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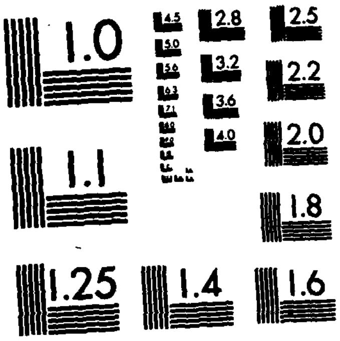

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Revised procedure for pavement design under seasonal frost conditions

Richard Berg and Thaddeus Johnson

Prepared for
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This report presents engineering guidance and design criteria for pavements at Army and Air Force facilities in seasonal frost areas. Design methods for controlling surface roughness and loss of subgrade strength during thawing periods are provided. Criteria for using thermal insulating materials and membrane encapsulated soil layers in seasonal frost areas are presented. Six design examples are included.

PREFACE

This report was prepared by Richard L. Berg, Research Civil Engineer, Geotechnical Research Branch and Thaddeus C. Johnson, Chief, Civil Engineering Research Branch, both of the Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory (USACRREL). It contains recently revised design criteria for road and airfield pavements in seasonal frost areas, and it has been prepared with the final objective of publication as a replacement for an existing engineering manual (Department of the Army Technical Manual TM 5-818-2 and Department of the Air Force Manual 88-6, Chapter 4 dated July 1965). It has been issued as a CRREL Special Report to promote dissemination of this knowledge to engineers concerned with pavement design in seasonal frost areas. The text and figures in this report are nearly the same as in the proposed Technical Manual. Comments for improvement are welcome.

Kenneth A. Linell and Edward F. Lobacz, both formerly of CRREL, prepared most of the original criteria.

The authors thank August Muller and Edwin Dudka, Office of the Chief of Engineers, and the many engineers and geologists from Corps of Engineers Districts and Divisions for technically reviewing the contents of this report. Engineers and geologists from the Air Force also technically reviewed the contents.

Special thanks is offered to Edward Perkins, Matthew Pacillo, Thomas Vaughan, William Bates, Nancy Richardson and Mark Hardenberg, all of the Technical Information Branch, USACRREL, for their invaluable assistance in the preparation of illustrations, typing and editing of this report.

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CONVERSION FACTORS: U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

These conversion factors include all the significant digits given in the conversion tables in the ASTM Metric Practice Guide (E380), which has been approved for use by the Department of Defense. Converted values should be rounded to have the same precision as the original (see E380).

Multiply	By	To obtain
mil	0.00254*	millimetre
inch	25.4*	millimetre
square inch	0.00064516	square metre
foot	0.3048*	metre
pound (avoirdupois)	0.4535924	kilogram
pound per cubic foot	16.01846	kilogram per cubic metre
pound per square foot	4.882428	kilogram per square metre
degrees Fahrenheit	$t_{°C} = (t_{°F}-32)/1.8$	degrees Celsius

*Exact.

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CHAPTER 1

GENERAL

1-1. Purpose and scope. This report presents criteria and procedures for the design and construction of pavements placed on subgrade or base course materials subject to seasonal frost action. When the report is officially published as a DA Technical Manual the criteria will be applicable to Air Force and Air National Guard airfields, to Army airfields and heliports, and to roads. The criteria also are applicable to pavements for civil works construction. The most prevalent modes of distress in pavements and their causes are listed in table 1-1. This report is concerned with modes unique to frost areas. The principal non-traffic-associated distress modes are distortion caused by frost heave and reconsolidation, and cracking caused by low temperatures. The principal traffic-load-associated distress modes are 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 report.

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

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.	

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-flyash (LCF), the term bound base is applicable in this report only if the mixture meets the requirements for cement-stabilized, lime-stabilized or LCF-stabilized soil set forth in TM 5-822-4 (AFM 88-7, Chap. 4) and in chapter 6 of this report.

(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.

(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 sub-

grade 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. Such soils are further defined in paragraph 2-4.

(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 flyash, 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 TM 5-822-4 (AFM 88-7, Chap. 4) and in chapter 6 of this report, and they must also meet the base course composition requirements set forth in chapter 5 of this report.

(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. Cohesionless materials such as crushed rock, gravel, sand, slag and cinders that do not experience significant detrimental ice segregation under normal freezing conditions. This is further discussed in paragraph 2-4. 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 one day, or the average of several temperature readings taken at equal time intervals, generally hourly, during one 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°F. The degree-days are minus when the average daily temperature is below 32°F

