwork is extremely weather sensitive. Although freezing temperatures affect both excavation and filling, usually no restrictions are imposed on excavation of frozen ground except those of practical and economic import. The major problem lies in the construction of fills and compaction of soil at freezing temperatures. It has been learned—and sadly relearned many times—that frozen soil is not good fill material. Failures and instability have been the principal result. In the most usual practice, specifications prohibit the placing of frozen soil in fills, and many design and construction agencies also restrict the placing of unfrozen material on frozen ground. The reason for these restrictions is obvious: frozen soil in the foundation or the fill cannot be compacted properly, and furthermore if it contains ice layers or lumps of snow or ice it will have reduced stability and will settle upon thawing.

Construction of fills using quarry-run rock, crushed rock, slag, or well-drained clean gravel can be accomplished during the winter with no loss in quality of the end result. Care should be taken to exclude large chunks of ice or snow from the fills. Underwater backfill for replacement of peat can be accomplished effectively in winter. Winter construction of embankments using preheated soils also is technically feasible, but can be economically justified only when absolutely necessary to complete the work at an early date. Surfaces of fills being constructed in cold weather can be protected overnight or over the weekend by placing calcium chloride on the surface or by covering with a layer of hay or other insulating material. Calcium chloride has been found effective only if the temperature does not drop below 25°F (−4°C).

In the United States most states require that frozen material shall not be used in construction of embankments or backfilling around structures, and that embankments shall not be constructed on frozen ground. A survey of state highway department practices (*191*) shows that the specifications of Maine, Pennsylvania, and Wisconsin permit frozen material to be placed in embankments. Maine's specification requirements are that "embankments may be formed when the depth of the fill plus the depth of the frozen ground does not exceed 5 feet." Maine's specifications also state that "base courses may be formed on frozen subgrade when the subgrade has been properly compacted prior to freezing." Pennsylvania allows the forming of embankments on ground frozen to a thickness no greater than 3 in. (75 mm). Wisconsin prohibits the formation of embankments in the fall or early winter except when the material is primarily granular. Five other state highway departments do not specifically exclude frozen material from embankments, but specify that embankments shall be constructed of material acceptable to the engineer, containing no unsuitable, perishable, or deleterious material.

On the other hand, embankments have been constructed successfully in freezing weather; but care and close control are required to place and compact nonfrozen material before it freezes. According to a summary of practices of 21 northern states given in Table 14 of Yoakum's (*191*) survey, many states have reported successful fills at temperatures below 32°F (0°C) when clean granular materials of low moisture content were used and when fills were permitted to consolidate for a year or more. A high embankment was reported successfully constructed during the winter of 1955 by the Massachusetts Turnpike Authority. The fill consisted of alternating layers of 8 in. (200 mm) of glacial till (30 to 35 percent passing the No. 200 sieve) and 8 in. of clean sand compacted to 9 percent of Standard Proctor density. During the day the soil did not freeze; however, overnight temperatures dropped as low as 0°F (−18°C) and the surface of the fill froze to a thickness of about 3 in. (75 mm). The thin frozen crusts were not removed. Since construction no noticeable settlements have been observed; it is believed that the sand layers acted as wicks for absorbing moisture and added stability to the embankment.

Salts can be used to depress the freezing point of soil moisture and keep it workable at subfreezing temperatures. Brine can be sprayed on the unfrozen soil after spreading each layer, or can be mixed into the soil at the borrow area. This approach can not be used if the soil is already frozen or already has a high moisture content. Furthermore, public concern about pollution and environmental protection would preclude extensive use of salt. The New York State Highway Department experimented with mixing calcium chloride into the soil to prevent freezing, but abandoned the practice because of the cost of the chemical and the mixing effort required (*192*). Other experience by New York indicates great difficulty in achieving specified densities at low temperatures, even in relatively clean cohesionless soils. It was concluded that when the temperature drops to 20°F (−7°C), it is extremely uneconomical to construct embankments in the winter.

Portland Cement Concrete Paving

The quality of portland cement concrete paving is adversely affected if it is frozen during setting and curing. For this reason specifications usually prohibit placement of concrete when the temperature is below about 40°F (4°C), or allow placement only with the permission of the responsible engineer. When concreting is permitted at temperatures below 40°F, specifications generally require that the water or the aggregate, or both, be heated and that the concrete be protected during curing by warmed enclosures, or by use of straw, hay, tarpaulins, or insulation (blankets), to retain heat given off during curing. Specifications of temperatures to be maintained during curing are varied. The recommendations of the American Concrete Institute (*193*) are comparatively liberal: concrete temperatures during curing should be above 40°F for 3 days (2 days for high-early-strength cement) and above freezing for 3 days thereafter. Salts or chemicals to lower the freezing point of mixing water are prohibited.

Accelerators such as calcium chloride are permitted in some cases and in limited proportions (about 2 percent, by weight, of the cement). The Russians have experimented with mixing and placing concrete containing mixtures of sodium chloride and calcium chloride totaling as much as 20 percent by weight of the mixing water to prevent freezing of concrete at temperatures down to 0°F (−18°C). Stormer (*194*) conducted experiments of a similar nature that appeared to support the validity of the Russian work; however, he suggested further investigations of freeze-thaw durability in the presence of water, coordination of reinforcement, and other critical factors. Lacking further supporting evidence, the prevalent view in the United States is that if portland cement concrete is to remain undamaged by frost in a moist exposure, it must be made with frost-resistant aggregates, air entrainment, and protected from freezing until it has developed approximately 3,500 psi (24 MPa) compressive strength.

Bituminous Paving

The requirements for placing bituminous paving mixtures in cold weather are as exacting as those for portland cement concrete. Increased viscosity upon cooling interferes with compaction of bituminous mixtures and makes application of prime coats and seal coats ineffectual at low temperatures. Bonding of adjacent paving lanes is adversely affected if constructed in cold weather, unless the second lane is paved soon after the first lane, wh_ the mixture temperature is still high. Many states have arbitrary cutoff dates in the fall after which no hot-mix asphaltic concrete may be placed. There are, however, numerous cases reported where bituminous work has been done successfully at subfreezing temperatures. The asphaltic surfacing of Calumet Skyway Bridge in Chicago was constructed through the winter of 1957 when temperatures were below freezing, with no apparent loss of strength or durability of the asphaltic pavement. Special precautions were taken in transporting the hot asphaltic mix in insulated trucks and in compacting it immediately. The only restriction was that the surface on which the mixture was placed must be free of ice and snow. Studies and experimentation by The Asphalt Institute have shown that hot-laid asphaltic mixtures are not injured by freezing weather and may be placed at temperatures as low as 0°F provided the mixture is compacted immediately after placing (*195*). Corlew and Dickson (*196*) developed an analytical method for predicting the temperature of hot-mix asphaltic concrete from the time it leaves the paver until compaction to the required density is complete.

Placement of deep-lift bituminous-stabilized base and full-depth hot-mix pavement at below-freezing temperatures has been reported. In these cases the hot mix is laid directly on the subgrade. Good results have been reported in New Jersey (*197*) and some midwestern states. Thicker sections of hot mix provide increased flexibility for cold-weather paving because of greater volumetric heat retention of the thicker layer (8 to 10 in.—200 to 250 mm—or greater). Beagle (*198*) states that "hot plant-mixed asphalt base can be placed on frozen subgrades, in subfreezing weather—if it can be kept hot enough during rolling."

MAINTENANCE AND REPAIR

Preventive Maintenance

Certain preventive maintenance measures can be taken to eliminate or minimize detrimental effects of frost action on pavements. Many agencies seal joints and cracks with bitumen (Appendix A). Table 24 summarizes responses to the question on whether low-temperature contraction cracks are sealed, and when they are sealed. Thirty-one agencies responded positively that cracks are sealed; however, there is no consensus as to the best season to accomplish this work. In general, agencies operating in the colder, more northerly, regions tend to fill cracks in the spring, summer, or fall; some states enjoying warmer climates indicated that cracks are filled in the winter as the cracks occur and as temperatures permit. A detailed treatment of joint and crack sealing has been presented by the Highway Research Board (*27*).

Although highway snow and ice control were excluded from the scope of this synthesis, it is pertinent to note (Appendix A) the large part of maintenance budgets that is devoted to these operations, which, in the states and provinces that responded to this question, ranges from insignificant to 50 percent. Several publications containing information on this subject are worthy of mention. The Highway Research Board (*199*) provided guidelines for maintenance personnel using the equipment and technology then available. Since that time, snowplow trucks have generally become larger and faster moving, necessitating design changes in the plows. New types of snowplows (such as the underbody plow) have been developed, and undoubtedly more chemicals are used now than were used in 1962. Byrd et al. (*200*) recommended snow removal and ice control techniques at interchanges and presented information on snowplows; the Highway Research Board (*201*) discussed research in the area of snow removal and ice control; and Mellor (*202*) discusses blowing snow. The last two references also consider the location of snow fencing. At least one issue of *Rural and Urban Roads* (*203*) was devoted almost entirely to articles discussing snow and ice control and equipment.

Spring load restrictions also may be considered preventive maintenance procedures; all of the agencies were polled to determine the extent of application of load restrictions. The replies of 42 states and provinces that experience some frost action are summarized in Appendix A. Twenty-four of the respondents do not impose special spring load restrictions (Table 25); of the 18 that do, 15 qualified their affirmative replies to indicate that the restrictions are applied only on older roads, secondary roads, or in other special cases. Nine of the 18 determine the load restrictions and their duration by experience, and most of the remainder make use of deflection measurements for this purpose. The plate loading test is used in Minnesota, but experience and Benkleman beam tests are also used. North Dakota plans to use the Dynaflect (*10*) to determine load restrictions. In Quebec the duration of the restrictions is controlled by

TABLE 24

SEALING OF LOW-TEMPERATURE TRANSVERSE CONTRACTION CRACKS IN ASPHALTIC PAVEMENTS

ITEM	NO. OF AGENCIES
Agencies reporting occurrence of low-temperature cracks	40
Reported effect of low-temperature cracks on pavement serviceability:	
Significant	33
Slight, or significant only in certain conditions	4
Not significant	3
Sealing of cracks:	
Not sealed	5
Sealed in:	
Spring	4
Summer	5
Fall	7
Winter	3
Spring or summer	1
Spring or fall	1
Spring, fall, or winter	3
Warm weather	2
Not stated	8
No information	1

TABLE 25

RESTRICTIONS ON AXLE LOADINGS IMPOSED IN SPRING [a]

QUESTIONS AND RESPONSES	NO. OF STATES AND PROVINCES
Are special restrictions imposed on maximum axle loadings allowed in the spring?	
No	24
Yes, unqualified	3
Yes, old roads, weaker roads, or secondary roads	8
Yes, certain roads	4
Yes, except Interstates or heavy-duty roads	2
Yes, on incompleted roads in stage construction	1
Basis for determination of restricted loads, and of duration of restrictions:	
Experience	9
Deflections, measured with Benkelman beam	5
Experience, deflections, and plate bearing tests	1
Visual inspection plus Dynaflect	1
Judgment of local authorities	1
No information	1

[a] From information summarized in Appendix A.

law, but restrictions are imposed normally for only a portion of the period, and that portion is determined by experience. Nebraska uses restrictions only on stage-constructed roads, with the restrictions being applied only until the design pavement thickness is achieved.

Several countermeasures are used by maintenance forces to minimize problems due to icings. Few of these can be classified as preventive maintenance measures, being instead measures that are applied as the problems arise. By far the most common practice is to melt openings through ice-filled culverts and ditches, by means of steam, hot water, burners of various types, or electrical heating devices. Steam is generally supplied from truck-mounted boilers, and applied through hoses and nozzles. Use of steam is reported by Alaska, British Columbia, Alberta, Saskatchewan, Montana, Minnesota, Wisconsin, Iowa, Ontario, Quebec, Vermont, New Hampshire, and Maine. In Alaska and parts of northern Canada, where icing of many culverts occurs regularly, permanent thaw pipes are installed within culverts, and periodically a steam hose is attached to a riser pipe at one end of the culvert, with a condensate-return hose attached to a riser pipe at the other end. Steaming operations can thus be performed quickly and efficiently. In addition to steam, Vermont and Maine also use hot water to thaw ice-filled drainage facilities.

Burners are reportedly used in Alaska, Idaho, North Dakota, and Wyoming. Except in Alaska these burners are portable units (such as weed burners or propane torches) employed as needed to thaw openings in blocked drains or culverts. In Alaska, however, a type of burner is used that remains in place throughout the winter, generally at the upstream end of a culvert. This type of burner, which uses a fuel oil-gasoline mixture, is often operated continuously, either during the entire winter, or at least during the late winter-early spring period. Burner techniques similar to those used in Alaska are also known to be employed in northern Canada.

A substantial improvement over the use of steam or burners is represented by the introduction of electrical heating devices. In the survey questionnaire, electrical heating was reported to be in use in Alaska and Saskatchewan. Mineral-insulated, metal-sheathed heating cables are generally employed. They are strung through culverts in much the same manner as permanent steam thaw pipes. The installations may be permanent or only for the duration of the winter season. Where electric power is available, a switch control is operated by maintenance personnel when thawing is required. In areas not served by power lines, truck-mounted generators must visit each site and actuate the heating device until the desired opening is thawed. In Alaska, some installations have been equipped with thermostatic controls, timers, or cycling devices, for the purposes of more closely matching power outputs with thawing requirements and achieving unattended operation. Power outputs depend on cable resistances and lengths, but most installations have performed well with power outputs of 50 watts per linear foot (160 W/m) or less.

Some highway departments (e.g., Colorado, Michigan, Indiana) have reported the use of NaCl or CaCl in clearing ice-blocked drainage facilities, but current water-quality criteria make the introduction of chemicals directly into waterways undesirable if not illegal. Hand or mechanical removal of ice, reported by Colorado and Delaware, but clearly used elsewhere, may involve blades, scarifiers, and rippers, as well as hand tools. These techniques are usually practiced as expedient or emergency measures, rather than as parts of a planned maintenance program.

Pavement Repairs

Repair of frost-damaged roads may be accomplished by any of three methods, singly or in combinations. The two most commonly used procedures involve (1) removal of frost-susceptible soil and replacement with nonfrost-susceptible material, and (2) installation of a subsurface drainage network. The drainage network generally consists of perforated pipe or tile underdrains, but may include french drains. Side ditches may be cleaned, regraded, or deepened to solve localized frost heaving problems. The following statement from Highway Research Board Synthesis 9 (27) summarizes these two procedures: "A correction of serious frost heaves involves removal of existing pavement and provision of uniformity of soils and drainage within the depth of zone of freezing."

