ENGINEER MANUAL

EM 1110-3-138
April 1984

ENGINEERING AND DESIGN

Pavement Criteria for Seasonal Frost Conditions

Mobilization Construction

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE CHIEF OF ENGINEERS

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, D.C. 20314

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Engineer Manual
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PAVEMENT CRITERIA FOR SEASONAL FROST CONDITIONS
Mobilization Construction

1. Purpose. This manual provides guidance for the design and construction of pavements placed on subgrade or base course materials subject to seasonal frost action. The criteria are applicable to Army airfields and heliports and to roads for U.S. Army mobilization facilities.

2. Applicability. This manual is applicable to all field operating activities having mobilization construction responsibilities.

3. Discussion. Criteria and standards presented herein apply to construction considered crucial to a mobilization effort. These requirements may be altered when necessary to satisfy special conditions on the basis of good engineering practice consistent with the nature of the construction. Design and construction of mobilization facilities must be completed within 180 days from the date notice to proceed is given with the projected life expectancy of five years. Hence, rapid construction of a facility should be reflected in its design. Time-consuming methods and procedures, normally preferred over quicker methods for better quality, should be de-emphasized. Lesser grade materials should be substituted for higher grade materials when the lesser grade materials would provide satisfactory service and when use of higher grade materials would extend construction time. Work items not immediately necessary for the adequate functioning of the facility should be deferred until such time as they can be completed without delaying the mobilization effort.

FOR THE COMMANDER:

Paul F. Kavanaugh

PAUL F. KAVANAUGH
Colonel, Corps of Engineers
Chief of Staff

DEPARTMENT OF THE ARMY
US Army Corps of Engineers
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CHAPTER 1

GENERAL

1-1. Purpose and scope. This manual presents criteria and procedures for the design and construction of pavements placed on subgrade or base course materials subject to seasonal frost action. The criteria are applicable to Army airfields and heliports and to roads for mobilization construction. The most prevalent modes of distress in pavements and their causes are listed in table 1-1. The principal modes unique to frost areas, with which this manual is concerned, are the non-traffic-associated distress modes of distortion caused by frost heave and reconsolidation, and of cracking caused by low temperatures, and the traffic-load-associated distress modes of cracking and distortion as affected by the extreme seasonal changes in supporting capacity of subgrades and bases that may take place in frost areas.

1-2. Definitions. The following frost terms are used in this manual.

a. Frost, soil, and pavement terms.

(1) Base or subbase course. All granular unbound, or chemical- or bituminous-stabilized material between the pavement surfacing layer and the untreated, or chemical- or bituminous-stabilized subgrade.

(2) Bound base. A chemical- or bituminous-stabilized soil used in the base and subbase course, consisting of a mixture of mineral aggregates and/or soil with one or more commercial stabilizing additives. Bound base is characterized by a significant increase in compressive strength of the stabilized soil compared with the untreated soil. In frost areas, bound base usually is placed directly beneath the pavement surfacing layer where its high strength and low deformability make possible a reduction in the required thickness of the pavement surfacing layer or the total thickness of pavement and base, or both. If the stabilizing additive is portland cement, lime or lime-cement-fly ash (LCF), the term bound base is applicable in this manual only if the mixture meets the requirements for cement-stabilized, lime-stabilized, or LCF-stabilized soil set forth in EM 1110-3-137 and in this manual.

(3) Boulder heave. The progressive upward migration of a large stone present within the frost zone in a frost-susceptible subgrade or base course. This is caused by adhesion of the stone to the frozen soil surrounding it while the frozen soil is undergoing frost heave; the stone will be kept from an equal, subsequent subsidence by soil that will have tumbled into the cavity formed beneath the stone. Boulders heaved toward the surface cause extreme pavement roughness and may eventually break through the surface, necessitating repair or reconstruction.

Table 1-1. Modes of distress in pavements.

Distress mode	General cause	Specific causative factor
Cracking	Traffic-load-associated	Repeated loading (fatigue) Slippage (resulting from braking stresses)
	Non-traffic-associated	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-associated	Rutting, or pumping and faulting (from repetitive loading) Plastic flow or creep (from single or comparatively few excessive loads)
	Non-traffic-associated	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 within the layered system or from abrasion by traffic. May also be triggered by freeze-thaw effects.	

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(4) Cumulative damage. The process by which each application of traffic load, or each cycle of climatic change, produces a certain irreversible damage to the pavement. When this is added to previous damage, the pavement deteriorates continuously under successive load applications or climatic cycles.

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

(6) Frost boil. The breaking of a small section of a highway or airfield pavement under traffic with ejection of soft, semi-liquid subgrade soil. This is caused by the melting of the segregated ice formed by frost action. This type of failure is limited to pavements with extreme deficiencies of total thickness of pavement and base over frost-susceptible subgrades, or pavements having a highly frost-susceptible base course.

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

(8) Frost-melting period. An interval of the year when the ice in base, subbase, or subgrade materials is returning to a liquid state. It ends when all the ice in the ground has melted or when freezing is resumed. In some cases, there may be only one frost-melting period, beginning during the general rise of air temperatures in the spring, but one or more significant frost-melting intervals often occur during a winter season.

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

(10) Granular unbound base course. Base course containing no agents that impart higher cohesion by cementing action. Mixtures of granular soil with portland cement, lime, or fly ash, in which the chemical agents have merely altered certain properties of the soil such as plasticity and gradation without imparting significant strength increase, also are classified as granular unbound base. However, these must meet the requirements for cement-modified, lime-modified, or LCF-modified soil set forth in EM 1110-3-137 and in this manual.

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

(12) Non-frost-susceptible materials. Cohesion less materials such as crushed rock, gravel, sand, slag, and cinders that do not experience significant detrimental ice segregation under normal

freezing conditions. Non-frost-susceptible materials also include cemented or otherwise stabilized materials that do not evidence detrimental ice segregation, loss of strength upon thawing, or freeze-thaw degradation.

(13) Pavement pumping. The ejection of water and soil through joints, cracks, and along edges of pavements caused by downward movements of sections of the pavement. This is actuated by the passage of heavy axle loads over the pavement after free water has accumulated beneath it.

(14) Period of weakening. An interval of the year that starts at the beginning of a frost-melting period and ends when the subgrade strength has returned to normal summer values, or when the subgrade has again become frozen.

b. Temperature terms.

(1) Average daily temperature. The average of the maximum and minimum temperatures for 1 day, or the average of several temperature readings taken at equal time intervals, generally hourly, during 1 day.

(2) Mean daily temperature. The mean of the average daily temperatures for a given day in each of several years.

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

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

(5) Design freezing index. The average air freezing index of the three coldest winters in the latest 30 years of record. If 30 years of record are not available, the air freezing index for the coldest winter in the latest 10-year period may be used.

(6) Mean freezing index. The freezing index determined on the basis of mean temperatures. The period of record over which temperatures are averaged is usually a minimum of 10 years, and

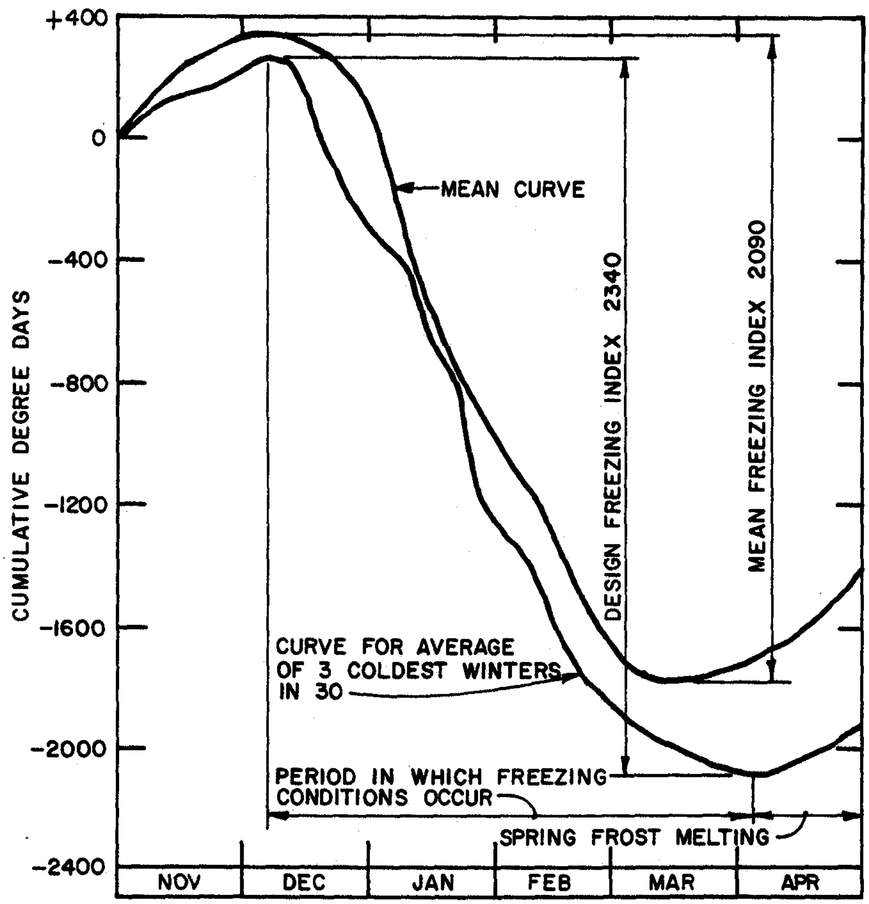

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FIGURE 1-1. DETERMINATION OF THE FREEZING INDEX preferably 30, and should be the latest available. The mean freezing index is illustrated in figure 1-1.

CHAPTER 2

FROST EFFECTS

2-1. Need for considering effects of frost in pavement design. The detrimental effects of frost action in subsurface materials are manifested by nonuniform heave of pavements during the winter and by loss of strength of affected soils during the ensuing thaw period. This is accompanied by a corresponding increase in damage accumulation and a more rapid rate of pavement deterioration during the period of weakening. Other related detrimental effects of frost and low temperatures are: possible loss of compaction, development of permanent roughness, restriction of drainage by the frozen strata, and cracking and deterioration of the pavement surface. Hazardous operating conditions, excessive maintenance, or pavement destruction may result. Except in cases where other criteria are specifically established, pavements should be designed so that there will be no interruption of traffic at any time due to differential heave or to reduction in load-supporting capacity. Pavements should also be designed so that the rate of deterioration during critical periods of thaw weakening, and during cold periods causing low-temperature cracking, will not be so high that the useful life of the pavements will be less than 5 years.

2-2. Conditions necessary for ice segregation. Three basic conditions of soil, temperature, and water must be present simultaneously for significant ice segregation to occur in subsurface materials.

a. Soil. The soil must be frost-susceptible, which usually implies that under natural climatic conditions the soil moisture becomes segregated into localized zones of high ice content. To some degree, all soils that have a portion of their particles smaller than about 0.05 millimeters are frost-susceptible. Temperature, moisture availability, surcharge pressure, and density act as additional influences that modify the basic frost-susceptibility of such soils.

b. Temperature. Freezing temperatures must penetrate the soil because the phase change from water to ice is largely responsible for drawing additional water from the surrounding soil toward the growing ice mass. The amount of water stored as segregated ice is usually observed to vary inversely with the rate of advance of the freezing isotherm.

c. Water. A source of water must be available to the zone of freezing. Usually the source will be an underlying ground water table, an aquifer or infiltration through overlying layers. If the supply of water to the freezing zone is restricted by distance from the external water sources or by low soil permeability, water will be drawn from the voids of the soil adjacent to the growing ice crystal or from soil below the freezing front.

d. Interrelationship among variables. A change in one or another of the three basic factors will vary the amount of ice segregated per unit volume of soil. Natural stratigraphic variations and construction details affect the relationship among these factors and therefore also influence the amount of segregated ice. A common example is a transition from cut to fill along a right-of-way, which represents a change in subgrade soils, in the pattern of subsurface water flow, and most likely in the freezing rate.

2-3. Description of ice segregation in soils. The process of ice segregation is a complex interaction of simultaneous heat and water flow through the mass of mineral and organic particles that make up the soil. Recent research has identified three distinct zones of the freezing process. Figure 2-1 illustrates the three zones when subfreezing temperatures have penetrated into the subgrade. The amount of unfrozen water varies with the type of soil, the soil particle surface characteristics, the gradation of the soil, and the temperature. In general, finer soils contain larger amounts of unfrozen water at a given subfreezing temperature than coarser soils and for a given soil the unfrozen moisture content decreases with lower subfreezing temperatures. While moisture movement in the frozen zone makes an insignificant contribution to frost heave, moisture movement induced by negative pore pressures developed in the freezing zone has a major impact on the magnitude of frost heave.

a. The lower boundary of the freezing zone in figure 2-1 is the depth at which the temperature is equal to the freezing point of the bulk water in the soil. This temperature is generally within one or two tenths of a degree below 32 degrees F., depending upon the chemical content of the soil water.

b. The upper boundary of the freezing zone in frost-susceptible soils is generally defined as the bottom of the growing ice lens. It is at this location where the negative pore pressure causing moisture movement to the ice lens is generated. An ice lens continues to grow in thickness in the direction of heat transfer until ice formation at a lower elevation cuts off the source of water, or until available water is depleted or it approaches a level at which sub-freezing soil temperatures no longer prevail. At this point, ice will stop forming.

2-4. Frost-susceptible soil. The potential intensity of ice segregation that may occur in a freezing season is dependent to a large degree on the size-range of the soil voids, which in turn is determined by the size and size distribution of the soil grains, soil density, and particle shape and orientation. As previously mentioned, at least a portion of the grains must be sufficiently small (less than about 0.05 millimeters in diameter) so that some of the water remains as unfrozen water films, providing channels for liquid flow. For pavement design, the potential ice segregation is often expressed as an empirical function of grain size as follows. Most inorganic soils containing 3

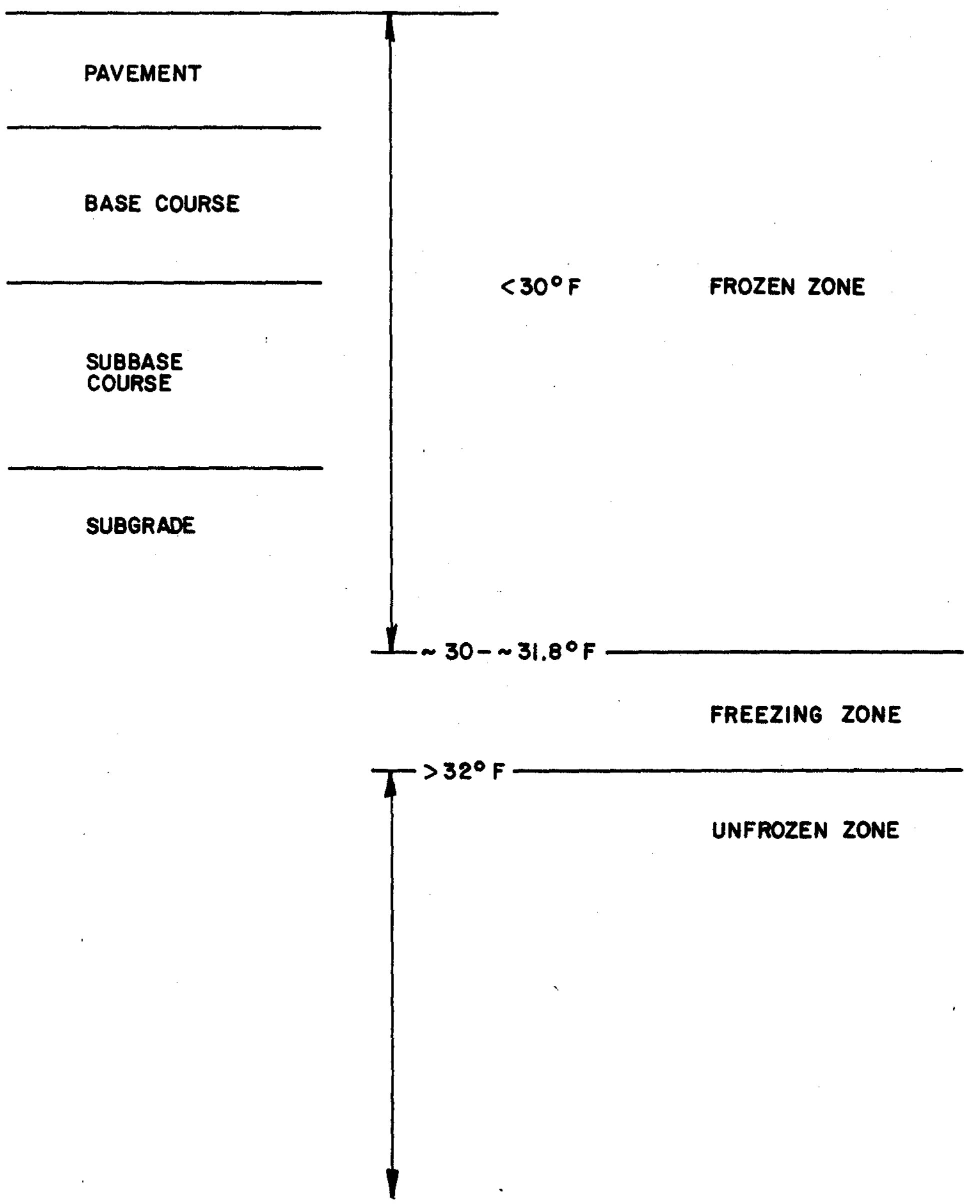

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FIGURE 2-1. FREEZING SEQUENCE IN A TYPICAL PAVEMENT PROFILE percent or more by weight of grains finer than 0.02 millimeters in diameter are frost-susceptible. Gravels, well-graded sands, and silty sands, especially those approaching the theoretical maximum density curve, that contain 1-1/2 to 3 percent of grains finer than the 0.02-millimeter size by weight should be considered as possibly frost-susceptible. Uniform sandy soils may have as much as 10 percent of their grains finer than 0.02 millimeters by weight without being frost-susceptible. However, their tendency to occur interbedded with other soils usually makes it impractical to consider them separately.

a. Standard laboratory freezing tests. Soil judged as potentially frost-susceptible under the above criteria may be expected to develop significant ice segregation if frozen at rates that are commonly observed in pavement systems (0.1 to 1.0 inches/day) and if free water is available (less than 5 to 10 feet below the freezing front). Figure 2-2 shows results of laboratory frost-susceptibility tests performed using a standardized freezing procedure on 6-inch high by 6-inch diameter specimens of soils ranging from well-graded gravels to fat clays. The soils that were tested are representative of materials found in frost areas. Test specimens were frozen with water made available at the base; this condition is termed "open-system" freezing, as distinguished from "closed-system" freezing in which an impermeable base is placed beneath the specimen and the total amount of water within the specimen does not change during the test. Appendix A contains a summary of results from freezing tests on natural soils. The data in appendix A can be used to estimate the relative frost-susceptibility of soils using the following two-step procedure: 1) select at least two soils having densities and grain-size distributions (the 0.074-, 0.02- and 0.01-millimeter sizes are most critical) similar to the soil in question, and 2) estimate the frost-susceptibility of that soil from those of the two similar soils.

(1) Diagrams a through d in figure 2-2 show individual test results for each of the four major soil groups: gravels, sands, silts, and clays. A family of overlapping envelopes is given in figure 2-3 showing the laboratory test results by various individual soil groupings, as defined by MIL-STD-619(CE). A frost-susceptibility adjective classification scale, relating the degree of frost-susceptibility to the exhibited laboratory rate of heave, is shown at the left side of figure 2-3. Because of the severity of the laboratory test, the rates of heave shown in figures 2-2 and 2-3 are generally greater than may be expected under normal field conditions. Soils that heave in the standard laboratory tests at average rates of up to 1 millimeter per day are considered satisfactory for use under pavements in frost areas, unless unusually severe conditions of moisture availability and temperature are anticipated.

(2) It can be seen in figures 2-2 and 2-3 that soils judged as non-frost-susceptible under the criteria given are not necessarily free of susceptibility to frost heaving. Also, soils that, although

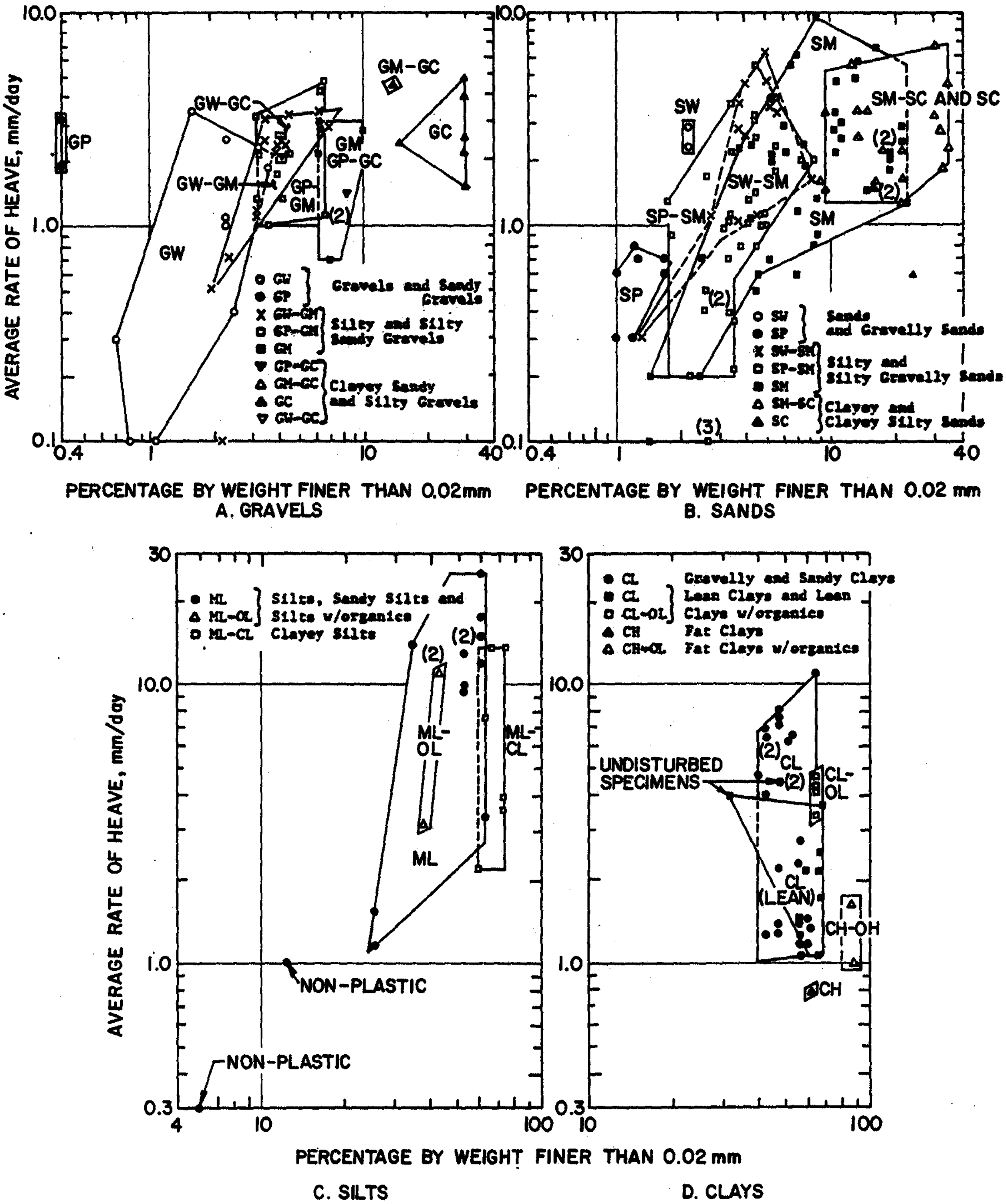

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FIGURE 2-2. RATES OF HEAVE IN LABORATORY FREEZING TESTS ON REMOLDED SOILS

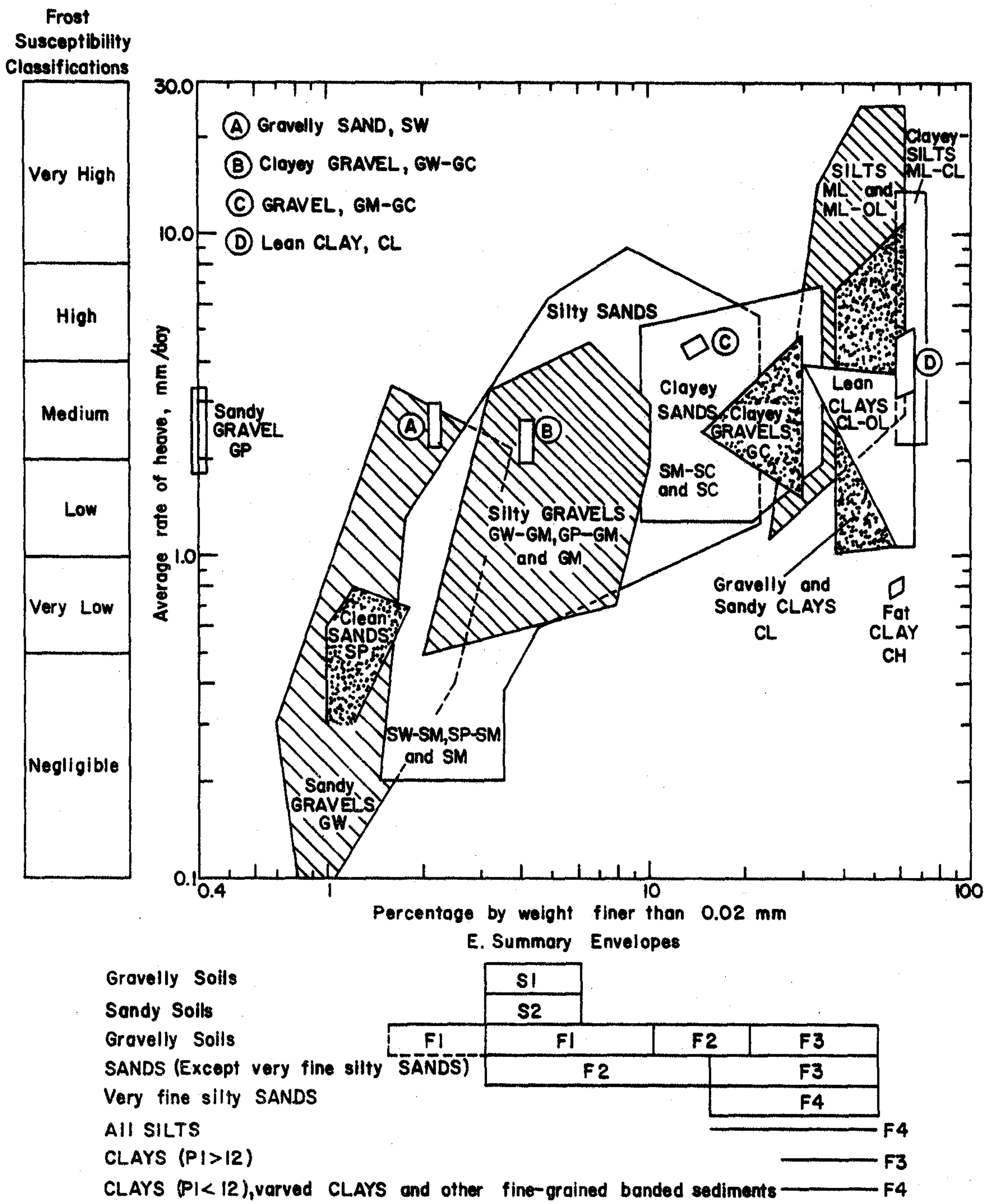

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FIGURE 2-3 . RATES OF HEAVE IN LABORATORY FREEZING TESTS ON REMOLDED SOILS indicated to be of negligible frost-susceptibility, approach a rate of heave of 1.0 millimeter per day in laboratory tests should be expected to show some measurable frost heave under average field conditions. These facts must be kept in mind when applying the criteria to other-than-normal pavement practice, and when considering subsurface drainage measures.

b. Frost-susceptibility classification. For frost design purposes, soils are divided into eight groups as shown in table 2-1. The first four groups are generally suitable for base course and subbase course materials, and any of the eight groups may be encountered as subgrade soils. Soils are listed in approximate order of decreasing bearing capacity during periods of thaw. There is also a tendency for the order of the listing of groups to coincide with increasing order of susceptibility to frost heave, although the low coefficients of permeability of most clays restrict their heaving propensity. The order of listing of subgroups under groups F3 and F4 does not necessarily indicate the order of susceptibility to frost heave of these subgroups. There is some overlapping of frost-susceptibility between groups. Soils in group F4 are of especially high frost-susceptibility.

(1) The S1 group includes gravelly soils with very low to medium frost-susceptibility classifications that are considered suitable for subbase materials. They will generally exhibit less frost heave and higher strength after freeze-thaw cycles than similar F1 group subgrade soils. The S2 group includes sandy soils with very low to medium frost-susceptibility classifications that are considered suitable for subbase materials. Due to their lower percentages of finer-than-0.02-millimeter grains than similar F2 group subgrade soils, they will generally exhibit less frost heave and higher strength after freeze-thaw cycles.

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

(3) Varved clays consisting of alternating layers of silts and clays are likely to combine the undesirable properties of both silts and clays. These and other stratified fine-grained sediments may be hard to classify for frost design. Since such soils are likely to

Table 2-1. Frost design soil classification.