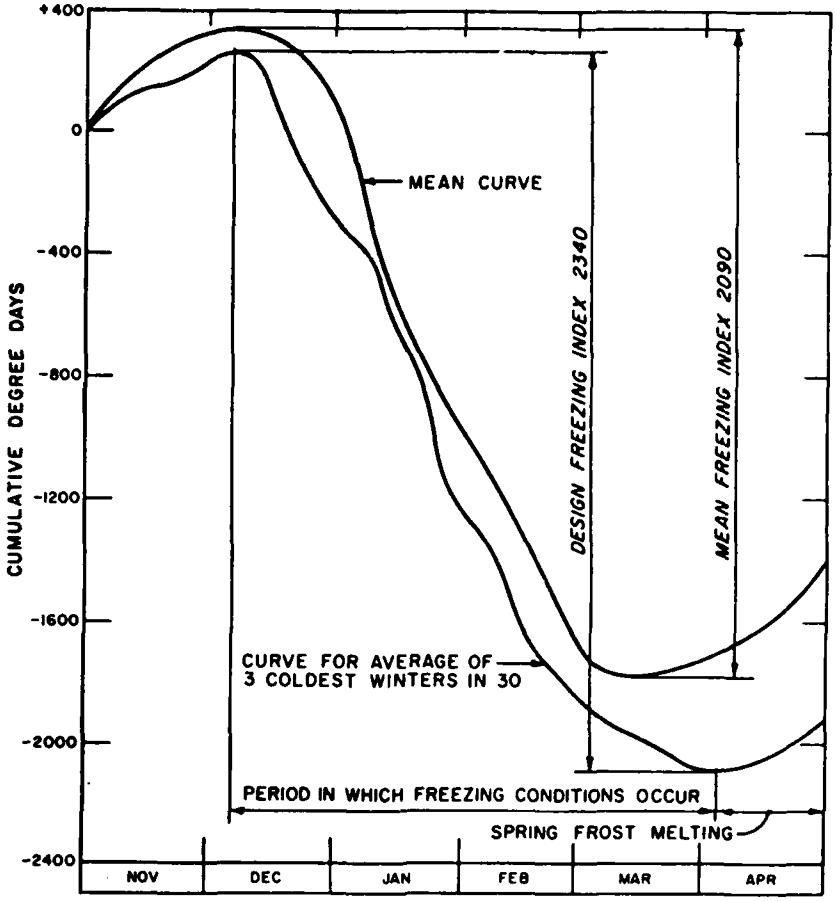


Figure 1-1. Determination of freezing index.

(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. To avoid the necessity for adopting a new and only slightly different freezing index each year, the design freezing index at a site with continuing construction need not be changed more than once in 5 years unless the more recent temperature records indicate a significant change in thickness design requirements for frost. The design freezing index is illustrated in figure 1-1.

(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 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. The detrimental effects of frost action are discussed in greater detail in paragraphs 2-5 and 2-6. 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 that that assumed as the design objective.

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 groundwater 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°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.

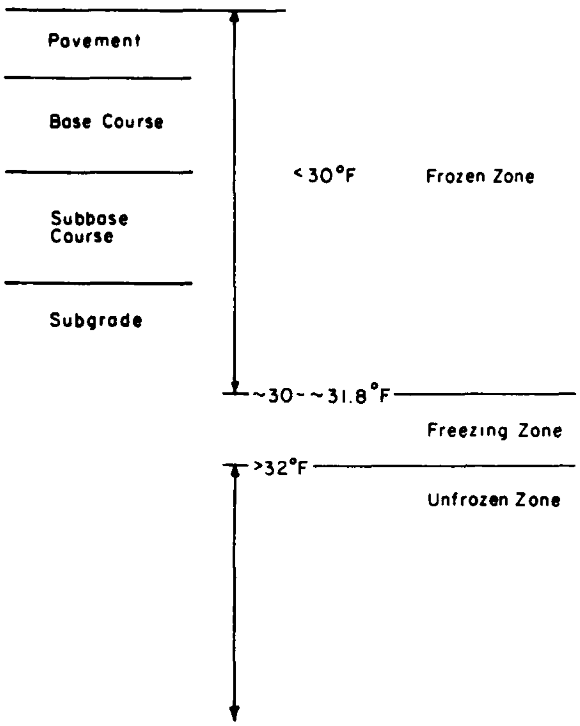
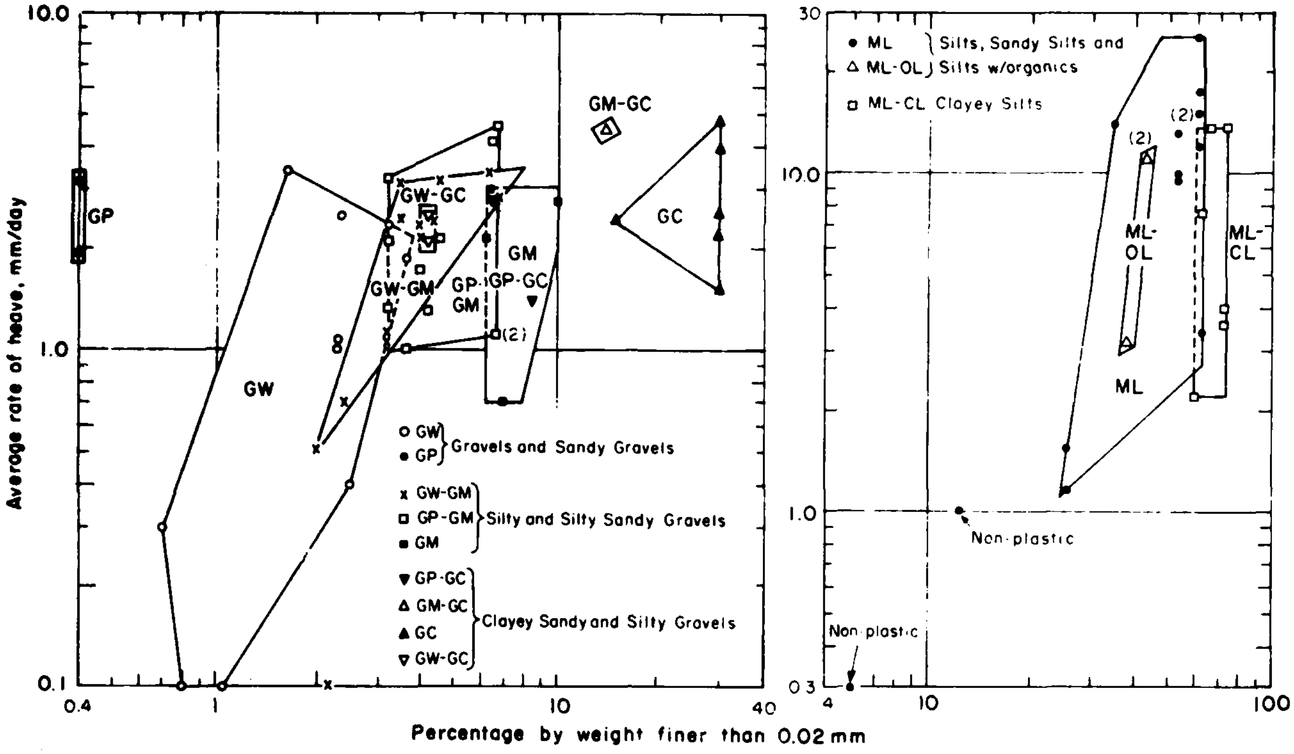


Figure 2-1. Freezing sequence in a typical pavement profile.