Recently, thermal-insulating materials have been used in reconstruction of badly frost-damaged roads. Either of two methods has been employed where insulation was used for this purpose. The first and most common procedure is to remove the pavement and all or a portion of the base material, and then repave the area. The second method has taken two forms. In the first, insulation is placed directly on the old pavement and new base material and pavement is placed over the insulation (Fig. 47). The principal advantages of this method are that no excavation is required

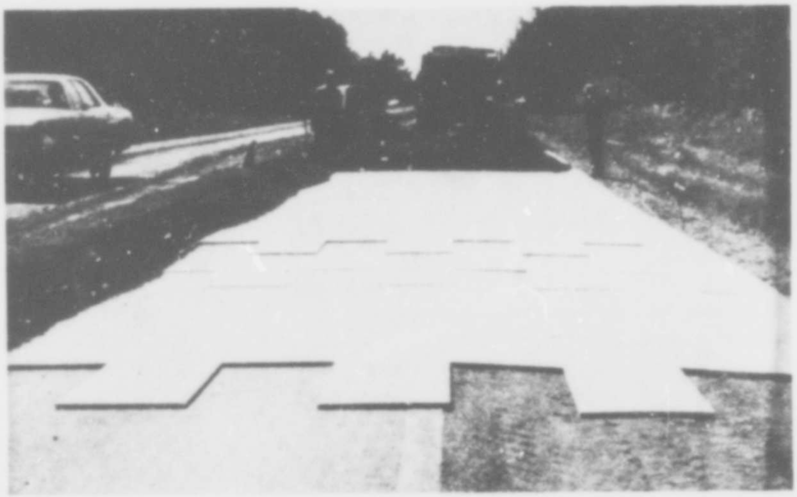


Figure 47. Overlaying frost-damaged pavement and shoulder with insulation and new base and pavement (204).

and traffic can be maintained easily by reconstructing one-half the width of the roadway and leaving the other half open to traffic. Williams (*224*) states that about 1,200 ft (365 m) of two-lane roadway can be reconstructed each day using this procedure. The primary disadvantage of this reconstruction procedure is that the finished grade is raised by about 2 ft.

In Norway a "top insulation" concept has recently been used (*205*). This concept is similar to that discussed by Williams except that a new pavement is placed directly on the insulating layer. Polyurethane and polystyrene having minimum compressive strengths of 114 psi (786 kPa) at 5 percent deflection have been used, and flexible pavements approximately 3¾ to 4¾ in. (95 to 120 mm) thick were placed directly over the insulating layer. The average rate of reconstruction of a 2.2-mile (3.54 km) segment of roadway using this concept was 430 ft per day (130 m/day); the maximum rate was 1,100 ft per day (335 m/day), which included placing the asphaltic pavement in three layers over the insulation. By using the "top insulation" concept, the final grade line is raised only 5 to 9 in. (125 to 230 mm), depending on the thicknesses of the insulating material and the pavement. Borg Hansen and Refsdal (*206*) state: "One obvious disadvantage with such a construction is the danger of ice forming on the surface because of the reduced heat flow toward the surface. Surface conditions were observed daily during the winter 1972-73, and although a certain degree of ice formation took place during the autumn, this was offset by light salting. From December on, the ice formation was more equal to that of uninsulated road; but when using this method, it must be a condition that salting takes place whenever required."

CHAPTER FIVE

RESEARCH NEEDS

The fundamental differences between roadway design in seasonal frost areas and nonfrost areas are that in frost areas consideration must be given to surface roughness and weakening of the layered system caused by frost effects and to cracking that develops when temperatures fall to low levels. These differences have been recognized for several decades, but a thorough understanding of the complex physical processes causing these effects has not yet been achieved. Mathematical models coupling heat and mass flux, stresses and strains in multilayered systems, and cumulative damage must be developed. With these models sensitivity tests should be conducted to ascertain the particular parameters and relationships that should be emphasized in continuing research. Current knowledge is inadequate in a number of broad topical areas, which may be grouped under the general headings of fundamentals of frost action and pavement design and performance. Research currently being conducted by the agencies surveyed (Appendix A) embraces both these general areas, and reflects an assessment by each of the agencies of the particular topics on which research is apt to be fruitful. The following is a brief assessment of the topics on which the current state of the art seems to be most deficient, and on which further research has the most promise of improving the cost effectiveness of roadway design in seasonal frost areas.

RESEARCH ON FACTORS FUNDAMENTAL TO FROST ACTION

The broad topic of frost action has engaged the interest of research workers for many years. Although much has been learned, understanding of the factors influencing frost action is still deficient. There is an acute need for applied research directed toward providing better answers for the following:

1. What are the soil factors that determine the severity of ice segregation, frost heave, and thaw-weakening, and how can they be measured and used to predict the severity of frost effects?
2. How can moisture migrations engendered by frost action be defined and forecast, and how can the moisture equilibrium conditions be predicted as a function of time, temperature, and space?
3. Given an accurately defined thermal regime, known soil properties, and a valid prediction of moisture equilibrium conditions, how can the volumetric expansion of a soil undergoing freezing be predicted?
4. What are the effective mechanisms of thaw-weakening and how can the resilient modulus and shear strength of soils be determined as a continuous function of temperature, freezing history, stress, and stress history?

RESEARCH ON DESIGN AND PERFORMANCE OF PAVEMENTS

Analysis and Design of Multilayered Systems

The most pressing need is that research workers who have access to methods of analysis of pavements based on mechanistic approaches translate those analytical approaches into design procedures accessible to pavement engineers. The choice of analytical models could be resolved initially in favor of either a linear elastic layered system analysis or a finite element model; the important criterion is that the model admit a mechanistic analytical approach; that is, an analysis of the transmitted forces and their action on the layered structure. This function of technology transfer will not be pioneering work, because mechanistic design systems already have been implemented on a small scale. Once they are more widely implemented, attention can be directed to research that will improve their application in seasonal frost areas. Improved application will come about through research and development in six topical areas, as follows:

1. Techniques for materials characterization, with modeling of strength, resistance to deformation, and volumetric equilibrium of the various materials, including the dependence of each of these properties on temperature, frost, and moisture.
2. Means of assessment of environmental influences, particularly the prediction of field moisture conditions in pavements as a function of time and space.
3. Yearly cumulative damage models, including both fatigue and distortion, that will account for variation of rate of damage incurred under repeated load applications, depending on temperature and on frozen, thawing, or thawed condition of the materials in the layered structure.
4. Coupling of mathematical models of temperature, moisture, and stresses and strains in the layered structure, with the cumulative damage model.
5. Means of predicting heave at a point on the pavement surface caused by frost action in the layered structure, in terms of temperature regime, properties and predicted moisture conditions of the layer or layers in which ice segregation occurs, and the thickness, stiffness, and weight of the materials overlying the layer in question.
6. Probabilistic models of surface roughness in terms of the predicted heave at a point on the surface, and of the properties of the layered structure.

Other Design Topics

There are countless other topics on which further research may be worthwhile. Four topics are mentioned that appear especially meritorious:

1. Low-temperature cracking of asphaltic pavements. Research in the last ten years has significantly advanced the state of the art in selection of asphalts that are less susceptible to embrittlement at low temperatures. The emphasis now should be in transferring this technology to the domain of design and construction. Further research also is needed on the low-temperature rheology of asphalts, methods for testing asphalts at low temperature, the effect of additives, and methods of rehabilitating cracked pavements.
2. Thermal barriers. Insulating layers for pavements and for culverts have great promise. Further research is needed to define the thermal regime around culverts and to examine and find solutions for the problem of surface icing of insulated pavements.
3. Encapsulation. This technique has promise as a means of utilizing poor-quality soil to replace better, but increasingly scarce, granular soil as base course. Laboratory and field experiments are needed to define limitations as to soil types suitable for encapsulation, and to determine the placement conditions, such as density and moisture content, that are necessary to minimize frost action.
4. Soil stabilization. Chemical and physical stabilization also offers a potential means of utilizing inferior and plentiful materials. Further research is needed to develop better approaches to mixture formulations that resist detrimental frost heave and freeze-thaw degradation. Encapsulation of soil containing small percentages of stabilizing additives also is a promising technique and a worthwhile topic for field experimentation.
5. Base-course drainage. The effectiveness of conventional base-course drainage that relies on horizontal flow within the base course needs to be assessed, and alternative systems providing vertical drainage tested. Subdrainage requirements need to be defined for wide paved areas (such as multilane highways), for full-depth asphaltic pavements, and for insulated pavement systems.

REFERENCES

1. LINELL, K. A., "Frost Action and Permafrost." Sect. 13, *Highway Engineering Handbook*, K. B. Woods (ed.), McGraw-Hill (1960).
2. CAREY, W. N., JR., and IRICK, P. E., "The Pavement Serviceability-Performance Concept." *HRB Bull. 250* (1960) pp. 40-58.
3. MONISMITH, C. L., Personal communication (1973).
4. PAINTER, L. J., "Analysis of AASHO Road Test Asphalt Pavement Data by the Asphalt Institute." *Hwy. Res. Record No. 71* (1965) pp. 15-38.
5. "Technical Manual—Roads and Air Fields." *TM5 250*, Dept. of the Army (1957).
6. "The AASHO Road Test: Report 5, Pavement Research." *HRB Spec. Rept. 61E* (1962) 352 pp.
7. KERSTEN, M. S., "Frost Penetration: Relationship to Air Temperature and Other Factors." *HRB Bull. 225* (1959) pp. 45-80.
8. "Investigation of Insulating Effect of Materials on Subgrade, Evaluation of First Year Data." *Interim Rept. 3-HPR-PR-1 (25)*, Maine State Highway Commission (1966).
9. LINELL, K. A., "Frost Design Criteria for Pavements." *HRB Bull. 71* (1953) pp. 18-32.
10. SCRIVNER, F. H., ET AL., "Detecting Seasonal Changes in Load-carrying Capabilities of Flexible Pavements." *NCHRP Report 76* (1969) 37 pp.
11. CHAMBERLAIN, E. J., "A Model for Predicting the Influence of Closed System Freeze-Thaw on the Strength of Thawed Clays." *OECD Oslo Symposium on Frost Action on Roads*, Vol. III (1973).
12. "Studies of Base Course Treatment to Prevent Frost Action." *TR-4*, Arctic Construction and Frost Effects Lab., New England Division, U.S. Army Engineers (1946).
13. "Studies of Base Course Treatment to Prevent Frost Action." *TR-11*, Arctic Construction and Frost Effects Lab., New England Division, U.S. Army Engineers (1947).
14. LINELL, K. A., and KAPLAR, C. W., "The Factor of Soil and Material Type in Frost Action." *HRB Bull. 225* (1959) pp. 81-126.
15. PENNER, E., "The Mechanism of Heaving in Soils." *HRB Bull. 225* (1959) pp. 1-22.
16. COOK, R. D., "Some Effects of Closed-System Freeze-Thaw Cycles on a Compacted Highly Plastic Clay." M.S. Thesis, Dept. of Civil Engineering, Univ. of Alberta (1963).
17. TITOV, V. P., "Strength of Thawing Ground." Kom. po. zem. pol. Bor'ba s puchinami na zheleznykh; avtomobil'nykh doragakh (1965) pp. 178-183; *Draft trans. TL 156*, CRREL (1970).
18. MIKHAILOV, G. D., and BREDYUK, G. P., "Shear Strength of Clayey Ground During Thawing (According to Laboratory and Field Studies)." Materially VIII vsesoyuznogo mezhduvedom stennogo soveschaniya po Geokriologii (Merzlotovedeniya), Akad. Nauk SSSR, Sibirskogoe otdelenie merzlotovedeniya, n. 5 (1966) pp. 51-60; *Draft trans. TL 265*, CRREL (1971).
19. GARSTKA, W. U., "Snow and Snow Survey." Sect. 10 *Handbook of Applied Hydrology*, V. T. Chow (ed.), McGraw-Hill (1964) 57 pp.
20. "Pavement Design for Frost Conditions." *TM5-818-2*, Dept. of the Army (1965).
21. OTIS, P., Personal communication (1970).
22. "Arctic and Subarctic Construction, Foundations of Structures." *TM5-852-4*, Dept. of the Army (in press).
23. LANE, K. S., "Treatment of Frost-Sloughing Slopes." *Proc. Second Internat. Conf. on Soil Mechanics and Foundation Engineering*, Rotterdam (1948).
24. FROMM, H. J., and PHANG, W. A., "A Study of Transverse Cracking of Bituminous Pavements." *Proc. Assoc. Asphalt Paving Tech.*, Vol. 41 (1972) pp. 383-423.
25. MCLEOD, N. W., "Relationship Between Pavement Structural Integrity and Hardness of the Asphalt Cement." *Proc. Third Internat. Conf. on the Structural Design of Asphalt Pavements* (1972).
26. SHAHIN, M. Y., and MCCULLOUGH, B. F., "Prediction of Low-Temperature and Thermal-Fatigue Cracking in Flexible Pavements." *Res. Rept. 123-14*, Center for Highway Research, Texas Transportation Inst., and Texas Highway Dept. (1972).
27. "Pavement Rehabilitation—Materials and Techniques." *NCHRP Synthesis 9* (1972) 41 pp.
28. KAPLAR, C. W., "Frost Action in Soils." *IR-92*, movie film, 35 min, CRREL (1964).
29. KAPLAR, C. W., "Stone Migration by Freezing of Soil." *Science*, 149, No. 3691 (1965).
30. CAREY, K. L., "Icings Developed from Surface Water and Ground Water." *Cold Regions Sci. and Eng. Monograph IIID3*, CRREL (1973) 69 pp.
31. "Arctic Construction." *TM5-349*, Dept. of the Army (1962).
32. LINELL, K. A., HENNION, F. B., and LOBACZ, E. F., "Corps of Engineers' Pavement Design in Areas of Seasonal Frost." *Hwy. Res. Record No. 33* (1963) pp. 76-136.
33. ALDRICH, H. P., JR., "Frost Penetration Below Highway and Airfield Pavements." *HRB Bull. 135* (1956) pp. 124-149.
34. "Calculation Methods for Determination of Depths of Freeze and Thaw in Soils." *TM5-852-6*, Dept. of the Army (1966).
35. THOMPSON, M. R., "Environmental Factors and Pavement Systems, State of the Art." *Proc. Allerton Park Conference on Systems Approach to Airfield Pave-*

ments, Tech. Rept. P-5, Construction Engineering Research Laboratory, U.S. Army (1973).