Frost group		Kind of soil	Percentage finer than 0.02 mm by weight	Typical soil types under Unified Soil Classification System
NFS**	(a)	Gravels Crushed stone Crushed rock	0-1.5	GW, GP
	(b)	Sands	0-3	SW,SP
PFS	(a)	Gravels Crushed stone Crushed rock	1.5-3	GW,GP
	(b)	Sands	3-10	SW,SP
S1		Gravelly soils	3-6	GW, GP, GW-GM, GP-GM
S2		Sandy soils	3-6	SW, SP, SW-SM, SP-SM
F1		Gravelly soils	6 to 10	GM, GW-GM, GP-GM
F2	(a)	Gravelly soils	10 to 20	GM, GW-GM, GP-GM,
	(b)	Sands	6 to 15	SM, SW-SM, SP-SM
F3	(a)	Gravelly soils	Over 20	GM, GC
	(b)	Sands, except very fine silty sands	Over 15	SM, SC
	(c)	Clays, PI less than 12	-	CL, CH
F4	(a)	All silts	-	ML, MH
	(b)	Very fine silty sands	Over 15	SM
	(c)	Clays, PI greater than 12	-	CL, CL-ML
	(d)	Varved clays and other fine-grained, banded sediments	-	CL and ML; CL, ML, and SM; CL, CH, ML and SM

** Non-frost-susceptible.
Possibly frost-susceptible, but requires laboratory test to determine frost design soil classification.

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heave and soften more readily than homogeneous soils with equal average water contents, the classification of the material of highest frost-susceptibility should be adopted for design. Usually, this will place the over-all deposit in the F4 category.

(4) Under special conditions, the frost group classification adopted for design may be permitted to differ from that obtained by application of the above frost group definitions. The difference is not to be greater than one frost group number justification for such differences should take into account special conditions of subgrade moisture or soil uniformity, in addition to soil gradation and plasticity, and should include data on performance of existing pavements near those proposed to be constructed.

2-5. Frost heaving. Frost heave, manifested by the raising of the pavement, is directly associated with ice segregation and is visible evidence on the surface that ice lenses have formed in the subgrade, in the base material, or in both. Detrimental frost heave can be effectively controlled by designing the pavement so that frost will penetrate to only a limited depth into frost-susceptible subgrade soil and by adequate subgrade preparation and transition details. If significant freezing of a frost-susceptible subgrade does occur, the heave may be uniform or nonuniform, depending on variations in the character of the soils and the ground water conditions underlying the pavement and the thermal properties of the paving materials.

a. Uniform heave. Uniform heave is the raising of adjacent areas of a pavement surface by approximately equal amounts. The initial shape and smoothness of the surface remain substantially unchanged. Conditions conducive to uniform heave may exist, typically, in a homogeneous section of pavement that is exposed to equal solar radiation and is constructed with a fairly uniform stripping or fill depth, and that has uniform ground water depth and horizontally uniform soil characteristics.

b. Nonuniform heave. Nonuniform heave causes objectionable unevenness or abrupt changes in grade at the pavement surface. Conditions conducive to irregular heave occur, for example, at changes in pavement types or sections, at locations where subgrades vary between clean non-frost-susceptible sands and silty frost-susceptible materials, at abrupt transitions from cut to fill sections with the ground water close to the surface, or where excavation cuts into water-bearing strata. On pavements with inadequate frost protection, some of the most severe pavement roughness is caused by differential heave at abrupt changes in subgrade soil type and at drains and culverts and by boulder heaves. Special techniques of subgrade preparation and adequate transition details are needed to minimize irregular heave from these causes.

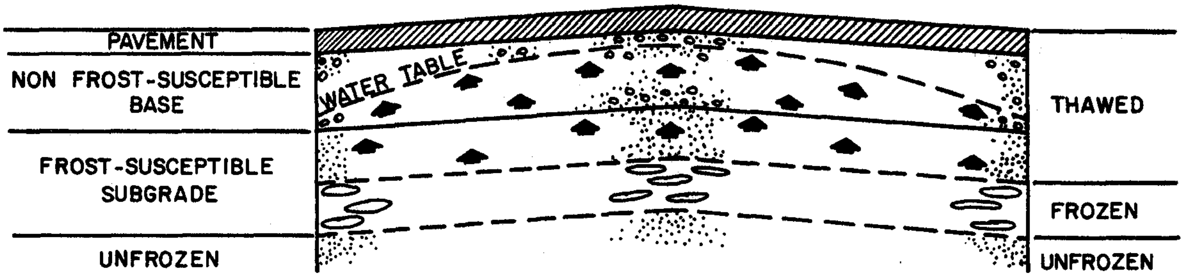

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FIGURE 2-4. MOISTURE MOVEMENT UPWARD INTO BASE COURSE DURING THAW

c. Supporting capacity may be reduced in clay subgrades even though significant heave has not occurred. Overconsolidation in clay soils occurs due to negative pressures generated in the freezing zone. As a result, the clay particles are reoriented into a more compact aggregated or layered structure with the clay particles and ice being segregated. The resulting consolidation may largely balance the volume of the ice lenses formed. Even clays that show no evidence of ice segregation, measurable moisture migration, or significant volume increase when frozen may significantly lose supporting capacity during the thaw period.

d. If frost-susceptible soil beneath a pavement will freeze, the effect of the reduced supporting capacity during frost-melting periods must be taken into account in designing the layered pavement structure.

CHAPTER 3

INVESTIGATION OF POTENTIAL FOR ICE SEGREGATION

3-1. Investigation procedure. The field and laboratory investigations conducted in accordance with EM 1110-3-141 will usually provide enough information to determine whether a given combination of soil and water conditions beneath the pavement will be conducive to frost action. Particular attention should be given to the degree of horizontal variation of subgrade conditions. This involves both soil and moisture conditions and is difficult to express simply and quantitatively. Subgrades may range from uniform conditions of soil and moisture that will result in negligible differences in frost heave, thaw settlement, and supporting capacity, to extremely variable conditions. These variable conditions may require extensive processing of subgrade materials to eliminate the frequent and very abrupt changes between high and low frost heave and high and low strength loss potentials. Following is a summary of procedures for determining whether or not the conditions of soil properties, temperature, and moisture that are necessary for ice segregation are present at a proposed site. In addition to assessing the potential for detrimental frost action, consider all reliable information about past frost heaving of airfield and highway pavements already built in the area.

3-2. Temperature. Air freezing index values should be based on actual air temperatures obtained from the meteorological station closest to the construction site. This is desirable because differences in elevation, topographical position, or nearness to bodies of water, cities, or other sources of heat may cause considerable variation in air freezing indices over short distances. These variations are of greater relative importance in areas of design freezing index of less than 1,000 degrees F.-days (i.e., mean air freezing index of less than about 500 degrees F.-days) than they are in colder climates.

a. Daily maximum and minimum and mean monthly air temperature records for all stations that report to the U.S. National Weather Service are available from Weather Service Centers. One of these centers is generally located in each state. The mean air freezing index may be based on mean monthly air tempertures, but computation of values for the design freezing index may be limited to only the coldest years in the desired cycle. These years may be selected from the tabulation of average monthly temperatures for the nearest first-order weather station. (A Local Climatological Data Summary, containing this tabulation for the period of record, is published annually by the National Weather Service for each of the approximately 350 U.S. first-order stations.) If the temperature record of the station closest to the construction site is not long enough to determine the mean or design freezing index values, the available data should be related, for the same period, to that of the nearest station or stations of adequate record. Site air freezing index values can then be computed based on this established relation and the indices for the more distant station or stations.

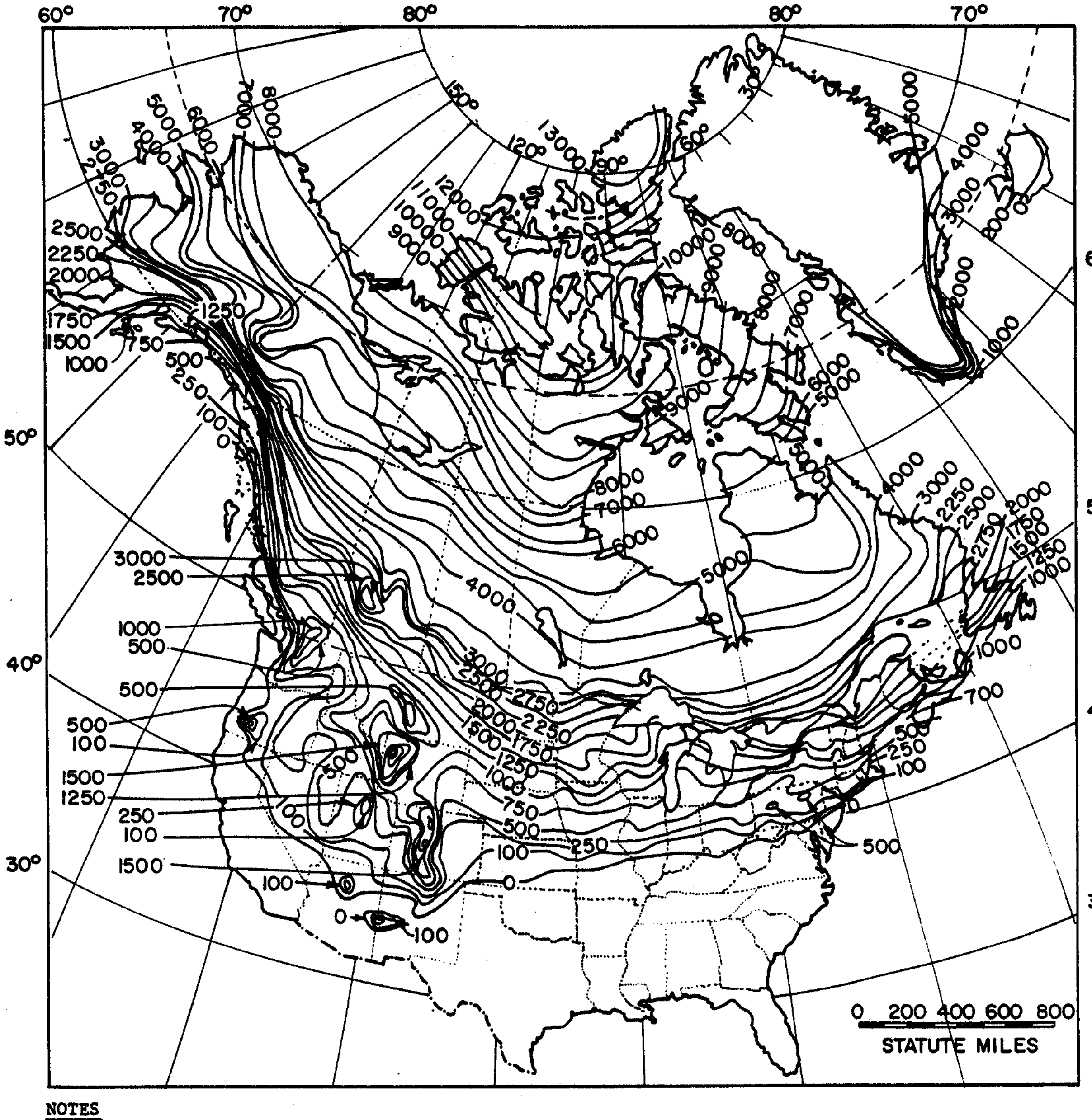
b. The distribution of freezing indices in North America is illustrated by figures 3-1 and 3-2. The figures show isolines of air freezing index values for the normal year (mean air freezing index), and the average of the 3 coldest years in 30 or the coldest year in 10 (design freezing index). Relationships between mean air freezing indices and values computed on various other statistical bases are shown in figure 3-3. For designing pavements, the design air freezing index should be calculated from available air temperatures or estimated from figure 3-2.

3-3. Depth of frost penetration. The depth of which subfreezing temperatures will penetrate below a pavement kept clear of snow and ice depends principally on the magnitude and duration of below-freezing air temperatures, on the properties of the underlying materials, and on the amount of water that becomes frozen. Curves in figure 3-4 may be used to estimate depths of frost penetration beneath paved areas kept free of snow and ice. They have been computed for an assumed 12-inch-thick rigid pavement, using the modified Berggren equation and correction factors derived by comparison of theoretical results with field measurements under different conditions. The curves yield the maximum depth to which the 32 degrees F. temperature will penetrate from the top of the pavement under the total winter freezing index values in homogeneous materials of unlimited depth for the indicated density and moisture content. Variations due to use of other pavement types and of rigid pavements of lesser thicknesses may be neglected.

a. The curves in figure 3-4 are not applicable for determining transient penetration depths under partial freezing indices. For specific problems of this type, the fundamental equations of heat transfer are applicable, for which various numerical solutions are available.

b. Maximum seasonal frost penetration depths obtained by use of figure 3-4 should be verified whenever possible by observations in the locality under consideration.

3-4. Water. A potentially troublesome water supply for ice segregation is present if the highest ground water table or a perched water table is, at any time of the year, within 5 feet of the proposed subgrade surface or of the top of any frost-susceptible subbase materials used. A water table less than 5 feet deep indicates potential ground moisture problems. When the depth to the top of the water table is in excess of 10 feet throughout the year, ice segregation and frost heave may be reduced, but special subgrade preparation techniques are still necessary to make the materials more uniform. Silt subgrades may retain enough moisture to cause significant frost heave and thaw weakening even when the water table is



NOTES

Design freezing index values are cumulative degree days of air temperature below 32 degrees F. for the coldest year in a 10-year cycle or the average of the 3 coldest years in a 30-year cycle.

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

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

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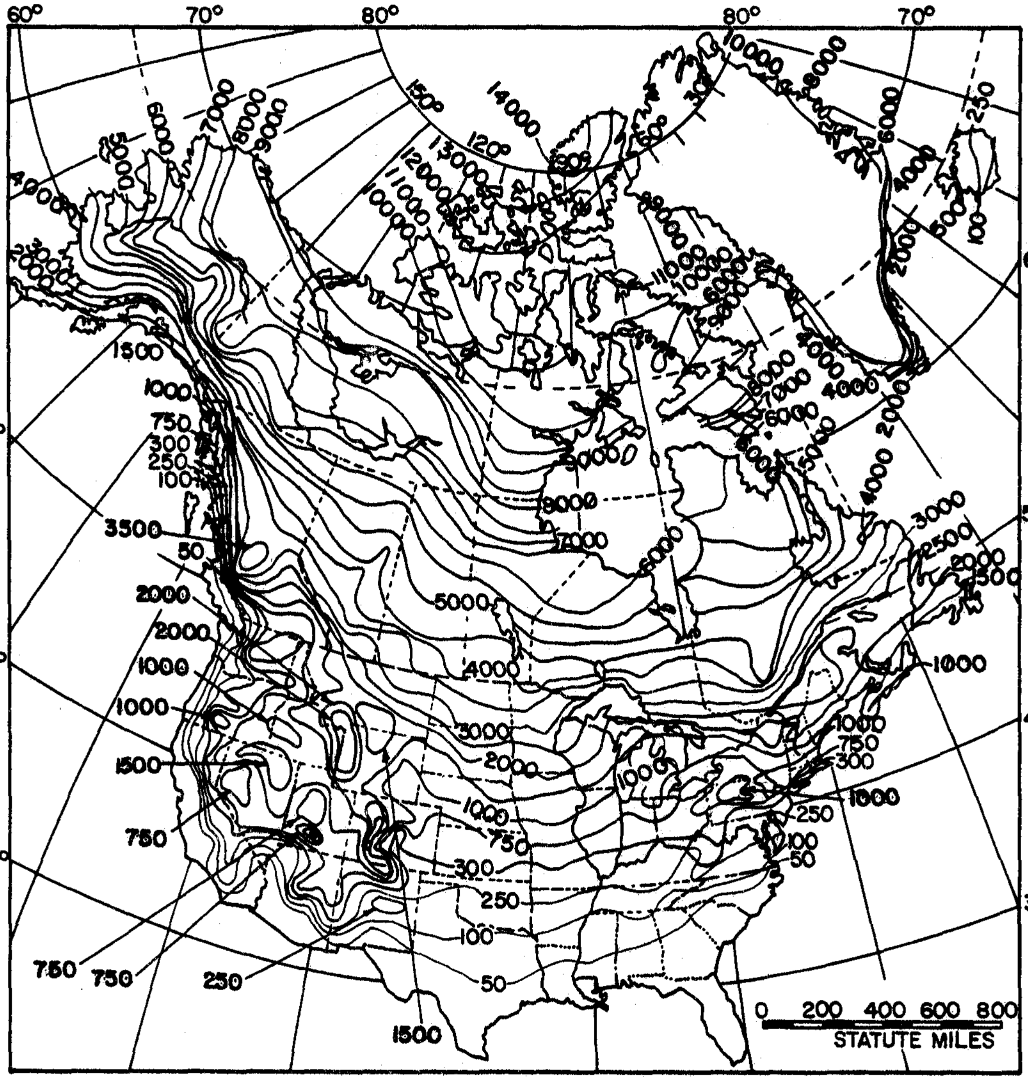
FIGURE 3-1. DISTRIBUTION OF MEAN AIR-FREEZING INDEX VALUES IN NORTH AMERICA



Notes

Design freezing index values are cumulative degree days of air temperature below 32 degrees F. for the coldest year in a 10-year cycle or the average of the 3 coldest years in a 30-year cycle.

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

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

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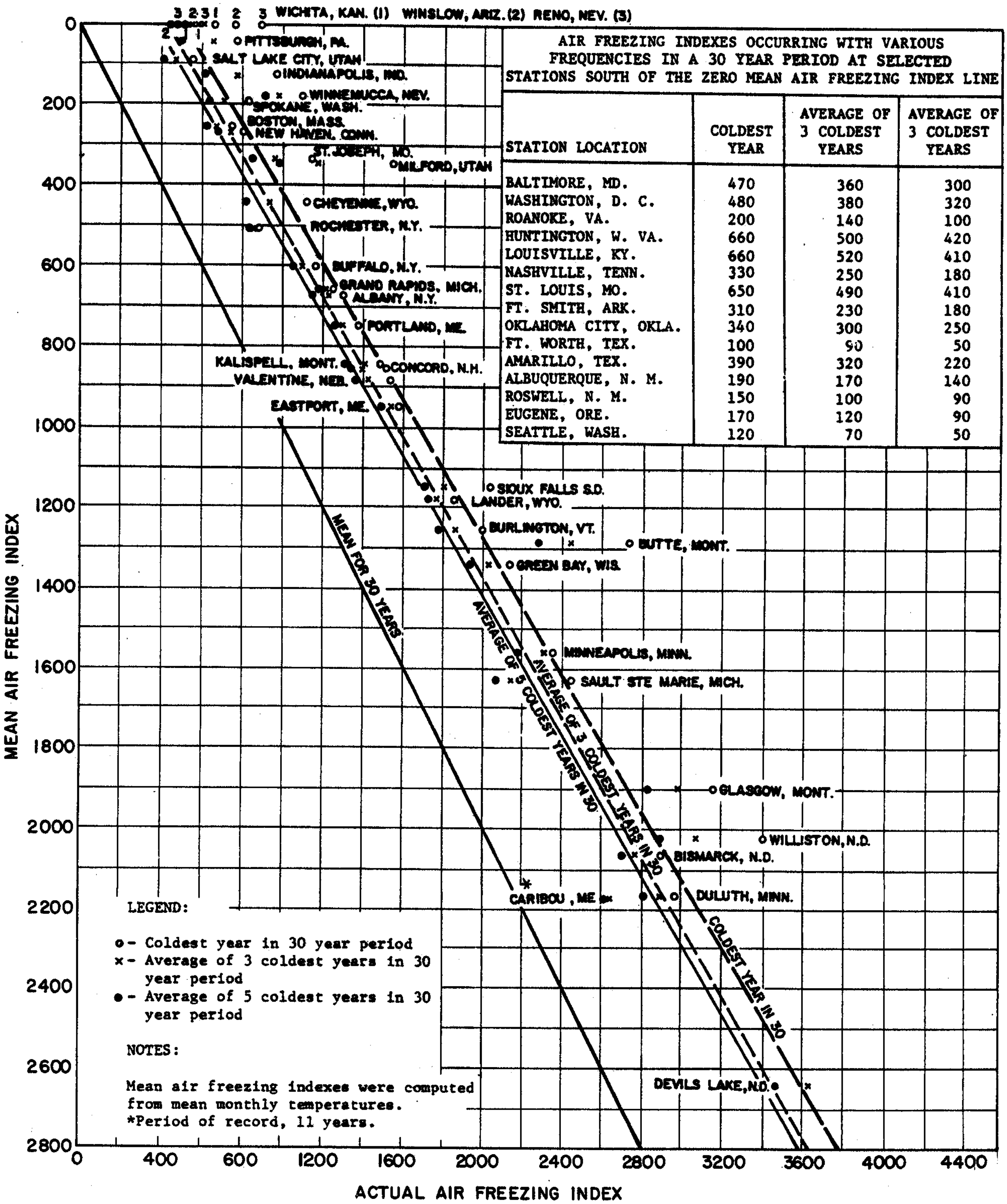
FIGURE 3-2. DISTRIBUTION OF DESIGN AIR-FREEZING INDEX VALUES IN NORTH AMERICA



AIR FREEZING INDEXES OCCURRING WITH VARIOUS FREQUENCIES IN A 30 YEAR PERIOD AT SELECTED STATIONS SOUTH OF THE ZERO MEAN AIR FREEZING INDEX LINE

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

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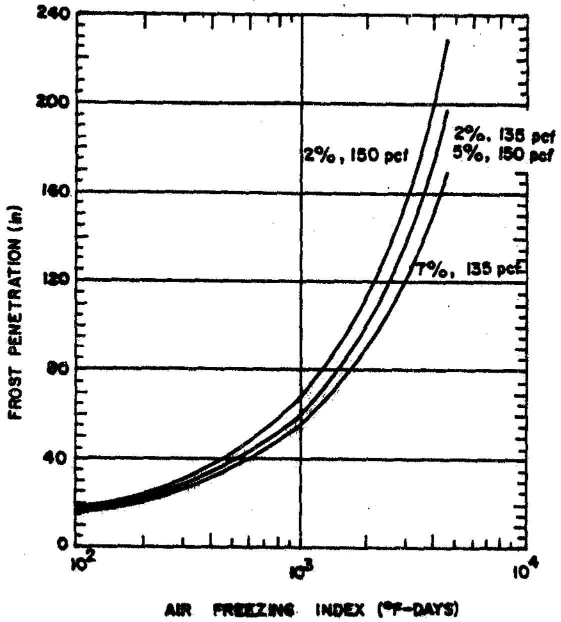
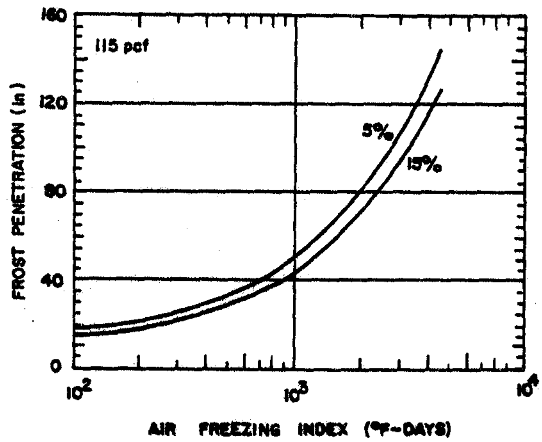
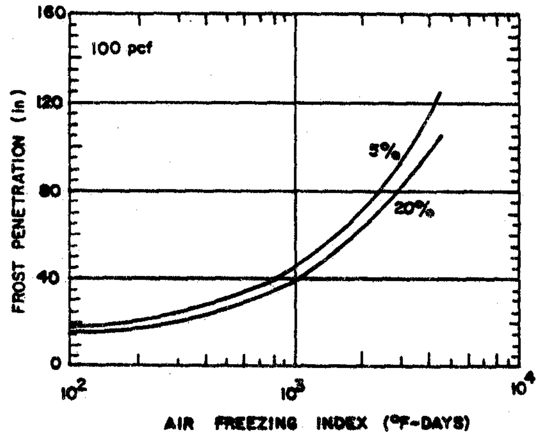
FIGURE 3-3. RELATIONSHIPS BETWEEN MEAN AND OTHER AIR-FREEZING INDICES



A. 135 pcf AND 150 pcf MATERIAL



B. 115 pcf MATERIAL



C. 100 pcf MATERIAL

NOTES:

1. Frost penetration depths are based on modified Berggren formula and computation procedures outlined in USACRREL Special Report 122.

2. Frost penetration depths are measured from pavement surface. Depths shown are computed for 12-inch PCC pavements kept free of snow and ice, and are good approximations for bituminous pavements over 6 to 9 inches of high quality base. Computations also assume all soil beneath pavements within depths of frost penetration is granular and non-frost-susceptible with indicated dry unit weight and moisture content.

3. It was assumed in computations that all soil moisture freezes when soil is cooled below 32 degrees F.

4. Dry unit weight and moisture content (in percent) given on figures.

5. For pavement design, use design freezing index (para 1-2b and 3-3).

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FIGURE 3-4. FROST PENETRATION BENEATH PAVEMENTS more than 10 feet below them. Special precautions must be taken when these soils are encountered and a relatively thin pavement section is planned, e.g., all-bituminous concrete. The water content that homogeneous clay subgrades will attain is usually sufficient to cause some ice segregation, even with a remote water table. Closed-system laboratory freezing tests that correspond to a field condition with a very deep water table usually indicate less severe heaving than will actually take place. This is because moisture contents near complete saturation may occur in the top of a frost-susceptible subgrade from surface infiltration through pavement and shoulder areas or from other sources.

CHAPTER 4

THICKNESS DESIGN OF LAYERED PAVEMENT STRUCTURE

4-1. Alternative methods of design. The thickness design process is the determination of the required thickness for each layer of a pavement system and of the combined thickness of all layers above the subgrade. Its objective is determining the lowest-cost pavement system whose rate of deterioration under traffic loads and environmental conditions will be acceptably low. In seasonal frost areas, the thickness design process must include the studies and analyses required by normal design, and it must also account for the effects of frost action. Two methods are prescribed here for determining the thickness design of a pavement that will have adequate resistance to 1) distortion by frost heave, and 2) cracking and distortion under traffic loads as affected by seasonal variation of supporting capacity, including possible severe weakening during frost-melting periods.

a. Limited subgrade frost penetration method. The first method is directed specifically to the control of pavement distortion caused by frost heave. It requires a sufficient thickness of pavement, base, and subbase to limit the penetration of frost into the frost-susceptible subgrade to an acceptable amount. Included also in this method is a design approach which determines the thickness of pavement, base and subbase necessary to prevent the penetration of frost into the subgrade. Prevention of frost penetration into the subgrade is nearly always uneconomical and unnecessary and will not be used to design pavements to serve conventional aircraft and motor vehicle traffic. For pavements where layers of synthetic insulation are permitted, full protection of the subgrade against freezing may be feasible (app B).

b. Reduced subgrade strength method. The second method does not seek to limit the penetration of frost into the subgrade but determines the thickness of pavement, base, and subbase that will adequately carry traffic loads over the design period of years, each of which includes one or more periods during which the subgrade supporting capacity is sharply reduced by frost melting. This approach relies on uniform subgrade conditions, adequate subgrade preparation techniques, and transitions for adequate control of pavement roughness resulting from differential frost heave.

4-2. Selection of design method. In most cases, the choice of the pavement design method will be made in favor of the one that gives the lower cost. Exceptions dictating the choice of the limited subgrade frost penetration method, even at higher cost, include pavements in locations where subgrade soils are so extremely variable (as, for example, in some glaciated areas) that the required subgrade preparation techniques could not be expected to sufficiently restrict differential frost heave. In other cases, special operational demands on the pavement facility might dictate unusually severe restrictions on tolerable pavement roughness, requiring that subgrade frost penetration be strictly limited or even prevented. If use of the limited subgrade frost penetration method is not required, tentative designs must be prepared by both methods for comparison of costs. Also, a tentative design must be prepared following the non-frost-design criteria of EM 1110-3-131 or EM 1110-3-141 since the thickness requirements under non-frost-criteria must be met in addition to the frost design requirements.

4-3. Design for limited subgrade frost penetration - airfields and roads. This method of design for seasonal frost conditions should be used where it requires less thickness than the reduced subgrade strength method. Its use is likely to be economical only in regions of low design freezing index or for pavements for heavy-load aircraft in regions of moderate to high freezing index.

a. The design freezing index should be used in determining the combined thickness of pavement, base, and subbase required to limit subgrade frost penetration. As with any natural climatic phenomenon, winters that are colder than average occur with a frequency that decreases as the degree of departure from average becomes greater. A mean freezing index cannot be computed where temperatures in some of the winters do not fall below freezing. A design method has been adopted, therefore, that uses the average air freezing index for the 3 coldest years in a 30-year period (or for the coldest winter in 10 years of record) as the design freezing index to determine the thickness of protection that will be provided.

b. The design method permits a small amount of frost penetration into frost-susceptible subgrades for the design freezing index year. The procedure is described in the following subparagraphs.

(1) Estimate average moisture contents in the base course and subgrade at the start of the freezing period and estimate the dry unit weight of base. As the base course may in some cases comprise successive layers containing substantially different fines contents, the average moisture content and dry unit weight should be weighted in proportion to the thicknesses of the various layers. Alternatively, the average may be assumed to be equal to the moisture content and dry unit weight of the material in the unbound base course.

(2) From figure 3-4, determine frost penetration a, which would occur in the design freezing index year in a base material of unlimited depth beneath a 12-inch thick rigid pavement or bituminous pavement kept free of snow and ice. Use straight line interpolation where necessary. For rigid pavements greater than 12 inches in thickness, deduct 10 degree-days for each inch of pavement exceeding 12 inches from the design freezing index before entering figure 3-4 to determine frost penetration a. Then add the extra concrete pavement thickness to the determined frost penetration.

(3) Compute base thickness c (fig 4-1) required for zero frost penetration into the subgrade as follows:

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

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

(5) Enter figure 4-1 with c as the abscissa and, at the applicable value of r, find in the left scale the design base thickness b that will result in the allowable subgrade frost penetration s shown on the right scale. For airfield runways, if computed r is equal to or exceeds 2.0, use 2.0 in figure 4-1. For other pavements, if r is equal to or exceeds 3.0, use 3.0 in figure 4-1.