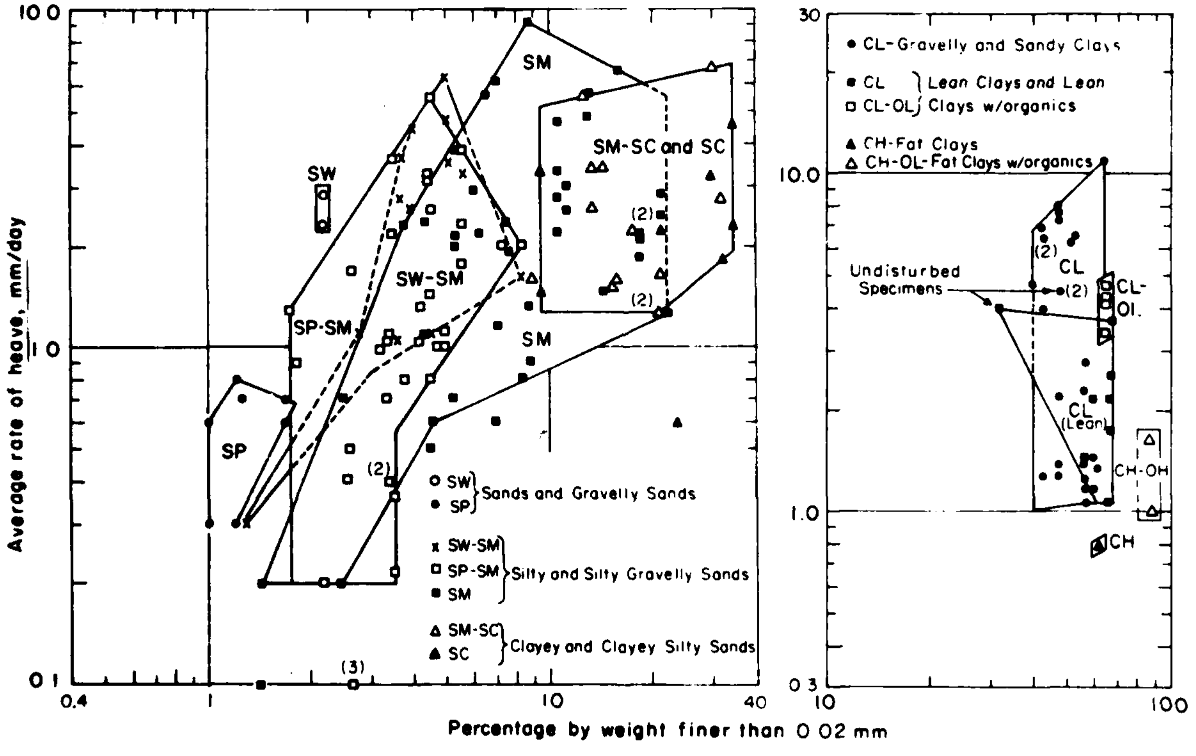
2-4. **Frost-susceptible soil.** The potential intensity of ice segregation that may occur in a freezing season is dependent to a large degree on the size-range of the soil voids, which in turn is determined by the size and size distribution of the soil grains, soil density and particle shape and orientation. 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 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. They should be subjected to a standard laboratory frost-susceptibility test to evaluate behavior during freezing. 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, or determined to be frost-susceptible by standard laboratory freezing tests, 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 E contains a summary of results from freezing tests on natural soils. The data in Appendix E 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, 2) estimate the frost-susceptibility of that soil from those of the two similar soils. A freezing test on a sample of the soil in question will give a direct evaluation of its frost-susceptibility.



a. Gravels

c. Silts



b. Sands

d. Clays

Figure 2-2. Rates of heave in laboratory freezing tests on remolded soils.