36. LINSLEY, R. K., JR., KOHLER, M. A., and PAULHUS, J. L. H., *Applied Hydrology*. McGraw-Hill (1949) 689 pp.
37. MACKAY, J. R., "A Frost Tube for the Determination of Freezing in the Active Layer above Permafrost." *Canad. Geotech. Jour.*, Vol. 10, No. 3 (1973).
38. JUMIKIS, A. R., *Thermal Soil Mechanics*. Rutgers Univ. Press (1966).
39. ALDRICH, H. P., and PAYNTER, H. M., "Derivation of a Rational Formula for the Prediction of Frost Penetration." *HRB Bull. 135* (1956) pp. 145-149.
40. SANGER, F. J., "Degree Days and Heat Conduction in Soils." *Proc. International Conference on Permafrost*, NAS-NRC (1963).
41. TABER, S., "Frost Heaving." *Jour. Geol.*, Vol. 37 (1929).
42. TABER, S., "A New Theory of Frost Heaving—Discussion." *Proc. HRB*, Vol. 11, Pt. 1 (1932) p. 173.
43. BESKOW, G., "Soil Freezing and Frost Heaving with Special Application to Roads and Railroads" (1935). Transl. by J. O. Osterberg, Tech. Inst., Northwestern Univ. (1947).
44. CASAGRANDE, A., "A New Theory of Frost Heaving—Discussion." *Proc. HRB*, Vol. 11, Pt. 1 (1932) pp. 168-172.
45. CASAGRANDE, A., "Effects of Frost in Soil." *Proc. 8th Congress*, Permanent International Association of Road Congresses (1938) 6th Question, p. 10.
46. RIIS, J., "Frost Damage to Roads in Denmark." *Proc. Second International Conference on Soil Mechanics and Foundation Engineering*, Vol. 2 (1948) p. 282.
47. "Data Report of Frost Investigations, 1943-49." *TR-20*, Vol. II, Arctic Construction and Frost Effects Laboratory, New England Division, U.S. Army Engineers (1947).
48. JACKSON, K. A., and CHALMERS, B., "Freezing of Liquids in Porous Media with Special Reference to Frost Heave in Soils." *Jour. Appl. Physics*, Vol. 29, No. 8 (1958).
49. CHALMERS, B., and JACKSON, K. A., "Experimental and Theoretical Studies of the Mechanism of Frost Heaving." *RR-199*, CRREL (1970).
50. EVERETT, D. H., "The Thermodynamics of Frost Damage to Porous Solids." *Trans. Faraday Soc.*, Vol. 57 (1961) pp. 1541-1551.
51. EVERETT, D. H., and HAYNES, J. M., "Capillary Properties of Some Model Pore Systems with References to Frost Damage." *RILEM Bull.*, New Series 27 (1965) pp. 31-38.
52. KAPLAR, C. W., "A Laboratory Freezing Test to Determine the Relative Frost Susceptibility of Soils." *Tech. Rept. TR250*, CRREL (1974).
53. TOWNSEND, D. L., and CSATHY, T. I., "Soil Type in Relation to Frost Action." *Rept. 15*, Ontario Joint Highway Program, Queen's University (1963).
54. PENNER, E., "Pressures Developed in a Porous Granular System as a Result of Ice Segregation." *HRB Spec. Rept. 40* (1958) pp. 191-199.
55. AITKEN, G. W., "Reduction of Frost Heave by Surcharge Loading." *Proc. International Conference on Permafrost* (1963).
56. "Subsurface Drainage Facilities for Airfields." *TM5-820-2*, Dept. of the Army (1965).
57. WILLIAMS, P. J., "Use of the Ice-Water Surface Tension Concept in Engineering Practice." *Hwy. Res. Record No. 393* (1972) pp. 19-29.
58. KAPLAR, C. W., "Phenomenon and Mechanism of Frost Heaving." *Hwy. Res. Record No. 304* (1970) pp. 1-13.
59. HOEKSTRA, P., CHAMBERLAIN, E., and FRATE, A., "Frost Heaving Pressures." *RR-176*, CRREL (1965).
60. WISSA, A. E. Z., and MARTIN, R. T., "Frost Susceptibility of Massachusetts Soils—Evaluation of Rapid Frost Susceptibility Tests." *Soils Publ. 320*, Massachusetts Inst. of Technology (1973).
61. TURNER, K. A., and JUMIKIS, A. R., "Loss of Bearing Capacity and Vertical Displacements of New Jersey Soils." *HRB Bull. 135* (1956) pp. 24-76.
62. LINELL, K. A., and KAPLAR, C. W., "Description and Classification of Frozen Soils." *TR-150*, CRREL (1966).
63. "Soil Mechanics for Road Engineers." Road Research Laboratory (1952) pp. 293-345.
64. KAPLAR, C. W., "New Experiments to Simplify Frost Susceptibility Testing of Soils." *Hwy. Res. Record No. 215* (1968) pp. 48-59.
65. BIDDESCOMBE, J. F., ZOLLER, J. H., and SANBORN, J. L., "Frost Susceptibility of New Hampshire Base Courses." Res. Proj. Rept., Civil Engineering Dept., Univ. of New Hampshire (1966).
66. WISSA, A. E. Z., and MARTIN, R. T., "Behavior of Soils Under Flexible Pavements: Development of Rapid Frost Susceptibility Tests." Dept. of Civil Engineering, Massachusetts Inst. of Technology (1968).
67. JESSBERGER, H. L., and CARBEE, D. L., "Influence of Frost Action on the Bearing Capacity of Soils." *Hwy. Res. Record No. 304* (1970) pp. 14-26.
68. BROMS, B., and YAO, Y. C., "Shear Strength of a Soil after Freezing and Thawing." *Jour. of Soil Mech. and Foundations*, Div. SM4, ASCE (1964).
69. CULLEY, R. W., "Effect of Freeze-Thaw Cycling on Stress-Strain Characteristics and Volume Change of a Till Subjected to Repetitive Loading." *Canad. Geotech. Jour.*, Vol. 8, No. 3 (1971) pp. 359-371.
70. QUINN, W. F., CARBEE, D., and JOHNSON, T. C., "Membrane Encapsulated Soil Layers (MESL) for Road Construction in Cold Regions." *OECD Oslo Symposium on Frost Action on Roads*, Vol. II (1973).
71. THOMSON, S., and LOBACZ, E. F., "Shear Strength at a Thaw Interface." *North American Contributions to the Second International Permafrost Conference* (1973) pp. 419-426, NAS.
72. CEDERGREN, H. L., "Seepage, Drainage, and Flow Nets." Wiley (1967).
73. *Moisture Equilibria and Moisture Changes in Soils Beneath Covered Areas*. G. D. Aitchison (ed.), Butterworth (1965).
74. "Water in Roads: Predictions of Moisture Content of Road Subgrades." OECD (1973).

75. "Water and Its Conduction in Soils: An International Symposium." *HRB Spec. Rept. 40* (1958) 338 pp.
76. JUMIKIS, A. R., "Some Concepts Pertaining to the Freezing Soil Systems." *HRB Spec. Rept. 40* (1958) pp. 178-190.
77. HOEKSTRA, P., "Water Movement Under Temperature Gradients with the Cold-Side Temperature Below Freezing." *Water Resources Res.*, Vol. 2, No. 2 (1966) pp. 241-250.
78. MCGAW, R., "Frost Heaving Versus Depth to Water Table." *Hwy. Res. Record No. 393* (1972) pp. 45-55.
79. PENNER, E., "Influence of Freezing Rate on Frost Heaving." *Hwy. Res. Record No. 393* (1972) pp. 56-64.
80. MCALPIN, G. W., and HOFMANN, W. P., "Soil Explorations." *Highway Engineering Handbook*, K. B. Woods (ed.), Sec. 10, McGraw-Hill (1960) 66 pp.
81. HOFMANN, W. P., "Design of Swiss Roads Against Frost Action—Discussion." *Hwy. Res. Record No. 33* (1964) pp. 168-169.
82. "Aerialphoto Interpretation in Classifying and Mapping Soils." *Agriculture Handbook 294*, Soil Conservation Service, U.S. Dept. of Agriculture (1966) 89 pp.
83. "Engineering Soils Survey of New Jersey." Reports 1 to 22, Bur. of Engineering Research, Rutgers University (1950-1957).
84. JUMIKIS, A. R., HOLMAN, W. W., and SCHUYLER, J. R., "The Engineering Soil Survey and its Relation to Engineering Problems." *HRB Bull. 213* (1959) pp. 1-11.
85. "Remote Sensing for Highway Engineering." *Hwy. Res. Record No. 421* (11 reports) (1972) 93 pp.
86. TANGUAY, M. C., and MILES, R. D., "Multispectral Data Interpretation for Engineering Soils Mapping." *Hwy. Res. Record No. 319* (1970) pp. 58-77.
87. "AASHO Interim Guide for Design of Pavement Structures." AASHO (1972) 125 pp.
88. "Thickness Design, Full-Depth Asphalt Pavement Structures for Highways and Streets." *MS-1*, The Asphalt Institute (1970).
89. "Development of Rigid Pavement Thickness Requirements for Military Roads and Streets." Ohio River Division Laboratories, Corps of Engineers, Dept. of the Army (1961).
90. "Thickness Design for Concrete Pavements." *Booklet No. IS010.02P*, Portland Cement Association (1966).
91. BERGAN, A. T., and MONISMITH, C. L., "Some Fatigue Considerations in the Design of Asphalt Concrete Pavements." Presented at Vancouver meeting, Canadian Technical Asphalt Association (1972).
92. PEATTIE, K. R., "A Fundamental Approach to the Design of Flexible Pavements." *Proc. First International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1962).
93. DORMON, G. M., and METCALF, C. T., "Design Curves for Flexible Pavements Based on Layered System Theory." *Hwy. Res. Record No. 71* (1965) pp. 69-84.
94. BROWN, S. F., and PELL, P. S., "A Fundamental Structural Design Procedure for Flexible Pavements." *Proc. Third International Conference on the Structural Design of Asphalt Pavements* (1972).
95. BERGAN, A. T., "Some Considerations in the Design of Asphalt Concrete Pavements for Cold Regions." Ph.D. dissertation, University of California, Berkeley (1972).
96. DEACON, J. A., "Materials Characterization—Experimental Behavior." *HRB Spec. Rept. No. 126* (1971) pp. 150-179.
97. SEED, H. B., CHAN, C. K., and LEE, C. E., "Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements." *Proc. First International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1962).
98. LAREW, H. G., and LEONARDS, G. A., "A Strength Criterion for Repeated Loads." *Proc. HRB*, Vol. 41 (1962) pp. 529-556.
99. PELL, P. S., and BROWN, S. F., "The Characteristics of Materials for the Design of Flexible Pavement Structures." *Proc. Third International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1972).
100. BERGAN, A. T., and MONISMITH, C. L., "Characterization of Subgrade Soils in Cold Regions for Pavement Design Purposes." *Hwy. Res. Record No. 431* (1973) pp. 25-37.
101. "Bituminous Aggregate Base Course: Survey of State Practices." *HRB Spec. Rept. No. 117* (1971) 67 pp.
102. MOULTON, L. K., and SCHAUB, J. H., "A Rational Approach to the Design of Flexible Pavements to Resist the Detrimental Effects of Frost Action." *Hwy. Res. Record No. 276* (1969) pp. 25-38.
103. "A Guide to the Structural Design of Flexible and Rigid Pavements in Canada." Roads and Transportation Assn. of Canada (1971).
104. "Summary Report of Frost Investigations, 1944-1947." *Internal Rept. IR-7*, New England Division, U.S. Army Engineers, Arctic Construction and Frost Effects Laboratory (1951).
105. SAYMAN, W. C., and GILMAN, G. D., "Field Investigations at Frost Test Area, Loring AFB, and Supplementary Investigations, 1950-1956." *Rept. IR-35*, CRREL (1967).
106. THOMPSON, M. R., "Durability and Frost Resistance of Lime and Cement Stabilized Soils." *Oslo Symposium on Frost Action on Roads*, Vol. II, OECD (1973).
107. HICKS, R. G., "Factors Influencing the Resilient Properties of Granular Materials." Ph.D. dissertation, Inst. of Transportation and Traffic Engineering, Univ. of California, Berkeley (1970).
108. BAZIN, P., and SAUNIER, J. B., "Deformability, Fatigue and Healing Properties of Asphalt Mixes." *Proc. Second International Conference on Structural Design of Asphalt Pavements*, Univ. of Michigan (1967).
109. HAYNES, J. H., and YODER, E. J., "Effects of Repeated Loading on Gravel and Crushed Stone Base Course Materials Used in the AASHO Road Test." *Hwy. Res. Record No. 39* (1963) pp. 82-96.

110. Kallas, B. F., and Riley, J. C., "Mechanical Properties of Asphalt Paving Materials." *Proc. Second International Conference on Structural Design of Asphalt Pavements*, Univ. of Michigan (1967).
111. Seed, H. B., Mitry, F. G., Monismith, C. L., and Chan, C. K., "Prediction of Flexible Pavement Deflections from Laboratory Repeated-Load Tests." *NCHRP Report 35* (1967) 117 pp.
112. Barenberg, E. J., "Behavior and Performance of Aggregate-Soil Systems Under Repeated Loads." Univ. of Illinois (1971).
113. Johnson, T. C., "Is Graded Aggregate Base the Solution in Frost Areas?" *Proc. Conference on Utilization of Graded Aggregate Base*, National Crushed Stone Association (in press).
114. Lovering, W. R., and Cedergren, H. R., "Structural Section Drainage." *Proc. International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1962).
115. Haas, R. C. G., et al., "Low-Temperature Pavement Cracking in Canada: The Problem and Its Treatment." *Proc. Canadian Good Roads Association* (1970).
116. van der Poel, C., "A General System Describing the Viscoelastic Properties of Bitumens and Its Relation to Routine Test Data." *Jour. Appl. Chem.*, Vol. 4 (1954).
117. Heukelom, W., and Klomp, A. G., "Road Design and Dynamic Loading." *Proc. Assn. of Asphalt Paving Technol.*, Vol. 33 (1964) pp. 92-125.
118. Hajek, J. J., and Haas, R. C. G., "Predicting Low-Temperature Cracking Frequency of Asphalt Concrete Pavements." *Hwy. Res. Record No. 407* (1972) pp. 39-54.
119. McLeod, N. W., Prepared Discussion on Ste. Anne Test Road. *Proc. Canadian Technical Asphalt Assn.* (1969).
120. Pfeiffer, J. P., and van Doormal, P. M., "The Rheological Properties of Asphaltic Bitumen." *Jour. Inst. Petrol. Tech.*, Vol. 22 (1936).
121. Halstead, W. J., and Welborn, J. Y., "Development of Specifications for Viscosity-Graded Asphalts." *Hwy. Res. Record No. 350* (1971) pp. 1-11.
122. Fromm, H. J., and Phang, W. A., "Viscosity Grading of Asphalts—Discussion." *Hwy. Res. Record No. 350* (1971) p. 63.
123. "Structural Design of Asphalt Concrete Pavement Systems." *HRB Spec. Rept. 126* (1971) 207 pp.
124. Dempsey, B. J., and Thompson, M. R., "Heat-Transfer Model for Evaluating Frost Action and Temperature Related Effects in Multi-Layered Pavement Systems." *Prog. Rept. IHR 401*, Dept. of Civil Engineering, Univ. of Illinois (1969).
125. Berg, R. L., "Thermoinsulating Media Within Embankments on Perennially Frozen Soil." Ph.D. dissertation, Univ. of Alaska (1973).
126. Barber, E. S., "Calculation of Maximum Pavement Temperatures from Weather Reports." *HRB Bull. 168* (1957) pp. 1-8.
127. Kasianchuk, D. A., "Fatigue Considerations in the Design of Asphalt Concrete Pavements." Ph.D. dissertation, Univ. of California, Berkeley (1968). (From Ref. 91).
128. Witczak, M. W., "Design of Full-Depth Asphalt Airfield Pavements." *Proc. Third International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1972).
129. Croney, D., and Coleman, J. D., "Soil Thermodynamics Applied to the Movement of Moisture in Road Foundations." *Proc. Seventh International Congress for Applied Mechanics* (1948).
130. Gurr, C. G., Marshall, T. J., and Hutton, J. T., "Movement of Water in Soil Due to a Temperature Gradient." *Soil Sci.*, Vol. 74 (1952).
131. Hoekstra, P., "Moisture Movement to a Freezing Front." CRREL, U.S. Army Material Command (1967).
132. Van Til, et al., "Evaluation of AASHO Interim Guides for Design of Pavement Structures." *NCHRP Report 128* (1972) 111 pp.
133. Lobacz, E. F., Gilman, G. D., and Hennion, F. B., "Corps of Engineers' Design of Highway Pavements in Areas of Seasonal Frost." *Oslo Symposium on Frost Action on Roads*, Vol. II, OECD (1973).
134. Dormon, G. M., and Edwards, J. M., "Pavement Design for Road Traffic." Appendix I to "Structural Design of Asphalt Pavements for Heavy Aircraft," Shell Construction Services (1970).
135. Barksdale, R. D., "Rutting of Pavement Materials." Unpubl. rept., Georgia Institute of Technology (1973).
136. Kasianchuk, D. A., Terrel, R. L., and Haas, R. C. G., "A Design System for Minimizing Fatigue, Permanent Deformation and Shrinkage Fracture Distress of Asphalt Pavements." *Proc. Third International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1972) pp. 629-655.
137. Heukelom, W., and Klomp, A. G. J., "Consideration of Calculated Strains at Various Depths in Connection with the Stability of Asphalt Pavements." *Proc. Second International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1967) pp. 155-168.
138. Casagrande, A., and Shannon, W. L., "Base Course Drainage for Airport Pavements." *Trans. ASCE*, Vol. 117 (1952) pp. 792-820.
139. Cedergren, H. R., and Lovering, W. R., "The Economics and Practicability of Layered Drains for Roadbeds." *Hwy. Res. Record No. 215* (1968) pp. 1-7.
140. Cedergren, H. R., and Ken O'Brien and Associates, "Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections." Federal Highway Administration (1972).
141. "Implementation Package for a Drainage Blanket in Highway Pavement Systems." Federal Highway Administration (1972).
142. Ring, G. W., "Seasonal Strength of Pavements." *Proc. Oslo Symposium on Frost Action on Roads*, Vol. III, OECD (1973).
143. Kersten, M. S., and Skok, E. L., "Application

AASHO Road Test Results to Design of Flexible Pavements in Minnesota." *Hwy. Res. Record No. 291* (1969) pp. 70-88.