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

d. When the maximum combined thickness of pavement and base required by this design procedure exceeds 60 inches, consideration should be given to alternatives such as the following:

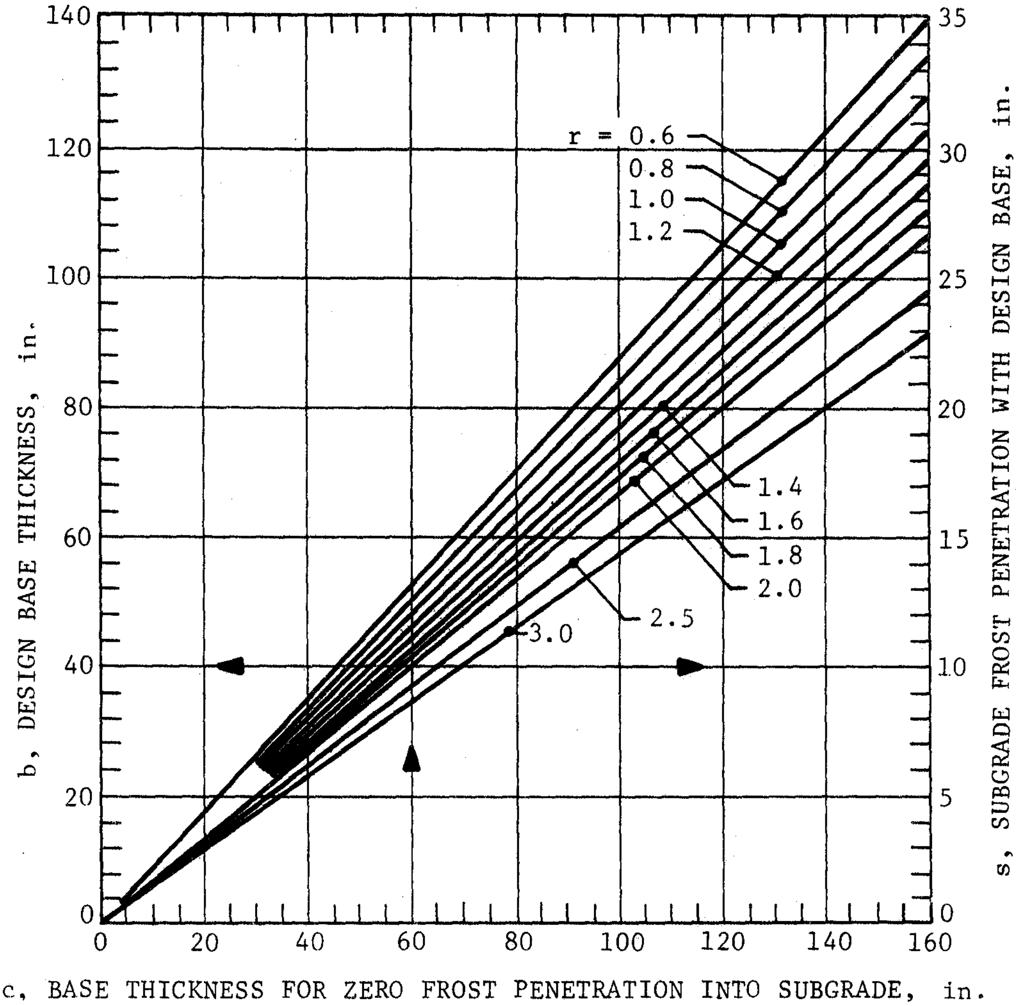
- Limiting total combined thickness to 60 inches and, in rigid-type pavements, using steel reinforcement to prevent large cracks.

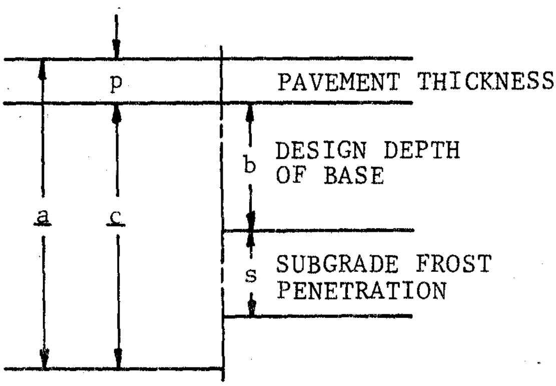
- Limiting total combined thickness to 60 inches and, in rigid-type pavements, limiting the maximum slab dimensions (as to 15 feet) without use of reinforcement.

- Reducing the required combined thickness by use of a subbase of uniform fine sand, with high moisture retention when drained, in lieu of a more free-draining material.

The first two of these alternatives would result in a greater surface roughness than obtained under the basic design method because of greater subgrade frost penetration. With respect to the third alternative, it should be noted that base course drainage requirements of EM 1110-3-136 must still be met.

e. If the combined thickness of pavement and base required by the non-frost-criteria exceeds the thickness given by the limited subgrade





a = Combined thickness of pavement and non-frost-susceptible base for zero frost penetration into subgrade

c = a-p

w_b= Water content of base

w_s= Water content of subgrade

$r = \frac{w_s}{w_b}$, Not to exceed 2.0 for Type A and B areas on airfields and 3.0 for other pavements

EXAMPLE: If c = 60" and r = 2.0, then b = 40" and s = 10"

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FIGURE 4-1: THICKNESS OF NON-FROST-SUSCEPTIBLE BASE FOR LIMITED SUBGRADE FROST PENETRATION frost penetration procedure of design, the greater thickness given by the non-frost-criteria will be adopted as the design thickness.

f. The base course composition requirements should be rigorously followed. The design base thickness determined is the total thickness of filter layers, granular unbound base and subbase, and any bound base. The thickness of the asphalt surfacing layer and of any bound base, as well as the CBR (California Bearing Ratio) requirements of each layer of granular unbound base, will be determined as set forth in EM 1110-3-131 and EM 1110-3-141. The thickness of rigid pavement slab will be determined from EM 1110-3-132 and EM 1110-3-142.

4-4. Design for reduced subgrade strength - airfields and roads. Thickness design may also be based on the seasonally varying subgrade support that includes sharply reduced values during thawing of soils that have been affected by frost action. Excepting pavement projects for heavyload aircraft or those that are located in regions of low design freezing index, this design procedure usually requires less thickness of pavement and base than that needed for limited subgrade frost penetration. The method may be used for both flexible and rigid pavements wherever the subgrade is reasonably uniform or can be made reasonably horizontally uniform by the required techniques of subgrade preparation. This will prevent or minimize significant or objectionable differential heaving and resultant cracking of pavements. When the reduced subgrade strength method is used for F4 subgrade soils, unusually rigorous control of subgrade preparation must be required. When a thickness determined by the reduced subgrade strength procedure exceeds that determined for limited subgrade frost penetration, the latter, smaller value should be used, provided it is at least equal to the thickness required for non-frost-conditions. In situations where use of the reduced subgrade strength procedure might result in objectionable frost heave, but use of the greater thickness of base course indicated by the limited subgrade frost penetration design procedure is not considered necessary, intermediate design thicknesses may be used. However, these must be justified on the basis of frost heaving experience developed from existing airfield and highway pavements where climatic and soil conditions are comparable.

a. Thickness of flexible pavements. In the reduced subgrade strength procedure for design, the curves in EM 1110-3-141 should be used to determine the combined thickness of flexible pavement and base required for aircraft wheel loads and wheel assemblies, and the design curves of EM 1110-3-131 should be used for highway and parking area design. In both cases, the curves should not be entered with subgrade CBR values determined by tests or estimates but instead with the applicable frost-area soil support index from table 4-1. Frost-area soil support indices are used as if they were CBR values; the term CBR is not applied to them, however, because, being weighted average values for an annual cycle, their value cannot be determined by CBR tests.

(1) General field data and experience indicate that on the relatively narrow embankments of highways, reduction in strength of subgrades during frost melting may be less in substantial fills than in cuts because of better drainage conditions and less intense ice segregation. If local field data and experience show this to be the case, then a reduction in combined thickness of pavement and base of up to 10 percent may be permitted for highways on substantial fills.

(2) EM 1110-3-141 and EM 1110-3-131 should also be used to determine the thicknesses of individual layers in the pavement system and to ascertain whether it will be advantageous to include one or more layers of bound base in the system.

Table 4-1. Frost-area Soil Support Indices for Flexible Pavement Design

Frost group of subgrade soil	F1	F2	F3 and F4
Frost-area soil support index	9.0	6.5	3.5

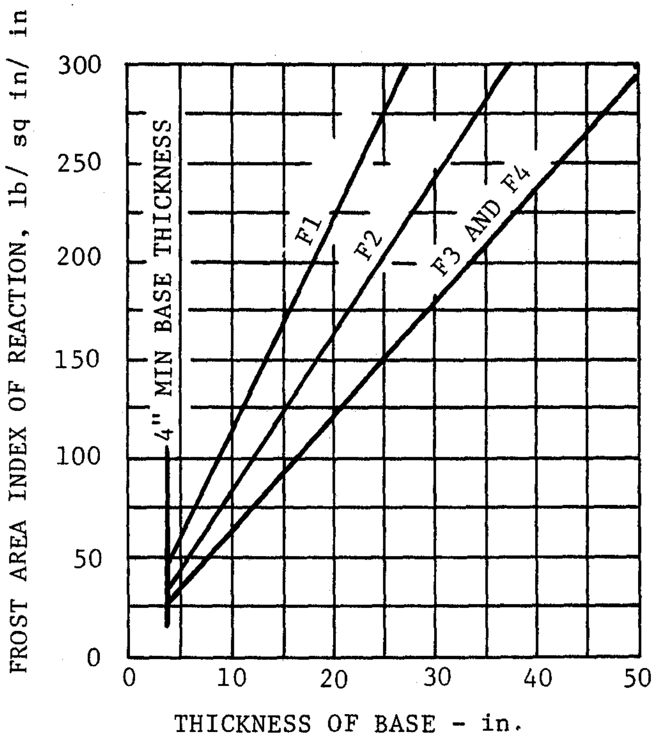
b. Thicknesses of rigid pavements. Where frost is expected to penetrate into a frost-susceptible subgrade beneath a rigid pavement, it is good practice to use a non-frost-susceptible base course at least equal in thickness to the slab. Experience has shown, however, that rigid pavements with only a 4-inch base have performed well in cold environments with relatively uniform subgrade conditions. Accordingly, where subgrade soils can be made reasonably uniform by the required procedures of subgrade preparation, the minimum thickness of granular unbound base should be 4 inches.

(1) Additional granular unbound base course, giving a thickness greater than the minimum specified above, will improve pavement performance, giving a higher frost-area index of reaction on the surface of the unbound base (fig 4-2) and permitting a pavement slab of less thickness. Bound base also has significant structural value and may be used to effect a further reduction in the required thickness of the pavement slab. EM 1110-3-142 and EM 1110-3-132 establish criteria for determination of the required thickness of rigid pavement slabs in combination with a bound base course. The provisions presented herein comprising requirements for granular unbound base as drainage and filter layers will still be applicable.

(2) The thickness of concrete pavement will be determined in accordance with EM 1110-3-142 for airfields and EM 1110-3-132 for roads and parking areas, using the frost-area index of reaction determined from figure 4-2. This figure shows the equivalent weighted average index of reaction values for an annual cycle that includes a period of thaw-weakening in relation to the thickness of base. Frost-area indices of reaction are used as if they were moduli of reaction, k, and have the same units. The term modulus of reaction is not applied to

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

NOTE: FOR DESIGN OVER F4 SUBGRADE SOILS SEE TEXT



FROST CONDITION REDUCED SUBGRADE STRENGTH DESIGN SUBGRADE MODULUS CURVES FOR RIGID AIRFIELD AND HIGHWAY PAVEMENTS

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FIGURE 4-2. FROST-AREA INDEX OF REACTION FOR DESIGN OF RIGID AIRFIELD AND HIGHWAY PAVEMENTS them, however, because being weighted average values for an annual cycle, they cannot be determined by a plate-bearing test. If the modulus of reaction, k, determined from tests on the equivalent base course and subgrade, but without frost melting, is numerically smaller than the index of reaction obtained from figure 4-2, the test value should govern the design.

4-5. Design of flexible pavement for runway overruns.

a. Frost condition requirements. A runway overrun pavement must be designed to withstand occasional emergency aircraft traffic in the form of short or long landings, aborted takeoffs, and possible barrier engagements. The pavement must also serve various maintenance vehicles such as crash trucks and snowplows. The design of an overrun must provide:

- Adequate stability for very infrequent aircraft loading during the frost-melting period.

- Adequate stability for normal traffic of snow-removal equipment and possibly other maintenance vehicles during frost-melting periods.

- Sufficient thickness of base or subbase materials of low heave potential to prevent unacceptable roughness during freezing periods.

b. Overrun design for reduced subgrade strength. To provide adequate strength during frost-melting periods, the flexible pavement and base course shall have the combined thickness given by the design curves in EM 1110-3-141 enter the curves with the applicable frost-area soil support index given in table 4-1. The thickness established by this procedure should have the following limitations:

- It should not be less than required for non-frost-condition design in overrun areas, as determined from EM 1110-3-141.

- It should not exceed the thickness required under the limited subgrade frost penetration design method.

- It should not be less than that required for normal operation of snowplows and other medium to heavy trucks.

The subgrade preparation techniques and transition details of this manual are required for overrun pavements. The composition of the layered pavement structure should conform with the applicable requirements of EM 1110-3-141, except that the composition of base courses should also conform with the requirements of this manual.

c. Overrun design for control of surface roughness. In locations with low to moderate design freezing indices, thicknesses smaller than those required by the reduced strength method may be given by the limited subgrade frost penetration method of design. If this happens, the latter should be used, but in no case will combined thicknesses smaller than those given for non-frost-design by EM 1110-3-141 be adopted. On the other hand, in some instances, local experience may indicate that a design thickness determined by the reduced subgrade strength method, coupled with the required subgrade preparation procedures and transitions will not restrict maximum differential frost heave to an amount which is reasonable for these emergency areas, generally not more than about 3 inches in 50 feet. In the selection of a design for restricting frost heave, consideration must be given to type of subgrade material, availability of water, depth of frost penetration, and local experience. Guidance is provided in the following subparagraphs.

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

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

(3) The criteria set forth for control of surface roughness apply only if they require a combined pavement and base thickness in excess of that needed for adequate load supporting capacity.

4-6. Design of shoulder pavements.

a. Pavement thickness design and composition of base courses. Where paved shoulders are required on airfields, the flexible pavement and base should have the combined thickness given by the design curve in EM 1110-3-141; enter the curve with the applicable frost-area soil support index shown in table 4-1. If the subgrade is highly susceptible to heave, local experience may indicate a need for a pavement section that incorporates an insulating layer or for additional granular unbound material to moderate the irregularity of pavement deformations resulting from frost heave.

b. Control of differential heave at small structures located within shoulder pavements. To prevent objectionable heave of small structures inserted in shoulder pavements, such as drain inlets and bases for airfield lights, the pavement substructure, extending at least 5 feet radially from them, should be designed and constructed entirely with non-frost-susceptible base and subbase course materials of sufficient thickness to prevent subgrade freezing. Gradual transitions are required. Alternatively, synthetic insulation could be placed below a base of the minimum prescribed thickness to prevent the advance of freezing temperatures into the subgrade; suitable transitions to the adjoining uninsulated pavement would be needed.

4-7. Use of state highway requirements for roads, streets, and open storage areas. To provide further flexibility in design options and to exploit economical local materials and related experience, state highway requirements may be used for pavements with a design index less than 4. Design index is defined in EM 1110-3-131 and EM 1110-3-132. The decision to use local state highway requirements will be based on demonstrated satisfactory performance of pavements in that state as determined by observation and experience. This should give reasonable assurance that the life cycle cost resulting from use of state highway requirements is comparable to that from use of Corps of Engineers criteria and procedures. If state requirements are used, the entire pavement should conform in every detail to the applicable state criteria.

CHAPTER 5

BASE COURSE COMPOSITION REQUIREMENTS

5-1. Free-draining material directly beneath bound base or surfacing layer. Base courses may be made up of either granular unbound materials or bound base materials or a combination of the two. However, a cement- or lime-bound base should not be placed directly beneath bituminous pavement. Also, an unbound base course will not be placed between two relatively impervious bound layers. If the combined thickness, in inches, of pavement and contiguous bound base courses is less than 0.09 multiplied by the design air freezing index (this calculation limits the design freezing index at the bottom of the bound base to about 20 degree-days), not less than 4 inches of free-draining material should be placed directly beneath the lower layer of bound base or, if there be no bound base, directly beneath the pavement slab or surface course. The free-draining material should contain 2.0 percent or less, by weight, of grains that can pass the No. 200 sieve, and to meet this requirement it probably will have to be screened and washed. The material in the 4-inch layer must also conform with filter requirements. If the structural criteria for design of the pavement do not require granular unbound base other than the 4 inches of free draining material, the material in the 4-inch layer must be checked for conformance with the filter requirements. If it fails the test for conformance, an additional layer meeting those requirements must be provided.

5-2. Other granular unbound base course. If the structural criteria for design of the pavement require more granular unbound base than the 4 inches of free draining material, the material should meet the applicable requirements of current guide specifications for base or subbase materials. In addition, the top 50 percent of the total thickness of granular unbound base must be non-frost-susceptible and must contain not more than 5 percent by weight of particles passing a No. 200 sieve. The lower 50 percent of the total thickness of granular unbound base may be either non-frost-susceptible material, S1 material, or S2 material. If the subgrade soil is S1 or S2 material meeting the requirements of current guide specifications for base or subbase, the lower 50 percent of granular base will be omitted. An additional requirement, if subgrade freezing will occur, is that the bottom 4-inch layer in contact with the subgrade must meet filter requirements, or a geotextile fabric meeting the filter requirements must be placed in contact with the subgrade. The dimensions and permeability of the base should satisfy the base course drainage criteria given in EM 1110-3-136 as well as the thickness requirements for frost design. Thicknesses indicated by frost criteria should be increased if necessary to meet subsurface drainage criteria. Base course materials of borderline quality should be tested frequently after compaction to insure that the materials meet these design criteria.

5-3. Use of F1 and F2 soils for base materials for roads and parking areas. A further alternative to the use of S1 and S2 base materials is permitted for roads and vehicle parking areas. Materials of frost groups F1 and F2 may be used in the lower part of the base over F3 and F4 subgrade soils. F1 materials may be used in the lower part of the base over F2 subgrades. The thickness of F2 base material should not exceed the difference between the reduced-subgrade-strength thickness requirements over F3 and F2 subgrades. The thickness of F1 base should not exceed the difference between the thickness requirements over F2 and F1 subgrades. Any F1 or F2 material used in the base must meet the applicable requirements of the guide specifications for base or subbase materials.

5-4. Filter over subgrade.

a. Granular filters. For both flexible and rigid pavements under which subgrade freezing will occur, at least the bottom 4 inches of granular unbound base should consist of sand, gravelly sand, screenings, or similar material. It should be designed as a filter between the subgrade soil and overlying base course material to prevent mixing of the frost-susceptible subgrade with the base during and immediately following the frost-melting period. This filter is not intended to serve as a drainage course. The gradation of this filter material should be determined in accordance with criteria presented in EM 1110-1-136, with the added overriding limitation that the material must be non-frost-susceptible or of frost group S1 or S2. Experience shows that a fine-grained subgrade soil will work up into a coarse, open-graded overlying gravel or crushed stone base course under the kneading action of traffic during the frost-melting period if a filter course is not provided between the subgrade and the overlying material. Experience and tests indicate that well-graded sand is especially suitable for this filter course. The 4-inch minimum filter thickness is dictated primarily by construction requirements and limitations. Greater thicknesses should be specified when required to suit field conditions. Over weak subgrades, a 6-inch or greater thicknesses may be necessary to support construction equipment and to provide a working platform for placement and compaction of the base course.

b. Geotextile fabric filters. The use of geotextile fabrics in lieu of a granular filter is encouraged. No structural advantage will be attained in the design when a geotextile fabric is used; it serves as a separation layer only.

5-5. Filter under pavement slab. For rigid pavements, all-bituminous-concrete pavements and pavements whose surfacing materials are constructed directly over bound base courses, not more than 85 percent of the filter or granular unbound base course material placed directly beneath the pavement or bound base course should be finer than 2.00 millimeters in diameter (U.S. standard No. 10 sieve) for a minimum thickness of 4 inches. The purpose of this requirement is to prevent loss of support by the pumping of soil through joints and cracks.

CHAPTER 6

USE OF STABILIZED SOILS IN FROST AREAS

6-1. Stabilizers and stabilized layers.

a. Additives. Asphalt, portland cement, lime, and LCF are the most common additives used in stabilized soils. The limitations of use, the basic requirements for mixture design, and the stabilization procedures using bituminous and chemical stabilizers are set forth in EM 1110-3-137. Special or supplemental requirements related to frost areas are outlined in the following paragraphs.

b. Limitations of use. In frost areas, stabilized soil in most cases will be used only in a layer or layers making up one of the upper elements of a pavement system. Usually, it will be placed directly beneath the pavement surfacing layer, where the added cost of stabilization is compensated for by its structural advantage in effecting a reduction in the required thickness of the pavement system. However, a cement, lime, or LCF-stabilized base should not be placed directly beneath bituminous pavements because cracking and faulting will be significantly increased. Treatment with a lower degree of chemical stabilization in layers placed at lower levels within the pavement system should be used in frost areas only with caution and after intensive tests. This is because weakly cemented material usually has less capacity to endure repeated freezing and thawing without degradation than firmly cemented material. A possible exception is the use of a low level of stabilization to improve a soil that will be encapsulated within an impervious envelope as part of a membrane encapsulated soil layer (MESL) pavement system (app C). The limited experience to date suggests that a soil that is otherwise unsuitable for encapsulation, because moisture migration and thaw weakening are excessive, may be made suitable for such use by moderate amounts of a stabilizing additive. Materials that are modified by small amounts of chemical additive also should be intensively tested to make sure that the improved material is durable through repeated freeze-thaw cycles and that the improvement is not achieved at the expense of making the soil more susceptible to ice segregation.

c. Construction cut-off dates. For materials stabilized with cement, lime, or LCF, whose strength increases with length of curing time, it is essential that the stabilized layer be constructed sufficiently early in the season to allow development of adequate strength before the first freezing cycle begins. Research has shown that the rate of strength gain is substantially lower at 50 degrees F., for example, than at 70 degrees or 80 degrees F. Accordingly, in frost areas it is not always enough to protect the mixture from freezing during a 7-day curing period as required by the applicable guide specifications. A construction cut-off date well in advance of the onset of freezing may be essential. General guidance for estimating reasonable cut-off construction dates that will allow time for development of frost-resistant bonds are presented in Transportation Research Records 442, 612, and 641.

6-2. Stabilization with lime and with LCF.

a. Bound base. Soils containing only lime as the stabilizer are generally unsuitable for use as base course layers in the upper layers of pavement systems in frost areas, except possibly in a MESL pavement system as mentioned above. Lime, cement, and a pozzolanic material such as flyash may be used in some cases to produce a cemented material of high quality that is suitable for upper base course and that has adequate durability and resistance to freeze-thaw action. In frost areas, LCF mixture design will be based on the procedures set forth in EM 1110-3-137, with the additional requirement that the mixture, after freeze-thaw testing as set forth below, should meet the weight-loss criteria specified in EM 1110-3-137 for cement-stabilized soil. The procedures of ASTM D 560 should be followed for freeze-thaw testing, except that the specimens should be compacted in a 6-inch diameter mold in five layers with a 10-pound hammer having an 18-inch drop and that the preparation and curing of the specimens should follow the procedures indicated in EM 1110-3-137 for unconfined compression tests on lime-stabilized soil.

b. Lime-stabilized soil. If it is economical to use lime-stabilized or lime-modified soil in lower layers of a pavement system, a mixture of adequate durability and resistance to frost action is still necessary. In addition to the requirements for mixture design of lime-stabilized and lime-modified subbase and subgrade materials set forth in EM 1110-3-137, cured specimens should be subjected to the freeze-thaw cycles of ASTM D 560 as modified by EM 1110-3-137 (but omitting wire-brushing) or other applicable freeze-thaw procedures.

6-3. Stabilization with portland cement. Cement-stabilized soil meeting the requirements set forth in EM 1110-3-137, including freeze-thaw effects tested under ASTM D 560, may be used in frost areas as base course or as stabilized subgrade. Cement-modified soil conforming with the requirements of EM 1110-3-137 also may be used in frost areas.

6-4. Stabilization with bitumen. Many different types of soils and aggregates can be successfully stabilized to produce a high-quality bound base with a variety of types of bituminous material. In frost areas, the use of tar as a binder should be avoided because of its high temperature-susceptibility. Asphalts are affected to a lesser extent by temperature changes, but a grade of asphalt suitable to the prevailing climatic conditions should be selected (app D). Excepting these special conditions affecting the suitability of particular types of bitumen, the procedures for mixture design set forth in EM 1110-3-137, EM 1110-1-131, and EM 1110-3-141 usually will insure that the asphalt-stabilized base will have adequate durability and resistance to moisture and freeze-thaw cycles.

CHAPTER 7

SUBGRADE PREPARATION AND TRANSITIONS FOR CONTROL OF FROST HEAVING AND ASSOCIATED CRACKING

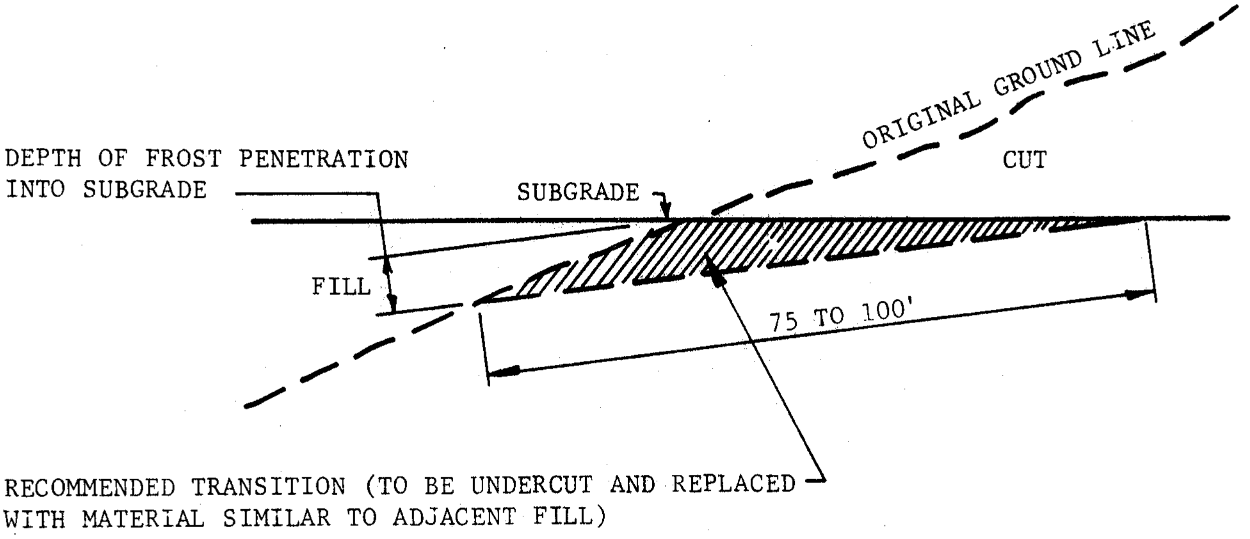
7-1. Subgrade preparation. It is a basic requirement for all pavements constructed in frost areas that subgrades in which freezing will occur should be especially prepared to achieve uniformity of soil conditions. In fill sections, the least frost-susceptible soils should be placed in the upper portion of the subgrade by temporarily stockpiling the better materials, cross-hauling, and selective grading. If the upper layers of fill contain frost-susceptible soils, the completed fill section should be subjected to the subgrade preparation procedures required for cut sections. In cut sections, the subgrade should be scarified and excavated to a prescribed depth, and the excavated material should be windrowed and bladed successively until thoroughly blended, and relaid and compacted. The depth of subgrade preparation, measured downward from the top of the subgrade, should be the lesser of either 24 inches, or two-thirds of the frost penetration given by figure 3-4 (except one-half of the frost penetration for airfield shoulder pavements and for roads, streets and open storage areas of Class D and E) less the actual combined thickness of pavement, base course, and subbase course, or 72 inches less the actual combined thickness of pavement, base, and subbase. At transitions from cut to fill, the subgrade in the cut section should be undercut and back-filled with the same material as the adjacent fill (fig 7-1). Refer to appendix E for field control of subgrade and base course materials.

a. Exceptional conditions. Exceptions to the basic requirement for subgrade preparation in the preceding paragraph are limited to the following:

(1) Subgrades known to be non-frost-susceptible to the depth prescribed for subgrade preparation and known to contain no frost-susceptible layers or lenses, as demonstrated and verified by extensive and thorough subsurface investigations and by the performance of nearby existing pavements, if any, are exceptions.