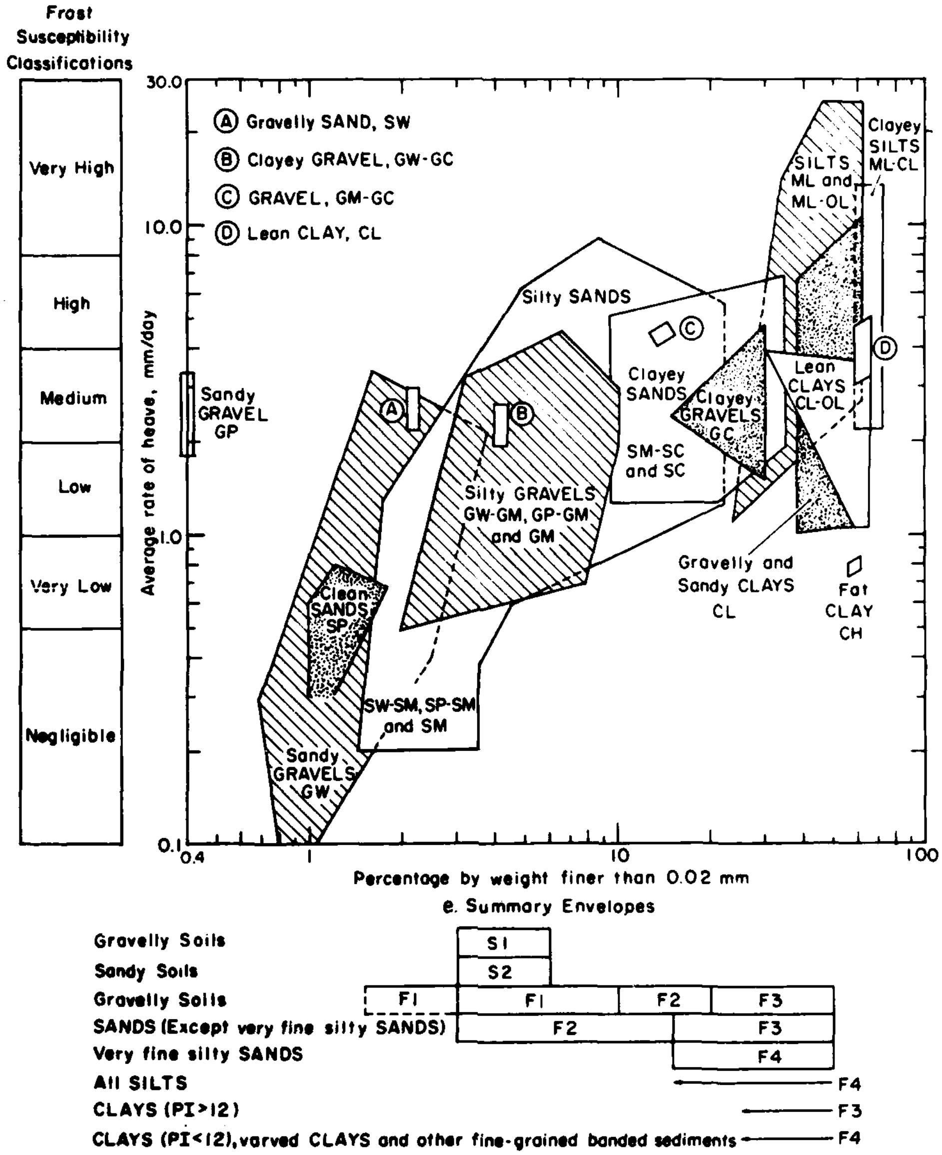


Figure 2-2 (cont'd).

(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 2-2e showing the laboratory test results by various individual soil groupings, as defined by Military Standard MIL-STD-619 B. 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 2-2e. Because of the severity of the laboratory test, the

rates of heave shown in figure 2-2 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 figure 2-2 that soils judged as non-frost-susceptible under the criteria given in paragraph 2-4 are not necessarily free of susceptibility to frost heaving. Also, soils that, although 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.

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 > 12	-	CL, CH
F4	(a) All silts	-	ML, MH
	(b) Very fine silty sands	Over 15	SM
	(c) Clays, PI < 12	-	CL, CL-ML
	(d) Varved clays and other fine-grained, banded sediments	-	CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML and SM

** Non-frost-susceptible.

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

(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 heave and soften more readily than homogeneous soils with equal average water contents, the classification of the material of highest frost-suscepti-

bility should be adopted for design. Usually, this will place the overall 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. This will, however, be subject to the specific approval of HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-) if the difference is not greater than one frost group number and if complete justification for the variation is presented. Such justification may take into account special conditions of subgrade moisture or soil uniformity, in addition to soil gradation and plasticity, and should include data on performance of 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 materials, 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 (chap. 4) and by adequate subgrade preparation and transition details (chap. 7). 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 groundwater 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 groundwater 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 groundwater 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 (chap. 7).

2-6. Thawing and reduction in pavement support capacity. Deterioration of pavements under repeated application of wheel loads is a process of cumulative damage; the rate of damage accumulation varies not only with traffic but also with seasonal changes in the supporting capacity of the various layers composing the pavement. One of the most critical conditions that develops in frost areas is the weakening of subgrade soils, base course and subbase during thawing. When ice segregation has occurred the strength of the soil is reduced during frost-melting periods. This causes a corresponding reduction in the load-supporting capacity of the pavement, particularly in winter partial thaws and early in the spring when thawing is taking place at the top of the subgrade and the rate of melting is rapid. The melting of segregated ice leaves the expanded soil in an under-consolidated condition, with a corresponding buildup of excess pore water pressure. Granular unbound base materials may also weaken significantly during frost-melting periods because of increased saturation combined with reduced density that is derived from expansion in the previously frozen state. The extent of weakening during thaw periods is greater in frost-susceptible base, subbase and subgrade materials that experience severe ice segregation.

a. As illustrated in figure 2-3, melting of the ice from the surface downward releases water that cannot drain through the still-frozen soil below or redistribute itself readily. Excess moisture from the wet and softened subgrade soil may move upward into the subbase and base, and migrate laterally to the nearest drain. If drainage provisions are inadequate, the base and subbase courses may become completely saturated. If this happens, the bearing capacity of the pavement system is substantially reduced, the effects of frost in subsequent freeze-thaw cycles are increased, water and fines may be pumped through joints and cracks, and the surface may deteriorate faster. Therefore, it is essential that base and subbase courses in frost regions be designed in strict accordance with the drainage criteria of TM 5-820-2 (AFM 88-5, Chap. 2) and with the further requirements set forth in chapter 5 of this