144. FREDRICKSON, F. C., DIETHELM, P. J., and ZWIERS, D. M., "Minnesota Department of Highways Flexible Pavement Design—1969." *Hwy. Res. Record No. 329* (1970) pp. 55-64.

145. CHRISTISON, J. T., and ANDERSON, K. O., "The Response of Asphalt Pavements to Low-Temperature Climatic Environments." *Proc. Third International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1972) pp. 41-52.

146. FROMM, H. J., and PHANG, W. A., "Temperature-Susceptibility Control in Asphalt-Cement Specifications." *Hwy. Res. Record No. 350* (1971) pp. 30-38.

147. WELBORN, J. Y., "Temperature-Susceptibility Control in Asphalt-Cement Specifications—Discussion." *Hwy. Res. Record No. 350* (1971) p. 39.

148. MAREK, C. R., and DEMPSEY, B. J., "A Model Utilizing Climatic Factors for Determining Stresses and Deflections in Flexible Pavement Systems." *Proc. Third International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan (1972) pp. 101-114.

149. YODER, E. J., *Principles of Pavement Design*. Wiley (1959).

150. *Highway Engineering Handbook*. K. B. Woods, D. S. Berry, and W. H. Goetz (ed.), McGraw-Hill (1960).

151. BARENBERG, E. J., "Mathematical Modeling of Pavement Systems: State of the Art." *Proc. Allerton Park Conference on Systems Approach to Airfield Pavements, Tech. Rept. P-5*, U.S. Army Construction Engineering Research Laboratory (1973).

152. WESTERGAARD, H. M. S., "Analysis of Stresses in Concrete Pavements Due to Variations in Temperature." *Proc. HRB*, Part I (1926) pp. 201-215.

153. WESTERGAARD, H. M. S., "Mechanics of Progressive Cracking in Concrete Pavements." *Pub. Roads*, Vol. 10, No. 4 (1929).

154. "Rigid Pavements for Roads, Streets, Walks and Open Storage Areas." *TM5-822-6*, Dept. of the Army (1969).

155. "Subgrades and Subbases for Concrete Pavements." *Booklet No. IS029.02P*, Portland Cement Association (1971).

156. HUTCHINSON, R. L., "Basis for Rigid Pavement Design for Military Airfields." *Misc. Paper No. 5-7*, Ohio River Division Laboratories, Corps of Engineers, Dept. of the Army (1966).

157. BARENBERG, E. J., "Response of Subgrade Soils to Repeated Dynamic Loads: State of the Art." *Proc. Allerton Park Conference on Systems Approach to Airfield Pavements, Tech. Rept. P-5*, U.S. Army Construction Engineering Research Laboratory (1973b).

158. LINELL, K. A., and HALEY, J. F., "Investigation of the Effect of Frost Action on Pavement Supporting Capacity." *HRB Spec. Rept 2* (1952) pp. 295-325.

159. SAYMAN, W. C., "Plate-Bearing Study of Loss of Pavement Supporting Capacity Due to Frost." *HRB Bull. 111* (1955) pp. 99-106.

160. CUMBERLEDGE, G., "A Study of Factors Influencing the Reduction of Highway Subgrade Support During the Spring Thaw Period." M.S. thesis, West Virginia Univ. (1967).

161. "A Charted Summary of Concrete Highway Pavement Practices in the United States and Canada." *Booklet No. IS0011.10*, Portland Cement Association (1969).

162. "Performance Study Report on Insulation Board (Polystyrene)." Subcommittee on Development, Evaluation and Recommendations of New Highway Materials, AASHO-ARBA (1970).

163. LACHENBRUCH, A. H., "Periodic Heat Flow in Stratified Medium with Application to Permafrost Problems." Geological Survey Bull. 1083-A (1959).

164. REFSDAL, G., "The Use of Insulating Materials in Highway Engineering." *Frost i Jord*, No. 9 (Mar. 1973).

165. KREBS, R. D., and WALKER, R. D., *Highway Materials*. McGraw-Hill (1971).

166. HAAS, W. M., "Symposium Summary." *Hwy. Res. Record No. 33* (1963) pp. 262-270.

167. ARMSTRONG, M. D., and CSATHY, T. I., "Frost Design Practice in Canada." *Hwy. Res. Record No. 33* (1963) pp. 170-198.

168. TAIVAINEN, O. A., "Preventive Measures to Reduce Frost Action on Highways in Finland." *Hwy. Res. Record No. 33* (1963) pp. 202-210.

169. RENGMARK, F., "Highway Pavement Design in Frost Areas in Sweden." *Hwy. Res. Record No. 33* (1963) pp. 137-150.

170. PROKSCH, H., "German Experiences with the Replacement of Granular Frost Blankets by Other Types of Construction." *Proc. Third International Conference on the Structural Design of Asphalt Pavements*, Univ. of Michigan, Vol. I (1972).

171. BENSON, J. R., "A Review of Factors to Be Considered in the Use of Asphalt Membrane Envelopes in Highway and Airfield Pavement Design." The Asphalt Institute (1953).

172. HARRIS, F. A., "Waterproofing Value of Asphalt Membrane in Earth Fills of Gulf Freeway." Texas Highway Dept. (1953).

173. PINCHBECK, G. R., "Alberta Highway Department Carries out an Experiment on Base Enveloping with Asphalt." *Roads and Eng. Constr.* (May 1954) pp 99-130.

174. BELL, J. R., and YODER, E. J., "Plastic Moisture Barriers for Highway Subgrade Protection." *Proc. HRB*, Vol. 36 (1957) pp. 713-735.

175. "Frost Investigations: Cold Room Studies." *TR-43*, Third Interim Report of Investigations, Arctic Construction and Frost Effects Laboratory, New England Division, U.S. Army Engineers (1958).

176. PEYTON, H. R., KENNEDY, C. F., and LUND, J. W., "Stabilization of Silty Soils in Alaska—Phase II." Alaska Dept. of Highways (1967).

177. BURNS, C. D., and BRABSTON, W. N., "Membrane-Envelope Technique for Waterproofing Soil Base Courses for Airstrips: Bare Base Support." *Misc.*

Paper S-68-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg (1968).

178. BURNS, C. D., BRABSTON, W. N., and GRAU, R. W., "Feasibility of Using Membrane-Enveloped Soil Layers as Pavement Elements for Multiple-Wheel Heavy Gear Loads." *Misc. Paper S-72-6*, U.S. Army Engineer Waterways Experiment Station, Vicksburg (1972).

179. SALE, J. P., PARKER, F., and BARKER, W. R., "Membrane Encapsulated Soil Layers." *Jour. Soil Mechanics and Foundations Div.*, ASCE, Vol. 99, SM 12 (1973).

180. LAMBE, T. W., KAPLAR, C. W., and LAMBIE, T. J., "Additives for Modifying the Frost Susceptibility of Soils." *TR 123*, Part II, CRREL (1971).

181. BRANDT, G. H., "Chemical Additives to Reduce Frost Heave and Water Accumulation in Soils." *Hwy. Res. Record No. 393* (1972) pp. 30-41.

182. WARNER, J. C., "Strength Properties of Chemically Solidified Soils." *Jour. Soil Mechanics and Foundations Div.*, ASCE, Vol. 98, SM 11 (1972).

183. CRONEY, D., and JACOBS, J. C., "The Frost Susceptibility of Soils and Road Materials." *Report LR 90*, Road Research Laboratory (1967) 68 pp.

184. HARDY, R. M., "Prevention of Frost Heaving by Injection of Spent Sulphite Liquor." *Proc. Third International Conference on Soil Mechanics*, Vol. II (1953) pp. 103-106.

185. THOMPSON, M. R., "Lime-Reactivity of Illinois Soils." *Jour. Soil Mechanics and Foundations Div.*, ASCE, Vol. 92, SM 5 (1966).

186. THOMPSON, M. R., "Soil Stabilization for Pavement Systems: State of the Art." *Tech. Rept. P-5, Proc. Allerton Park Conference on Systems Approach to Airfield Pavements*, Construction Engineering Research Lab., Univ. of Illinois (1973).

187. THOMPSON, M. R., "Lime-Treated Soils for Pavement Construction." *Jour. Highway Div.*, ASCE, HW2 (1968).

188. "Soil Cement Laboratory Handbook." Portland Cement Association (1959).

189. HERRIN, M., "Bituminous-Aggregate and Soil Stabilization." Section III, *Hwy. Engrg. Handbook*, K. B. Woods (ed.), McGraw-Hill (1960).

190. AUGHENBAUGH, N. B., JOHNSON, R. B., and YODER, E. J., "Degradation of Base Course Aggregates During Compaction." *Tech. Rept. TR-166*, CRREL (1966).

191. YOAKUM, D., "A Survey of Winter Construction Practices, Earthwork, Concrete, and Asphalt." *S. R. 76*, CRREL (1966).

192. YOAKUM, D., "Winter Construction of Earthwork and Foundations." *Civil Eng.* (Aug. 1967) p. 50.

193. "Recommended Practice for Cold Weather Concreting." No. 306-6, American Concrete Inst. (1966).

194. STORMER, C. D., "Cold Concrete." *TR 220* (AD 705561) CRREL (1970).

195. MACATEE, W. R., "Cold Weather Paving of Asphaltic Roads." *Roads and Bridges*, Vol. 8, No. 2 (1943).

196. CORLEW, J. S., and DICKSON, P. F., "Methods for Calculating Temperature Profiles of Hot-Mix Asphaltic Concrete as Related to the Construction of Asphalt Pavement." *Proc. Assn. Asphalt Paving Tech.*, Vol. 37 (1968) p. 101.

197. BEAGLE, C. W., "Compaction of Deep Lift Bituminous Stabilized Base." *Proc. Assn. Asphalt Paving Tech.*, Vol. 35 (1966) pp. 549-566.

198. BEAGLE, C. W., "Cold Weather Paving—Temperature—Density." *Proc. Assn. Asphalt Paving Tech.*, Vol. 38 (1969) pp. 519-536.

199. "Current Practices for Highway Snow and Ice Control." *Current Road Problems No. 9-4R*, Highway Research Board (1962) 33 pp.

200. BYRD, TALLAMY, MACDONALD, and LEWIS, "Snow Removal and Ice Control Techniques at Interchanges." *NCHRP Report 127* (1971) 90 pp.

201. "Snow Removal and Ice Control Research." *HRB Spec. Rept. No. 115* (1970) 282 pp.

202. MELLOR, M., "Blowing Snow." *Monograph III-A3c*, CRREL (1965) 79 pp.

203. Several staff reports on snow and ice control. *Rural and Urban Roads*, Vol. II, No. 7 (July 1973).

204. WILLIAMS, W. G., "Development and Use of Plastic Foam Insulation to Prevent Frost Action Damage to Transportation Facilities." *Proc. of Symposium on Civil Engineering*, Univ. of Alaska (1971) pp. 227-276.

205. BORG HANSEN, P., "Frost Protection of Existing Roads by a Top Insulating Layer." *Frost I Jord*, No. 9 (1973) pp. 7-19.

206. BORG HANSEN, P., and REFSDAL, G., "Thermal Insulation of Roads—Design Aspects Regarding Strength, Moisture Absorption and Insulation Thickness." *Proc. Oslo Symposium on Frost Action in Roads*, Vol. II, OECD (1973).

207. GILMAN, G., "Distribution of Mean Annual Air Temperatures and Freezing and Thawing Indexes in North America." Tech. Note, CRREL (1968) (Unpubl.)

APPENDIX A

SUMMARY OF INFORMATION FROM QUESTIONNAIRES RETURNED BY 62 AGENCIES

Freezing Index, Frost Depth and Frost Susceptibility Criteria*

State, province or agency	Approximate range in design freezing index†	Portion of state or province in which frost considered in design	Does variation in freezing index or depth of frost penetration directly affect thickness of pavement structure?	Criteria or tests used to identify frost susceptible subgrade soils	Criteria or tests used to restrict frost susceptibility of unbound base/subbase materials	Depth to which frost susceptible subgrades are undercut below normal subgrade and material with which replaced
Alabama	0-100	None. Max. measured frost depth is 9 inches.				
Alaska	1000-9000	All	Only in that designs for seasonal frost and permafrost differ, and that freezing index affects design of insulated sections.	GSD, after C of E criteria.	GSD and plasticity.	NI
Arizona (NI)	50-750					
Arkansas	50-300	None				
British Columbia	100-6000	All	No	NI	GSD	NI
California	0-1000	Severe climate areas.	No	Soils with > 5% passing No. 200 sieve are considered FS.	None	NI
Connecticut	500-1000	All	Yes, total thickness 2/3 to 3/4 of frost depth for coldest year in 5 years.	GSD, Casagrande criteria.	GSD, Casagrande criteria.	Undercut to admit total thickness pavement and subbase - 2/3-3/4 frost depth. Replace with gravel, 0-10% passing No. 200.
Delaware	100-250	None				
District of Columbia	300	None	No	Soil classification.	GSD and Atterberg limits.	None
Florida	0	None				
Georgia	0-100	None				
Hawaii	0	None				
Illinois	300-1400	All	Yes, depth of frost can have limited effect on thickness of subbase.	All A-4 soils and other soils with silt content above 70% are FS.	GSD and NaSO, soundness.	If free water present, FS soil within frost depth is removed, or subbase is thickened.
Indiana	400-1000	No specific areas.	No	None	None	None
Iowa	1000-2000	All	No	GSD, may be FS if > 15% passing No. 200.	None	FS soil may be removed if within 5 ft of subgrade and GW present. 2-3 ft select glacial till placed in most cuts.
Kansas	400-800	All	No	Silty soils are considered FS.	GSD, ≤ 15% passing No. 200.	None
Kentucky	200-500	None				
Louisiana	0-50	None				
Manitoba (NI)						
Massachusetts	500-1300	All	NI	Soils with > 12% passing No. 200 are removed and replaced.	GSD, ≤ 10% passing No. 200.	Generally two feet, replace with special borrow containing ≤ 10% passing No. 200.
Michigan	800-2400	All	No, but standard sections account for frost by including thick granular subbase.	Visual identification.	GSD, loss by washing ≤ 7%.	Undercutting determined by soils engineer for each cut. Replaced with granular backfill.
Minnesota	2000-4000	All	May affect depth of undercutting of FS subgrades.	All fine-grained soils considered FS.	GSD, ≤ 10% passing No. 200.	Where differential heaving probable, remove all FS material to depth of frost, soils engineer selects replacement material.