(2) Fine-grained subgrades containing moisture well in excess of the optimum for compaction, with no feasible means of drainage nor of otherwise reducing the moisture content, and which consequently cannot feasibly be scarified and recompacted, are also exceptions.

b. Treatment of wet fine-grained subgrades. If wet fine-grained subgrades exist at the site, it will be necessary to achieve equivalent frost protection with fill material. This may be done by raising the grade by an amount equal to the depth of subgrade preparation that otherwise would be prescribed or by undercutting and replacing the wet fine-grained subgrade to that same depth. In either case, the fill or



SOURCE: MAINE STATE HIGHWAY COMMISSION

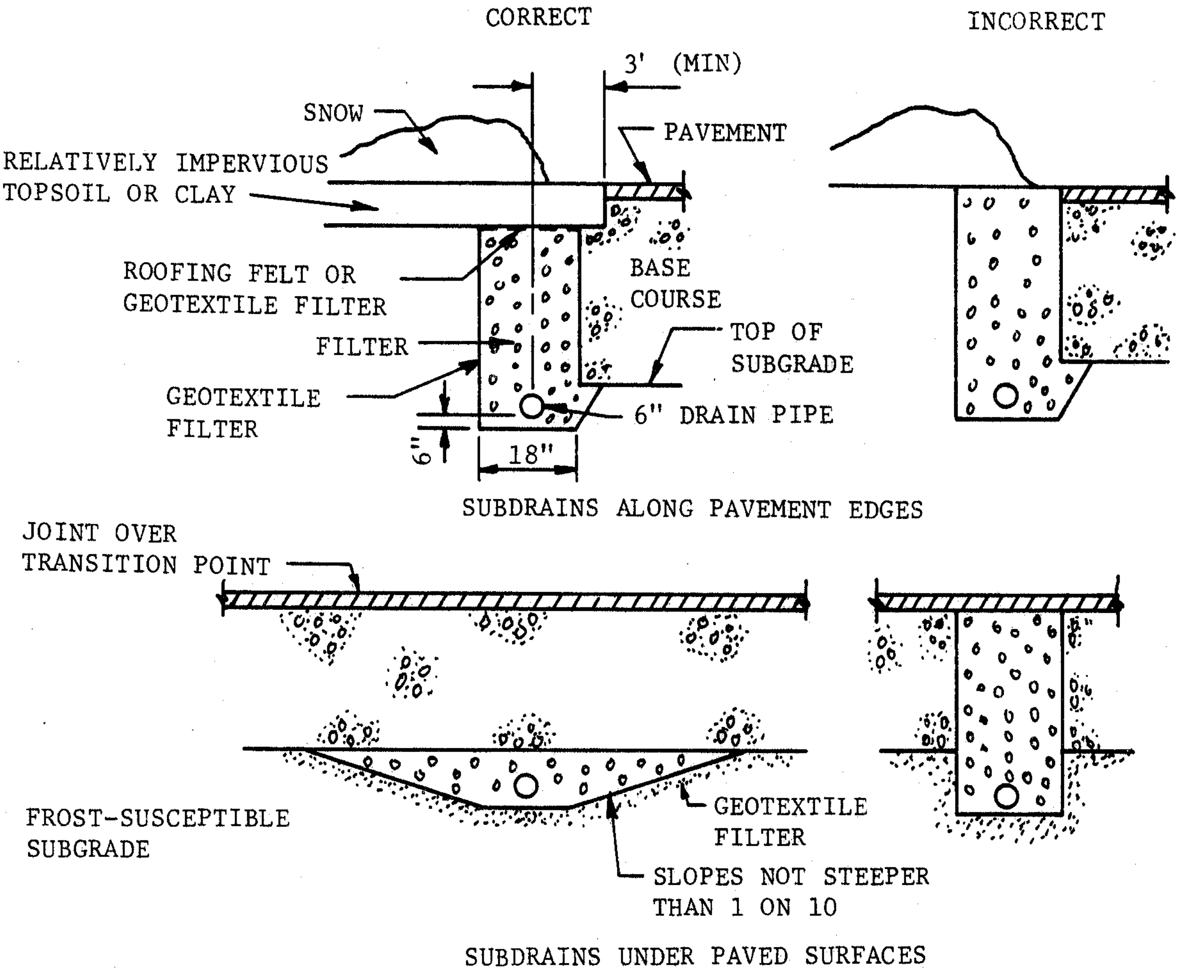
FIGURE 7-1. TAPERED TRANSITION USED WHERE EMBANKMENT MATERIAL DIFFERS FROM NATURAL SUBGRADE IN CUT backfill material may be non-frost-susceptible material or frost-susceptible material meeting specified requirements. If the fill or backfill material is frost-susceptible, it should be subjected to the same subgrade preparation procedures prescribed above.

c. Boulder removal. It is essential that all stones more than about 6 inches in diameter be removed from frost-susceptible subgrades to prevent boulder heaves from damaging the pavement. In the process of constructing fills, all large stones should be removed from subgrade materials that will experience freezing. In cut sections, all large stones should be removed from the subgrade to the same depth as the special subgrade preparation outlined in the preceding paragraphs.

7-2. Control of differential heave at drains, culverts, ducts, inlets, hydrants, and lights.

a. Design details and transitions for drains, culverts, and ducts. Drains, culverts, or utility ducts placed under pavements on frost-susceptible subgrades frequently experience differential heaving. Wherever possible, the placing of such facilities beneath pavements should be avoided. Where this cannot be avoided, construction of drains should be in accordance with the "correct" method indicated in figure 7-2, while treatment of culverts and large ducts should conform with figure 7-3. All drains or similar features should be placed first and the base and subbase course materials carried across them without break so as to obtain maximum uniformity of pavement support. The practice of constructing the base and subbase course and then excavating back through them to lay drains, pipes, etc., is unsatisfactory as a marked discontinuity in support will result. It is almost impossible to compact material in a trench to the same degree as the surrounding base and subbase course materials. Also, the amount of fines in the excavated and backfilled material may be increased by incorporation of subgrade soil during the trench excavation or by manufacture of fines by the added handling. The poor experience record of combination drains--those intercepting both surface and subsurface water--indicates that the filter material should never be carried to the surface as illustrated in the "incorrect" column in figure 7-2. Under winter conditions, this detail may allow thaw water accumulating at the edge of the pavement to feed into the base course. This detail is also undesirable because the filter is a poor surface and is subject to clogging, and the drain is located too close to the pavement to permit easy repair. Recommended practice is shown in the "correct" column in figure 7-2.

b. Frost protection and transitions for inlets, hydrants, and lights. Experience has shown that drain inlets, fueling hydrants, and pavement lighting systems, which have different thermal properties than the pavements in which they are inserted, are likely to be locations of abrupt differential heave. Usually, the roughness results from progressive movement of the inserted items. To prevent these damaging



NOTES:
1. FOR ADDITIONAL DETAILS ON DESIGN AND DEPTH OF SUBDRAINS AND DEPTH OF SUBDRAINS AND FILTERS COURSES SEE EM 1110-3-136.
2. GRANULAR OR GEOTEXTILE FABRICS FILTER MAY BE NECESSARY BETWEEN BASE COURSE AND SUBGRADE (PARA 5-4).
3. UPPER 4 INCHES OF BASE COURSE MUST HAVE FREE-DRAINING CHARACTERISTICS (PARA 5-1).

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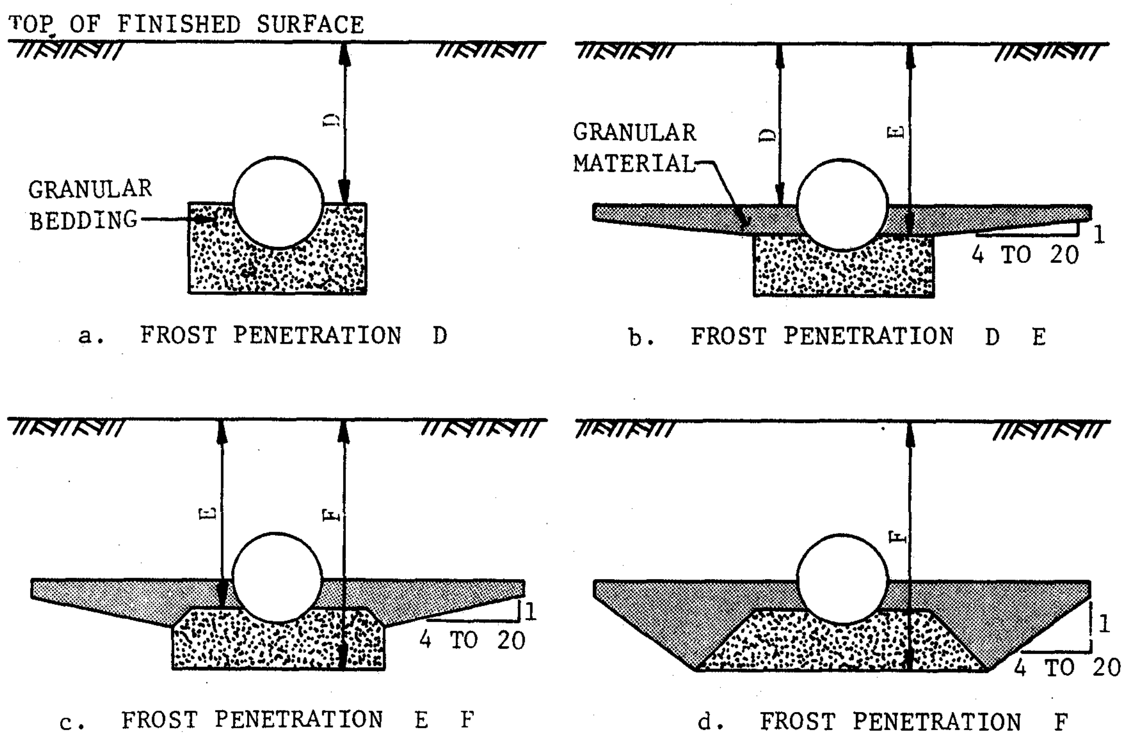
FIGURE 7-2. SUBDRAIN DETAILS FOR COLD REGIONS



SOURCE: MINNESOTA DEPARTMENT OF HIGHWAYS

FIGURE 7-3. TRANSITIONS FOR CULVERTS BENEATH PAVEMENTS movements, the pavement section beneath the inserts and extending at least 5 feet radially from them should be designed to prevent freezing of frost-susceptible materials by use of an adequate thickness of non-frost-susceptible base course, and by use of insulation. Consideration should also be given to anchoring footings with spread bases at appropriate depths. Gradual transitions are required to surrounding pavements that are subject to frost heave.

7-3. Pavement thickness transitions.

a. Longitudinal transitions. Where interruptions in pavement uniformity cannot be avoided, differential frost heaving should be controlled by use of gradual transitions. Lengths of longitudinal transitions should vary directly with the speed of traffic and the amount of heave differential; for rigid pavements, transition sections should begin and end directly under pavement joints, and should in no case be shorter than one slab length. As an example, at a heavy-load airfield where differentials of heave of 1 inch may be expected at changes in combined thickness of pavement and base, or at changes from one subgrade soil condition to another, gradual changes in base thicknesses should be effected over distances of 200 feet for the runway area, 100 feet for taxiways, and 50 feet for aprons. The transition in each case should be located in the section having the lesser total thickness of pavement and base. Pavements designed to lower standards of frost-heave control, such as roads, shoulders, and overruns, have less stringent requirements, but may nevertheless need transition sections.

b. Transverse transitions. A need for transitions in the transverse direction arises at changes in total thickness of pavement and base, and at longitudinal drains and culverts. Any transverse transition beneath pavements that carry the principal wheel assemblies of aircraft traveling at moderate to high speed should meet the same requirements applicable to longitudinal transitions. Transverse transitions should be sloped not steeper than 10 horizontal to 1 vertical. Transverse transitions between pavements carrying aircraft traffic and adjacent shoulder pavements should be located in the shoulder and should not be sloped steeper than 4 horizontal to 1 vertical.

7-4. Other measures. Other possible measures to reduce the effects of heave are use of insulation to control depth of frost penetration and use of steel reinforcement to improve the continuity of rigid pavements that may become distorted by frost heave. Reinforcement will not reduce heave nor prevent the cracking resulting from it, but it will help to hold cracks tightly closed and thus reduce pumping through these cracks. Transitions between cut and fill, culverts and drains, changes in character or stratification of subgrade soils, as well as subgrade preparation and boulder removal should also receive special attention in field construction control (app E).

7-5. Pavement cracking associated with frost action. One of the most detrimental effects of frost action on a pavement is surface distortion as the result of differential frost heave or differential loss of strength. These may also lead to random cracking. For airfield pavements, it is essential that uncontrolled cracking be reduced to the minimum. Deterioration and spalling of the edges of working cracks are causes of uneven surface conditions and sources of debris that may seriously damage jet aircraft and engines. Cracking may be reduced by control of such elements as base composition, uniformity and thickness, slab dimensions, subbase and subgrade materials, uniformity of subsurface moisture conditions, and, in special situations, by use of reinforcement and by limitation of pavement type. The importance of uniformity cannot be overemphasized. Where unavoidable discontinuities in subgrade conditions exist, gradual transitions as outlined in preceding paragraphs are essential.

CHAPTER 8

EXAMPLES OF PAVEMENT DESIGN

8-1. Example 1. Heavily trafficked road. Design flexible and rigid pavements for the following conditions:

- Class B (rolling terrain within the "built-up area").

- Category III.

- Design index: 4 (for flexible pavements).

  3 (for rigid pavements).

- Design air freezing index: 700 degree-days.

- Subgrade material: uniform sandy clay, CL; plasticity index, 18; frost group, F3; water content, 20 percent (average); normal-period CBR, 10; normal-period modulus of subgrade reaction k = 200 psi/inch on subgrade and 400 psi/inch on 24 inches of base course.

- Base course material: crushed gravel (GW), normal-period CBR=80, 30 percent passing no. 10 sieve, 1 percent passing No. 200 sieve.

- Subbase course material: course to fine silty sand (SP-SM), normal-period CBR=20, 11 percent passing No. 200 sieve, 6 percent finer than 0.02 millimeters, frost classification S2, meets filter criteria for material in contact with subgrade.

- Average dry unit weight (good quality base and subbase): 135 pcf.

- Average water content after drainage (good quality base and subbase): 5 percent.

- Highest ground water: about 4 feet below surface of subgrade.

- Concrete flexural strength: 650 psi.

Since this pavement has a design index of 4 or less, criteria in local highway department requirements may be used in lieu of criteria in EM 1110-3-131 and EM 1110-3-132. Local experience with existing pavements indicates that frost heave has been relatively uniform.

a. Flexible pavement design by limited subgrade frost penetration method. From figure 3-4, the combined thickness a of pavement and base to prevent freezing of the subgrade in the design freezing index year is 45 inches. According to criteria in EM 1110-3-131, the minimum pavement thickness is 2 inches over a CBR=80 base course that must be at least 4 inches thick. The ratio of subgrade to base water content is r = 20/5 = 4. Since this is a highway pavement, the maximum allowable r of 3 is used in figure 4-1 to obtain the required thickness of base b of 24 inches, which would allow about 6 inches of frost penetration into the subgrade in the design year. Subgrade preparation would not be required since the combined thickness of pavement and base is more than one-half the thickness required for complete protection.

b. Flexible pavement design by reduced subgrade strength method. From table 4-1 the frost-area soil support index is 3.5, which from the design curve in EM 1110-3-131, yields a required combined thickness of pavement and base of 19 inches. Since this is less than the (2 + 24) 26-inch thickness required by the limited subgrade frost penetration method, the 19-inch thickness would be used. The pavement structure could be composed of the following: 2 inches of asphalt concrete, 9 inches of crushed gravel, (since the crushed gravel contains only 1 percent passing the No. 200 sieve, it also serves as the free-draining layer directly beneath the pavement) and 9 inches of the silty sand subbase material. Subgrade preparation would be required to a depth of 1/2 x 45 - 19 = 3-1/2 inches.

c. Rigid pavement design by limited subgrade frost penetration method. From EM 1110-3-132 the required pavement thickness p, based on the normal-period k = 400 psi per inch, the concrete flextural strength of 650 psi and the design index of 3, is 5.0 inches. From figure 3-4, the combined thickness of pavement and base is 45 inches, equivalent to that for the flexible pavement. By use of r = 3 in figure 4-1, the required thickness of base b is 23 inches, which would allow about 6 inches of frost penetration into the subgrade in the design year. No subgrade preparation would be required.

d. Rigid pavement design by the reduced subgrade strength method. Since frost heave has not been a major problem, a minimum of 4 inches of the free-draining base course material could be used, plus 4 inches of the subbase that will serve as a filter material on the subgrade. For this case, the frost-area index of reaction would be 50 psi per inch (fig 4-2), requiring a pavement slab 8 inches thick. Subgrade preparation to a depth of 1/2 x 45 - 16 = 6-1/2 inches would be required.

e. Alternative designs. Other designs using stabilized layers, including all bituminous concrete pavements, should be investigated to determine whether they are more economical than the designs presented above.

8-2. Example 2. Lightly trafficked road. Design flexible pavements for the following conditions:

- Class E (flat terrain within the "open" area)

- Category III

- Design index: 2 (from EM 1110-3-131).

- Design air freezing index: 1,500 degree-days.

- Subgrade material: fine silty sand, SM, nonplastic; frost group, F4; water content, 15 percent (average); normal-period CBR, 15.

- Base course material: gravel (GW), normal-period CBR=80, 30 percent passing No. 10 sieve and 3 percent passing the No. 200 sieve.

- Subbase course material: Coarse to fine silty sand (SP-SM), normal-period CBR=20, 10 percent passing No. 200 sieve, 5 percent finer than 0.02 millimeters, frost classification S2, meets filter criteria for material in contact with subgrade.

- Average dry unit weight of the base and subbase: 125 pcf.

- Average water content of the base and subbase after drainage: 7 percent.

- Select borrow material: Silty sand (SM), normal period CBR=15, 25 percent passing No. 200 sieve, 15 percent finer than 0.02 millimeters; frost classification F2, meets filter criteria for materials in contact with subgrade.

- Highest ground water: approximately 3 feet below surface of subgrade.

a. Limited subgrade frost penetration method. By use of the procedure outlined in example 1, paragraph 8-1, the combined thickness of pavement and base a to prevent freezing of the subgrade in the design year is 70 inches, which was determined by interpolation between the soils having densities of 115 and 135 pcf. From EM 1110-3-131, the minimum pavement thickness over an 80 CBR base course is 1-1/2 inches. From figure 4-1, the design base thickness is 48 inches for r = 15/7 = 2.1. This would allow about 12 inches of frost penetration into the subgrade in the design year. No subgrade preparation would be required since the thickness is greater than 1/2 x 70 = 35 inches.

b. Reduced subgrade strength design method. From table 4-1, the frost area soil support index is 3.5, which from the design curve in EM 1110-3-131, yields a required thickness of pavement and base of 15 inches. This is substantially less than the thickness required by the limited subgrade frost penetration method. Subgrade penetration would be required to a depth of 1/2 x 70 - 15 = 20 inches. The pavement structure could be composed of 1-1/2 inches of pavement, 8 inches of base course, and 5.5 inches of subbase course plus the 20 inches of prepared subgrade. Since the base course material contains more than 2 percent passing the No. 200 sieve, material in at least the upper 4 inches must be washed to reduce the amount passing the No. 200 sieve to 2 percent or less.

c. All-bituminous concrete (ABC) pavement. The pavement structure from paragraph 8-2.b. can be used to obtain the thickness required through the use of equivalency factors listed in EM 1110-3-131. For the base course, the equivalency factor is 1.15, and 8 inches/1.15 = 7.0 inches of bituminous concrete that could be substituted for the base course. The equivalency factor for the subbase is 2.30, and 5.5 inches/2.30 = 2.4 inches of bituminous concrete that could be substituted for the subbase. The all-bituminous concrete pavement would be 1.5 + 7.0 + 2.4 = 10.9 inches or 11 inches thick. A filter course a minimum of 4 inches thick is required beneath the pavement. Subgrade preparation would be required to a depth of 1/2 x 70 - 15 = 20 inches. The required thickness of the pavement may also be determined from elastic modulus values for the pavement and subgrade. The procedure for obtaining the modulus values can be found in U.S. Army Engineer Waterways Experiment Station Technical Report No. 5-75-10. Figure 8-1 is used to obtain the pavement thickness when the modulus values have been obtained. For this example, a subgrade modulus, E_2, of 4,000 psi and a pavement modulus, E_1, of 200,000 psi will be used. The minimum pavement thickness is 7.5 inches. This thickness is substantially less than that determined using the equivalency values. A 4-inch thick filter course is required beneath this pavement, and the depth of subgrade preparation would be 24 inches.

d. Use of F2 soil. Use of the available F2 borrow material will allow reduced thicknesses of base and subbase and, if desired, could also be used to reduce the depth of preparation of the F4 subgrade. The reduced subgrade strength design method is used to determine the minimum thickness of pavement and base above the F2 soil which has a frost area soil support index of 6.5 (table 4-1). The design curve in EM 1110-3-131 yields a required thickness of pavement and base of 10 inches above the F2 soil. Therefore, the pavement structure could be composed of 1-1/2 inches of pavement, 5 inches of washed base course, 4.5 inches of subbase, and at least 7 inches of F2 soil above the subgrade to comply with the minimum of 17 inches of cover which was required over the F4 subgrade. The pavement structure outlined above would still require processing and preparation of the upper 18 inches of the F4 subgrade. This depth could be reduced by increasing the thickness of F2 soil. For example, if 12 inches of F2 soil was used, preparation to a depth of only 13 inches would be necessary in the F4 soil.

e. Use of local highway design criteria. Local state highway design criteria and standards may be used. If the state criteria are used, however, they must be completely adapted. Portions of the state criteria and portions of the Corps of Engineers criteria should not be mixed.

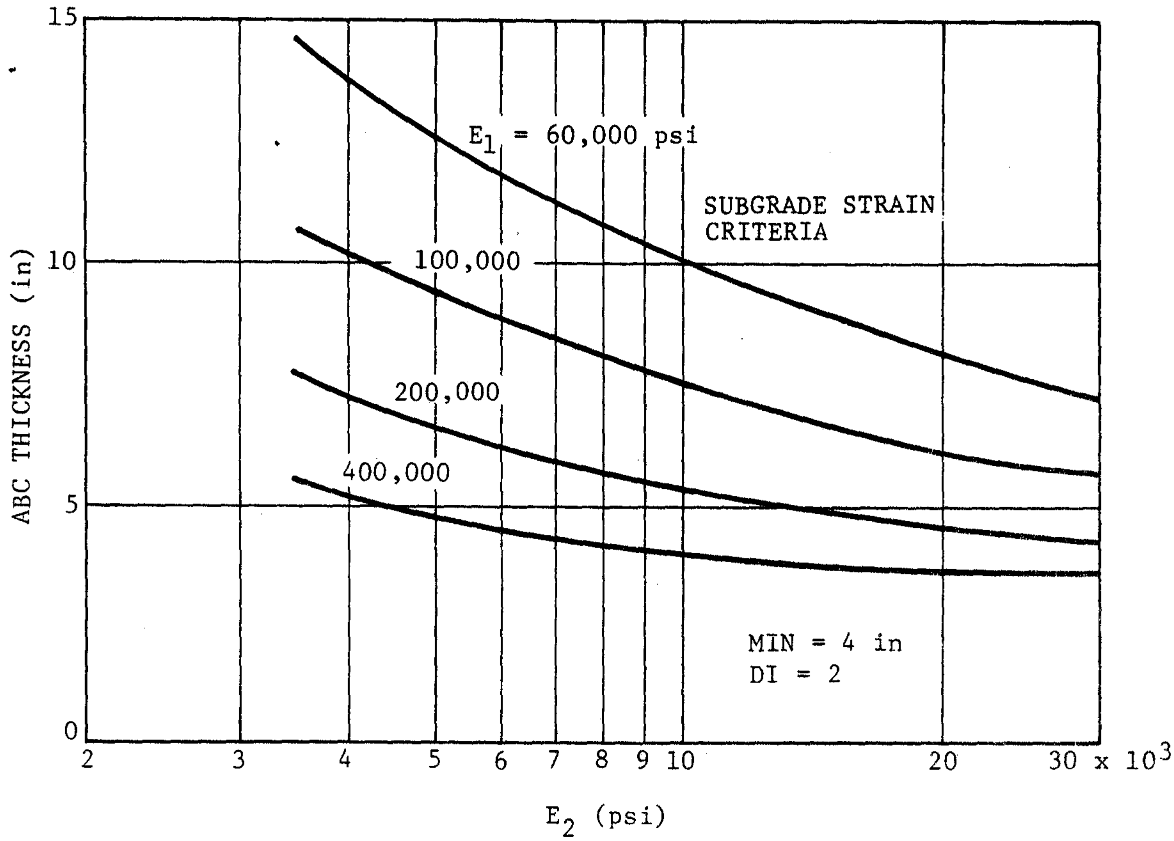


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FIGURE 8-1. DESIGN CURVES FOR ABC ROAD PAVEMENT

APPENDIX A

SUMMARY OF RESULTS OF FREEZING TESTS ON NATURAL SOILS

A-1. Introduction. The U.S. Army Cold Regions Research and Engineering Laboratory (USACRREL) has conducted frost-susceptibility tests on scores of soils. Generally, these were base course materials proposed for use in roads or airfield pavements. Most soils came from construction projects within the United States, but some came from Canada, Greenland, Antarctica, Africa, and Asia. In addition, many fine-grained soils were obtained for special studies at USACRREL and have been tested. They are included in the tables of this appendix. These data are presented for general guidance for estimation of the relative frost-susceptibility of similar soils. It should be noted, however, that a freezing test on a sample of a specific soil will give a more accurate evaluation.

A-2. Presentation of test data and results. Table A-1 contains the test data of soil specimens grouped according to the Unified Soil Classification System. The soils are positioned within each group according to the increasing percentage of grains finer than the 0.02-millimeter size by weight present in the soil. Other data include the physical properties of the material, the results of freezing tests, and the relative frost-susceptibility classification as shown in figure 2-2. Table A-1 contains the test results on 1) soils that met the test specification of having a dry unit weight of 95 percent or greater than that obtained by the appropriate compactive procedure used or specified, and 2) soils that had an initial moisture content before freezing equal to or greater than 85 percent of full saturation. The test results listed in table A-1 (average rate of heave versus percentage by weight of grains finer than the 0.02-millimeter size) are plotted in figure 2-2, in envelopes according to soil type. Table A-2 contains data grouped similarly in every respect to those in table A-1, except that they do not meet the compaction criterion of 95 percent or greater and the initial degree of saturation. Table A-3 contains heave rate data on specimens tested under a lower load pressure than specimens in tables A-1 and A-2. Data from tables A-2 and A-3 have not been plotted in figure 2-2.

A-3. Discussion.

a. Two heave rates have been computed for each specimen presented in the tables: an average heave rate and a maximum heave rate, in millimeters per day. This is done to measure the maximum degree of variability, if any, occurring during each test. The degree of variability is expressed as a heave rate variability index. The reason for high variability is not known. It may be reflective of several variables either in some portion of the specimen or in the test controls, such as specimen inhomogeneity (density, layer discontinuities or other internal influencing factors), friction between the soil and container, rate of heat extraction, and interruption of water supply (internal and external). A large variability index could be indicative of dominance of several counter forces during tests. Such a test result might be assigned a smaller degree of confidence than one whose test variability index is low.

b. Recent experimentation at USACRREL indicates that some variable degree of friction may exist between the specimen and its container during freezing and heaving. Freezing tests of specimens performed in horizontally segmented (multi-ring) cells usually showed higher heave rate than those of counterpart specimens in inside-tapered, solid-walled cells. The inside-tapered cells were a great improvement over straight-walled soil cells.

c. More recent investigations at USACRREL to simplify and shorten the time interval for the frost-susceptibility test revealed that soil specimens in cylinders made of segmented rings 1 inch high usually gave considerably higher heave rates than their counterparts in inside-tapered solid-walled cylinders, especially at the highest rates of frost penetration. Studies to simplify and reduce time for frost-susceptibility testing are still in the development and evaluation stage. When sufficient data are available from segmented ring cylinders, it may be possible to correlate these data with the maximum heave rate.

d. For each specimen listed in tables A-1 through A-3, a detailed temperature and heave versus time plot for the complete period of freezing is available in the USACRREL data files. A plot of moisture content distribution with depth after freezing for each inch of specimen height is also available. The tabular data presented in this appendix give only the overall initial and final average water content, the percentage of heave, and the rates of heave computed in the manner detailed in the notes within the tables.

e. Figure 2-3 presents a summary grouping of the individual envelopes shown in figures 2-2a-c. There are no distinct, neat groupings, nor is there a unique heave rate for any given percentage of 0.02-millimeter grains in the gradation. The groupings overlap considerably, and it should be noted that the Unified Soil Classification System was not developed for frost classification but is used here because of its wide acceptance in soils engineering.

Table A-1. Summary of Frost-Susceptibility Tests on Natural Soils-Open System Nominal Load Pressure 0.5 psi

The following table contains tests results on soils having dry weights equal to or greater than 95 percent of compactive density and initial moisture content of 85 percent or greater of saturation. Heave rate data from table A-1 is presented in figure 2-2.