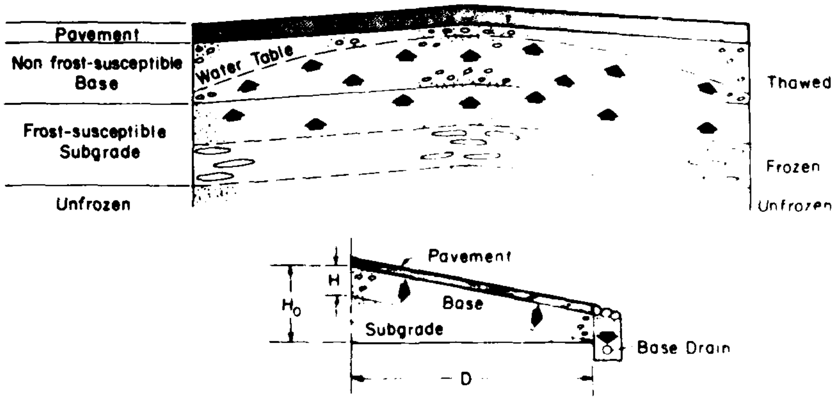


Figure 2-3. Moisture movement upward into base course during thaw.

report. The possible effects of restriction of subsurface drainage by frozen soils should be considered at all points in drainage design.

_b_. Soil is weakened during a frost-melting period principally because increased pore water pressures develop due to release of moisture. The degree of strength loss during a frost-melting period and the length of the reduced strength period depend on the rates at which the heaved soil can consolidate, the pore water pressures can be dissipated, and moisture tension can develop. These rates in turn depend on the type of soil, temperature conditions during freezing and thawing, the amount and type of traffic during frost-melting, rainfall during the previous fall and winter, spring rainfall, drainage conditions and atmospheric humidity.

_c_. Supporting capacity may be reduced in clay subgrades even though significant heave has not taken place. 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. Design methods are presented in chapter 4.

CHAPTER 3

INVESTIGATION OF POTENTIAL FOR ICE SEGREGATION

3-1. Investigation procedure. The field and laboratory investigations conducted in accordance with TM 5-825-2 (AFM 88-6, Chap. 2) 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, which 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. Soil. As stated in paragraph 2-4, the frost-susceptibility of soils may be estimated from the percentage of grains finer than 0.02 millimeters by weight or may be determined by laboratory freezing tests.

Such freezing tests will be carried out by or under the supervision of the U.S. Army Cold Regions Research and Engineering Laboratory (USACRREL), Hanover, New Hampshire. A period of 6 to 8 weeks must be allowed for a complete frost-susceptibility test but interim results are usually available for design guidance in about 4 weeks.

3-3. 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 or 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 1000 °F-days (i.e. mean air freezing index of less than about 500 °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 temperatures, 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

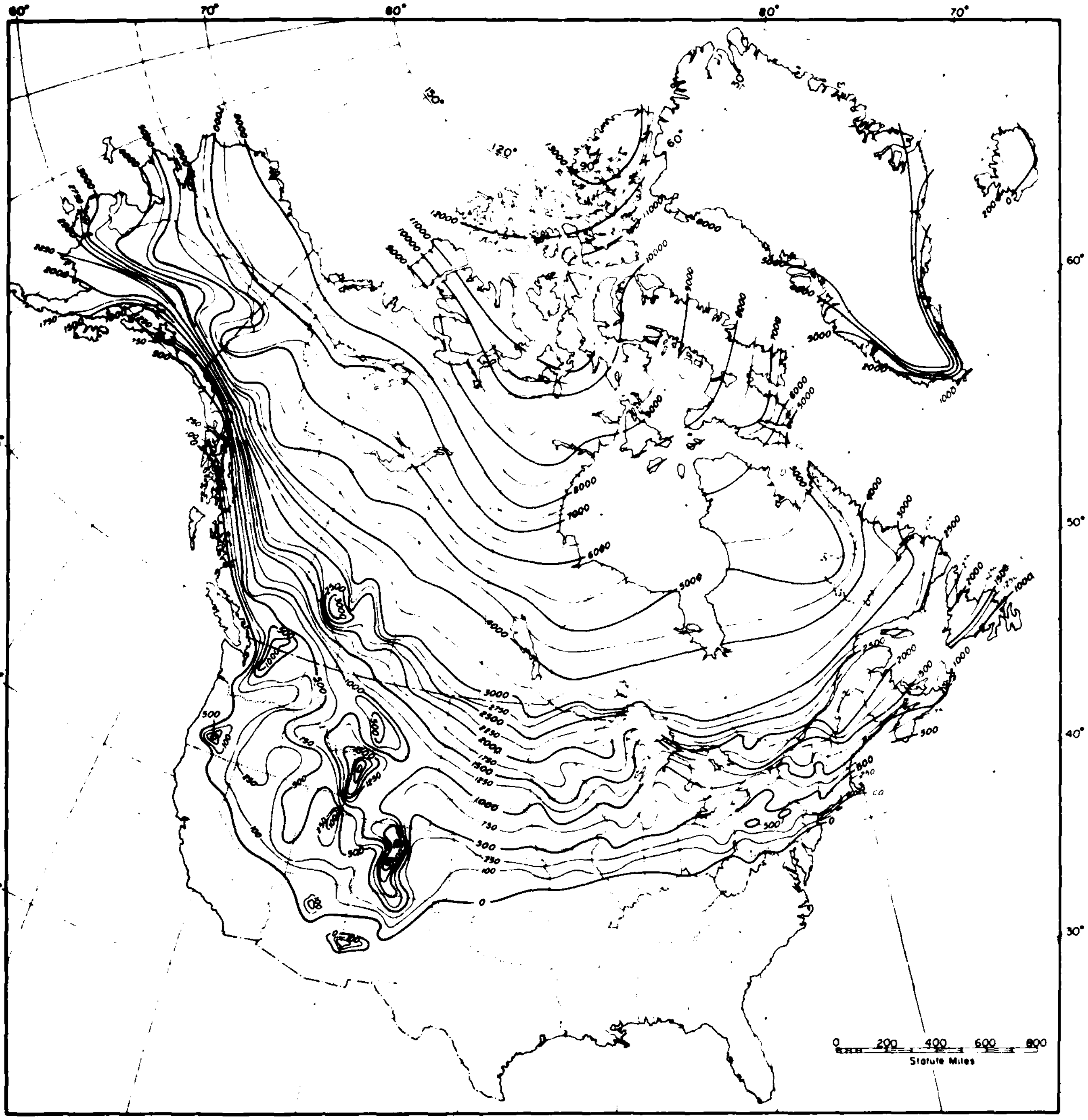


Figure 3-1. Distribution of mean air freezing indices in North America.