Freezing Index, Frost Depth and Frost Susceptibility Criteria*

State, province or agency	Approximate range in design freezing index†	Portion of state or province in which frost considered in design	Does variation in freezing index or depth of frost penetration directly affect thickness of pavement structure?	Criteria or tests used to identify frost susceptible subgrade soils	Criteria or tests used to restrict frost susceptibility of unbound base/subbase materials	Depth to which frost susceptible subgrades are undercut below normal subgrade and material with which replaced
Mississippi	0-100	None				
Missouri	250-1000	None				
Montana	1000-3500	All	Yes, in severe frost areas have increased the thickness by 20%.	GSD, after C or E criteria.	Limited PI and percentage passing No. 200.	NI
Nevada	50-1500	Northern half of state.	Yes, total thickness ≥ 1/2 estimated frost depth.	Careful study made of material of high silt content.	None	As necessary to make total thickness ≥ estimated frost depth.
New Brunswick (NI)						
Newfoundland (NI)						
New Jersey (NI)						
New Mexico	50-1500	Mountainous portion.	Only insofar as it affects the regional factor.	None	None, other than aggregate soundness tests.	None
North Carolina (NI)						
North Dakota	2500-4000	All	NI	None	None	NI
Nova Scotia	800-1500	All	No	None	GSD and plasticity.	NI
Ohio	300-1000	All	No, but FS soil not permitted in top 3 ft of embankments.	A-4 soil with > 50% silt and and PI < 10 especially FS.	GSD, ≤ 15% passing No. 200.	Special treatment of FS soils in cut sections.
Oklahoma	100-400	None				
Ontario	800-6000	All	No, but depth of frost is measured and used as an aid in design.	For fills acceptable NFS soils have < 40% finer than .05 mm and < 45% finer than .1 mm.	0-8% passing No. 200 (sometimes 0-10 or 0-12) and PI = 0.	Remove A and B horizons at transitions cut-fill; in-place shattering of rock cuts at transitions to rock fill.
Oregon	50-1500	Cascade mountains and eastward.	Yes, total thickness of NFS materials ≥ half the maximum frost penetration.	Soils with > 8% passing No. 200 are FS.	Subbase < 8% passing No. 200, sand equivalent > 25; Base: sand equivalent ≥ 30, LL ≤ 33, PI ≤ 6.	Sometimes undercut FS soil and where water table high; also increase subbase, for total thickness > standard 1/2 frost depth.
Pennsylvania	300-1000	All	Yes, total thickness of NFS materials ≥ 1/3 to 3/4 the frost penetration in coldest year in 10.	Percentage finer than .02 mm.	GSD	Usually raise grade, but sometimes undercut FS soils.
Prince Edward Island	1500-1800	None	NI	None	None	None
Quebec	2000-7000	All	NI	Soils with > 10% passing No. 200 and > 3% passing No. 270.	GSD	NI
Rhode Island (NI)						
South Carolina	0-50	None	No	None	None	None
South Dakota (NI)						
Tennessee	100-250	None	No	None	None	None
Texas	0-300	None	No	None	Atterberg limits.	None
Utah	200-1500	None, except use regional factors.	No	None	None, except exclude compressible materials.	Undercut only to remove compressible soils.

Freezing Index, Frost Depth and Frost Susceptibility Criteria*

State, province or agency	Approximate range in design freezing index†	Portion of state or province in which frost considered in design	Does variation in freezing index or depth of frost penetration directly affect thickness of pavement structure?	Criteria or tests used to identify frost susceptible subgrade soils	Criteria or tests used to restrict frost susceptibility of unbound base/subbase materials	Depth to which frost susceptible subgrades are undercut below normal subgrade and material with which replaced
Vermont	1300-2200	All	NI	A-4 and finer soils considered FS.	GSD	Undercut wet FS soil 1-2 feet, depending on gradation and moisture.
Virginia	50-400	None				
Washington	50-1000	All	Yes, use total thickness ≥ 1/2 recorded frost depth.	Soils with > 10% passing No. 200 are FS.	GSD, ≤ 10% passing No. 200.	In frost zones, undercut to provide total thickness ≥ 1/2 frost depth.
West Virginia	250-600	All	NI	Based on Unified Soil Classification System.	GSD, classification, abrasion, soundness.	None, pavement is designed for frost if top 12 in. of subgrade is FS.
Wyoming	900-2500	Where soils are alluvium, and mountain soils.	NI	Soil classification.	GSD and plasticity index.	Depends on moisture and soil conditions.
National Research Council of Canada	(Not engaged in design and maintenance of roads, commented only on research needs.)					
USDA-Forest Service	0-6000	Wherever frost action is significant on roads in our 154 U.S. National Forests.	In some areas, yes.	GSD, Atterberg limits. Sometimes use C of E criteria.	GSD, Atterberg limits. Sometimes use C of E criteria.	If frost is a problem either remove or blend with NFS material, to achieve uniformity.
USDI-Bureau of Reclamation	(We design our road relocations in conformance with design requirements of states in which roads are located.)					
USDD-Department of the Navy	(We follow reasonably closely the practices of the C of E, and of the highway departments of the states wherein the proposed road will be located.)					
USDT-Federal Highway Admin., Region 15	NI	Scenic and forest roads in northeastern U.S., all roads are secondary class and commercial traffic is banned.	NI	GSD, after C of E criteria.	NI	To estimated depth of frozen zone, also in frost areas use 6-12 in. select soil, A-2 (4) or better, beneath base course.
Massachusetts Turnpike Authority	500-1100	All of Mass. Turnpike.	Entire road designed for frost.	GSD, after C of E criteria.	GSD, ≤ 10% passing No. 200.	As necessary for total thickness of pavement and base equal to 40 in.
New York State Thruway Authority	500-1000	(No longer engaged in design or construction, maintenance only.)				
Asphalt Institute			No	None	≤ 7% passing No. 200 recommended for high quality granular base.	For control of differential frost heave, abrupt changes in subgrade conditions must be avoided, by cross-hauling and intermixing, and by removing and replacing or reworking highly FS soils in localized areas.
Portland Cement Association			No	None	GSD, after AASHO M147.	For control of differential frost heave, uniform subgrade support essential; best achieved by grading, blending, cross-hauling etc. and by removal of FS silt pockets and replacement with surrounding soil.

* All states, provinces and agencies that were canvassed by questionnaire and were not visited are listed here.

† Ref. (207).

Flexible Pavement Design*

State, province or agency	Subgrade support value: If so, how is it characterized?	Subgrade support value: Does it attempt to represent spring conditions?	Traffic analysis period, years	AADT and axle loads	Terminal serviceability index or equivalent	Type of base or subbase, e.g. whether bound or unbound	Environmental or regional factor	Have you used full-depth asphaltic concrete pavements?
	Does design thickness vary with the following?							
Alaska	Yes. FS classification.	Yes	No. 20 in all cases.	No, only on DHV.	No, not used.	NI	No	No
British Columbia	Yes, by measured deflection (CGRA procedure).	Yes, deflection measured in spring.	No	No (CGRA procedure).	Yes	Yes	No	Yes, used occasionally.
California	Yes. R-value.	No, but R-value measured on saturated specimens.	No, normally 20 yrs.	Yes, traffic index.	No, not used.	Yes	No	No
Connecticut	Yes. Subbase thickness depends on soil classification and frost depth.	No	No, 20 in all cases.	Yes, on ADT and DHV.	No	No. Standard sections.	No	No
Illinois	Yes, CBR.	Specimens are soaked 4 days before testing.	Yes, 15-20 yrs.	Yes, on ADT of 18-kip axle loads.	Yes	Yes, layer coefficients.	Yes, time/traffic exposure factor.	Yes, on a few secondary roads.
Indiana	Yes, CBR.	Yes, specimens soaked 4 days.	No, 20 years.	Yes, 18-kip axle loads.	Yes	Yes, layer coefficients.	No	Yes
Iowa	Yes, soil classification.	No	Yes, 10-20 years.	Yes	Yes	Yes, layer coefficients. Bases ATB or CTB. Subbases cement- or lime-treated.	Yes	Yes, all primary and most secondary roads.
Kansas	Yes, triaxial modulus.	Not explicity, but samples tested at saturation.	Yes, 10-20 years.	Yes	No	Yes, only ATB used. Equivalencies based on triaxial moduli.	Only with respect to rainfall regime.	Yes, all our flexible pavements are full-depth.
Massachusetts	Yes, DBR (similar to CBR).	No	Yes, 10-20 years.	Yes, on ADT of 18-kip axle loads.	Yes	Yes, layer coefficients.	Yes, regional factor.	No
Michigan	No, but development of standard sections was based on estimated CBR values.		No, 20 years.	Yes, ADT.	Used to develop standard sections.	Aggregate base and ATB are used.	Regional factor ranges from 3.0-4.5.	Only recently for widening and intersection betterments.
Minnesota	Yes, R-value and soil classification.	Yes, because mathematical model used for design is based on spring BB deflections.	No, 35 in all cases.	Yes, ADT and CADT.	No	Yes, gravel equivalents.	No	Yes, on all classes of roads.
Montana	Yes, R-value.	No	20 years.	Yes, ADT of 18-kip axle loads.	Yes	Yes, layer coefficients.	Yes, regional factor.	Yes
Nevada	Yes, R-value.	NI	No, 20 for all roads.	Yes, ADT.	Yes	Yes, layer coefficients.	Yes, regional factor.	No
New Mexico	Yes, R-value.	No	No, 20 in all cases.	Yes	Yes	Yes, layer coefficients.	Yes, reg. factor 0.2-3.5.	No
North Dakota	Yes, support value and classification.	NI	20 years.	NI	Yes	NI	No	No
Nova Scotia	Yes, CBR.	No	Yes, 17-20 years.	Yes	Yes	Yes, gravel equivalents.	No	No
Ohio	Yes, soil classification and, occasionally, CBR.	Yes	No, 20 in all cases.	Yes, ADT of 18-kip loads.	No, 2.5 in all cases.	Yes, layer coefficients.	No, reg. factor 1.1 in all cases.	Yes, on intermediate and and secondary roads.

Flexible Pavement Design*

State, province or agency	Subgrade support value: If so, how is it characterized?	Subgrade support value: Does it attempt to represent spring conditions?	Does design thickness vary with the following? Traffic analysis period, years	AADT and axle loads	Terminal serviceability index or equivalent	Type of base or subbase, e.g. whether bound or unbound	Environmental or regional factor	Have you used full-depth asphaltic concrete pavements?
Ontario	Yes, textural soil classification.	Yes because design equation is based on limiting the spring deflections to certain performance-proven levels.	Yes, estimated service life.	Yes, ADT of 18-kip axle loads.	Yes	Use concept of gravel equivalents, but most bases and subbases are unbound.	No	Yes, heavy duty roads only.
Oregon	Yes, R-value.	Yes, R-value measured at saturation or minimum in-situ water content, and minimum specified compaction.	No, 20 in all cases.	Yes, traffic coefficient.	No, 2.5 in all cases.	Yes, equivalencies in terms of crushed base aggregate.	No	No
Pennsylvania	Yes, CBR.	Specimens are soaked 4 days before testing.	No, 20 in all cases.	Yes, ADT of 18-kip axle loads.	No, 2.5 in all cases.	Yes, layer coefficients.	Yes, but 1.5 in most cases.	Only several demonstration sections on secondary roads.
Quebec	Yes, CBR and soil classification.	Yes, specimens are soaked.	No, 20 in all cases.	No	No	NI	No	No
Vermont	No		No	No	No	No	No	No
Washington	Yes, R-value.	No, but test specimens on wet side of optimum.	No, 10 in all cases.	Yes, ADT of 5-kip wheel loads.	No, not used.	Yes, equivalency factors.	No	Yes
West Virginia	Yes, R-value.	Yes, for soils with ~30% silt content use exudation R-value at 240 psi.	No, 10 in all cases.	Yes, ADT of 5-kip wheel loads.	No, not used.	Yes, gravel equivalencies.	No	No
Wyoming	Yes, R-value	No, use R-value at standard exudation of 300 psi.	No, 20 in all cases.	Yes, ADT of 18-kip axle loads.	Yes	Yes, layer coefficients.	Yes	Yes, on all classes of roads.
USDA-Forest Service	Yes, many different methods depending on geographic area.	For all-season roads yes, for summer roads or with spring load restrictions, no.	Varies.	Yes, ADT on our roads generally < 400.	When AASHO method used, yes.	Yes, layer coefficients.	Yes	Yes, on roads carrying < 400 ADT in Northeastern U.S.A.
USDT-Federal Highway Admin., Region 15	Yes, R-value.	Yes, test specimens are saturated.	No, 20 in all cases.	Yes, typical ADT of equiv. 18-kip axle loads: 10.	No, 2.0 in all cases.	Yes, layer coefficients.	Yes	One designed, but not yet constructed.
Massachusetts Turnpike Authority	CBR	No	20 years.	18-kip ADT - 1100.	2.5	Layer coefficients.	Seasonality factor.	No
Asphalt Institute	Yes, CBR, R-value or bearing value from plate bearing test.	Yes, through use of soaked CBR and experience in selecting proper value. Pavements given by our design will perform satisfactorily in spring and will have excess carrying capacity in other seasons.	Yes	Yes, ADT of equiv. 18-kip axle loads.	No, 2.5 in all cases.	Yes, substitution ratios.	No, but special local experience may justify special designs.	Yes, full-depth AC is our standard design for all classes of pavements; substitution ratios used for bases other than AC.
Portland Cement Association								

* All states, provinces and agencies that reported frost is considered in roadway design, and that were not visited, are listed here.