Table A-1. Summary of Frost-Susceptibility Tests on Natural Soils -Open System Nominal Load Pressure 0.5 psi[1]

Material Source	Unified Soil Classification Symbol (2)	Maximum Size in.	SOIL GRADATION DATA (AS FROZEN) Percent Finer, mm 4.76	0.42	0.074	0.02	0.01	0.005	Coefficients (3) C_u	C_c	PHYSICAL PROPERTIES OF BASIC SOIL Atterberg Limits (4) LL	PI	Specific Gravity
CLAYEY SANDY GRAVELS													
Washington, D.C.	GW-GC	1-1/4	37	16.0	6.4	4.2	-	-	57.0	2.5	26.0	9.0	2.65
Washington, D.C.		1-1/4	37	16.0	6.4	4.2	-	-	57.0	2.5	26.0	9.0	2.65
Present Isle	GP-GC	3/4	37	14.0	11.0	6.6	5.0	3.2	145.0	16.0	24.9	8.7	2.72
Present Isle		3/4	33	15.0	12.0	8.7	6.9	-	315.0	32.0	22.3	8.1	2.75
CLAYEY SILTY GRAVELS													
Clinton County	GM-GC	1-1/2	54	30.0	20.0	15.0	9.0	5.0	485.0	1.9	24.88	6.8	2.74
CLAYEY GRAVELS													
Great Falls	GC	1-1/2	48	36.0	22.0	17.0	15.0	12.0	4000.0	1.2	42.6	24.6	2.66
Loring		3/4	68	52.0	41.0	30.0	25.0	18.0	945.0	0.1	22.1	7.8	2.73
Loring		3/4	68	52.0	41.0	30.0	25.0	18.0	945.0	0.1	22.1	7.8	2.73
Loring		3/4	68	52.0	41.0	30.0	25.0	18.0	945.0	0.1	22.1	7.8	2.73
Loring		3/4	68	52.0	41.0	30.0	25.0	18.0	945.0	0.1	22.1	7.8	2.73
Loring		3/4	68	52.0	41.0	30.0	25.0	18.0	945.0	0.1	22.1	7.8	2.73
SANDS AND GRAVELLY SANDS													
Stewart	SW	2	58	15.0	4.9	2.3	1.5	1.1	23.0	1.3			2.72
Stewart		2	58	15.0	4.9	2.3	1.5	1.1	23.0	1.3			2.72
Plattsburg	SP	1-1/2	59	20.0	2.1	1.0	0.8	0.5	24.0	0.3			2.67
Plattsburg		1-1/2	59	20.0	2.1	1.0	0.8	0.5	24.0	0.3			2.67
Plattsburg		1	72	7.0	3.0	1.3	0.9	0.5	5.3	2.0			3.20
Fairchild		2	85	8.6	3.6	1.3	1.2	-	3.4	0.2			2.74
Fairchild		2	70	6.9	3.4	1.4	1.3	-	4.7	1.3			2.74
Plattsburg		1-1/2	72	36.0	4.5	1.8	1.4	1.0	5.1	0.7			2.67
Plattsburg		1-1/2	72	36.0	4.5	1.8	1.4	1.0	5.1	0.7			2.67
GRAVELS AND SANDY GRAVELS													
Alaska Highway	GW	1	41	11.0	2.0	1.0	0.6	0.4	24.0	1.1			2.75
Loring		3/4	45	9.0	3.9	1.8	1.5	1.2	16.0	1.3			2.71
Loring		3/4	46	10.0	4.4	3.4	2.9	2.1	18.0	1.4			2.71
Koflavik	GP	2	37	9.0	3.0	1.0	-	-	38.0	0.8			2.81
Alaska Highway		1	44	11.0	2.6	1.2	0.7	0.5	26.0	0.9			2.73
Koflavik		3	38	12.0	4.1	1.6	0.9	0.5	91.0	0.5			2.64
SILTY SANDY GRAVELS													
Koflavik	GW-GM	3	32	14.0	6.0	2.1	1.1	0.1	159.0	2.7			2.65
Loring		3/4	53	10.0	6.2	4.9	4.4	3.4	15.0	1.0			2.71
Loring		3/4	53	10.0	6.2	4.9	4.4	3.4	15.0	1.0			2.71
Loring		3/4	53	10.0	6.2	4.9	4.4	3.4	15.0	1.0			2.71
Loring		3/4	53	10.0	6.2	4.9	4.4	3.4	15.0	1.0			2.71
Loring		2	41	9.0	6.4	5.3	4.4	3.4	22.0	1.3	24.0	5.5	2.71
Marble Point	GP-GM	2	56	32.0	11.0	3.7	3.0	2.0	101.0	0.3			2.74
Marble Point		2	38	21.0	10.0	3.9	-	-	185.0	5.7			2.75
Marble Point		2	38	21.0	10.0	3.9	-	-	185.0	5.7			2.75
Project Blue Jay		3/4	54	32.0	10.0	4.0	2.2	1.5	139.0	0.2			2.73
CLAYEY GRAVELS													
Loring	GC	3/4	68	52.0	41.0	30.0	25.0	18.0	945.0	0.1	22.1	7.8	2.73
Loring		3/4	68	52.0	41.0	30.0	25.0	18.0	945.0	0.1	22.1	7.8	2.73
Loring		3/4	68	52.0	41.0	30.0	25.0	18.0	945.0	0.1	22.1	7.8	2.73
SANDS AND GRAVELLY SANDS													
Plattsburg	SP	3/8	60	1.0	0.1	<0.1	<0.1	<0.1	3.8	0.9			2.96
SILTY GRAVELLY SANDS													
Hutchinson's Pit	SW-SM	2	57	20.0	8.7	5.0	3.5	2.0	43.0	1.3			2.75
Hutchinson's Pit		2	57	20.0	8.7	5.0	3.5	2.0	43.0	1.1			2.75
Thule	SP-SM	3/4	65	41.0	8.6	2.8	2.0	1.4	35.0	0.3			2.75
Tobyhanna		1-1/2	59	39.0	8.5	4.5	2.5	1.6	6.0	0.2			2.72
SILTY SANDS													
Alaska Highway	SM	-	100	100.0	33.0	2.5	-	-	1.6	1.0			2.79
Tobyhanna		1-1/2	79	45.0	14.0	5.5	4.0	3.1	24.0	0.7			2.72
Dow		3/4	61	27.0	14.0	7.8	5.5	3.8	160.0	2.7	17.6	3.1	2.72
Fairchild		3/4	71	34.0	23.0	11.0	6.3	4.0	95.0	2.2	21.6	2.9	2.79
Ball Mountain		3/4	88	58.0	28.0	12.0	7.5	3.6	36.0	1.2			2.77
Ball Mountain		3/4	88	58.0	28.0	12.0	7.5	3.6	36.0	1.2			2.77
Hill Field		-	100	95.0	28.0	13.0	10.0	7.5	17.0	4.3			2.64
Project Blue Jay		3/4	70	54.0	31.0	19.0	12.0	8.5	147.0	0.4	16.0	3.7	2.70
Project Blue Jay		3/4	70	54.0	31.0	19.0	12.0	8.5	147.0	0.4	16.0	3.7	2.70

Table A-1. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure o.5 psi[1] (Cont.)

PHYSICAL PROPERTIES OF BASIC SOIL		SPECIMEN DATA (AS MOLDED)				FREEZING TEST DATA						
Compaction Data (5)						Avg. Water Content						
Maximum Dry Unit Weight	Optimum Moisture Content	Dry Unit Weight	Degree of Compaction	Void Ratio	G. at Start of Test (6)	Before Test	After Test	Total Heave (7)	Rate of Heave mm/day (8)		Heave Rate Var. Index (9)	Frost Susc. Class (10)
pcf	%	pcf	%	%	%	%	%	%	Avg.	Mos.		
133.9(d)	4.7	135	101	0.220	97	8.0	12.7	15.6	2.1	3.0	1.42	M
133.9(d)	4.7	136	101	0.218	96	7.7	12.0	15.5	2.6	3.3	1.26	M
136.8(b)	-	134	98	0.265	97	9.7	22.3	42.5	2.9	3.7	1.28	M
-	-	131	96	C.250	98	8.8	16.2	19.9	1.5	2.0	1.33	L
130.2(a)	9.0	129	99	0.320	100	11.7	30.3	65.6	4.6	5.7	1.24	M
140.0(d)	5.6	133	95	0.252	100	9.5	21.0	28.0	2.4	5.0	2.08	M-H
135.8(d)	7.5	129	95	0.320	100	11.6	34.8	84.4	4.0	6.8	1.70	H
135.8(d)	7.5	132	97	0.290	100	10.3	19.0	30.2	2.3	3.7	1.60	M
135.8(d)	7.5	136	100	0.250	100	9.0	17.6	28.5	1.5	2.7	1.80	L-M
135.8(d)	7.5	134	99	0.270	100	9.7	24.3	42.5	2.6	4.0	1.54	M
135.8(d)	7.5	132	97	0.290	95	10.0	32.0	81.9	4.9	13.2	2.69	H-VH
139.9(b)	-	136	97	0.254	100	9.7	18.1	20.6	2.9	4.0	1.38	M
139.9(b)	-	136	97	0.250	100	9.3	21.4	32.3	2.4	3.8	1.58	M
132.8(b)	-	130	98	0.281	100	10.5	11.2	6.0	0.6	0.7	1.16	VL
132.8(b)	-	130	98	0.283	100	10.6	12.8	9.6	0.3	0.4	1.33	M
139.1(b)	-	139	100	0.440	86	11.7	11.7	7.5	0.3	0.4	1.33	M
119.2(b)	-	116	98	0.469	100	17.0	19.0	10.4	0.8	1.6	2.00	VL-L
132.1(b)	-	125	95	0.368	100	13.4	15.4	10.8	0.7	1.1	1.57	VL-L
125.2(b)	-	124	99	0.338	95	12.0	12.0	5.3	0.6	0.8	1.33	VL
125.2(b)	-	125	100	0.329	90	12.3	13.9	9.8	0.7	0.9	1.28	VL
143.9(b)	-	136	95	0.261	80	7.6	11.6	11.3	0.7	1.0	1.42	VL
143.8(d)	6.1	123	86	0.374	91	12.5	14.4	13.8	1.3	1.8	1.38	L
143.8(d)	6.1	130	90	0.300	98	10.8	11.1	8.3	1.1	1.8	1.64	L
112.0(b)	-	112	100	0.562	82	16.6	16.6	9.3	0.1	0.1	1.00	M
143.3(b)	-	127	95	0.341	83	10.4	13.9	9.6	0.7	1.3	1.86	VL-L
145.5	-	137	94	0.390	81	11.4	11.4	7.8	0.5	1.4	2.80	VL-L
138.6(b)	-	137	98	0.380	70	8.7	8.7	2.7	0.3	0.7	2.33	M-VL
139.1(b)	-	139	100	0.210	74	6.0	11.1	24.6	1.5	3.0	2.00	L-M
139.1(b)	-	133	96	0.273	72	7.4	14.9	23.9	1.8	3.5	1.94	L-M
139.1(b)	-	126	91	0.342	75	9.5	13.1	20.0	1.5	2.8	1.86	L-M
139.1(b)	-	120	86	0.409	73	11.0	13.3	11.4	1.1	1.7	1.54	L
139.1(b)	-	135	94	0.256	100	9.5	17.7	33.1	2.9	3.8	1.31	H
150.8(b)	-	141	93	0.213	100	7.8	12.8	17.0	1.4	2.2	1.57	L-M
145.6(b)	-	137	94	0.252	100	9.2	9.6	3.5	0.3	0.8	2.66	VL
145.6(b)	-	137	94	0.252	100	8.6	11.0	7.7	0.6	1.0	1.66	VL
143.4(b)	-	138	96	0.238	79	6.9	24.6	47.4	3.3	5.2	1.58	L-H
135.8(d)	-	120	88	0.420	97	15.1	69.7	134.3	8.0	13.8	1.72	VH
135.8(d)	-	122	90	0.394	94	13.5	58.5	106.5	8.5	10.3	1.58	H-VH
135.8(d)	-	127	93	0.290	98	12.3	56.8	111.3	6.6	10.8	1.64	H-VH
126.7(b)	-	127	100	0.455	80	12.3	12.3	1.4	0.1	0.1	1.00	M
143.3(c)	5.3	141	98	0.220	71	5.7	29.5	61.7	4.3	5.3	1.23	H
143.3(c)	5.3	140	98	0.231	78	6.5	34.1	73.8	4.8	5.8	1.20	H
143.7(b)	-	135	94	0.271	100	9.8	12.9	10.6	0.8	1.0	1.25	VL
140.6(b)	-	132	94	0.280	100	10.0	20.5	24.6	1.4	2.8	2.00	M
105.7(b)	-	104	98	0.672	78	18.9	24.0	7.0	0.3	0.5	1.66	H
140.6(b)	-	130	92	0.300	100	11.1	27.2	38.0	2.6	5.5	2.12	M-H
136.7(b)	-	135	99	0.254	60	5.5	35.7	70.5	4.0	5.8	1.45	H
144.4(b)	-	134	93	0.287	100	10.3	30.3	56.0	3.0	5.2	1.73	M-H
141.8(d)	5.6	133	94	0.300	100	10.8	38.5	77.3	6.1	7.2	1.18	H
141.8(d)	5.6	132	94	0.307	100	11.1	30.4	45.6	5.3	7.2	1.36	H
120.4(d)	-	113	94	0.460	95	15.6	26.2	16.8	1.9	2.7	1.42	L-H
137.3(d)	7.5	136	99	0.238	73	6.4	14.8	16.7	1.6	2.7	1.68	L-H
137.3(d)	7.5	132	96	0.275	68	6.9	26.9	37.4	3.0	5.8	1.93	L-H

Table A-1. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

Material Source	Unified Soil Classification Symbol (2)	Maximum Size	SOIL GRADATION DATA (AS FROZEN)								PHYSICAL PROPERTIES OF BASIC SOIL		
			Percent finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity
		in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI	
SILTY SANDS (CONT.)													
Truax		3/4	92.	79.0	35.0	22.0	15.0	1.9	55.0	1.9	14.4	1.6	2.72
Truax		3/4	92	79.0	35.0	22.0	15.0	1.9	55.0	1.9	14.4	1.6	2.72
Truax		3/4	92	79.0	35.0	22.0	15.0	1.9	55.0	1.9	14.4	1.6	2.72
Truax		3/4	92	79.0	35.0	22.0	15.0	1.9	55.0	1.9	14.4	1.6	2.72
Truax		3/4	92	79.0	35.0	22.0	15.0	1.9	55.0	1.9	14.4	1.6	2.72
Plattsburg	SM	-	100	95.0	28.0	1.5	1.2	0.9	2.5	0.9			2.68
Plattsburg		-	100	95.0	28.0	1.5	1.2	0.9	2.5	0.9			2.68
Alaska		-	100	100.0	33.0	2.5	-	-	1.6	1.0			2.79
Westover		-	100	86.0	20.0	2.5	-	-	4.1	1.2			2.69
Minnesota		-	100	95.0	20.0	3.8	2.2	-	3.7	1.3			2.68
Bradley		3/4	79	27.0	14.0	4.2	2.6	-	47.0	1.9			2.76
Bradley		3/4	67	31.0	14.0	4.4	2.6	-	62.0	0.9			2.76
Bethel		-	100	100.0	21.0	4.5	2.5	1.0	3.0	1.1			2.68
Bethel		-	100	100.0	21.0	4.5	2.5	1.0	3.0	1.1			2.68
Westover		-	100	86.0	26.0	5.1	-	-	27.0	1.3			2.69
Greenland		3/4	66	45.0	17.0	5.2	3.7	2.4	47.0	0.4			2.73
Greenland		3/4	66	45.0	17.0	5.2	3.7	2.4	47.0	0.4			2.73
Greenland		3/4	66	45.0	17.0	5.2	3.7	2.4	47.0	0.4			2.73
Project Blue Jay		3/4	82	53.0	21.0	6.0	5.2	2.8	25.0	0.6			2.71
Afghanistan		1	57	23.0	12.0	6.2	3.9	2.7	111.0	3.1			2.69
Korea		2	88	43.0	18.0	6.5	4.0	2.5	28.6	1.3			2.58
Westover		-	100	85.0	27.0	7.0	-	-	6.9	1.2			2.71
Westover		-	100	85.0	27.0	7.0	-	-	6.9	1.2			2.71
M.I.T.		2	84	47.0	13.0	7.5	5.3	3.6	17.0	1.9			2.70
M.I.T.		2	76	49.0	17.0	7.8	4.5	3.0	28.0	1.4			2.70
Portsmouth		3/4	98	94.0	29.0	8.2	5.4	3.7	4.0	1.8			2.73
Minnesota		-	100	97.0	48.0	8.8	4.5	-	4.4	0.8			2.72
Westover		2	58	27.0	14.0	8.9	7.5	6.0	250.0	2.2	21.9	3.0	2.70
Volk Field		-	100	88.0	13.0	11.0	9.5	7.7	20.0	7.5			2.72
Mansfield Hollow		3/4	78	53.0	23.0	11.0	7.5	4.5	38.0	1.3			2.70
Mansfield Hollow	SM	3/4	78	53.0	23.0	11.0	7.5	4.5	38.0	1.3			2.70
Fairchild		3/4	71	34.0	23.0	11.0	6.3	4.0	95.0	2.2	21.6	2.0	2.79
Thule		3/4	73	47.0	20.0	12.0	9.0	6.9	71.0	1.8	14.1	2.2	2.88
Thule		3/4	73	47.0	20.0	12.0	9.0	6.9	71.0	1.8	14.1	2.2	2.88
Portsmouth		3/4	68	45.0	23.0	14.0	9.1	1.2	14.0	1.2			2.71
Westover		3/4	97	75.0	38.0	14.0	7.0	-	17.0	0.8			2.65
Truax		3/4	90	79.0	28.0	15.0	12.0	9.0	36.0	4.2			2.70
Minnesota		3/4	97	73.0	31.0	17.0	14.0	13.0	280.0	18.0	18.3	2.8	2.73
Truax		1/2	82	71.0	32.0	19.0	13.0	9.4	50.0	4.6	14.0	2.0	2.72
M.I.T.		1-1/2	81	58.0	38.0	19.0	12.0	6.5	56.0	0.9	20.7	0.9	2.70
Truax		3/4	82	71.0	32.0	19.0	13.0	9.5	50.0	4.6	14.4	1.6	2.72
Truax		3/4	92	79.0	35.0	22.0	15.0	12.0	55.0	1.9	14.4	1.6	2.72
Truax		3/4	92	79.0	35.0	22.0	15.0	12.0	55.0	1.9	14.4	1.6	2.72
Truax		3/4	92	79.0	35.0	22.0	15.0	12.0	55.0	1.9	14.4	1.6	2.72
Truax		3/4	92	79.0	35.0	22.0	15.0	12.0	55.0	1.9	14.4	1.6	2.72
CLAYEY SILTY SANDS													
Sioux Falls	SM-SC	1	71	28.0	16.0	9.0	6.0	4.3	108.0	3.7	24.1	5.9	2.72
Loring		1/2	87	22.0	15.0	13.0	11.0	8.0	260.0	34.0	24.0	6.0	2.72
Thule		3/4	65	39.0	22.0	14.0	10.0	7.0	310.0	0.9	16.1	4.3	2.87
Thule		3/4	65	39.0	22.0	14.0	10.0	7.0	310.0	0.9	16.1	4.3	2.87
Casper		1-1/2	91	48.0	23.0	15.0	13.0	11.0	225.0	13.0	22.0	4.6	2.64
Patterson		1-1/2	62	33.0	22.0	15.0	10.0	5.5	400.0	2.7	22.0	6.1	2.74
Casper		1-1/2	98	62.0	21.0	16.0	14.0	12.0	137.0	14.0	21.8	6.0	2.65
Casper		3/4	98	68.0	29.0	18.0	16.0	14.0	195.0	11.0	22.0	4.6	2.66
Bong	SM-SC	1-1/2	94	75.0	44.0	21.0	15.0	10.0	33.0	1.3	16.8	5.1	2.76
Bong		1-1/2	94	75.0	44.0	21.0	15.0	10.0	33.0	1.3	16.8	5.1	2.76
Bong		1-1/2	94	75.0	44.0	21.0	15.0	10.0	33.0	1.3	16.8	5.1	2.76
Loring		1-1/2	83	63.0	46.0	30.0	25.0	18.0	188.0	0.8	21.1	6.0	2.71
Loring		1-1/2	87	62.0	48.0	32.0	24.0	15.0	100.0	0.2	21.1	6.0	2.71
Fairchild		3/4	76	29.0	17.0	9.5	7.0	4.5	55.0	7.2	24.6	6.3	2.77
Loring		1-1/2	83	60.0	47.0	34.0	27.0	20.0	320.0	0.3	21.1	6.0	2.71
CLAYEY SANDS													
Pierre	SC	1-1/2	67	31.0	17.0	8.7	7.0	4.3	100.0	3.0	25.3	7.3	2.77
Fargo		3/4	98	33.0	17.0	9.5	7.5	5.5	50.0	5.2	30.7	10.5	2.70
Fargo		3/4	98	33.0	17.0	9.5	7.5	5.5	50.0	5.2	30.7	10.5	2.70
Fargo		3/4	98	33.0	17.0	9.5	7.5	5.5	50.0	5.2	30.7	10.5	2.70
Fargo		3/4	98	33.0	17.0	9.5	7.5	5.5	50.0	5.2	30.7	10.5	2.70

Table A-1. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

PHYSICAL PROPERTIES OF BASIC SOIL		SPECIMEN DATA (AS MOLDED)				FREEZING TEST DATA						
Compaction Data (5)						Avg. Water Content						
Maximum Dry Unit Weight	Optimum Moisture Content	Dry Unit Weight	Degree of Compaction	Void Ratio	G. at Start of Test (6)	Before Test	After Test	Total Heave (7)	Rate of Heave mm/day (8)		Heave Rate Var. Index (9)	Frost Susc. Class (10)
pcf	%	pcf	%	%	%	%	%	%	Avg.	Mos.		
137.3(d)	5.6	129	94	0.315	94	10.9	23.2	28.2	3.3	4.2	1.27	H-H
137.3(d)	5.6	119	87	0.423	91	14.3	16.5	7.4	1.1	1.7	1.54	L
137.3(d)	5.6	126	91	0.350	90	11.7	17.0	13.0	2.0	3.0	1.50	M
137.3(d)	5.6	126	91	0.348	100	12.8	27.0	41.2	2.8	3.5	1.25	H
137.3(d)	5.6	118	86	0.431	100	15.9	24.0	20.2	1.4	2.0	1.42	L
110.3(b)	-	107	97	0.567	85	18.6	18.7	4.4	0.2	0.5	2.50	M
110.3(b)	-	109	99	0.540	96	19.2	19.2	4.4	0.1	0.2	2.00	M
106.4(b)	-	108	101	0.605	100	21.7	24.6	9.4	0.7	1.5	2.14	VL-L
119.9(b)	-	115	96	0.458	99	18.5	17.6	4.3	0.2	0.3	1.50	M
114.2(d)	-	114	100	0.434	99	16.0	42.8	49.8	2.3	9.3	4.04	M-VH
133.6(b)	-	133	100	0.300	100	10.9	20.6	20.0	1.2	1.6	1.33	L
143.1(b)	-	143	100	0.202	100	7.3	21.9	21.9	2.4	3.3	1.38	M
106.4(d)	-	106	100	0.578	96	20.7	21.6	21.6	0.5	1.0	2.00	VL
106.4(d)	-	105	99	0.593	88	19.4	21.6	21.6	0.6	1.0	1.66	VL
114.3(b)	-	114	100	0.467	100	17.7	23.9	14.2	0.7	1.0	1.42	VL
137.9(b)	-	135	98	0.258	100	9.5	22.6	35.3	2.2	2.7	1.22	M
137.9(b)	-	137	99	0.244	97	8.6	31.6	60.2	3.8	5.5	1.44	M-H
137.9(b)	-	136	99	0.252	100	9.2	22.9	38.4	2.0	2.9	1.45	M
136.0(c)	7.0	129	95	0.312	88	10.1	28.5	36.9	2.9	3.8	1.31	M
144.6(b)	-	144	99	0.155	98	6.0	13.9	24.7	2.2	3.7	1.68	M
120.0(b)	-	120	99	0.398	96	12.5	27.2	39.5	5.5	8.0	1.45	H
116.4(b)	-	117	100	0.450	100	16.5	26.5	19.6	1.2	1.8	1.50	L
116.4(b)	-	111	95	0.521	100	19.2	22.4	10.8	0.6	1.3	2.16	VL-L
123.0(d)	13.2	123	100	0.374	96	13.2	21.9	22.4	2.3	3.2	1.39	M
122.1(d)	14.2	122	100	0.384	100	14.2	25.3	28.3	1.9	2.7	1.42	L-M
111.2(d)	-	109	98	0.560	96	19.8	26.2	13.5	0.8	1.5	1.86	VL-M
126.0(b)	-	120	95	0.419	99	15.3	22.0	18.3	1.4	3.3	2.35	L-M
129.4(b)	-	128	> 99	0.312	100	11.5	12.4	8.5	0.9	1.2	1.33	VL-L
119.5(b)	-	114	95	0.375	100	16.7	33.5	37.2	2.2	3.0	1.36	M
136.0(d)	-	131	96	0.290	98	10.5	30.1	45.0	3.3	4.0	1.21	M
136.0(d)	-	131	96	0.291	88	9.5	24.2	35.3	4.6	6.5	1.41	M-H
142.9(b)	-	136	95	0.280	100	10.0	22.2	27.1	2.8	4.8	1.71	M-H
150.9(b)	-	145	96	0.243	100	8.4	30.2	58.0	2.6	5.3	2.04	M-H
150.9(b)	-	144	95	0.248	98	8.4	34.1	66.4	3.0	6.5	2.16	M-H
128.6(b)	-	127	99	0.333	100	12.3	46.1	81.8	5.6	8.8	1.57	H-VH
-	-	112	> 95	0.483	97	17.6	73.1	116.9	4.9	7.4	1.51	H
137.3(d)	5.6	130	95	0.300	93	10.2	19.5	23.2	1.5	2.7	1.80	L-M
130.2(b)	-	124	95	0.374	100	13.7	63.3	118.0	6.3	10.2	1.62	H-VH
-	-	129	> 95	0.311	95	10.8	14.8	10.8	2.2	3.0	1.36	M
119.0(d)	15.0	119	100	0.404	100	15.0	30.2	35.4	2.1	2.8	1.33	M
139.0(c)	5.3	136	98	0.246	100	9.1	17.2	21.0	1.8	3.5	1.94	L-M
137.3(d)	5.6	130	95	0.303	100	11.1	18.2	22.0	2.5	2.8	1.12	M
137.3(d)	5.6	134	98	0.265	98	9.6	14.9	14.7	2.5	2.8	1.12	M
137.3(d)	5.6	139	102	0.216	100	8.0	16.3	15.5	1.3	1.7	1.30	L
137.3(d)	5.6	132	96	0.280	100	10.3	23.3	37.1	2.9	3.3	1.34	M
137.0(d)	7.2	131	96	0.292	100	10.6	15.7	16.6	1.7	2.7	1.58	L-M
139.1(b)	-	134	96	0.265	99	9.7	29.1	56.7	5.5	7.5	1.36	H
152.5(b)	-	148	97	0.215	100	7.5	31.0	61.4	2.6	4.7	1.80	M-H
152.5(b)	-	146	96	0.223	100	7.8	35.9	68.8	3.3	6.5	1.96	M-H
120.8(d)	7.2	120	99	0.378	100	14.2	19.6	17.1	1.5	2.7	1.80	L-M
-	-	135	95	0.267	100	9.7	26.0	44.4	3.3	4.2	1.27	M-H
120.8(d)	7.2	118	98	0.403	100	15.2	21.7	17.7	1.6	2.3	1.44	L-M
120.8(d)	7.2	119	99	0.393	95	14.0	22.0	20.0	2.2	3.2	1.45	M
139.5(c)	-	135	97	0.290	100	10.8	13.1	13.1	1.3	2.0	1.54	L
139.5(c)	-	136	97	0.267	100	9.7	12.8	12.8	1.3	1.7	1.30	L
139.5(c)	-	134	96	0.282	100	10.2	16.2	21.8	1.7	3.7	2.18	L-M
-	-	127	> 95	0.334	100	12.3	44.0	77.6	6.7	8.7	1.30	H-VH
-	-	127	> 95	0.334	100	12.3	50.1	56.9	2.8	3.3	1.18	H
142.1(b)	-	131	92	0.314	94	10.7	22.7	29.0	3.2	5.7	1.78	H-H
135.8(d)	-	123	91	0.369	100	13.5	78.8	159.4	15.4	21.3	1.38	H
134.5(d)	6.9	123	91	0.381	100	14.0	16.5	9.7	0.6	0.7	1.16	VL
127.2(d)	9.0	113	89	0.494	87	15.9	40.5	52.6	5.0	7.8	1.56	H
127.2(d)	9.0	117	92	0.438	89	14.4	37.6	40.8	3.5	5.5	1.57	H-W
127.2(d)	9.0	103	81	0.641	99	25.1	74.9	80.0	2.9	5.0	1.72	M-H
127.2(d)	9.0	107	84	0.581	100	23.1	33.8	28.8	1.9	2.8	1.47	L-M