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

Figure 3-2. Distribution of design air freezing indices in North America.

freezing index 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). Figure 3-3 shows mean freezing indices for northern Eurasia. Relationships between mean air freezing indices and values computed on various other statistical bases are shown in figure 3-4.

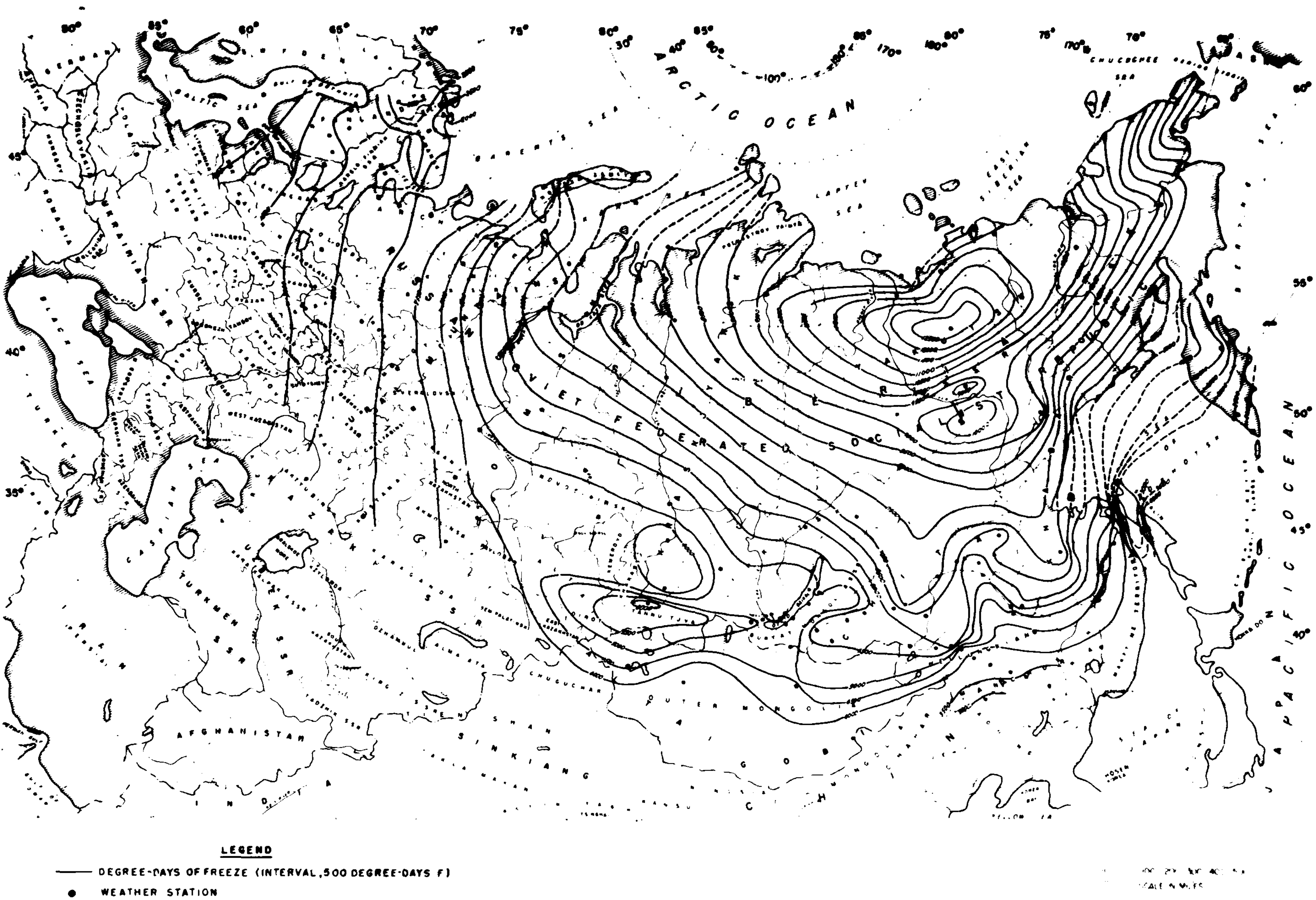
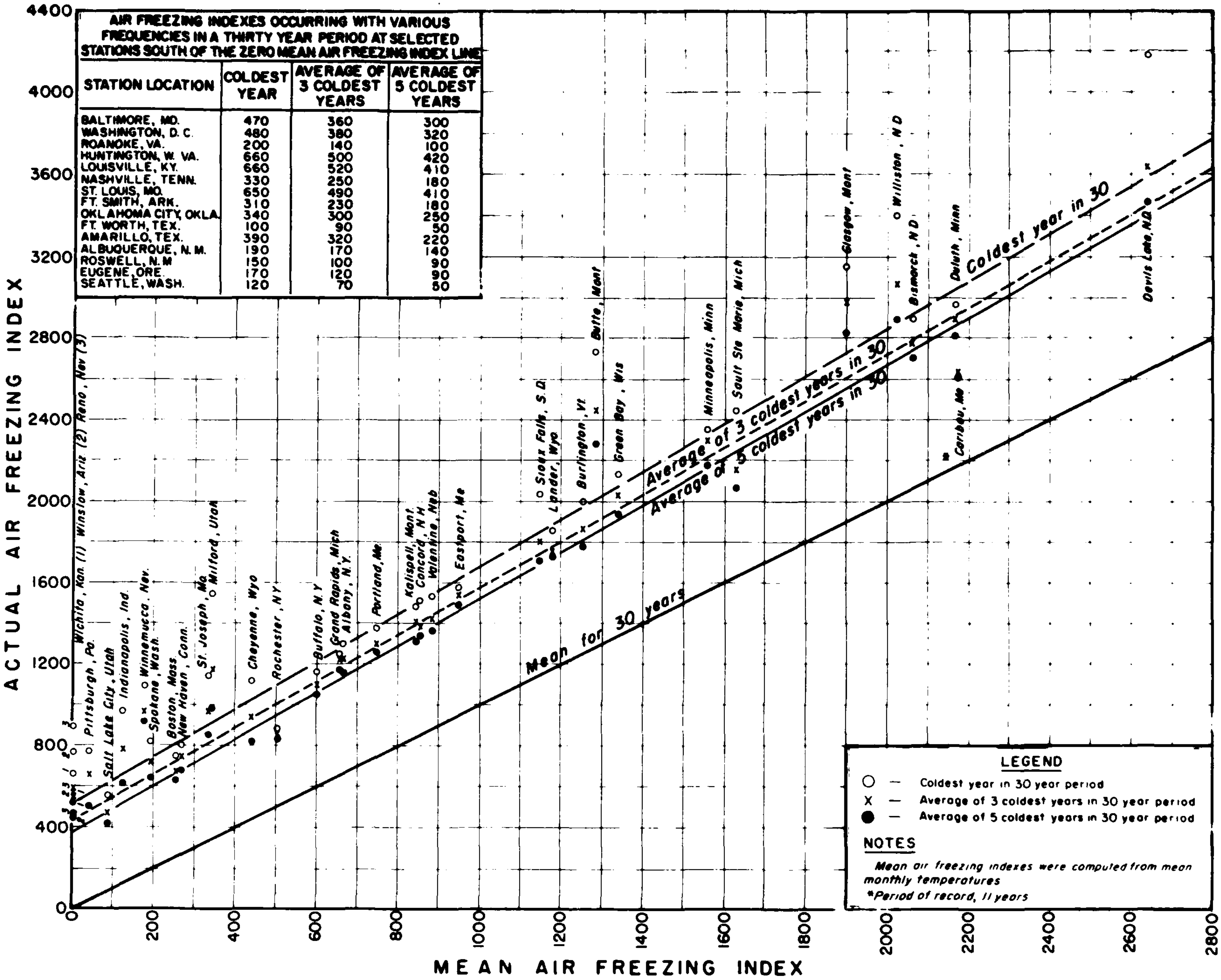