Rigid Pavement Design*

State, province or agency	Does design thickness of PCC slab vary with the following? — Modulus of subgrade reaction (k): If so, how is it determined?	Does it attempt to represent spring conditions?	Does its value vary depending on thickness and type of subbase?	Traffic analysis period, years	AADT and axle loads	Terminal serviceability index or equivalent	Subbase: What types are used?	Is subbase intended to Control pumping?	Is subbase intended to Reduce frost heave?	How is thickness determined?
Alaska	Rigid pavements not used.									
British Columbia	No. Standard design.			No	No	No	CTB	Yes	Yes	Varies with subgrade strength.
California	Yes, estimated from measured R-value.	No	Yes, on thickness	No, 20 years	Yes	No	CTB	Yes	No	0.4 ft. minimum.
Connecticut	No. Design based on experience.			No, 20 years	Yes, or DHV.	No	Granular	Yes	Yes	In relation to frost depth.
Illinois	Yes, CBR.	Specimens are soaked 4 days before testing	No, based only on subgrade, but credit given for subbase in design charts.	No, 20 years.	Yes, on ADT of 18-kip axle loads.	Yes	Granular, ATB, CTB, lime-flyash.	Yes	No	Standard 4-in. thickness used.
Indiana	Yes	NI	No, based only on subgrade.	No, 20 years	Yes, 18-kip axle loads.	Yes	6-inch granular or 4-inch stabilized.	Yes	No	Standard.
Iowa	Yes	No	Yes, increased with subbase.	Yes, 10-20 years.	Yes	No	ATB or CTB.	Yes	No	Standard 4-inch.
Kansas	Yes, from triaxial tests.	Test samples are saturated.	No	No, 20 years.	No, design allows for infinite 18-kip loadings.	No	Granular	Yes	No	Standard 4-inch.
Massachusetts	Rigid pavements not generally used.									
Michigan	No. Standard sections	Development of standard sections was based on estimated spring moduli.	For standard sections assumed modulus increases with thickness of granular subbase.	No, 20 years.	Yes, ADT.	Used to develop standard sections	Granular	Yes	No	Standard thicknesses.
Minnesota	Yes, standard sections for K = 150 and 300 pci for which choice is based on soil classification.	Yes, estimated subgrade moduli of 150 or 300 refer to spring conditions.	No, value estimated for subgrade alone.	No, 35 years.	Yes, CADT.	No	Granular	Yes	No	Standard thickness 3 to 6 inches.
Montana	Estimated from correlation with R-value.	No	Yes, on thickness.	20 years	Yes, ADT of 18-kip axle loads.	Yes	CTB	NI	NI	Standard thickness, usually 4 in.
Nevada	Yes, estimated from correlation with R-value.	No	Yes, on thickness.	No, 20 years	Yes, ADT	No 2.5 in all cases.	CTB above, gravel below.	5-in. CTB is for pumping.	Gravel is for frost heave.	Depends on frost depth.
New Mexico	Yes	No, PCC not used in severe frost areas.	No	No, 20 years.	No	NI	CTB	NI	NI	Standard thickness 4-in.
North Dakota	NI	No	No	20 years.	NI	Yes	Leveling course only.	No	No	As a leveling course.
Nova Scotia	Rigid pavements not used									
Ohio	Yes	Yes	Yes, on thickness and strength.	No, 20 in all cases.	Yes, ADT of 18-kip loads.	No, 2.5 in all cases.	Granular	Yes	Yes	6 in. if for pumping. 12-18 in. over FS silts.
Ontario	No. Standard design			No	No	No	CTB	Yes	No	Standard 6-inch.
Oregon	No, standard 8-inch thickness of CRC.		Combined slab and and subbase thickness ≥ 1/2 frost depth.	No	No	No	Granular	No, pumping not a problem with CRC.	Yes	For combined thickness ≥ 1/2 frost depth.
Pennsylvania	Yes, use CBR from laboratory tests.	Yes	Yes, on thickness exceeding prescribed 6 and 8-inch minimum thicknesses.	No, 20 in all cases.	Yes, ADT of 18-kip loads.	No, 2.5 in all cases.	Granular	Yes	Yes	For combined thickness 1/3-1/2 frost depth.
Quebec	No			No, 35 in all cases.	No	No	Soil cement	Yes	Yes	From soil classification.
Vermont	Rigid pavements not used.									
Washington	Yes	No	Yes	No, 25 in all cases.	Yes, ADT of 5-kip wheel loads.	Not, not used.	Granular, ATB, CTB.	Yes	Yes	For frost penetration.
West Virginia	No, standard design.			No	No	No	Granular	Yes	Yes	Standard 6-inch.
Wyoming	Yes, estimated from correlation with R-value.	No, R-value taken at standard 300 psi exudation pressure.	Yes	No, 20 in all cases.	Yes, ADT of 18-kip axle loads.	No	Granular or lightly treated with cement or lime.	Yes	No	Standard 4 to 6-inch.
USDA-Forest Service	Rigid pavements not used.									
USDT-Federal Highway Admin., Region 15	Rigid pavements not used.									
Massachusetts Turnpike Authority	Rigid pavement not used on Massachusetts Turnpike.									
Portland Cement Association	Yes, determine k-value by plate bearing tests or by correlation with other tests.	No, fatigue analyses show that reduced spring support offset by high winter values.	Yes, increases with increasing thickness, and much higher with CTB.	Yes, can be varied, but normally 40 yrs.	Yes, fatigue under each load group analyzed separately.	Not used.	Granular and CTB.	Yes	No, but subbase also provides some frost protection.	4 to 6 inches is recommended.

* All states, provinces and agencies that reported frost considered in roadway design, and that were not visited, are listed here.

Maintenance and Planning

State, province or agency	Do you impose special restrictions on maximum axle loadings allowed in the spring?	Does surface roughness caused by frost heave adversely affect the serviceability of your pavements?	Low-temperature contraction cracks in asphaltic pavements: Do they adversely affect serviceability of your pavements?	Do you seal the cracks, and if so, when?	What precautions are taken by your designers to minimize such cracking?	Culvert blockage by ice: Does it occur on your roads?	How is drainage restored?	Approximately what part of maintenance budget is for snow removal and ice control?
Alabama								
Alaska	Yes, on older, under-designed roads.	Yes	Yes	Yes, early spring.	We use 120-150 pen. asphalt. Research needed. Our gravel roads also crack.	Yes	Electrical cables and steam thawing.	50%
Alberta	Yes, on certain roads.	Not frequently or severely.	Yes	Yes, early spring.	Use high viscosity, high pen. asphalt cements, and SC road oils.	Yes	Steam thawing.	20-25%
British Columbia	Yes, to keep spring deflection < .05 in.	Yes	Yes	Yes	Select asphalt to have stiffness $\leq$ 30,000 psi at lowest temperature expected.	Yes	Steam thawing.	15%
California	No	Problem areas are minimal.	Yes, brittleness has adverse effect.	Yes, in fall.	Use softer asphalt (120-150 pen.) at higher elevations.	No		9%
Colorado	No	Yes	Yes	Yes, in warm weather.	Use softer grades at request of District Eng.	Yes	Calcium chloride	20%
Connecticut	No	No	No	Yes, in summer.	None	No		26%
Delaware	No	No	Opinion divided.	Yes, in winter.	None	Yes	Hand or mech. clearing.	5%
Georgia	No	No	Yes	No	None	No		
Idaho	Yes, certain roads, by judgment.	Yes	Yes	Yes, in summer.	Use softer asphalts (120-150 and 200-300 pen).	Yes	Thaw with weed burners.	20-25%
Illinois	Yes, by local agencies, for secondary roads.	Not generally, but in some cases yes.	Yes	Yes, in fall.	None	No		10%
Indiana	No	No	Yes	Yes, when they occur.	NI	Yes	Calcium chloride	20-30%
Iowa	No	Yes, on pre-1940 pavements.	No	Yes, when crack > ¼ inch.	None	Yes	Steam thawing.	32%
Kansas	No	Only where pavement insufficient.	Yes, severe problem.	Yes, warm winter days, late fall or early spring.	Using mostly 85-100 and 100-120 pen.	Seldom		9%
Maine	Yes, on inadequate roads > 20 years old.	Yes	Yes, severe problem.	Now no, previously yes.	Using softer asphalts, Ontario's method for critical mix.	Yes	Steam and hot water.	40%
Maryland	No	Yes	No		None	Yes, limited degree.	No action needed.	24%
Massachusetts	No	Yes	Yes	Yes, in fall.	None	No		35%
Michigan	Yes, on older roads.	Yes	Yes	Yes, in late fall.	Specifying both penetration and minimum limit for viscosity, for control of susceptibility.	Yes	Salt	35%

Maintenance and Planning

State, province or agency	Do you impose special restrictions on maximum axle loadings allowed in the spring?	Does surface roughness caused by frost heave adversely affect the serviceability of your pavements?	Low-temperature contraction cracks in asphaltic pavements: Do they adversely affect serviceability of your pavements?	Do you seal the cracks, and if so, when?	What precautions are taken by your designers to minimize such cracking?	Culvert blockage by ice: Does it occur on your roads?	How is drainage restored?	Approximately what part of maintenance budget is for snow removal and ice control?
Minnesota	Yes, except not on heavy duty roads.	Yes. Repeated heaving causes cracking.	Yes	Yes, spring, fall and winter.	No criteria established.	Yes	Steam	25-35%
Missouri	No	No	No	Yes, at temperature $\leqq 40°$	None	No		10%
Montana	Yes, in some cases.	Yes	Yes	Yes, in good weather.	Use 120-150 pen. asphalt in rural areas.	Yes	Steam	20%
Nebraska	Yes, on incompleted stage constructed roads.	Not generally, but only in frost boil locations.	Yes, but not severe.	Yes; spring, fall or winter.	None	Rarely		9%
Nevada	No	No, it is a minor problem.	Yes	Yes, in spring.	Use high pen. asphalt; rutting is a problem.	NI	NI	NI
New Hampshire	Yes, only on feeder roads.	Yes	Yes	Yes, in winter.	Use low filler and high bitumen content. Use high float emulsions in resurfacing.	Yes	Steam	50%
New York	No	Yes, differential heave is detrimental.	Yes, but not severe.	No	None, but experimenting with high float emulsion.	Yes	No action taken.	33%
New Mexico	No	Yes	Yes	Yes	Generally use 85-100 pen., but one district uses 120-150.	No		NI
North Dakota	Yes, except Interstates and some primary highways.	Yes	Yes	Yes, in spring.	None, except use of bitumen adapted to state.	Yes	Propane torches	12%
Nova Scotia	Yes, on secondaries, 75% · of normal loads.	No	No		NI	Yes	No action needed.	20%
Ohio	No	No	No		Use 70-80 and 85-100 pen. asphalts.	No		20%
Ontario	Yes, on some weaker roads.	Yes	Yes	Sometimes, in summer.	Limit penetration index ($\geqq -1.0$) by specifying moderately high viscosity; also use softer grades, to penetration 300-400.	Yes	Steam	50%
Oregon	No	Yes, on roads with insufficient base and subbase.	No, provided smoothness is satisfactory.	Yes, in cold fall periods.	Adjust aggregate grading to allow thick films of high viscosity bitumen.	No		30%
Pennsylvania	No	No	No		None	Yes	NI	40%
Quebec	Yes	Yes, in some cases.	Yes	Sometimes, in summer.	None, but problem under study.	Yes	Steam	40-50%

Maintenance and Planning

State, province or agency	Do you impose special restrictions on maximum axle loadings allowed in the spring?	Does surface roughness caused by frost heave adversely affect the serviceability of your pavements?	Low-temperature contraction cracks in asphaltic pavements: Do they adversely affect serviceability of your pavements?	Do you seal the cracks, and if so, when?	What precautions are taken by your designers to minimize such cracking?	Culvert blockage by ice: Does it occur on your roads?	How is drainage restored?	Approximately what part of maintenance budget is for snow removal and ice control?
South Carolina	No	No	Yes	Yes, in winter.	None	No		Insignificant
Saskatchewan	No	Yes	Yes	Yes, in spring every 3 years.	Use high viscosity asphalt cement at 200 ± pen.; also use SC 3000 road oil in hot mix.	Yes	Steam and electrical heating.	21%
Tennessee	No	No	Yes	Yes, in fall.	None	No		5%
Texas	No	No	Yes	Yes	None	No		<1%
Utah	Yes	Yes	Yes	Yes, spring and summer.	Recently changed from pen. to viscosity grading.	Yes	No action taken.	27%
Vermont	No	Yes	Yes	Yes, starting this year.	Experimenting with softer grades of asphalt.	Yes	Hot water and steam.	NI
Washington	No	To a degree.	No, but cracks do occur.	No	None; only recently recognized problem.	Seldom		20%
West Virginia	No	Yes	Yes	Yes, in summer.	None	No		5-7%
Wisconsin	Yes, on older and inadequate roads.	Yes	Yes	NI	None	Yes	Steam	30%
Wyoming	Only occasionally, based on local experience.	Yes, in some areas.	No, but cracks do occur.	Yes	Have used a 200-300 pen. cushion under overlays.	Yes	Weed burners.	30%
USDA-Forest Service	Yes, on certain roads not designed for unrestricted use.	No, our roads are low speed, low standard, low volume roads.	No, but cracks do occur.	Yes, in summer.	None	Yes	No action taken.	Negligible (many roads are closed in winter).
USDT-Federal Highway Admin., Region 15	No	(Our agency performs no maintenance.)						
Massachusetts Turnpike Authority	No	Yes	Yes	Yes, spring or fall.	None	No		NI
New York State Thruway Authority	No	Yes	Yes (reflection cracks).	Yes, in fall.	NI	Yes	No action needed.	20%
Asphalt Institute		Not on adequately designed roads.	Adverse effects have been found in some areas of Canada.		None, but research is underway on this topic.			
Portland Cement Association	No, not required for properly designed rigid pavements.	Yes, but seasonal roughness does not necessarily decrease progressively the serviceability.						