Table A-1. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

Material Source	Unified Soil Classification Symbol (2)	Maximum Size in.	SOIL GRADATION DATA (AS FROZEN) Percent finer, mm 4.76	0.42	0.074	0.02	0.01	0.005	Coefficients (3) C_u	C_c	PHYSICAL PROPERTIES OF BASIC SOIL Atterberg Limits (4) LL	PI	Specific Gravity
CLAYEY SANDS (CONT.)													
Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70
Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70
Project Blue Jay		3/4	73	55	35	23.0	20.0	15.0	500	1.7	24.7	3.1	2.73
Lowry		-	100	86	39	25.0	21.0	17.0	150	6.9	24.5	7.3	2.64
Lowry		-	100	86	39	25.0	21.0	17.0	150	6.9	24.5	7.8	2.64
Lowry		-	100	86	39	25.0	21.0	17.0	150	6.9	24.5	7.3	2.64
Lowry		-	100	90	44	32.0	28.0	22.0	150	1.5	24.5	7.3	2.64
SILTS AND SANDY SILTS													
New Hampshire	ML	-	100	99	97	60.0	22.0	10.0	-	-	26.6	0.1	2.70
New Hampshire		-	100	99	97	60.0	22.0	10.0	-	-	26.6	0.1	2.70
New Hampshire		-	100	99	97	60.0	22.0	10.0	-	-	26.6	0.1	2.70
New Hampshire		-	100	99	97	60.0	22.0	10.0	-	-	26.6	0.1	2.70
New Hampshire		-	100	99	97	60.0	22.0	10.0	-	-	26.6	0.1	2.70
CLAYEY SILTS													
Dow Field	ML-CL	3/4	88	76	66	40.0	30.0	20.0	-	-	22.0	0.9	2.71
Loring		3/4	84	70	59	44.0	35.0	27.0	-	-	21.1	6.0	2.70
Loring		1	90	73	61	48.0	40.0	30.0	-	-	21.1	6.0	2.70
New Hampshire		-	100	96	90	67.0	36.0	16.0	-	-	24.8	5.1	2.70
New Hampshire		-	100	96	90	67.0	36.0	16.0	-	-	24.8	5.1	2.70
New Hampshire		-	100	96	90	67.0	36.0	16.0	-	-	24.8	5.1	2.70
New Hampshire		-	100	97	93	67.0	39.0	26.0	-	-	26.5	6.0	2.71
New Hampshire		-	100	97	93	67.0	39.0	26.0	-	-	26.5	6.0	2.71
New Hampshire		-	100	97	93	67.0	39.0	26.0	-	-	26.5	6.0	2.71
New Hampshire		-	100	97	93	67.0	39.0	26.0	-	-	26.5	6.0	2.71
SILTS WITH ORGANICS													
Fairbanks	ML-OL	-	100	100	95	32.0	16.0	10.0	-	-	28.4	4.4	2.72
Fairbanks		-	100	100	95	32.0	16.0	10.0	-	-	28.4	4.4	2.72
Fairbanks		-	100	100	95	32.0	16.0	10.0	-	-	28.4	4.4	2.72
Ladd Field		-	100	100	91	38.0	13.0	6.0	-	-	31.6	0.0	2.75
Ladd Field		-	100	100	91	38.0	13.0	6.0	-	-	31.6	0.0	2.75
Ladd Field		-	100	100	91	38.0	13.0	6.0	-	-	31.6	0.0	2.75
Fairbanks		-	100	100	94	40.0	23.0	13.0	-	-	25.3	3.3	2.67
Fairbanks		-	100	100	94	40.0	23.0	13.0	-	-	25.3	3.3	2.67
Fairbanks		-	100	100	94	40.0	23.0	13.0	-	-	25.3	3.3	2.67
Fairbanks		-	100	100	97	42.0	22.0	12.0	-	-	25.3	3.3	2.68
LEAN CLAYS													
Portsmouth	CL	-	100	98	91	33.0	24.0	19.0	-	-	28.0	12.0	2.71
Crosby		-	100	98	91	58.0	41.0	31.0	-	-	36.5	16.8	2.78
Greenland		-	100	100	97	60.0	43.0	34.0	-	-	31.3	15.2	2.79
Yukon		-	100	100	100	67.0	37.0	29.0	-	-	28.0	3.6	2.74
Yukon		-	100	100	100	67.0	37.0	29.0	-	-	28.0	3.6	2.74
Yukon		-	100	100	100	67.0	37.0	29.0	-	-	28.0	3.6	2.74
Yukon		-	100	100	100	67.0	37.0	29.0	-	-	28.0	3.6	2.74
Yukon		-	100	100	100	67.0	37.0	29.0	-	-	28.0	3.6	2.74
LEAN CLAYS WITH ORGANICS													
Malad, Idaho	CL-OL	-	100	99	96	65.0	48.0	35.0	-	-	37.0	13.0	2.58
Malad, Idaho		-	100	99	96	65.0	48.0	35.0	-	-	37.0	13.0	2.58
Malad, Idaho		-	100	99	96	65.0	48.0	35.0	-	-	37.0	13.0	2.58
Malad, Idaho		-	100	99	96	65.0	48.0	35.0	-	-	37.0	13.0	2.58
FAT CLAYS													
Frederick	CH	-	100	99	74	61.0	52.0	43.0	-	-	55.0	37.0	2.38

Table A-1. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

PHYSICAL PROPERTIES OF BASIC SOIL		SPECIMEN DATA (AS MOLDED)				FREEZING TEST DATA						
Compaction Data (5)						Avg. Water Content						
Maximum Dry Unit Weight	Optimum Moisture Content	Dry Unit Weight	Degree of Compaction	Void Ratio	G. at Start of Test (6)	Before Test	After Test	Total Heave (7)	Rate of Heave mm/day (8)		Heave Rate Var. Index (9)	Frost Susc. Class (10)
pcf	%	pcf	%	%	%	%	%	%	Avg.	Mos.		
127.2(d)	9.0	108	85	0.560	100	20.8	60.2	65.0	3.9	6.2	1.58	M-H
127.2(d)	9.0	112	88	0.507	100	18.8	41.9	49.0	3.0	4.3	1.43	M-H
133.1(d)	9.4	128	96	0.334	83	10.0	44.4	77.1	5.1	9.2	1.80	H-VL
121.0(d)	-	112	92	0.468	100	17.7	34.3	37.8	2.7	4.3	1.59	M-H
121.0(d)	-	111	91	0.491	100	18.6	38.1	42.8	3.2	4.0	1.25	H-M
121.0(d)	-	112	92	0.467	98	17.4	27.4	32.1	2.9	3.8	1.31	M
121.0(d)	-	112	92	0.472	100	17.8	57.1	103.3	5.8	8.0	1.38	H
106.7(c)	16.5	90	85	0.872	100	32.3	72.0	60.4	8.3	12.8	1.54	VH
106.7(c)	16.5	95	89	0.773	100	28.5	63.7	68.8	9.3	11.7	1.26	VH
106.7(c)	16.5	98	92	0.712	100	26.0	123.2	72.7	6.2	12.7	2.04	H-VH
106.7(c)	16.5	95	89	0.781	100	26.8	166.6	105.6	11.4	15.7	1.38	VH
106.7(c)	16.5	97	91	0.742	100	27.4	185.4	144.4	15.9	19.0	1.19	VH
127.6(d)	-	119	93	0.418	100	15.4	87.1	155.4	11.4	16.3	1.42	VH
133.8(d)	8.3	112	84	0.506	99	18.5	78.0	164.4	13.1	19.6	1.47	VH
133.8(d)	8.3	113	85	0.502	81	15.0	47.1	82.1	7.4	15.0	2.82	H-VH
106.7(c)	16.5	100	94	0.685	100	25.4	166.3	262.2	12.3	16.5	1.34	VH
106.7(c)	16.5	99	93	0.702	100	26.0	103.3	139.3	13.3	20.5	1.54	VH
106.7(c)	16.5	100	94	0.685	100	25.3	95.8	119.1	11.5	17.0	1.48	VH
109.9(d)	15.6	105	96	0.605	70	15.7	164.6	275.5	27.6	36.0	1.30	VH
109.9(d)	15.6	105	96	0.605	82	18.2	138.9	221.7	22.7	21.8	1.26	VH
109.9(d)	15.6	106	96	0.600	61	13.4	161.3	275.8	26.2	33.7	1.28	VH
109.9(d)	15.6	104	94	0.631	100	23.3	142.1	226.4	24.7	31.3	1.26	VH
112.5(d)	15.7	85	75	1.000	100	36.6	34.4	2.9	0.5	1.0	2.00	VL
112.5(d)	15.7	90	80	0.890	100	32.6	34.6	7.9	0.7	1.5	2.14	VL-L
112.5(d)	15.7	98	87	0.740	100	26.9	29.2	12.4	0.5	1.7	3.40	VL-L
101.6(d)	18.1	84	83	1.040	98	37.1	38.4	7.8	0.6	1.5	2.50	VL-L
101.6(d)	18.1	90	89	0.899	97	31.6	35.8	11.2	0.6	1.0	1.66	VL
101.6(d)	18.1	94	93	0.811	99	29.4	39.8	25.5	1.8	2.0	1.11	L
107.4(d)	17.1	94	88	0.702	96	25.0	65.5	124.0	4.5	8.7	1.93	H-VH
107.4(d)	17.1	98	91	0.703	100	26.2	65.8	81.8	7.4	8.7	1.76	H-VH
108.4(d)	17.1	97	91	0.717	100	26.8	82.1	102.1	8.0	9.7	1.21	VH
108.5(d)	14.8	99	91	0.695	86	22.4	30.1	10.4	0.7	1.2	1.71	VL-L
113.4(d)	-	113	100	0.474	92	16.3	38.0	47.1	4.0	4.8	1.20	H
119.3(a)	13.5	117	98	0.485	100	17.5	24.6	17.7	1.4	2.3	1.64	L-M
119.4(d)	15.0	116	97	0.518	100	18.3	30.1	26.8	2.2	5.3	2.40	M-M
121.4(d)	12.8	117	96	0.460	89	15.0	22.0	24.0	1.1	2.5	2.27	L-M
121.4(d)	12.8	118	97	0.448	94	15.4	33.0	45.7	3.8	5.3	1.39	M-M
121.4(d)	12.8	123	101	0.385	100	14.1	29.5	38.5	2.1	4.0	1.90	M
121.4(d)	12.8	120	98	0.424	100	15.5	29.1	34.3	1.8	3.7	2.06	L-M
121.4(d)	12.8	115	95	0.476	94	16.5	36.6	46.2	2.5	4.2	1.68	M-M
99.6	21.0(a)	99	99	0.630	100	24.4	31.4	20.9	3.4	4.0	1.18	M
99.6	21.0(a)	96	96	0.676	100	26.3	60.8	61.0	4.6	7.3	1.58	M
99.6	21.0(a)	98	98	0.644	100	25.0	42.5	42.3	4.1	5.2	1.26	M
99.6	21.0(a)	99	99	0.627	100	24.3	45.0	45.0	4.2	5.0	1.19	M
106.7	19.5(a)	105	98	0.715	86	21.2	38.4	39.0	0.8	1.7	2.12	VL-L

Table A-1. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

NOTES:

1. Data based on 6-inch molded specimens frozen under laboratory conditions of 85 percent or greater saturation before freezing, molded dry wt. - 95 percent or greater of applicable standard, penetration rate of 1/4 or 1/2 inch/day at 32 degrees, and free water at specimen base (38 degrees).

2. Soil classifications according to MIL-STD-619 (CE).

3. Gradation coefficients (MIL-STD-619 (CE)): $C_u = \frac{D_{60}}{D_{10}}$ and $C_c = \frac{(D_{30})^2}{(D_{60})\ (D_{10})}$

4. Atterberg limits on plastic materials only. Test on material passing No. 40 sieve only.

5. Natural soil maximum dry weight and optimum moisture for compaction test type: a) AASHTO T99 Method A, b) Providence vibrated density test, c) AASHTO T180 Method D, d) AASHTO T180 Method A, e) Harvard minature compaction.

6. Saturation percent at start of freezing test (drained for 24 hours).

7. Based on original frozen height.

8. Average rate of heave determined from maximum representative portion of heave versus time plot (minimum 5 consecutive days).

9. Maximum heave rate (average of 3 highest daily heave rates)/average heave rate (see Note 7).

10. Definition of classes by rate of heave (mm/day): N (negligible) 0-0.5, VL (very low) 0.5-1.0, L (low) 1.0-2.0, M (medium) 2.0-4.0, H (high) 4.0-8.0, VH (very high) above 8.0.

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Table A-2. Summary of Supplementary Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi

The following table contains tests results on soil samples that did not meet the criteria of 95 percent dry weight or 85 percent moisture content.

Table A-2. Summary of Supplementary Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1]

Material Source	Unified Soil Classification Symbol (2)	Maximum Size in.	SOIL GRADATION DATA (AS FROZEN) Percent Finer, mm 4.76	0.42	0.074	0.02	0.01	0.005	Coefficients (3) C_u	C_c	PHYSICAL PROPERTIES OF BASIC SOIL Atterberg Limits (4) LL	PI	Specific Gravity
SILTY GRAVELLY SANDS													
Koflavik	SW-SM	3/4	57	16	5.0	1.4	-	-	27.0	1.1			2.81
B.P.R. Alaska		1	68	12	5.6	2.9	2.3	1.8	10.0	1.0			2.75
Spokane		1-1/2	68	11	7.0	3.5	2.3	1.2	6.7	1.4			2.80
Minnesota		2	69	20	9.6	3.8	-	-	28.0	1.8			2.73
Stewart		2	68	26	9.1	4.0	2.9	1.8	31.0	1.1	19.3	4.3	2.70
Stewart	SW-SM	2	68	26	9.1	4.0	2.9	1.8	31.0	1.1	19.3	4.3	2.70
M.I.T.		1-1/2	70	29	9.7	4.4	3.2	2.5	24.0	1.2			2.70
Hutchinson's Pit		1	57	20	8.7	5.0	3.5	2.0	43.0	1.1			2.75
Hutchinson's Pit		1	57	20	8.7	5.0	3.5	2.0	43.0	1.1			2.75
Hutchinson's Pit		1	57	20	8.7	5.0	3.5	2.0	43.0	1.1			2.75
Loring		1/2	62	13	7.2	5.7	5.0	4.0	15.0	1.3			2.71
Rapid City		1-1/2	57	30	12.0	8.7	7.1	5.8	183.0	1.1	19.0	2.0	2.75
Afghanistan		2	58	23	8.2	3.7	2.3	1.8	48.0	1.2			2.71
Greenland	SP-SM	3/4	60	39	9.7	1.8	0.8	-	62.0	0.2			2.73
Fairchild		2	84	11	5.3	1.9	1.7	-	4.0	1.6			2.75
Minnesota		-	100	71	8.8	2.2	1.3	-	4.3	1.5			2.70
Volk Field		-	100	86	5.0	2.6	2.4	1.8	2.0	0.9			2.66
Indiana		-	100	100	6.3	2.6	2.2	1.7	1.9	1.0			2.65
Indiana		-	100	100	6.3	2.6	2.2	1.7	1.9	1.0			2.65
Indiana		-	100	100	6.3	2.6	2.2	1.7	1.9	1.0			2.65
Minot		1-1/2	73	11	5.2	2.7	2.2	1.6	8.1	0.9			2.73
Dow Field		3/4	66	18	6.0	2.8	1.7	1.0	15.0	0.9			2.72
Selfridge		1-1/2	74	25	6.9	3.2	2.7	1.8	15.0	0.6			2.70
Selfridge		1-1/2	77	27	7.1	3.3	3.0	2.6	13.0	0.7			2.70
Schenectady		3/4	99	84	10.0	3.3	3.0	2.0	3.4	1.8			2.68
Schenectady	SP-SM	3/4	99	84	10.0	3.3	3.0	2.0	3.4	1.8			2.68
Kinross		1-1/2	98	80	8.8	3.3	2.0	0.9	2.8	1.4			2.62
Kinross		-	100	82	9.0	3.4	2.0	0.9	2.8	1.4			2.62
Kinross		-	100	82	9.0	3.4	2.0	0.9	2.8	1.4			2.63
Hutchinson's Pit		2	56	17	6.0	3.5	2.4	-	28.0	0.7			2.74
Korea		2-1/2	58	28	9.4	3.5	2.1	1.4	111.0	0.3			2.61
Korea		2-1/2	56	28	9.4	3.5	2.1	1.4	111.0	0.3			2.61
Lincoln		1	66	22	6.5	3.9	2.7	2.0	17.0	0.9			2.65
Spokane		3/4	79	13	8.1	4.1	2.7	1.5	6.4	3.2			2.80
Spokane		3/4	79	13	8.1	4.1	2.7	1.5	6.4	3.2			2.30
Kinross		3/4	92	67	9.0	4.5	2.9	1.8	4.2	1.2			2.65
Kinross		3/4	92	67	9.0	4.5	2.9	1.8	4.2	1.2			2.65
Kinross		1-1/2	92	67	9.0	4.5	2.9	1.8	4.2	1.2			2.65
Project Blue Jay		3/4	71	46	10.0	4.5	4.0	1.8	20.0	0.3			2.70
Project Blue Jay		3/4	71	46	10.0	4.5	4.0	1.8	20.0	0.3			2.70
Tobyhanna		1-1/2	59	39	8.5	4.5	2.5	1.6	6.0	0.2			2.72
Lincoln		1	80	24	6.5	4.9	3.8	3.0	15.0	0.8			2.65
Lincoln		1	63	30	7.0	5.0	3.0	2.0	28.0	0.4			2.65
Lincoln		1	72	27	7.8	5.0	4.0	3.2	16.0	0.6			2.65
Cape Dyer		2	61	29	9.7	5.1	4.2	3.1	52.0	0.7			2.68
Afghanistan	SP-SM	1	71	32	11.9	5.5	3.9	2.8	-	-			2.73
West Virginia		1-1/2	57	33	10.0	5.6	4.5	-	81.0	0.3			2.70
Volk Field		2	94	83	10.0	5.6	5.0	3.6	3.0	1.5			2.62
Loring		1	65	14	8.0	7.1	6.2	4.6	260.0	34.0	24.0	6.0	2.71
Loring		1	88	17	10.0	8.2	7.2	5.8	20.0	4.9			2.72
CLAYEY SANDS													
Fargo	SC	3/4	98	33	17.0	9.5	7.5	5.5	50.0	5.2	30.7	10.5	2.70
Fargo		3/4	98	33	17.0	9.5	7.5	5.5	50.0	5.2	30.7	10.5	2.70
Project Blue Jay		3/4	73	55	35.0	23.0	20.0	15.5	500.0	1.7	24.7	8.1	2.70
Breed's Hill (EDT)		3/4	76	60	41.0	24.0	-	-	191.0	1.1	24.0	11.0	2.75
Westover		3	82	66	48.0	30.0	23.0	17.0	115.0	0.9	20.7	7.2	2.71
Minnesota		3/4	97	78	48.0	31.0	-	-	-	-	28.7	10.7	2.70
Project Blue Jay		3/4	80	58	44.0	35.0	31.0	22.0	310.0	0.1	18.6	9.2	2.75
Project Blue Jay		3/4	80	58	44.0	35.0	31.0	22.0	310.0	0.1	18.6	9.2	2.75

Table A-2. Summary of Supplementary Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

PHYSICAL PROPERTIES OF BASIC SOIL		SPECIMEN DATA (AS MOLDED)				FREEZING TEST DATA						
Compaction Data (5)						Avg. Water Content						
Maximum Dry Unit Weight	Optimum Moisture Content	Dry Unit Weight	Degree of Compaction	Void Ratio	G. at Start of Test (6)	Before Test	After Test	Total Heave (7)	Rate of Heave mm/day (8)		Heave Rate Var. Index (9)	Frost Susc. Class (10)
pcf	%	pcf	%	%	%	%	%		Avg.	Max.		
112.0(b)	-	111	99	0.532	100	19.5	19.3	2.0	0.3	0.5	1.66	L
123.1(b)	-	117	95	0.467	93	15.8	21.6	15.7	1.2	1.8	1.50	L
-	-	128	> 95	0.365	100	13.0	15.8	13.6	1.1	2.0	1.82	L
135.6(d)	6.5	135	100	0.268	97	9.4	22.0	37.0	2.8	4.3	1.54	M-M
141.7(b)	-	139	98	0.214	100	8.6	18.2	27.9	4.4	6.0	1.35	H
141.7(b)	-	138	98	0.224	100	8.5	20.4	32.2	2.7	4.3	1.39	M-M
137.9(b)	-	131	95	0.285	97	10.2	20.7	21.9	1.2	2.0	1.66	L
143.3(c)	5.3	144	101	0.179	99	6.7	24.7	49.7	6.1	7.7	1.26	M
143.3(c)	5.3	141	98	0.221	87	7.0	37.0	81.3	4.8	5.8	1.20	M
143.3(c)	5.3	138	96	0.242	99	8.7	23.3	43.8	3.5	4.8	1.37	M-M
139.1(b) -	-	135	97	0.256	98	9.2	24.7	37.4	3.3	4.7	1.42	M-M
-	-	137	> 95	0.253	98	8.9	13.9	16.4	1.7	2.5	1.47	L-M
146.7(b)	-	147	>100	0.150	100	5.4	16.4	31.3	3.7	5.3	1.43	M-M
138.0(d)	5.3	137	99	0.246	100	9.0	16.9	20.4	1.3	1.8	1.38	L
123.5(b)	-	121	98	0.421	100	15.3	17.9	10.8	0.9	1.5	1.66	VL-L
114.4(b)	-	114	100	0.473	100	16.8	16.0	2.4	0.2	0.5	2.50	L
115.6(b)	-	115	100	0.450	100	15.3	16.3	2.8	0.1	0.5	5.00	M
107.1(b)	-	109	102	0.516	100	19.3	16.7	0.7	0.1	0.5	5.00	M
107.1(b)	-	109	98	0.576	100	21.8	19.8	1.3	0.1	0.5	5.00	M
107.1(b)	-	109	102	0.514	100	19.4	20.5	2.3	0.4	0.5	1.25	M
130.5(b)	-	129	99	0.316	100	11.5	14.3	8.8	0.5-	1.0	2.00	VL
137.6(b)	-	133	97	0.278	100	10.7	21.8	27.6	1.8	3.3	1.83	L-M
126.8(b)	-	127	100	0.329	100	12.2	19.9	18.3	1.0	1.7	1.70	L
126.8(b)	-	127	100	0.329	100	12.2	20.7	18.0	1.2	2.2	1.83	L-M
113.0(b)	-	113	100	0.484	98	17.7	25.5	16.5	1.1	2.2	2.00	L-M
113.0(b)	-	112	99	0.487	100	18.2	26.2	17.5	1.0	2.0	2.00	L
109.0(b)	-	108	99	0.518	100	19.8	21.5	6.2	0.7	1.2	1.71	VL-L
109.0(b)	-	106	98	0.542	100	20.6	20.9	3.3	0.4	0.7	1.75	M-VL
109.0(b)	-	105	97	0.552	100	20.4	21.1	4.0	0.4	0.8	2.00	M-VL
141.0(b)	-	140	99	0.222	100	8.1	18.2	28.1	3.7	5.5	1.48	M
127.0(b)	-	128	100	0.268	96	9.6	13.0	46.2	2.2	3.5	1.59	M
127.0(b)	-	124	98	0.310	99	11.9	17.3	20.8	3.8	5.0	1.32	M-M
134.0(d)	-	134	100	0.238	100	9.0	13.7	14.0	0.8	1.4	1.75	VL-L
-	-	128	< 95	0.361	100	12.6	17.7	13.5	1.1	1.4	1.27	L
-	-	128	< 95	0.351	90	11.3	18.5	16.3	1.4	2.3	1.64	L-M
120.4(b)	-	115	95	0.438	100	16.5	19.3	8.2	0.8	1.7	2.12	VL-L
120.4(b)	-	119	98	0.396	100	14.9	31.1	36.5	2.7	3.7	1.37	M
120.4(b)	-	120	100	0.367	99	13.9	32.9	44.4	5.4	7.8	1.44	M
142.6(b)	-	138	97	0.215	100	8.0	25.8	29.0	3.1	4.5	1.45	MH
142.6(b)	-	137	96	0.230	100	8.5	37.8	69.4	3.2	5.8	1.81	MH
140.4(b)	-	134	96	0.280	100	9.9	20.2	21.8	1.5	2.3	1.53	L-M
133.1(b)	-	135	101	0.228	98	8.6	13.5	14.9	1.0	1.4	1.40	L
133.1(b)	-	137	103	0.212	100	8.0	12.7	15.8	1.0	1.4	1.40	L
133.1(b)	-	132	99	0.250	98	9.3	15.6	19.6	1.2	1.7	1.42	L
134.8(h)	-	130	97	0.289	94	10.1	24.1	37.3	2.1	3.3	1.57	M
143.2(b)	-	141	-	0.205	96	7.2	19.8	35.5	3.9	7.1	1.82	MH
129.1(b)	-	125	97	0.349	87	11.3	23.4	27.6	1.8	3.3	1.83	L-M
121.6(b)	-	120	97	0.364	100	13.9	31.2	39.2	2.3	3.2	1.39	M
139.1(b)	-	135	97	0.254	93	8.6	14.6	17.4	2.0	3.8	1.90	M
139.1(b)	-	135	97	0.259	99	8.5	31.6	59.8	2.0	4.2	2.10	MH
127.2(d)	9.0	123	97	0.374	100	13.9	21.5	18.7	1.5	2.7	1.80	L-M
127.2(d)	9.0	118	93	0.424	100	15.7	32.8	42.4	3.3	4.5	1.36	M-H
133.1(c)	9.8	134	101	0.272	100	8.0	17.9	25.3	2.2	2.8	1.27	M
138.7(c)	7.2	139	100	0.237	94	8.0	10.5	7.3	0.6	1.0	1.66	VL
-	-	130	> 95	0.297	100	10.9	22.3	31.5	3.1	4.6	1.48	M-M
-	-	114	> 95	0.478	91	16.2	32.0	38.6	1.8	2.5	1.38	L-M
139.6(c)	7.0	139	100	0.234	100	8.5	17.3	26.3	2.2	3.8	1.72	M
139.6(c)	7.0	132	95	0.301	100	10.9	34.7	83.0	4.6	8.3	1.80	H-VM

Table A-2. Summary of Supplementary Frost-Susceptibility tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

Material Source	Unified Soil Classification Symbol (2)	SOIL GRADATION DATA (AS FROZEN)										PHYSICAL PROPERTIES OF BASIC SOIL		
		Maximum Size	Percent Finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity
		in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI	
SILTS AND SANDY SILTS													
Goose Bay	ML	-	100	99	54	6.0	-	-	-	-			2.74
Westover		-	100	91	53	13.0	-	-	-	-			2.69
Labrador		-	100	100	95	27.0	10	-	-	-	26.0	3.0	2.77
Labrador		-	100	100	95	27.0	10	-	-	-	26.0	3.0	2.77
Valparaiso		-	100	100	99	54.0	25	15.0	-	-	23.7	4.0	2.72
Valparaiso		-	100	100	99	54.0	25	15.0	-	-	23.7	4.0	2.72
Hanover		-	100	100	98	35.0	18	8.0	-	-	29.5	12.7	2.75
Dow Field		3/4	95	91	87	54.0	40	28.0	-	-	32.8	8.1	2.66
Minnesota	ML	3/4	97	92	83	63.0	44	28.0	-	-	36.0	5.1	2.6
New Hampshire		-	100	99	97	60.0	22	10.0	-	-	26.6	0.1	2.70
New Hampshire		-	100	99	97	60.0	22	10.0	-	-	26.6	0.1	2.70
New Hampshire		-	100	99	97	60.0	22	10.0	-	-	26.6	0.1	2.70
New Hampshire		-	100	99	97	60.0	22	10.0	-	-	26.6	0.1	2.70
CLAYEY SILTS													
Yukon	ML-CL	-	100	100	98	60.0	37	22.0	-	-	25.3	5.8	2.73
New Hampshire		-	100	100	86	61.0	34	14.0	-	-	24.1	5.9	2.76
New Hampshire		-	100	96	90	67.0	36	16.0	-	-	25.0	6.0	2.70
New Hampshire		-	100	93	85	73.0	47	23.0	-	-	26.0	5.0	2.70
New Hampshire		-	100	100	99	73.0	37	13.0	-	-	23.7	6.0	2.70
New Hampshire		-	100	100	99	73.0	37	13.0	-	-	23.7	6.0	2.70
SILTS AND ORGANICS													
Ladd Field	ML-OL	-	100	100	91	38.0	13	6.0	-	-	31.6	0.0	2.75
Fairbanks		-	100	100	97	42.0	22	12.0	-	-	32.6	6.2	2.67
Fairbanks		-	100	100	97	42.0	22	12.0	-	-	32.6	6.2	2.67
GRAVELLY AND SANDY CLAYS													
Dow Field	CL	3/4	82	70	62	40.0	31	23.0	-	-	25.6	7.9	2.73
Fort Belvoir		3/4	95	87	64	43.0	36	30.0	-	-	41.0	18.0	2.70
East Boston		3/4	84	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
East Boston		3/4	84	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
East Boston		3/4	84	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
East Boston	CL	3/4	84	72	56	43.0	35	25.0	-	-	23.0	7.0	2.75
Fort Belvoir		1/4	98	90	61	49.0	41	34.0	-	-	43.8	20.3	2.73
Fort Belvoir		1/4	98	90	61	49.0	41	34.0	-	-	43.8	20.3	2.73
Fort Belvoir		1/4	98	90	61	49.0	41	34.0	-	-	43.8	20.3	2.73
Fort Belvoir		1/4	98	98	61	49.0	41	34.0	-	-	43.8	20.3	2.73
Portsmouth		-	100	100	96	49.0	38	30.0	-	-	30.0	11.7	2.73
East Boston		3/4	86	73	57	49.0	42	30.0	-	-	21.0	7.0	2.76
East Boston		3/4	86	73	57	49.0	42	30.0	-	-	21.0	7.0	2.76
East Boston		3/4	86	73	57	49.0	42	30.0	-	-	21.0	7.0	2.76
East Boston		3/4	86	73	57	49.0	42	30.0	-	-	21.0	7.0	2.76
Dow Field		3/4	96	93	86	51.0	38	27.0	-	-	26.4	8.4	2.69
Dow Field		3/4	85	82	78	53.0	40	30.0	-	-	27.6	9.5	2.73
AASHTO		1-1/4	95	87	74	58.0	48	38.0	-	-	27.6	11.9	2.74
AASHTO		1-1/4	95	87	74	58.0	48	38.0	-	-	27.3	11.9	2.74
AASHTO		1-1/4	95	87	74	58.0	48	38.0	-	-	27.3	11.9	2.74
AASHTO		1-1/4	95	87	74	58.0	48	38.0	-	-	27.3	11.0	2.74
AASHTO		1-1/4	95	87	74	58.0	48	38.0	-	-	27.3	11.0	2.74
Bong		3/4	97	90	80	60.0	48	36.0	-	-	28.6	12.6	2.80
Bong		3/4	97	90	80	60.0	48	36.0	-	-	28.6	12.6	2.80
Bong		3/4	97	91	81	61.0	50	35.0	-	-	29.6	13.6	2.80
Dow Field		1-1/2	94	88	80	64.0	52	37.0	-	-	30.0	12.0	2.71
Dow Field		1-1/2	94	88	80	64.0	52	37.0	-	-	30.0	12.0	2.71
East Boston	CL	3/4	86	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
East Boston		3/4	86	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
East Boston		3/4	86	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
East Boston		3/4	86	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
East Boston		3/4	86	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
East Boston		3/4	86	72	56	43.0	35	25.0	-	-	23.0	7.0	2.76
Portsmouth		-	100	100	92	46.0	36	30.0	-	-	30.0	11.7	2.73
Portsmouth		-	100	100	92	46.0	36	30.0	-	-	30.0	11.7	2.73
East Boston		3/4	86	72	56	49.0	42	30.0	-	-	21.0	7.0	2.76
East Boston		3/4	86	72	56	49.0	42	30.0	-	-	21.0	7.0	2.76
AASHO		1-1/2	95	88	75	58.0	49	37.0	-	-	27.3	11.9	2.74