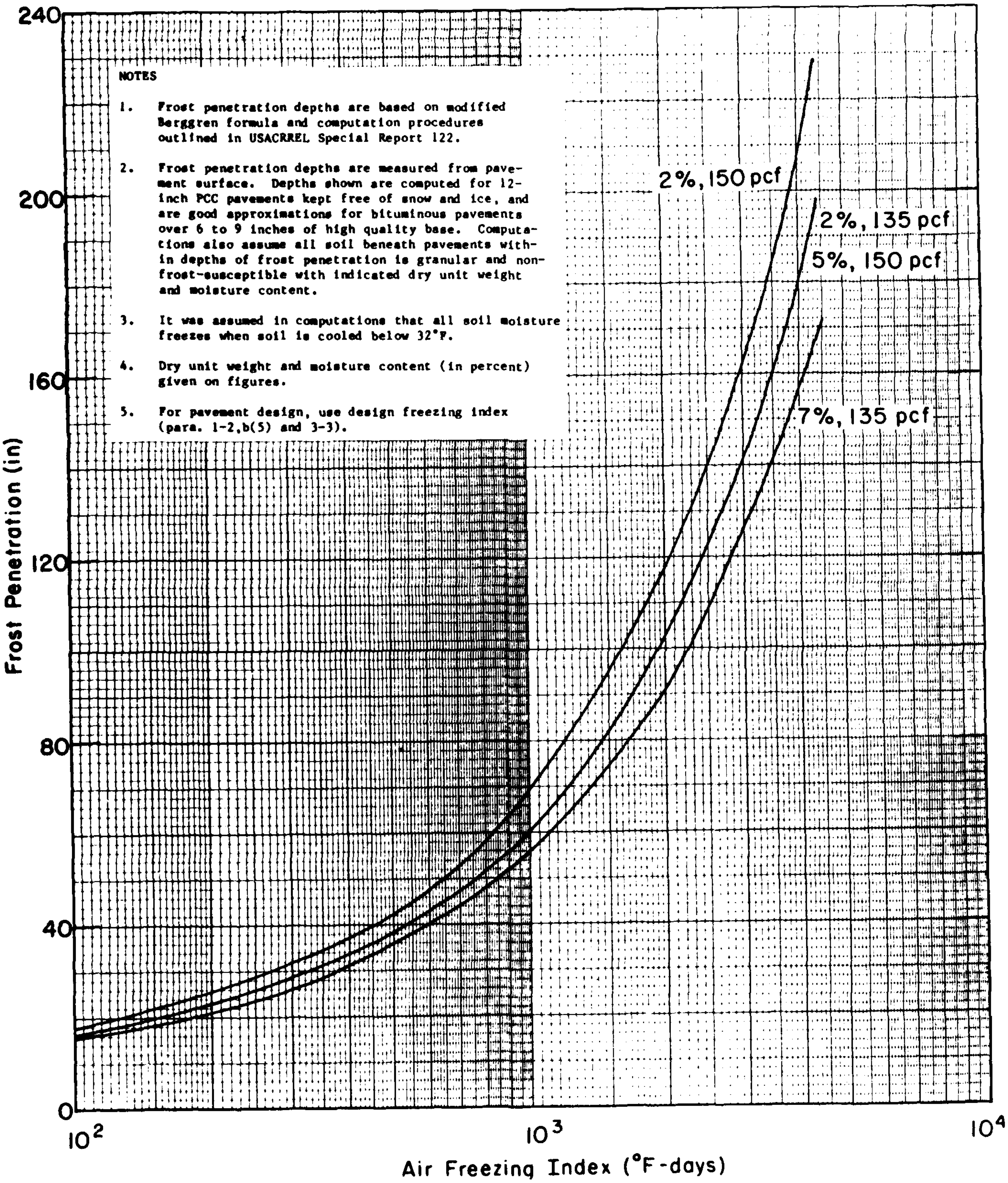
Figure 3-3. Distribution of mean air freezing indices in northern Eurasia.