Research on Roadway Design in Seasonal Frost Areas

State, province or agency	*Topics of recent or current work by your organization*	*What are the principal unresolved problems affecting roadway design in seasonal frost areas?*	*Which unresolved problems are the most promising areas for research?*
Alabama	Measured depth of frost penetration under Alabama roads.	NI	NI
Alaska	FS of base courses, culvert icings, vegetation for cut slopes, insulation.	Thermal cracking, heave predictions, thickness design.	Thermal cracking, frost heave and heave pressures.
Alberta	Seasonal deflections, snow and ice symposium.	Thermal cracking, freeze-thaw deterioration, seasonal strength changes.	Thermal cracking.
British Columbia	Regression analysis of deflection data.	Relationship of strength to soil type, topography, drainage, freezing index etc.	Field studies of surface strength.
California	Plan to incorporate a climatic or geographic variable in design method.	Frost not a severe problem in California.	NI
Colorado	None	FS classification, insulation transitions.	NI
Connecticut	Sponsoring development of rapid test for identification of FS soil.	Greater sub-drainage problem on multi-lane pavements.	Allowable retention time for moisture under pavements.
Delaware	None	NI	NI
Georgia	None	NI	NI
Idaho	Measurement of frost heaves, insulation, Benkelman beam deflection.	Heave control drainage, effect of thaw weakening on design method.	Heave control and thaw weakening.
Illinois	Freeze-thaw effects on durability of stabilized materials and on soil resiliency, moisture movements within pavement systems.	Durability in relation to climate, thaw weakening, differential frost heave.	Thaw weakening.
Indiana	Polystyrene insulation.	Identification and mapping of FS soils.	Identification and mapping of FS soils.
Iowa	None	None, design seems adequate.	NI
Kansas	NI	Insufficient use of available knowledge.	Better dissemination of information.
Maine	Aggregates, frost action, insulation, pavement life, studded tire wear.	Difficulty in achieving uniformity of materials, life of bituminous pavements.	Durability and skid resistance of pavements.
Maryland	None	Frost is not a major problem in Maryland.	NI
Massachusetts	Rapid test of FS of soil.	Thermal cracking, and longitudinal construction joints in flexible pavements.	Need for better longitudinal joints.
Michigan	Stability, drainability and environmental factors in granular materials.	Drainability vs. saturation, and stability vs saturation, in granular materials.	Study of granular materials.
Minnesota	Insulation of centerline culverts.	Advance identification of areas that will heave.	Reduce frost effects by subgrade insulation.
Missouri	None	NI	NI
Montana	Study of pavement cracking.	NI	NI
Nebraska	Dynaflect in overlay design, regional factors.	NI	NI
Nevada	None	NI	NI
New Hampshire	Development of rapid frost heaving test.	Positive identification of areas that will heave.	Insulation to prevent subgrade freezing without surface conditions; plastic moisture barriers.
New York	Pavement design, insulated pavements including surface icing.	Simple criteria for FS of soils; additives to reduce FS; simple tests for thaw weakening.	All those listed under the foregoing heading.
New Mexico	BB deflection tests	NI	NI
North Dakota	BB and Dynaflect tests to study rate of recovery after spring thaw weakening.	Rate of recovery of normal subgrade strength.	Use of Dynaflect to determine recovery period.
Nova Scotia	BB deflection studies.	Effect of frost action on subgrade strength.	NI
Ohio	None	NI	NI

Research on Roadway Design in Seasonal Frost Areas

State, province or agency	*Topics of recent or current work by your organization*	*What are the principal unresolved problems affecting roadway design in seasonal frost areas?*	*Which unresolved problems are the most promising areas for research?*
Ontario	Thermal cracking; Brampton test roads for field evaluation of various bases and pavement systems; seasonal strength variations; design systems.	Drainage of surface water in winter, quality control on construction, designing for uniform heaving.	Keeping moisture away from the pavement structure.
Oregon	None	(Pavements built by our current design standards have not been a problem with respect to frost.)	NI
Pennsylvania	Frost action in soils (at M.I.T.), pavement evaluation.	Finding economical solutions to frost problem.	NI
Quebec	Seasonal variations in BB deflections.	Pavement failure caused by winter rains and thaws.	Permeability effects in strength recovery.
South Carolina	None	NI	NI
Saskatchewan	BB deflection; thermal cracking; effects of swelling clay on serviceability.	Thermal cracking, effects of swelling clay on serviceability.	Use of soft grades of asphalt of high viscosity to control thermal cracking.
Tennessee	None	NI	NI
Texas	Loss of support in the spring.	Distress due to frequent freeze-thaw cycles.	NI
Utah	Loss of support in the spring.	NI	NI
Vermont	Loss of support in the spring.	NI	NI
Washington	None	Inadequate drainage.	Inadequate drainage.
West Virginia	Relationship among temperature, frost penetration, and pavement performance in W. Va.	NI	NI
Wisconsin	Seasonal BB deflections; freezing index to predict deflections.	Discovery of silt pockets contained in sand subgrade strata; frost tenting (heave at cracks).	Those listed under the foregoing heading.
Wyoming	Plan to study deflections and spring load restrictions.	Recognition, treatment and design for frost.	NI
National Research Council of Canada	Thermal and structural behavior of insulated roads.	Predicting depth of frost, and FS criteria.	Correlating GSD, freezing tests, and field behavior.
USDA-Forest Service	None	Identification of FS materials, frequent and cyclic freeze-thaw, need for rational design.	Development of simple rational frost design procedure.
USDT-Federal Highway Admin., Region 15	None	Determination of a realistic soil support value.	Determination of a realistic soil support value.
Massachusetts Turnpike Authority	None	Frost action at culverts, and in shade of overpass structures.	NI
New York State Thruway Authority	None	Cut-fill transitions, high culverts, shoulders, shadow areas.	Shoulders
Asphalt Institute	Low-temperature contraction cracking.	Heave predictions in relation to soil, temperature, and GW conditions; defining what differential heave constitutes failure; development of yearly cumulative damage model to account for frozen and thawed conditions.	Yearly cumulative damage model.*
Portland Cement Association	Limited research on insulating layers.	NI	NI

* **Suggested approach:** (1) Use multilayered elastic stress-strain analysis. (2) Characterize each material by resilient modulus, determined in frozen state and at various water contents in unfrozen state. (3) Combine multilayered analysis with model for frost penetration under various thermal regimes. (4) Compute critical strains for various time intervals throughout yearly period to check cumulative damage model.

APPENDIX B

SELECTED INFORMATION ON ROADWAY DESIGN PRACTICES IN SEASONAL FROST AREAS (11 AGENCIES)

The information contained in this summary was obtained from questionnaires and visits to 10 states and provinces, and from practices of the U. S. Army Corps of Engineers.

Abbreviations Used

BB	Benkelman beam
C of E	U.S. Army Corps of Engineers
AADT	Annual average daily traffic
CADT	ADT of commercial vehicles
DHV	Design hourly volume (traffic)
DFI	Design freezing index; corresponds to a cold year with recurrence interval of 10 years
FS	Frost susceptible, frost susceptibility
NFS	Nonfrost susceptible
FS group	Frost susceptibility classification of C of E, F-1, F-2, F-3, or F-4
GSD	Grain size distribution
GW	Groundwater
DGA	Dense-graded aggregate (base)
AC	Asphaltic concrete
ATB	Asphalt-treated base
CTB	Cement-treated base
PCC	Portland cement concrete (unreinforced)
RC	Reinforced concrete
CRC	Continuously reinforced concrete
PCA	Portland Cement Association
USCS	Unified Soil Classification System
Blank space	No information

State or province	Range in DFI *	Portion of state or province in which frost considered in design	Does variation in freezing temperature regime or freezing index directly affect thickness of pavement structure?	Soil classification system in use for subgrade soils	Classification, criteria or tests used to characterize frost susceptibility of subgrade soils	to restrict frost susceptibility of unbound base/subbase material	percentage passing No. 200 sieve in unbound subbase and base materials of various classes	Depth to which FS subgrades are undercut below normal subgrade and material with which replaced
Alberta	2000-6000	All	No	USCS	GSD, after C of E, for soil with PI < 12. Clays with PI > 25- "low" FS, PI 12-25 "medium".	< 10% passing #200 sieve, max. PI 5-6%.	2-10%	All cuts undercut 1 foot, if FS, 2-3 feet. Replace with less FS material, striving for uniformity of subgrade.
Colorado	500-2500	Mountain areas above 9000 ft MSL	No. It affects only indirectly the regional factor.	AASHO system and Group Index	None		Usually 5-12%	Undercutting to remove FS materials not usually practiced.
Idaho	1000-2500	All	Yes. Freezing index and precipitation were used to determine climatic factors	AASHO, but now changing to USCS.	None	Sand equivalent ≥ 30 if more than 5% passing No. 200 sieve	3-10%	Undercutting entire cut to remove FS materials not practiced; but all cuts are grade-pointed, i.e. at transitions from cut to fill undercut 12" ± over distance of 100 ft, and replace with base material.
Maryland	250-500	In northern and western parts of state we use regional factors greater than 1.0.	Yes, it was used to determine the range of regional factors.	Similar to AASHO, modified by expansion of A-5 group, and by including C of E FS groups.	GSD, based on an interpretation and correlation of AASHO classification with C of E FS classification.	Only crushed and processed materials are used.	Usually 0-12%	Depends on GW conditions. Replace with "granular cap" material, either A-1, A-2, or A-2-4.
Maine	1000-2700	All	No, but thicker section being used on I-95 north of Bangor.	FS groups of C of E	GSD, after C of E criteria	C of E criteria	Base: 0-5% Subbase: 0-7%	All cuts undercut 6", replaced with gravel. Also additional undercutting as necessary for boulder removal and to replace isolated pockets of FS material.
Nebraska	750-1500	Frost action affects performance of pavements statewide except where subgrades are sands.	No, we feel depth of frost is not a significant determinant of pavement performance.	Group Index, similar to AASHO's but modified by adding -4, -3, -2 and -1 for the best soils and 21 to 32 for the worst soils.	None. All Nebraska soils except clean fine and coarse sands are considered FS.	PI ≤ 6, and ≤ 15% passing No. 200 sieve.	5-13% for soil aggregate base. 8-12% for granular foundation course.	Potential frost boil areas undercut and backfilled with granular material or treated with lime.
New Hampshire	1000-2300	All	Yes. Prefer use NFS layers, to full depth of frost, but not always feasible.	AASHO, with subdivision of A-2's based on percentage passing No. 200.	Casagrande criteria, ≤ 3% finer than .02 mm. Rapid freezing test (HRR 215).	≤ 3% finer than .02 mm. Rapid freezing test if FS is borderline.	Crushed stone 0-8% Sand, Gravel, Crushed gravel: 0-12% of fraction passing No. 4.	Varies. Undercuts replaced with sand with ≤ 12% of -#4 passing No. 200 sieve.
New York	500-2000	All	No	Textural	GSD, Casagrande criteria based on % finer than .02 mm, used as a guide only.	Base: None. Use AC base only. Subbase: ≤ 10% passing No. 200 sieve.	Unbound base not used. Subbase: 0-10%	Undercut up to 2 feet to provide subgrade uniformity and prevent boulder heaves. Rock may be either undercut or fragmented by blasting. Backfill with granular fill or excavated material, maximum size 6 inches.
Saskatchewan	3000-7000	All	No	USCS and Group Index	Judgment, based on experience with the various soil types found within the province.	None	Base: Usually 7-10% Subbase: Usually 0-20%	All cuts undercut 2 feet, replacing with same material well mixed to break up inclined stratification, or with granular material with 0-15% passing No. 200.
Wisconsin	1100-2800	All	Frost is severe throughout the state. If conditions unusually severe, regional engineers can recommend additional granular base.	AASHO, modified to down-grade the group indices of highly frost susceptible (F-4) soils.	GSD, after C of E criteria.	≤ 5% passing No. 200 sieve.	Std. Specs: 0-10%, 2-12% or 8-15%, but in practice generally limit the minus No. 200 to 5% as a maximum.	Remove FS pockets or thicken granular base throughout the cut. Cuts in igneous rock undercut 6", or ripped and recompacted, or fragmented by blasting to 6-foot depth.
USA Corps of Engineers	0-5000+	Wherever DFI exceeds about 100	Yes, particularly for low DFI (< 500±). Where DFI is higher, thickness depends instead on FS of subgrade.	USCS and FS groups	GSD, with Groups F1, F2, F3, and F4 dependent on percentage finer than .02 mm. Standard laboratory FS test, from which degree of FS dependent upon rate of heaving during slow freezing.	≤ 3% finer than .02 mm. Freezing test if FS is borderline.	Base: 0-10% Subbase: 0-15% or 0-25%.	Undercut only to extent necessary so that required thickness pavement and NSF base and subbase can be achieved. Replaced with NFS material.

* Ref. (207).

FLEXIBLE PAVEMENTS

Does design thickness of Heavy Duty (HD), Intermediate (I) and Secondary (S) roads vary with the following?

State or province	Subgrade support value: How is it characterized?	Subgrade support value: Does it attempt to represent spring conditions?	Traffic analysis period, years	AADT and axle loads (range in ADT)	Terminal serviceability index or equivalent	Type of base or subbase, e.g. whether bound or unbound	Regional factor	Essence of design method, by which thickness for given traffic and serviceability conditions is determined as a function of subgrade support and frost conditions
Alberta	Yes. Soil classification and BB deflection.	Yes. BB deflection is measured in spring.	Yes HD 15-18 S 10-15	Yes HD > 4000 I, S 200-4000	Yes	Yes. Gravel equivalents.	No	Emphasis is on achieving uniformity of subgrade conditions. Use stage construction. First construct subgrade and 2-3" gravel. Second measure BB deflection which determines base design. Construct base plus 2" "oil-bound" (temporary cold-mix pavement with MC asphalt). Third (1-4 years later) measure BB deflection which determines surface course design. Construct asphaltic concrete surface course.
Colorado	Yes. R-value	Yes. Samples at low density are compressed to saturation.	20 in all cases	Yes HD, I > 750 S < 750	Yes	Yes. Layer coefficients.	Yes. 0.75-3.0	AASHO interim guides, using R-value and regional factor. Design adjusted during construction after measuring R-value of subgrade in completed cuts and fills.
Idaho	Yes. R-value and expansion pressure.	Yes. Samples molded at low density and compressed to saturation.	20 in all cases	Yes HD 500 (CADT) I 200 (CADT) S 40 (CADT)	No. Not used.	Yes. Substitution ratios.	Yes. Climatic factors 1.0-1.15	Thickness (in gravel equivalent) determined from R-value of subgrade and traffic index. Climatic factors are direct multipliers on gravel equivalent. Adjusted gravel equivalent converted to standard thicknesses of AC surface and ATB or CTB, to determine required thickness additional base of unbound gravel.
Maryland	Yes. CBR	Yes. CBR samples are soaked.	20 in all cases	Yes	2.5 in all cases.	Yes. Layer coefficients for AC. Substitution ratios for other bound and unbound materials.	Yes. 0.88-1.65	Design equation of AASHO interim guides used to determine thickness of AC plus equivalent DGA. Substitution ratios used to convert DGA to other types of bound and unbound bases. Frost conditions enter design equation only through regional factor and FS classification of subgrade. FS subgrades are reinforced with 12" granular cap (assigning it a CBR of 7) or stabilized with cement.
Maine	Yes. CBR, but design must meet minimum thickness requirements for the applicable FS group.	Yes. CBR samples are soaked, and FS groups are keyed to thickness requirements for spring conditions.	Yes HD 20 S 15	Yes, daily 18k axles	Yes. 2.5 HD 2.0 S	Yes. Layer coefficients.	Yes. 2.5-3.0	AASHO interim guides used for interstates, but checked also against thickness requirement from C of E design curve for 20k wheel load in relation to FS group. Usual thickness for HD roads about 32", but I-95 north of Bangor has additional 24" gravel. Thickness I and S roads determined principally from C of E curve related to FS groups.
Nebraska	Yes. Group index.	No. Our worst shale and clay subgrades are treated with lime to minimize frost action.	20 in all cases	Yes, daily 18k axles HD 110 I 60 S 25	Yes. 2.5 primary 2.0 other	Yes. Layer coefficients.	Yes. 0.5-3.0, independent of temperature regime.	Our usual design is "full-depth AC", whose thickness is determined as the AC surface course thickness from AASHO interim guides, plus black base to achieve the structural number required by the interim guides. Subgrade support evaluated by correlation with Group Index. Frost conditions do not enter design equation. Full-depth used even on heavy clays, but only if treated with lime. Flexible pavements used only on intermediate and secondary roads.
New Hampshire	Yes. CBR and soil classification.	Yes. CBR samples are soaked.	20 in all cases	Yes	2.0 all cases	Yes. Layer coefficients.	Yes. 2.0-3.0	Prefer provide NFS material to full depth of frost but actually get some subgrade freezing because interstates have 5½-10" AC plus 48" NFS, and other highways less. Thickness AC and granular base standardized for particular values of AADT, but thickness NFS subbase varies from 0 to 24" on recommendation of soils engineer.
New York	No.	—	20 in all cases	Yes. DHV: HD > 500 I 200-500 S < 200	No. Used only to determine priorities.	No. Bound base always used.	No	Standard designs depending only on class of highway and DHV, whose thicknesses were determined by evaluation of performance of pavements under similar terrain and environmental conditions. Emphasis is on achieving uniformity of subgrade conditions.
Saskatchewan	Yes. Measured CBR, and measured Group Index correlated with CBR. Recently started measuring resilient modulus after 3 cycles of freeze-thaw.	Yes. CBR samples are soaked, and resilient modulus is after freeze-thaw.	Yes. 15 in most cases. Sometimes, for stage construction, use 5 years.	Yes. Total 18k axles.	Yes. 2.5 primary 2.0 secondary	Yes. Shell Oil procedure, under which equivalencies vary with layer thickness and with subgrade strength.	No	Shell Oil procedure, which utilizes layered system theory. Stiffness value used for asphaltic mixtures is a function of properties of bitumen and temperature. Stiffness of unbound bases varies with thickness and with strength of subgrade. Stiffness of subgrade taken as 1500 × CBR, after adjustment of CBR downward by 2% in cuts, downward severely in lake basin sediments and upward by 2% where drainage is excellent.
Wisconsin	Yes. Group Index modified for frost susceptible soils	Yes, because it is modified by C of E frost classification of frost susceptible soils.	20 in all cases	Yes	Yes. 2.5 primary 2.0 secondary	Yes. Layer coefficients.	No. 3.0 throughout the state.	AASHO interim guides, using soil support value determined by correlation with design Group Index.
USA Corps of Engineers	CBR if subgrade freezing will be prevented; otherwise characterized only by FS group.	Yes. FS group is determined by rate of frost heave, but considered to be related also to strength during thaw and a thaw-weakened CBR is assigned to each.	Yes	Yes	No. Not used.	No. Bound base not generally used, but if so its use does not affect thickness required above an underlying weaker layer.	No. Not used.	Thickness required above subbase, and above base, determined by empirical design curves, from their respective CBR values. Total thickness required above subgrade determined as the lesser of: (1) thickness required to limit frost penetration into subgrade, which depends principally on DFI; or (2) thickness required over thaw-weakened subgrade to maintain structural adequacy of pavement, which depends on FS group.