Table A-2. Summary of Supplementary Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

PHYSICAL PROPERTIES OF BASIC SOIL		SPECIMEN DATA (AS MOLDED)				FREEZING TEST DATA						
Compaction Data (5)						Avg. Water Content						
Maximum Dry Unit Weight"	Optimum Moisture Content	Dry Unit Weight	Degree of Compaction	Void Ratio	G. at Start of Test (6)	Before Test	After Test	Total Heave (7)	Rate of Heave mm/day (8)		Heave Rate Var. Index (9)	Frost Susc. Class (10)
pcf	%	pcf	%	%	%	%	%	%	Avg.	Mos.		
102.0(c)	7.9	102	100	0.688	100	24.4	25.6	7.0	0.3	1.0	3.33	M-VL
113.6(d)	11.0	112	99	0.484	100	18.0	26.0	17.3	1.0	1.0	1.60	L
102.0(d)	18.1	106	104	0.626	100	22.7	27.3	11.4	1.2	1.5	1.25	L
102.0(d)	18.1	103	102	0.668	94	22.4	30.0	16.3	1.5	2.3	1.53	L-M
115.8(d)	13.5	113	98	0.601	98	18.4	62.1	95.3	9.8	11.5	1.17	VH
115.8(d)	13.5	113	98	0.601	100	18.0	65.5	100.0	10.0	11.9	1.33	VH
103.6(d)	16.9	101	96	0.695	100	24.8	19.0	105.6	14.1	16.8	1.19	VH
107.1(c)	-	104	98	0.500	100	22.2	136.8	104.0	13.0	23.3	1.68	VH
-	-	101	95	0.611	99	23.0	50.6	84.6	3.5	5.8	1.66	M-H
106.7(c)	16.5	106	90	0.580	100	21.7	79.6	126.2	15.9	20.7	1.30	VH
106.7(c)	16.5	104	98	0.618	100	22.8	120.7	130.1	15.9	18.0	1.13	VH
106.7(c)	16.5	108	101	0.567	100	20.9	99.9	190.6	26.0	28.3	1.08	VH
106.7(c)	16.5	105	98	0.611	100	22.6	116.7	239.2	12.8	19.7	1.54	VH
124.5(d)	11.5	123	99	0.389	100	14.2	28.9	37.0	2.2	3.5	1.51	M
106.7(c)	16.5	105	98	0.643	88	20.5	78.1	150.2	7.9	15.8	2.00	M-VH
106.7(c)	16.5	101	95	0.662	100	24.5	84.6	117.6	14.0	18.3	1.30	VH
106.7(c)	16.5	101	95	0.674	100	25.0	86.8	235.3	14.0	15.5	1.10	VH
110.1(c)	14.7	107	97	0.577	99	21.2	42.4	50.8	3.7	4.5	1.22	M-H
110.1(c)	14.7	106	96	0.596	100	22.0	36.4	29.8	4.0	5.3	1.32	H
101.6(d)	18.1	98	97	0.737	100	26.8	45.7	36.5	3.1	4.0	1.20	M
107.4(c)	17.1	101	95	0.646	100	24.2	112.6	223.6	11.3	14.0	1.24	VH
107.4(c)	17.1	111	103	0.505	100	18.9	105.7	281.2	11.5	13.7	1.19	VH
-	-	133	95	0.352	100	12.8	42.7	73.0	4.8	10.3	2.14	M-VH
114.9(a)	15.6	115	100	0.468	94	16.3	25.1	25.0	1.3	2.0	1.54	L
130.8(d)	-	126	96	0.371	100	13.4	46.5	95.3	6.5	10.5	1.62	H-VH
130.8(d)	-	130	99	0.324	100	11.7	30.2	47.7	4.0	5.5	1.38	H
130.8(d)	-	125	96	0.374	100	13.6	22.9	122.9	7.0	11.1	1.58	H-VH
130.8(d)	-	130	99	0.328	100	11.0	34.0	81.1	6.5	7.5	1.15	H
114.9(a)	15.6	110	96	0.536	100	19.8	81.6	188.4	7.7	12.0	1.56	VH
114.9(a)	15.6	117	102	0.456	90	15.0	22.3	18.2	1.6	3.7	2.46	L-M
114.9(a)	15.6	113	98	0.504	100	18.5	27.4	22.1	1.3	3.0	2.30	L-M
114.9(a)	15.6	118	103	0.441	100	16.2	27.0	27.6	2.2	3.2	1.45	M
110.3(a)	17.7	109	99	0.569	95	10.8	60.7	112.7	4.5	12.8	2.84	H-VH
130.8(d)	-	129	98	0.336	100	12.2	37.4	72.5	7.8	12.7	1.62	H-VH
130.8(d)	-	130	99	0.328	100	11.9	29.9	48.3	7.3	37.2	1.2	H-VH
130.8(d)	-	129	98	0.336	100	12.2	32.0	61.2	8.0	10.6	1.31	VH
130.8(d)	-	131	100	0.317	100	11.5	18.3	22.8	4.6	5.7	1.24	M
119.8(d)	14.0	118	98	0.424	100	15.7	49.1	67.0	6.2	10.7	1.72	H-VH
119.8(d)	14.0	119	99	0.429	100	15.8	66.0	125.0	6.6	11.0	1.66	M-VH
121.0(a)	13.5	117	97	0.467	100	17.2	26.4	28.4	2.3	4.3	1.86	V-M
121.0(a)	13.5	120	99	0.420	100	15.5	20.2	17.1	1.3	2.0	1.54	L
121.0(a)	13.5	125	103	0.367	100	13.5	18.6	13.9	1.1	2.3	2.03	L-M
121.0(a)	13.5	119	98	0.442	100	16.3	26.2	27.0	2.8	3.8	1.36	M
121.0(a)	13.5	126	104	0.360	100	13.3	18.0	10.1	1.2	1.3	1.08	L
128.8(c)	-	125	98	0.395	96	13.5	16.1	16.1	1.5	1.7	1.13	L
128.8(c)	-	125	98	0.403	100	14.5	17.7	17.4	1.2	1.7	1.42	L
128.8(c)	-	126	98	0.389	97	13.6	16.7	16.7	1.4	1.5	1.07	L
119.8(d)	14.0	117	98	0.448	100	16.4	69.7	124.3	10.1	12.0	1.20	VH
119.8(d)	14.0	118	98	0.431	100	16.1	42.4	70.7	3.3	3.8	1.15	M
130.8(d)	-	110	84	0.565	100	20.5	76.9	109.1	7.7	10.7	1.38	H-VH
130.8(d)	-	120	91	0.435	100	15.8	122.2	145.0	9.8	12.7	1.30	VH
130.8(d)	-	110	84	0.565	100	20.5	65.8	42.2	6.8	10.2	1.50	H-VH
130.8(d)	-	120	91	0.435	100	15.8	84.1	101.8	8.2	12.2	1.48	VH
130.8(d)	-	120	91	0.430	87	13.6	49.5	45.7	2.4	3.2	1.33	M
130.8(d)	-	110	84	0.561	88	17.8	47.4	27.4	1.9	2.8	1.47	L-M
110.3(c)	17.3	96	-	0.772	98	27.7	62.2	95.3	4.2	7.0	1.66	H
110.3(c)	17.3	95	-	0.798	95	27.7	73.3	114.9	4.1	8.0	1.95	H
130.8(d)	-	120	91	0.433	100	15.7	60.4	96.2	4.1	9.8	2.39	H-VH
130.8(d)	-	110	84	0.561	100	20.3	56.0	70.0	2.6	7.3	2.80	M-H
121.0(a)	13.5	110	91	0.553	100	20.3	90.2	156.8	7.2	11.3	1.56	M-VH

Table A-2. Summary of Supplementary Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

Material Source	Unified Soil Classification Symbol (2)	Maximum Size	SOIL GRADATION DATA (AS FROZEN) Percent Finer, mm						Coefficients (3)		PHYSICAL PROPERTIES OF BASIC SOIL Atterberg Limits (4)		Specific Gravity
		in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI	
LEAN CLAYS													
Greenland	CL	-	100	100	97	60	43	34	-	-	36.5	16.3	2.78
Volk Field		-	100	100	93	77	70	58	-	-	45.0	24.4	2.75
Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77
Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77
Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77
Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77
Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77
Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77
Boston Blue C		-	100	100	100	84	74	63	-	-	43.3	21.6	2.72
Boston Blue C		-	100	100	100	84	74	63	-	-	43.3	21.6	2.72
Dow		-	100	100	100	89	75	57	-	-	33.8	16.4	2.79
Dow		-	100	100	100	89	75	57	-	-	33.8	16.4	2.79
Dow		-	100	100	100	89	75	57	-	-	33.8	16.4	2.79
Dow		-	100	100	100	89	75	57	-	-	33.8	16.4	2.79
Boston Blue C		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72
Boston Blue C		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72
Boston Blue C		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72
Boston Blue C		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72
LEAN CLAYS WITH ORGANICS													
Malad, Idaho	CL-OL	-	100	99	96	65	48	35	-	-	36.9	13.3	2.58
Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58
Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58
Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58
Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58
Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58
FAT CLAYS													
Volk Field	CH	-	100	98	78	68	65	59	-	-	55.5	38.0	2.76
Boston Blue C		-	100	100	100	94	88	81	-	-	52.7	26.1	2.78
Boston Blue C		-	100	100	100	94	88	81	-	-	52.7	26.1	2.78
Niagara		-	100	100	100	94	92	86	-	-	59.3	37.0	2.79
Niagara		-	100	100	100	96	95	91	-	-	60.0	37.4	2.79
Niagara		-	100	100	100	96	95	91	-	-	60.0	37.4	2.79
FAT CLAYS WITH ORGANICS													
Fargo	CH-OH	-	100	100	98	86	76	64	-	-	67.8	45.8	2.76
Fargo		-	100	100	98	86	76	64	-	-	67.8	45.8	2.76

Table A-2. Summary of Supplementary Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

PHYSICAL PROPERTIES OF BASIC SOIL: Compaction Data (5)		SPECIMEN DATA (AS MOLDED)				FREEZING TEST DATA						
Maximum Dry Unit Weight	Optimum Moisture Content	Dry Unit Weight	Degree of Compaction	Void Ratio	S. at Start of Test (6)	Avg. Water Content: Before Test	Avg. Water Content: After Test	Total Heave (7)	Rate of Heave mm/day (8)		Heave Rate Var. Index (9)	Frost Susc. Class (10)
pcf	%	pcf	%	%	%	%	%	%	Avg.	Mos.		
119.4(c)	15.0	92	77	0.930	99	31.3	52.8	41.3	2.9	5.3	1.82	M-H
-	-	101	-	0.683	100	24.8	28.5	19.3	1.0	1.5	1.50	L
-	-	99	-	0.742	96	25.6	214.1	182.2	8.6	12.8	1.48	VH
-	-	99	-	0.753	100	27.2	69.0	131.3	4.7	6.7	1.43	M
-	-	96	-	0.804	93	27.0	46.7	47.2	2.1	3.8	1.80	M
-	-	96	-	0.806	94	27.3	127.3	240.3	8.4	14.0	1.66	VH
-	-	98	-	0.755	98	27.3	88.5	155.2	6.2	7.7	1.24	H
-	-	98	-	0.755	98	26.8	47.5	38.6	2.5	3.7	1.48	H
106.2(c)	20.2	82	-	1.083	94	37.3	60.4	38.4	4.6	5.3	1.15	H
106.2(c)	20.2	79	-	1.162	100	42.5	107.6	141.8	12.6	17.8	1.41	VH
117.0(d)	-	100	85	0.739	87	23.0	115.4	173.4	15.4	21.2	1.38	VH
117.0(d)	-	103	88	0.684	94	23.0	109.2	188.8	19.8	22.8	1.15	VH
117.0(d)	-	105	90	0.660	92	21.8	54.3	67.7	8.6	11.0	1.28	VH
117.0(d)	-	102	87	0.706	93	23.4	87.3	127.8	13.3	17.8	1.54	VH
106.2(c)	20.2	80	-	1.197	97	41.3	124.7	83.3	8.1	11.2	1.38	VH
106.2(c)	20.2	80	-	1.186	98	41.2	124.2	130.8	9.5	15.7	1.65	VH
106.2(c)	20.2	78	-	1.245	98	43.2	96.5	78.1	8.9	12.0	1.34	VH
106.2(c)	20.2	80	-	1.200	100	42.7	93.1	84.7	7.9	11.7	1.48	H-VH
99.6(a)	21.0	92	92	0.745	100	28.9	53.2	63.3	5.4	6.8	1.26	H
99.6(a)	21.0	90	90	0.790	100	30.6	56.0	58.6	5.1	6.3	1.24	H
99.6(a)	21.0	80	80	1.012	100	39.7	94.4	110.7	6.0	9.5	1.58	H-VH
99.6(a)	21.0	84	84	0.913	99	35.7	78.6	90.5	5.2	8.7	1.67	H-VH
99.6(a)	21.0	88	88	0.828	100	32.4	99.1	116.1	5.8	9.2	1.58	H-VH
99.6(a)	21.0	90	90	0.788	100	30.3	101.6	129.9	6.5	9.7	1.49	H-VH
-	-	108	-	0.592	100	21.3	21.3	5.8	0.4	0.5	1.25	H
106.2(c)	20.2	85	<95	1.031	97	36.1	101.8	111.8	4.1	8.3	2.02	H-VH
106.2(c)	20.2	87	<95	0.989	100	35.3	61.2	58.9	2.4	4.8	2.00	M-H
-	-	95	88	0.835	95	29.8	43.7	43.9	2.4	3.0	1.25	M
-	-	93	86	0.874	100	31.4	41.6	35.7	1.5	2.3	1.53	L-M
-	-	94	87	0.845	100	30.4	-	36.8	1.5	2.8	1.86	L-M
-	-	89	<95	0.988	100	35.7	44.5	18.4	1.0	2.0	2.00	L
-	-	89	<95	0.988	100	35.7	48.0	24.0	1.5	2.0	1.33	L

Table A-2. Summary of Supplementary Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.5 psi[1] (Cont.)

NOTES:

1. Data based on 6-inch molded specimens frozen under laboratory conditions of penetration rate of 1/4 to 1/2 inch/day at 32 degrees and free water at specimen base (38 degrees).

2. Soil classifications according to MIL-STD-619 (CE).

3. Gradation coefficients (MIL-STD-619 (CE)):

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

4. Atterberg limits on plastic materials only. Tests on material passing No. 40 sieve only.

5. Natural soil maximum dry weight and optimum moisture for compaction test type a) AASHTO T99 Method A; b) Providence vibrated density test; c) AASHTO T180 Method D; d) AASHTO T180 Method A; e) Harvard minature compaction.

6. Saturation percent at start of freezing test (drained for 24 hours).

7. Based on original frozen height.

8. Average rate of heave determined from maximum representative portion of heave versus time plot (minimum 5 consecutive days).

9. Maximum heave rate (average of 3 highest daily heave rates)/ average heave rate (see Note 7).

10. Definition of classes by rate of heave (mm/day): N (negligible) 0-0.5; VL (very low) 0.5-1.0; L (low) 1.0-2.0; M (medium) 2.0-4.0; H (high) 4.0-8.0; VH (very high) above 8.0.

Table A-3. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.073 psi

The following table contains data on soil samples tested at lower load pressures.

Table A-3. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.073 psi[1]

Material Source	Unified Soil Classification Symbol (2)	SOIL GRADATION DATA (AS FROZEN)									PHYSICAL PROPERTIES OF BASIC SOIL		
		Maximum Size	Percent Finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity
		in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI	
SANDY GRAVELS													
Alaska Highway	GW	2	40	10	3.7	1.9	1.5	0.9	22.0	1.6	-	-	2.64
SILTY SANDY GRAVELS													
Alaska Highway	GP-GM	2	27	10	5.2	3.1	2.0	1.2	40.0	4.7	38.6	2.7	2.73
Alaska Highway	GW-GM	2	44	16	7.2	5.4	3.8	2.4	67.0	2.2	38.6	2.7	2.73
Alaska Highway	GP-GM	2	34	18	11.0	6.2	4.2	2.7	440.0	3.6	25.7	3.6	2.72
Alaska Highway	GP-GM	2	37	20	12.0	8.5	6.5	5.1	310.0	3.1	25.7	3.6	2.70
SILTY GRAVELS													
Ball Mountain Till	GM	2	91	35	18.0	7.0	-	-	250.0	0.3	-	-	2.81
GRAVELLY SANDS													
Alaska Highway	SW	2	53	13	3.8	1.8	1.4	0.9	20.0	1.0	-	-	2.65
Alaska Highway	SC	-	100	100	33.0	2.5	-	-	1.6	1.0	-	-	2.79
Alaska Highway		-	100	100	33.0	2.5	-	-	1.6	1.0	-	-	2.79
CLAYEY SANDS													
Till	SM-SC	3/4	84	65	49.7	36.0	30.0	21.0	225.0	1.0	21.1	6.0	2.72
CLAYEY SILTS													
Valparaiso, Indiana	ML	-	100	100	99.0	54.0	25.0	15.0	-	-	23.7	4.0	2.72
Silt		-	100	100	99.0	54.0	25.0	15.0	-	-	23.7	4.0	2.72
New Hampshire Silt		-	100	99	97.0	60.0	22.0	10.0	-	-	26.6	0.1	2.70
SILTS													
Ladd Field Silt	ML-CL	-	100	100	91.0	38.0	13.0	6.0	-	-	31.6	0.2	2.75
Fairbanks Silt		-	100	100	97.0	42.0	22.0	12.0	-	-	32.6	6.2	2.67
SANDY CLAYS													
East Boston Till	CL	3/4	84	72	56.0	43.0	35.0	25.0	-	-	23.0	7.0	2.76
AASHTO Road Test		1-1/2	95	87	74.0	58.0	48.0	38.0	-	-	27.3	11.9	2.74
AASHTO Road Test		1-1/2	95	87	74.0	58.0	48.0	38.0	-	-	27.3	11.9	2.74
AASHTO Road Test		1-1/2	95	87	74.0	58.0	48.0	38.0	-	-	27.3	11.9	2.74
Yukon Silt		-	100	100	100.0	67.0	37.0	29.0	-	-	28.0	8.6	2.74
Yukon Silt		-	100	100	100.0	67.0	37.0	29.0	-	-	28.0	8.6	2.74

NOTES:

1. Data based on 6-inch molded specimens frozen under laboratory conditions of penetration rate of 1/4 to 1/2 inch/day at 32 degrees and free water at specimen base (38 degrees).

2. Soil classifications according to MIL-STD-619 (CE).

3. Gradation coefficients (MIL-STD-619 (CE)):

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

4. Atterberg limits on plastic materials only. Tests on material passing No. 40 sieve only.

5. Natural soil maximum dry weight and optimum moisture for compaction test type a) AASHTO T99 Method A, b) Providence vibrated density test, c) AASHTO T180 Method D, d) AASHTO T180 Method A, e) Harvard miniature compaction.

6. Saturation percent at start of freezing test (drained for 24 hours).

Table A.3. Summary of Frost-Susceptibility Tests on Natural Soils - Open System Nominal Load Pressure 0.073 psi[1] (Cont.)

PHYSICAL PROPERTIES OF BASIC SOIL		SPECIMEN DATA (AS MOLDED)				FREEZING TEST DATA						
Compaction Data (5)						Avg. Water Content						
Maximum Dry Unit Weight	Optimum Moisture Content	Dry Unit Weight	Degree of Compaction	Void Ratio	S. at Start of Test (6)	Before Test	After Test	Total Heave (7)	Rate of Heave mm/day (8)		Heave Rate Var. Index (9)	Frost Susc. Class (10)
pcf	%	pcf	%	%	%	%	%	%	Avg.	Mos.		
133.4(b)	-	132	99	0.249	100	9.4	11.6	1.9	0.9	1.3	1.45	VL-L
123.6(b)	-	121	98	0.401	100	14.7	18.3	17.6	1.1	2.5	2.27	L-M
118.5(b)	-	121	102	0.401	100	10.6	20.8	17.6	2.4	3.8	1.65	M
127.0(b)	-	126	99	0.336	77	9.5	20.8	30.5	1.9	3.7	1.95	L-M
126.7(b)	-	128	101	0.315	94	11.0	19.6	29.7	1.9	3.3	1.74	L-M
-	-	147	-	0.195	100	5.6	11.7	17.4	1.4	3.8	2.71	L-M
132.9(b)	-	129	97	0.277	100	10.5	12.2	10.2	1.0	1.7	1.70	L
106.4(b)	-	112	105	0.551	92	18.2	32.8	20.0	2.0	3.0	1.50	M
106.4(b)	-	111	105	0.565	100	20.3	29.3	11.1	1.1	1.7	1.54	L
133.8(d)	8.3	133	99	0.279	100	10.2	17.1	24.7	1.4	2.7	1.93	L-M
115.8(d)	13.5	112	96	-	72	13.5	53.1	81.4	6.8	11.0	1.62	H-VH
115.8(d)	13.5	112	96	-	94	17.7	45.2	142.3	5.6	11.5	1.98	H-VH
106.7(c)	16.5	105	99	0.609	100	22.5	105.8	155.1	11.7	17.8	1.52	VH
101.6(d)	16.1	99	92	0.724	100	26.4	66.1	93.2	7.1	9.5	1.34	H-VH
107.4(c)	17.1	102	95	0.602	100	24.8	61.0	55.7	5.5	11.3	2.05	H-VH
130.8(d)	-	125	96	0.380	100	13.8	63.9	130.1	11.5	14.0	1.28	VH
121.0(a)	13.5	116	96	0.481	100	17.6	31.2	34.9	3.1	3.3	1.06	M
121.0(a)	13.5	114	94	0.497	100	11.2	29.0	31.4	3.5	4.3	1.03	M-H
121.0(a)	13.5	122	105	0.414	100	15.3	43.8	72.7	2.5	3.7	1.48	M
121.4(d)	12.8	120	99	0.443	91	15.3	26.2	33.1	1.6	2.8	1.75	L-H
121.4(d)	12.8	118	97	0.775	99	15.1	27.2	24.3	4.2	4.5	1.07	H

7. Based on original frozen height.

8. Average rate of heave determined from maximum representative portion of heave versus time plot (minimum 5 Consecutive days).

9. Maximum heave rate (average of 3 highest daily heave rates)/average heave rate (see Note 7).

10. Definition of classes by rate of heave (mm/day): N (negligible) 0-0.5, VL (very low) 0.5-1.0, L (low) 1.0-2.0, M (medium) 2.0-4.0, H (high) 4.0-8.0, VH (very high) above 8.0.

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APPENDIX B

USE OF INSULATION MATERIALS IN PAVEMENTS

B-1. Insulating materials and insulated pavement systems. The only acceptable insulating material for use in roads and airfields is extruded polystyrene boardstock. Results from laboratory and field tests have shown that extruded polystyrene does not absorb a significant volume of moisture and that it retains its thermal and mechanical properties for several years, at least. The material is manufactured in board stock ranging from 1 to 4 inches thick.

a. Experience has shown that surface icing may occur on insulated pavements at times when uninsulated pavements nearby are ice-free and vice versa. Surface icing creates possible hazards to fast-moving aircraft and motor vehicles. Accordingly, in evaluating alternative pavement sections, the designer should select an insulated pavement only in special cases not sensitive to differential surface icing. Special attention should be given to the need for adequate transitions to pavements having greater or lesser protection against subgrade freezing.

b. An insulated pavement system comprises conventional surfacing and base above an insulating material of suitable thickness to restrict or prevent the advance of subfreezing temperatures into a frost-susceptible subgrade. Unless the thickness of insulation and overlying layers is sufficient to prevent subgrade freezing, additional layers of granular materials are placed between the insulation and the subgrade to contain a portion of the frost zone that extends below the insulation. In consideration of only the thermal efficiency of the insulated pavement system, an inch of granular material placed below the insulating layer is much more effective than an inch of the same material placed above the insulation. Hence, under the design procedure outlined below, the thickness of the pavement and base above the insulation is determined as the minimum that will meet structural requirements for adequate cover over the relatively weak insulating material, and the determination of the thickness of insulation and of additional granular material is predicated on the placement of the latter beneath the insulation.

B-2. Determination of thickness of cover above insulation. On a number of insulated pavements in the civilian sector, the thickness of material above the insulation has been established to limit the vertical stress on the insulation caused by dead loads and wheel loads to not more than one-third of the compressive strength of the insulating material. The Boussinesq equation should be used for this determination.

B-3. Design of insulated pavement to prevent subgrade freezing. Once the thickness of pavement and base above the insulation has been determined, it should be ascertained whether a reasonable thickness of insulation will keep subfreezing temperatures from penetrating through the insulation. Calculations for this purpose make use of the design air and surface freezing indexes and the mean annual soil temperature at the site. If the latter is unknown, it may be approximated by adding 7 degrees F. to the mean annual air temperature. If the design surface freezing index cannot be calculated from air temperature measurements at the site, or cannot be estimated using data from nearby sites, it may be estimated by multiplying the design air freezing index, calculated as described in paragraphs 1-2b and 3-2b, by the appropriate n-factor. For paved surfaces kept free from snow and ice, and n-factor of 0.75 should be used. For calculating the required thickness of insulation, the design surface freezing index and the mean annual soil temperature are used with figure B-1 to determine the surface temperature amplitude A. The initial temperature differential v_0 is obtained by subtracting 32 degrees F. from the mean annual soil temperature, or it also may be read directly from figure B-1. The ratio v_0/A is then determined. Figure B-2 is then entered with the adopted thickness of pavement and base to obtain the thickness of extruded polystyrene insulation needed to prevent subgrade freezing beneath the insulation. If the required thickness is less than about 2 to 3 inches, it will usually be economical to adopt for design the thickness given by figure B-2, and to place the insulation directly on the subgrade. If more than about 2 to 3 inches of insulation are required to prevent subgrade freezing, it usually will be economical to use a lesser thickness of insulation, underlain by subbase material (S1 or S2 materials in table 2-1). Alternative combinations of thicknesses of extruded polystyrene insulation and granular material (base and subbase) to completely contain the zone of freezing can be determined from figure B-3, which shows the total depth of frost for various freezing indices, thicknesses of extruded polystyrene insulation, and base courses. The thickness of subbase needed to contain the zone of freezing is the total depth of frost penetration less the total thickness of pavement, base and insulation.

B-4. Design of insulated pavement for limited subgrade freezing. It may be economically advantageous to permit some penetration of frost into the subgrade. Accordingly, the total depth of frost penetration given by figure B-3 may be taken as the value a in figure 4-1, and a new combined thickness b of base, insulation, and subbase is determined that permits limited frost penetration into the subgrade. The thickness of subbase needed beneath the insulation is obtained by subtracting the previously established thicknesses of base, determined from structural requirements, and of insulation, determined from figure B-3. Not less than 4 inches of subbase material meeting the requirements of paragraph 5-4 should be placed between the insulation and the subgrade. If less than 4 inches of subbase material is necessary, consideration should be given to decreasing the insulation thickness and repeating the process outlined above.