RIGID PAVEMENTS

Does design thickness of PCC slab for Heavy Duty (HD), Intermediate (I) and Secondary (S) roads vary with the following?

State or province	Modulus of subgrade reaction (k): How is it determined?	Modulus of subgrade reaction (k): Does it attempt to represent spring conditions?	Modulus of subgrade reaction (k): Does its value vary depending on thickness and type of subbase?	Traffic analysis period, years	AADT and axle loads (range in AADT)	Terminal serviceability index or equivalent	Thickness and type of subbase: What thicknesses and types are used?	Thickness and type of subbase: Is additional subbase provided specifically for frost protection?	Steel reinforcement
Alberta	Rigid Pavements Not Used								
Colorado	Yes. Estimated from measured R-value.	Yes. R-value specimen is of low density, compressed to saturation.	Yes, on both. R-value represents combined stiffness of subgrade and subbase.	20 in all cases	Yes. HD, I ≥ 750; S < 750	Yes 2.0 or 2.5	Yes. Unbound, CTB, and ATB. Thickness determined from required k-value for 8-inch slab.	No	No. Only plain concrete used.
Idaho	Yes.* Estimated by correlation with measured R-value.	Yes. R-value specimen is of low density and is compressed to saturation.	No, determined for subgrade alone, but all rigid pavements have 4" CTB.	20 in all cases	Yes*	No	No. 4" CTB always used. One new project has 4" ATB.	No, except standard practice of grade-pointing, at transitions from cut to fill, is followed.	No
Maryland	Yes.* Estimated from correlation with CBR value.	Yes, because correlated with soaked CBR.	Yes, k is increased to account for the 6" granular subbase that is always used.	25 in all cases	Yes*	No	No. 6" granular subbase always used.	No	Yes. CRC thickness 1" less than RC.
Maine	Yes. Estimated from measured CBR value.		Yes, k is increased depending on thickness of base.	20 in all cases	Yes	No. 2.5 all cases	Yes. 24-30" granular	Yes. The subbase is provided for frost protection and to increase the k-value.	
Nebraska	Yes. Estimated	No	If 3" CTB used, k-value is increased by 100 pci.	20 in all cases	Yes	Yes, 2.5 primary, 2.0 other	Yes. None on best subgrades, otherwise either: 3" CTB or 4" granular.	No	Yes. Plain concrete, RC, and CRC (one project) are used.
New Hampshire	Rigid Pavements Not Used								
New York	No	—	—	20 in all cases	Yes. DHV: HD > 550, I 200-500, S < 200	No. Used only to determine priorities.	No. 12" granular material always used.	No.	All rigid pavement is RC.
Saskatchewan	Rigid Pavements Not Used								
Wisconsin	Yes. Inferred from GSD, plasticity and volume change.	Yes	No, except if base very thick may adjust (k) moderately upward.	20 in all cases	Yes	Yes, 2.5 primary, 2.0 secondary.	No. 6-8" granular, sometimes with top 3" stabilized with asphalt cement.	Have used 9-10" subbase if regional engineer requests it.	Yes. Interstates are 9" RC or 8" CRC.
USA Corps of Engineers	Yes. Inferred from FS group of subgrade, adjusted upwards with increasing thickness of subbase.	Yes, FS group is believed to characterize particular levels of thaw-weakened modulus.	Yes. On the thickness of unbound NFS subbase.	Yes	Yes	No. Not used.	Yes, with thickness. Minimum 4" granular, but thickness usually at least equal to thickness of slab.	Yes, under the design procedure whose objective is to limit frost penetration into subgrade.	Yes. RC pavements may be 0.5-2 inches thinner than plain concrete, depending on amount of steel.

* Design procedure shows a dependency, but actually slab thicknesses statewide are standardized, independently of the indicated parameter.

State or province	Essence of rigid pavement design method, by which thickness for given traffic and serviceability conditions is determined as a function of subgrade support and frost conditions	Usage to date of flexible/rigid pavements	Have you used full-depth asphaltic concrete pavements?
Alberta	Rigid pavements not used	All flexible	Yes, have used 5 years. Have about 300-400 miles on primary roads and 300-400 miles on secondaries.
Colorado	PCA procedure, used to 1969, gave 8-inch slab. All interstates built to date are 8 inches. Subbase thickness is designed to give required k-value. Now using AASHO method. Rigid pavement not used in mountains (most severe frost) because flexible pavements economical.	2/3/1/3	Yes, about 50 miles, on primary and secondary roads. Choice based strictly on cost, except not used on interstates, where always require unbound subbase.
Idaho	Standard design – 8" PCC over 4" CTB or ATB, plus 12" granular at grade points.	All flexible except 70-100 miles of interstate.	No, but expect to use over good subgrades.
Maryland	PCA procedure (1951 PCA manual)	1/2/1/2	Use thick AC pavements extensively, but always use 4" crushed stone unless subgrade is gravel.
Maine	AASHO interim guides. Subbase thickness, for frost protection, determined by experience.	Stopped building rigid 6 years ago, but now are advertising a rigid pavement for a section of interstate.	No, but now using greater thickness of asphaltic material.
Nebraska	AASHO interim guides.	Flexible seldom used for ADT > 2700.	Yes, our present design for intermediate and secondary roads is full-depth.
New Hampshire	Rigid pavements not used	All flexible	No. Concerned about possible greater tendency for low-temperature contraction cracks, and reluctant to place AC directly on FS subgrade.
New York	Standard designs, either 9" or 8" RC, depending only upon DHV.	Previously mostly rigid, now flexible used also.	No. This concept objectionable because violates principle that strength needs decrease with depth.
Saskatchewan	Rigid pavements not used	All flexible	Yes. About 200 miles, from 7½-9½".
Wisconsin	AASHO interim guides.	Only 12 miles flexible pavements on interstates (of total 450 miles).	Yes, two projects of 5 miles each. One is 9½", the other 12".
USA Corps of Engineers	Design curves based on Westergaard equations. k-value determined by measurement if only limited frost penetration into subgrade, otherwise k-value inferred from FS group and from thickness subbase.	Both have been used, depending on economics.	No, except experimental pavements.

State or province	Rigid pavements, selected design sections* — Interstate class	Rigid pavements — Other	Flexible pavements, selected design sections* to serve indicated traffic — Interstate class — Granular Base or CTB	Interstate class — Full-depth AC	Other heavy to intermediate — Granular base	Other heavy to intermediate — Full-depth AC	Light or secondary — Granular base	Light or secondary — Full-depth AC
Alberta	Rigid pavements not used		15000 ADT 2" AC 2" cold mix 10" CTB	5000-15000 ADT 10-12" AC	5000 ADT 2" AC 2" cold mix 12" granular† †or 8" soil cement	1000-5000 ADT 7-10" AC	600 ADT 2" AC 2" cold mix 5" grnlr	600 ADT 4" AC
Colorado	8" PCC ATB or CTB		5" AC 4" granular 4-16" subbase	Not used		100 18 k-axles 6¼-11" AC	3" bit. mat 4" grnlr base 9" subbase	1000 ADT 4-6½" AC
Idaho	8" PCC 4" CTB or ATB	Not used	5" AC 5" ATB 2½-6" granular	Not used		Not used		Not used
Maryland	8-9" CRC 6" granular	9" RC or 8" CRC 6" granular (Heavy duty roads only)	12¼-14" AC 4" granular	Not used				
Maine	9" PCC 4" granular 8" granular 12-18" granular	Not used	9-10" AC 4" granular 18" granular 24" granular 3" AC 3" bit. mac. 9" granular 18" granular	Not used	5" AC 2" granular 18" granular	Not used	2" road mix 1" granular 18" granular	Not used
Nebraska	10" PCC 4" grnlr; 9" RC 4" grnlr; 8" CRC 3" CTB; 8" RC —; 6" PCC —		Flexible pavement seldom used for ADT > 2700			< 2700 ADT 7¼" AC		5¼-6" AC
New Hampshire	Rigid pavements not used		15000 ADT 6¼-10¼" AC 12" granular 12" granular 0-24" NF subbase	Not used	4000 ADT 3" AC 12" grnlr 12" grnlr 0-24" NF subbase	Not used	1500 ADT 2" AC 12" grnlr 6-18" NF subbase	Not used
New York	9" RC 12" granular	8" RC 12" granular	> 550 DHV 2¼" AC 8" ATB 12" granular	Not used	200-550 DHV 2¼" AC 6" ATB 12" granular	Not used	< 200 DHV 1½" AC 3" ATB 12" granular	Not used
Saskatchewan	Rigid pavements not used		4½" AC 7" granular 10" sand	9" AC		7¼" AC		
Wisconsin	8" CRC or 9" RC 3" ATB 3-5" granular		Only 12 miles flexible pavement used on interstates. Its section is: 7" AC 6" CTB 6" granular 5" granular	Not used	12000 ADT 5" AC 8" granular	9¼-12" AC (only 2 projects)	1000 ADT 5¼" AC 6" granular 9" granular	Not used
USA Corps of Engineers	8" or 9" PCC (usual range) 4" granular Additional granular extremely variable		12000 ADT 4½" AC 6" granular 12" granular	Not used	3-4" AC 4" granular Subbase varies	Not used	1½-2½" AC 4" granular Subbase varies	Not used

*Selected sections are not necessarily those typically used, but are sections either recently constructed or indicated by design procedure. They are not comparable because traffic, subgrade and frost conditions differ.

Low-temperature transverse cracking in flexible pavements

State or province	Does low-temperature transverse cracking affect serviceability of your asphaltic pavements?	In what season are cracks filled?	What design practices are followed to minimize low-temperature cracking?	What grades or types of asphalts are used in your flexible pavements?	Remarks
Alberta	Yes	Filled in spring	Use high viscosity asphalt cement. Also use SC-3000 in hot-mix pavements.	AC-275, viscosity 275 poises @ 140°F and penetration 250 @ 77°F; SC-3000; emulsions SS-1 and AE-200 used with sand in base courses.	Still getting 135+ cracks per mile. SC-3000 mix has fewer cracks but has caused bleeding. Low-temperature cracking is priority research item.
Colorado	Yes	Filled during warm periods	Now changing to viscosity grading specification and will use softer grades if requested by district engineer.	AC-5, viscosity 400-600 poises @ 140°F and penetration $\geq$ 120 @ 77°F; AC-10, viscosity 800-1200 poises @ 140°F and penetration $\geq$ 70 @ 77°F.	Getting 50-100 cracks per mile, particularly in high mountains, but do not affect ride.
Idaho	Yes	Filled in summer	New policy calls for use of softer grades of asphalts.	120-150 and 200-300 penetration for base and surface courses, except surface course over ATB uses 85-100. Also use MC cutbacks on secondary roads.	Transverse cracks do not affect ride so we do not consider them a serious problem except when base is CTB, which for this reason we rarely use now for flexible pavements.
Maryland	No	—	—	—	
Maine	Yes	Formerly filled in spring with AE-90 (high float emulsion), but stopped filling because cracks reopen next winter.	Use softer grades.	Central and northern: AC-10; penetration usually is 100-140. Southern: AC-20, viscosity 1600-2400 poises @ 140°F, penetration $\geq$ 60 (but usually is 60-100).	Cracking is a severe problem. For re-surfacing we use ¾" hot mix made with high float emulsion, which is less thermally sensitive. Research is needed to achieve longer useful life of asphaltic pavements.
Nebraska	Yes	Filled in spring and summer with cationic emulsion or in winter with 60-70 pen. grade.	None	Mostly 85-100 penetration.	A recent survey indicates we are getting 40-48 cracks per mile, with a tendency for greater severity in northern than in southern part of state.
New Hampshire	Yes	Filled in winter with RS-1 emulsion.	Use low filler and high bitumen contents. Also using high float emulsions for resurfacing.	AC-10 and AC-20 depending only on ambient temperature during construction. Also high float emulsions for road mix and hot plant mix for resurfacing.	We believe up to 4" AC on clean strong base will not crack, but that thicker AC pavements have greater propensity for cracking. Cracking is not serious on our thicker (to 10½") pavements but they are quite new; cracking is becoming serious on some older 5½" pavements.
New York	Yes, but not severe.	Not filled	None, but experimenting with high float emulsion.	85-100 penetration.	High float emulsion has promise for surface courses since it can be formulated to have low thermal sensitivity.
Saskatchewan	Yes	Filled in spring, at 3-year intervals, with MC-0.	Using high viscosity asphalt cements. Also use SC road oils in hot AC mixtures.	Principally AC-6, viscosity 470-800 @ 140°F, penetration 170-240 @ 77°F. Also using SC-6 road oil (similar to SC-3000) in hot mixes.	Some cracks (spaced 200-300 feet) extend into subgrade; others (10 to 20 feet) only in pavement. Hot mixes made with SC-6 appear not to have propensity for cracking. Low-temperature cracking is priority area for research.
Wisconsin	Yes		None	85-100 and 120-150 penetration, and cutbacks.	We get 125 cracks per mile or more. We look to more use of the softer 120-150 grade. Pavement heave at cracks is a severe problem and requires research.
USA Corps of Engineers	Yes	Varies. Pavement maintenance often is by other agencies.	Use softer grade of asphalt in northern areas.	120-150 penetration for severe frost areas, 85-100 penetration for moderate frost areas.	Low-temperature cracking appears particularly pronounced in AC mixes of low bitumen content and high stability.

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