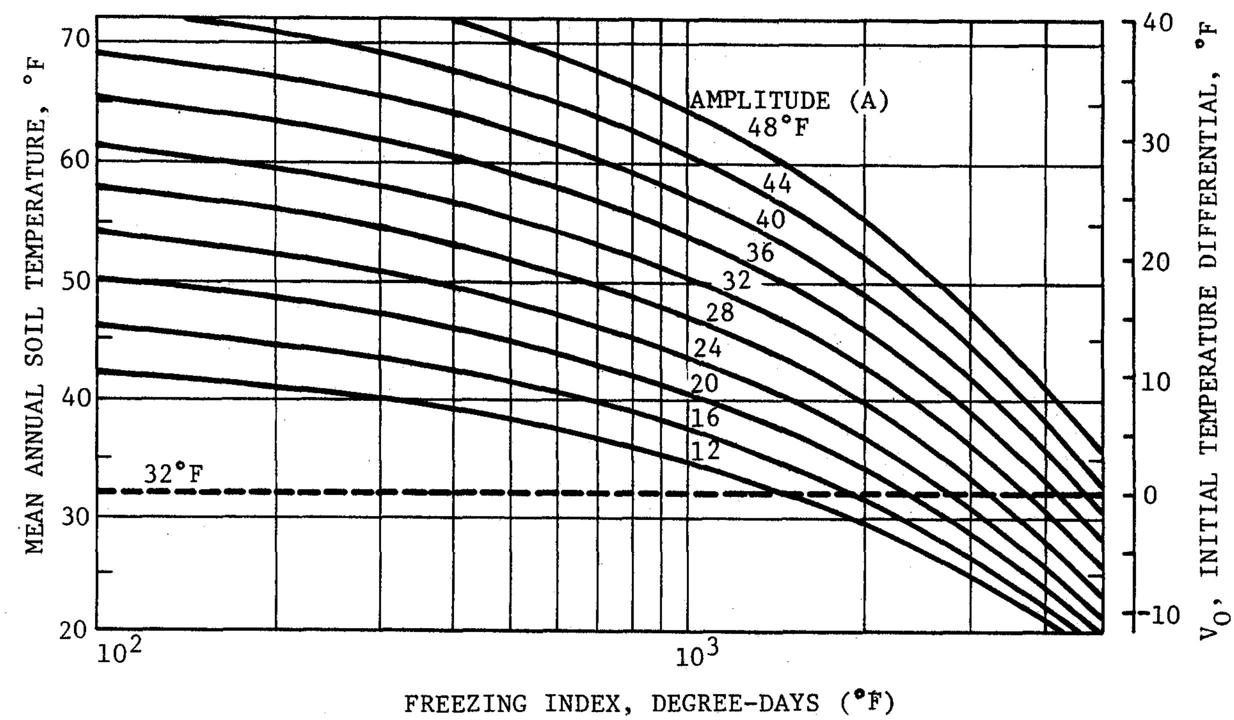

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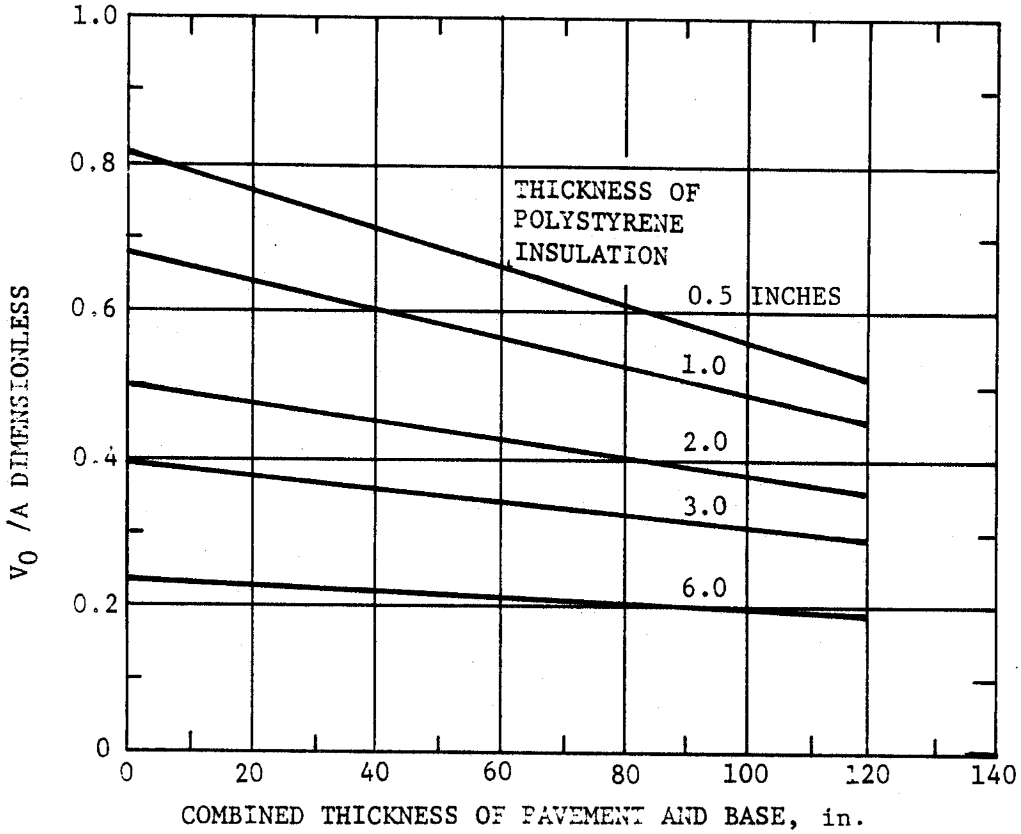
FIGURE B-1. EQUIVALENT SINUSOIDAL SURFACE TEMPERATURE AMPLITUDE A AND INITIAL TEMPERATURE DIFFERENCE, V_0



NOTE: DESIGN CURVES BASED ON THE FOLLOWING MATERIAL PROPERTIES:
PAVEMENT: SAME THERMAL PROPERTIES AS UPPER BASE
BASE: Y_d = 135 pcf, W = 7 PERCENT
EXTRUDED POLYSTYRENE INSULATION

$Y_d = 2.0$ pcf, $K = 0.21 \dfrac{\text{Btu in.}}{\text{ft}^2 \text{ hr } ^\circ\text{F.}}$

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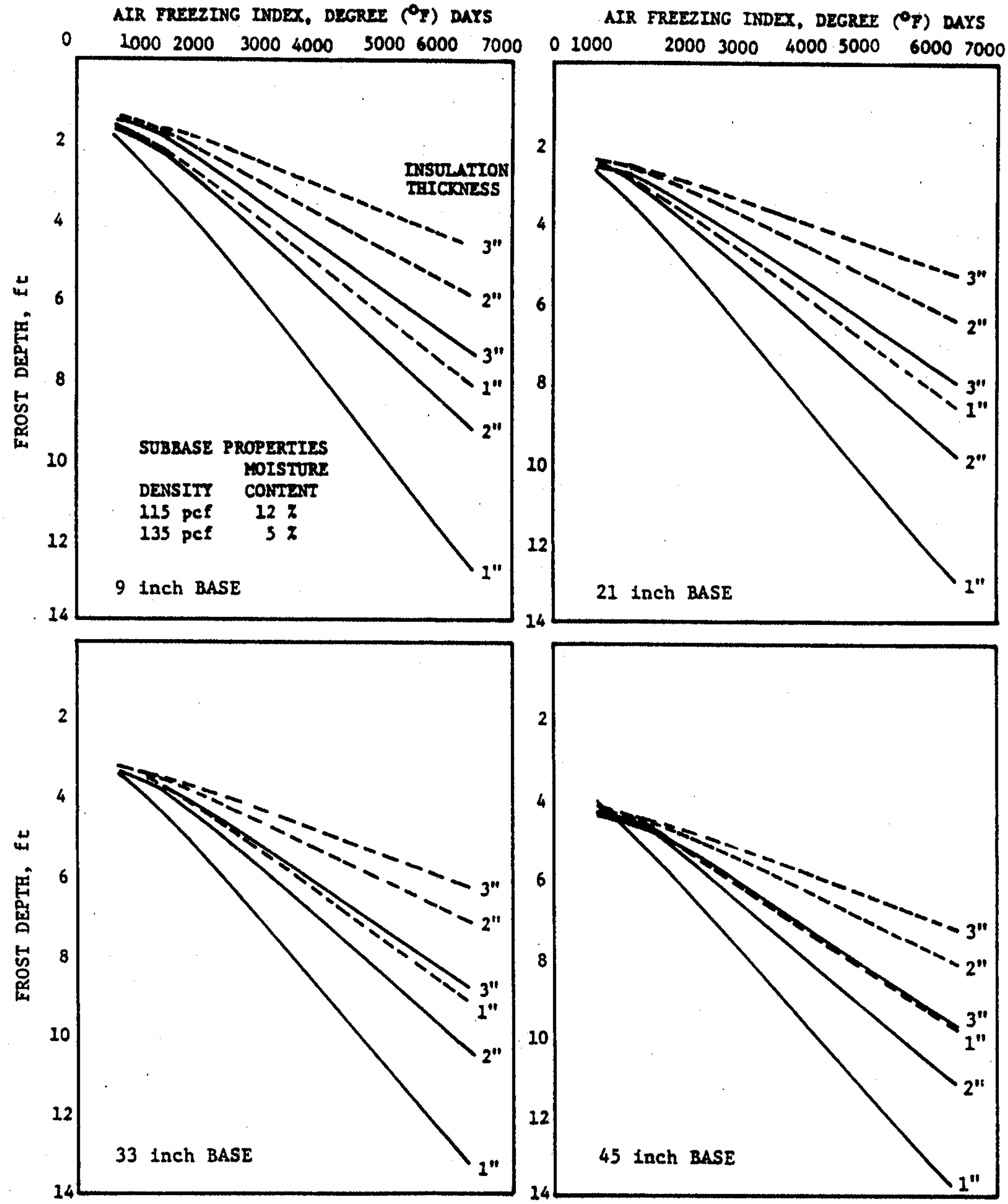
FIGURE B-2. THICKNESS OF POLYSTYRENE INSULATION TO PREVENT SUBGRADE FREEZING



NOTES: PAVEMENT: 3 INCH BITUMINOUS
BASE COURSE: Y_d = 135 pcf, W = 5 percent

INSULATION: EXTENDED POLYSTYRENE

Y_d = 2 pcf; $K = 0.21 \frac{\text{Btu in}}{\text{ft}^2 \text{ hr } ^\circ\text{F.}}$

UNDERLYING GRANULAR MATERIAL:
——— Y_d = 115 pcf, W = 12 percent
– – – – Y_d = 135 pcf, W = 5 percent

SURFACE TRANSFER COEFFICIENT = 0.75

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FIGURE B-3. EFFECT OF INSULATION THICKNESS AND BASE ON FROST PENETRATION

B-5. Construction practice. While general practice has been to place insulation in two layers with staggered joints, this practice should be avoided at locations where subsurface moisture flow or a high ground water table may be experienced. In the latter cases it is essential to provide means for passage of water through the insulation to avoid possible excess hydrostatic pressure in the soil on which the insulating material is placed. Free drainage may be provided by leaving the joints between insulating boards slightly open, or by drilling holes in the boards, or both.

APPENDIX C

MEMBRANE-ENCAPULATED SOIL LAYERS (MESL)

C-1. Concept of encapsulation. Fine-grained soils exhibit high strength and low deformability (high stiffness) when well compacted at moisture contents below optimum. The membrane-encapsulated soil layer (MESL) is a developing technique to assure the permanence of these desirable properties by preserving the moisture content at its initial low level. Full-scale test sections have indicated excellent structural performance of a lean clay MESL serving as either base or subbase course in pavement systems in a warm climate. Experimental pavements undergoing tests in New Hampshire and Alaska also indicate that under favorable conditions MESL may serve as an acceptable replacement for granular material. Laboratory tests on fine-grained soils have shown that freezing under a closed system, i.e., preventing inflow of water from sources outside the moist soil specimen being tested, causes much less frost heave than freezing of similar specimens in the open system, i.e., with water fully available. Loss of supporting capacity during thaw also is much reduced in fine-grained soils that have been compacted at low moisture contents, because less moisture is available during freezing.

C-2. Testing requirements. If a MESL is proposed to be used in a pavement system in a frost area, any soil intended to be encapsulated should be thoroughly tested to determine classification index properties and CBR-moisture-density relationships. Representative samples should be tested to determine the effect of closed-system freezing on volume expansion, moisture migration, and reduction of resilient modulus, CBR, or other measure of supporting capacity, and to ascertain the moisture content at which the material must be placed to acceptably limit adverse frost effects. The results of the testing together with pavement design criteria in EM 1110-3-131 and EM 1110-3-141 will also serve to indicate at what levels in the layered pavement system the MESL may be used.

C-3. Materials.

a. Fine-grained soils. As guidance in the preliminary appraisal of the feasibility of MESL at a given location that experiences subfreezing temperatures, tests to date have shown that, among the fine-grained soils, soils of higher plasticity tend to respond most favorably to closed-system freezing. In general, it will be necessary to compact the soil on the dry side of optimum moisture content. Even nonplastic silts are substantially altered in their response to freezing by closed-system conditions, but tests to date indicate it will be necessary to place such soils at moisture contents several percentage points below the optimum values. The need for placement of encapsulated soil at low moisture contents establishes regional limits for the economical application of the MESL concept. Suitable soil existing at a low moisture content must be available within economical haul distance, or the climate and rainfall regime must be such that reduction of moisture contents of the soil be economically feasible.

b. Membrane materials. From tests performed to date, it is considered that the most suitable membranes for use in cold regions are the same materials used in temperate climates. Successful experimental use has been made of a lower membrane of clear, 6-mil polyethylene, and an upper membrane of polypropylene cloth, field-treated with cationic emulsified asphalt conforming to ASTM D 2397, Grade CRS-2.

C-4. Construction practice. Construction techniques for encapsulation of soil have been developed in experimental projects. The recommended construction procedures have been summarized in a report for the Federal Highway Administration (Implementation Package 74-2). Special requirements for frost areas, not covered in the referenced report, relate to the rigorous control of moisture contents to meet the limiting values determined as outlined in paragraph C-2.

APPENDIX D

MINIMIZING LOW-TEMPERATURE CONTRACTION CRACKING OF BITUMINOUS PAVEMENTS

D-1. Causes and effects of low-temperature contraction cracks. In cold regions, one of the most prevalent and objectionable modes of distress, affecting only bituminous pavements, is thermal cracking. This type of cracking includes thermal fatigue cracking caused by repeated (often diurnal) cycles of high and moderately low temperatures, and low-temperature contraction cracking, which results from thermal contraction of the bituminous-stabilized layer. The thermal contraction induces tensile stresses in the cold and relatively brittle bituminous mixture in the layer because it is partially restrained by friction along the interface with the supporting layer. In very cold regions, some of the cracks may penetrate through the pavement and down into the underlying materials. Unfortunately, in the winter, when the most severe tensile stresses develop, flexible pavements are less ductile and more brittle than in other seasons. Closely spaced thermal cracks are particularly detrimental in airfield pavements because the crack edges may ravel and produce surface debris that can damage jet engines. The ingress of water through the cracks also tends to cause loss of bond, increasing the rate of stripping, and resulting in some cases in a depression at the crack brought about by raveling of the lip of the crack and pumping of the fine fraction of base material. During the winter months when the entire pavement and substructure 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 form an ice lens below the crack that produces an upward lipping of the crack edges. Both of these effects result in rough-riding qualities and often secondary cracks are produced that parallel the major crack. Pavement roughness at low-temperature contraction cracks can be especially severe where subgrade soils are expansive clays; moisture entering the cracks causes localized swelling of subgrade soil, which results in upheaval of the pavement surface at and adjacent to each crack.

D-2. Effect of penetration and viscosity of asphalt. Currently, the most effective means available to minimize low-temperature contraction cracking is the use of asphalt that becomes less brittle at low temperatures. This may be accomplished in part by use of soft grades of asphalt such as AC-5 and AC-2.5. It may also be accomplished in part by use of asphalt of low temperature-susceptibility. A useful measure of temperature-susceptibility of asphalt cement is the pen-vis number (PVN) which may be determined from the penetration at 77 degrees F. and the kinematic viscosity at 275 degrees F. (fig D-1). Current Corps of Engineers specifications for asphalt for use in pavements in cold regions require a PVN not lower than -0.5. For airfields and major roadways in severely cold climates, asphalt cement is to be

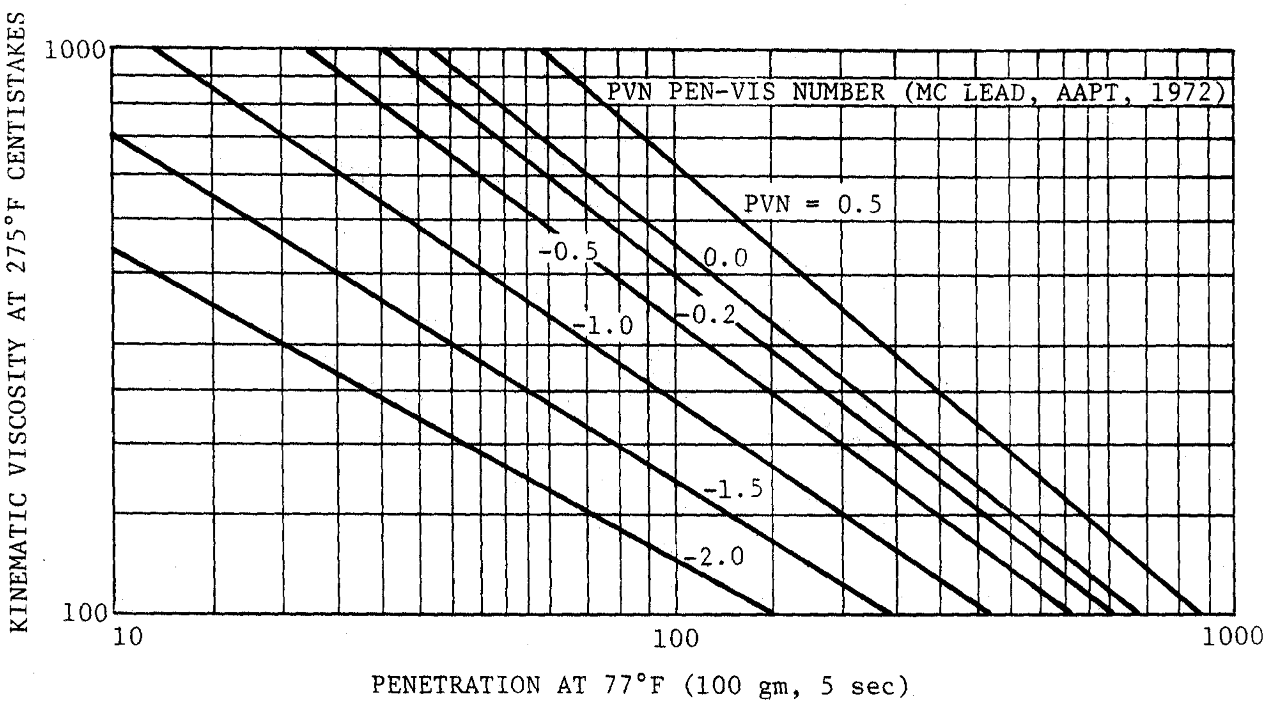

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FIGURE D-1. PEN-VIS NUMBERS OF ASPHALT CEMENT selected and specified in accordance with the requirements for special grades having a minimum PVN of -0.2.

D-3. Selection of asphalt. Figure D-2 is a useful guide for selection of asphalts that will resist low-temperature cracking for various minimum temperatures. To minimize low-temperature contraction cracking during a pavement's service life, a grade of asphalt should be selected that lies to the right of the diagonal line representing the lowest temperature expected during the service life at a depth of 2 inches below the pavement surface. In the absence of temperature data from nearby pavements, the minimum temperature at 2 inches below the surface may be taken as the lowest air temperature in the period of record (not less than 10 years), plus 5 degrees F. It can be seen from figure D-2 that if asphalt of relatively high PVN can be obtained, selection of extremely soft grades of asphalt will be unnecessary, except in the most severely cold environments. Asphalt of grades AC-2.5, -5, or -10, or the equivalent AR grades, should be selected for airfield pavements and roads in cold regions. For roads with a design index of 4 or less in extremely cold regions, slow-curing road oil SC-3000 also is acceptable.

D-4. Effect of mix design variables. It may not always be possible to use the extremely soft grades indicated by figure D-2 for very low temperatures and still produce mixtures meeting the requirements of EM 1110-3-131 and EM 1110-3-141. In that event, the softest grade that will still meet those requirements should always be selected. In designing asphalt-aggregate mixtures in accordance with EM 1110-3-131 and EM 1110-3-141, it should be realized that age-hardening of asphalt, which leads to increasing incidence of low-temperature cracking, will be retarded if air voids are maintained near the lower specified limit. Consequently, mix design and compaction requirements are especially critical for pavements that will experience low temperatures. Asphalt content in most cases should be set at a level above the optimum value, and it may be necessary to readjust the aggregate gradation slightly to accommodate the additional asphalt.

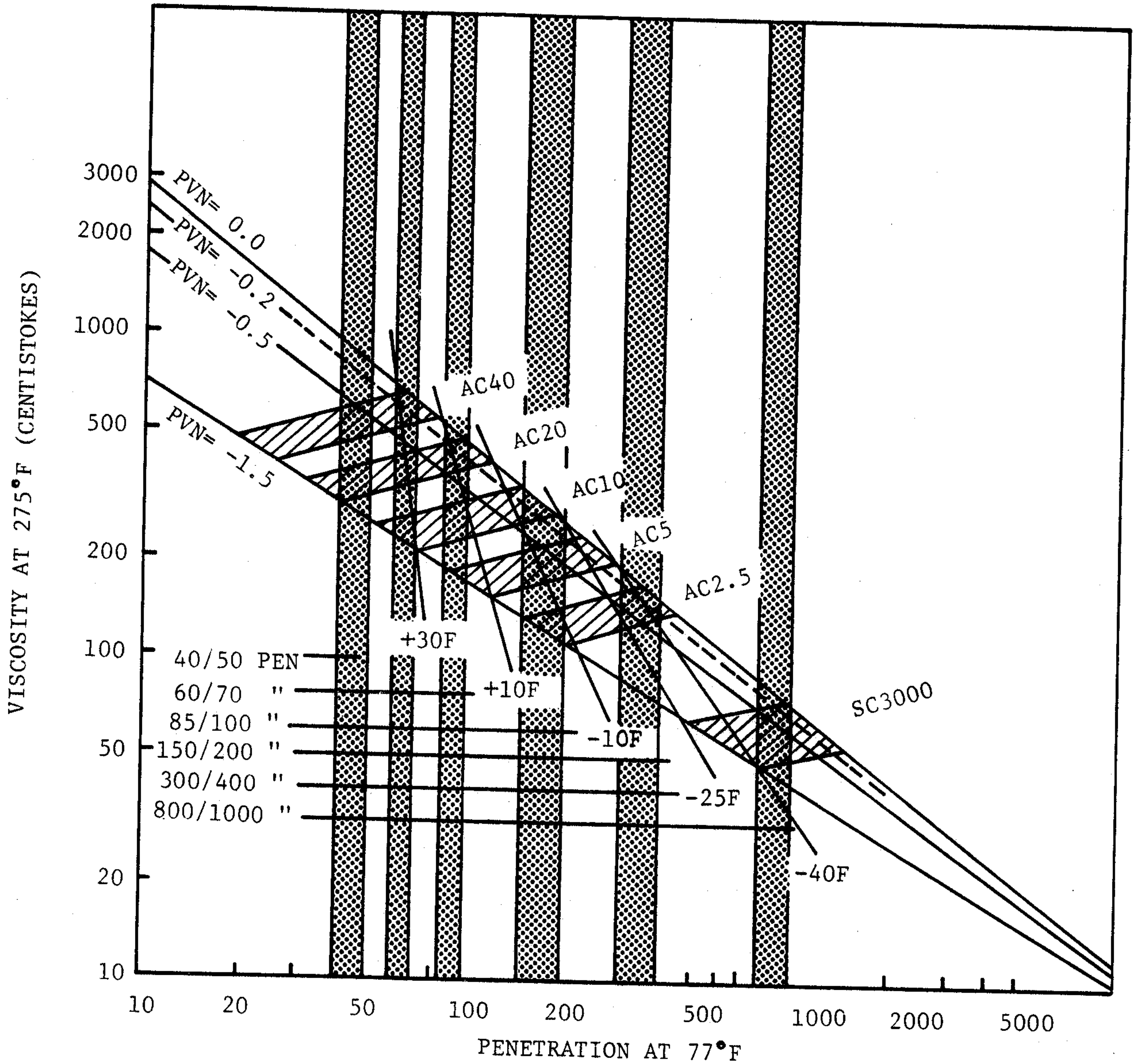

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FIGURE D-2. GUIDE TO SELECTION OF ASPHALT FOR PAVEMENTS IN COLD REGIONS

APPENDIX E

FIELD CONTROL OF SUBGRADE AND BASE COURSE CONSTRUCTION FOR FROST CONDITIONS

E-1. General. Field control of airfield and highway pavement construction in areas of seasonal freezing should give specific consideration to conditions and materials that will result in detrimental frost action. The contract plans and specifications should require the subgrade preparation work established in this manual in frost areas. They also should provide for special treatments, such as removal of unsuitable materials encountered, with sufficient information included to identify those materials and specify necessary corrective measures. However, construction operations quite frequently expose frost-susceptible conditions at isolated locations of a degree and character not revealed by even the most thorough subsurface exploration program. It is essential, therefore, that personnel assigned to field construction control be alert to recognize situations that require special treatment, whether or not anticipated by the designing agency. They must also be aware of their responsibility for such recognition.

E-2. Subgrade preparation. The subgrade is to be excavated and scarified to a predetermined depth, windrowed and bladed successively to achieve adequate blending, and then relaid and compacted. The purpose of this work is to achieve a high degree of uniformity of the soil conditions by mixing stratified soils, eliminating isolated pockets of soil of higher or lower frost-susceptibility, and blending the various types of soils into a single, relatively homogeneous mass. It is not intended to eliminate from the subgrade those soils in which detrimental frost action will occur, but to produce a subgrade of uniform frost-susceptibility and thus create conditions tending to make both surface heave and subgrade thaw-weakening as uniform as possible over the paved area. The construction inspection personnel should be alert to verify that the processing of the subgrade will yield uniform soil conditions throughout the section. To achieve uniformity in some cases, it will be necessary to remove highly frost-susceptible soils or soils of low frost-susceptibility. In that case the pockets of soil to be removed should be excavated to the full depth of frost penetration and replaced with material of the same type as the surrounding soil.

a. A second, highly critical condition requiring the rigorous attention of inspection personnel is the presence of cobbles or boulders in the subgrades. All stones larger than about 6 inches in diameter should be removed from fill materials for the full depth of frost penetration, either at the source or as the material is spread in the embankments. Any such large stones exposed during the subgrade preparation work also must be removed, down to the full depth to which subgrade preparation is required. Failure to remove stones or large roots can result in increasingly severe pavement roughness as the stones or roots are heaved gradually upward toward the pavement surface. They eventually break through the surface in extreme cases, necessitating complete reconstruction.

b. Abrupt changes in soil conditions must not be permitted. Where the subgrade changes from a cut to a fill section, a wedge of subgrade soil in the cut section with the dimensions shown in figure 7-1 should be removed and replaced with fill material. Tapered transitions also are needed at culverts beneath paved areas (fig 7-3), but in such cases, the transition material should be clean, non-frost-susceptible granular fill. Other under-pavement pipes should be similarly treated, and perforated-pipe underdrains should be constructed as shown in figure 7-2. These and any other discontinuities in subgrade conditions require the carefullest attention of construction inspection personnel, as failure to enforce strict compliance with the requirements for transitions may result in serious pavement distress.

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

d. In areas where rock excavation is required, the character of the rock and seepage conditions should be considered. In any case, the excavations should be made so that positive transverse drainage is provided, and so that no pockets are left on the rock surface that will permit ponding of water within the depth of freezing. The irregular ground water availability created by such conditions may result in markedly irregular heaving under freezing conditions. It may be necessary to fill drainage pockets with lean concrete. At intersections of fills with rock cuts, the tapered transitions mentioned above and shown in figure 7-1 are essential. Rock subgrades where large quantities of seepage are involved should be blanketed with a highly pervious material to permit the escape of water. Frequently, the fractures and joints in the rock contain frost-susceptible soils. These materials should be cleaned out of the joints to the depth of frost penetration and replaced with non-frost-susceptible material. If this is impractical, it may be necessary to remove the rock to the full depth of frost penetration.

e. An alternative method of treatment of rock subgrades--in-place fragmentation--has been used effectively in road construction. Blast holes 3 to 6 feet deep are commonly used. They are spaced suitably for achieving thorough fragmentation of the rock to permit effective drainage of water through the shattered rock and out of the zone of freezing in the subgrade. A tapered transition should be provided between the shattered rock cut and the adjacent fill.

E-3. Base course construction. Where the available base course materials are well within the limiting percentages of fine material set forth in chapter 5 of this manual, the base course construction control should be in accordance with normal practice. In instances where the material selected for use in the top 50 percent of the total thickness of granular unbound base is borderline with respect to percentage of fine material passing the No. 200 sieve, or is of borderline frost-susceptibility (usually materials having 1-1/2 to 3 percent of grains finer than 0.02 millimeters by weight), frequent gradation checks should be made to insure that the materials meet the design criteria. If it is necessary for the contractor to be selective in the pit in order to obtain suitable materials, his operations should be inspected at the pit. It is more feasible to reject unsuitable material at the source when large volumes of base course are being placed. It may be desirable to stipulate thorough mixing at the pit and, if necessary, stockpiling, mixing in windrows, and spreading the material in compacted thin lifts in order to insure uniformity. Complete surface stripping of pits should be enforced to prevent mixing of detrimental fine soil particles or lumps in the base material.

a. The gradation of materials taken from the base after compaction, such as density test specimens, should be determined frequently, particularly at the start of the job, to learn whether or not fines are being manufactured in the base under the passage of the base course compaction equipment. For base course materials exhibiting possibly serious degradation characteristics, construction of a test embankment may be warranted to study the manufacture of fines under the proposed or other compaction efforts. Mixing of base course materials with frost-susceptible subgrade soils should be avoided by making certain that the subgrade is properly graded and compacted prior to placement of base course, by insuring that the first layer of base course filters out subgrade fines under traffic, and by eliminating the kneading caused by overcompaction or insufficient thickness of the first layer of base course. Experience has shown that excessive rutting by hauling equipment tends to cause mixing of subgrade and base materials. This can be greatly minimized by frequent rerouting of material-hauling equipment.

b. After completion of each course of base, a careful visual inspection should be made before permitting additional material placement to insure that areas with high percentages of fines are not present. In many instances these areas may be recognized both by examination of the materials and by observation of their action under compaction equipment, particularly when the materials are wet. The materials in any areas that do not meet the requirements of the specifications, which will reflect the requirements of this manual, should be removed and replaced with suitable material. A leveling course of fine-grained material should not be used as a construction expedient to choke open-graded base courses, to establish fine grade, or to prevent overrun of concrete. Since the base course receives high stresses from traffic, this prohibition is essential to minimize weakening during the frost-melting period. Action should be taken to vary the base course thickness so as to provide transition, when this is necessary, to avoid abrupt changes in pavement supporting conditions.

APPENDIX F

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