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

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

Figure 3-4. Relationships between mean and other air freezing indices.



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°F.
4. Dry unit weight and moisture content (in percent) given on figures.
5. For pavement design, use design freezing index (para. 1-2,b(5) and 3-3).

Figure 3-5. Frost penetration beneath pavements.

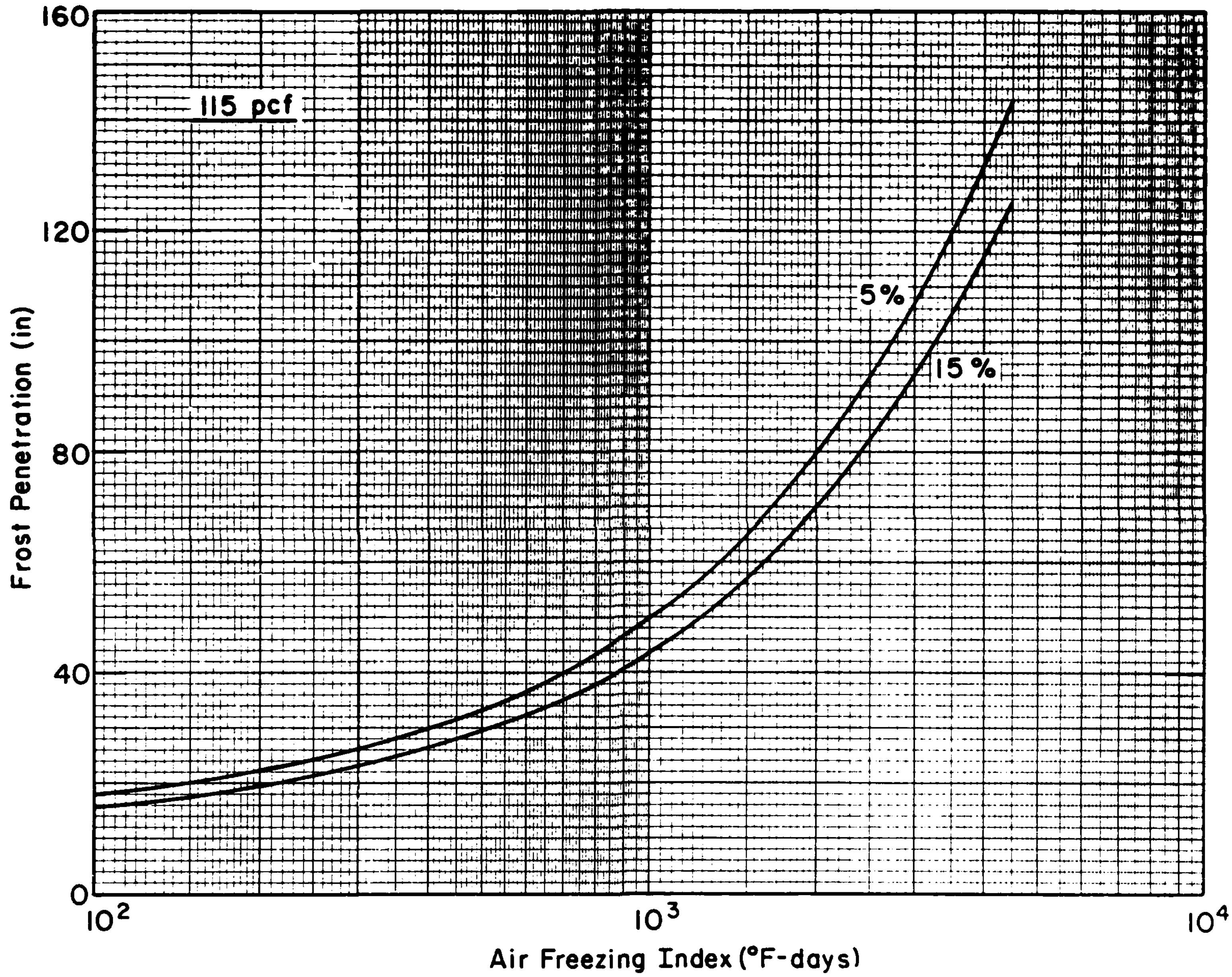


Figure 3-5 (cont'd). Frost penetration beneath pavements.

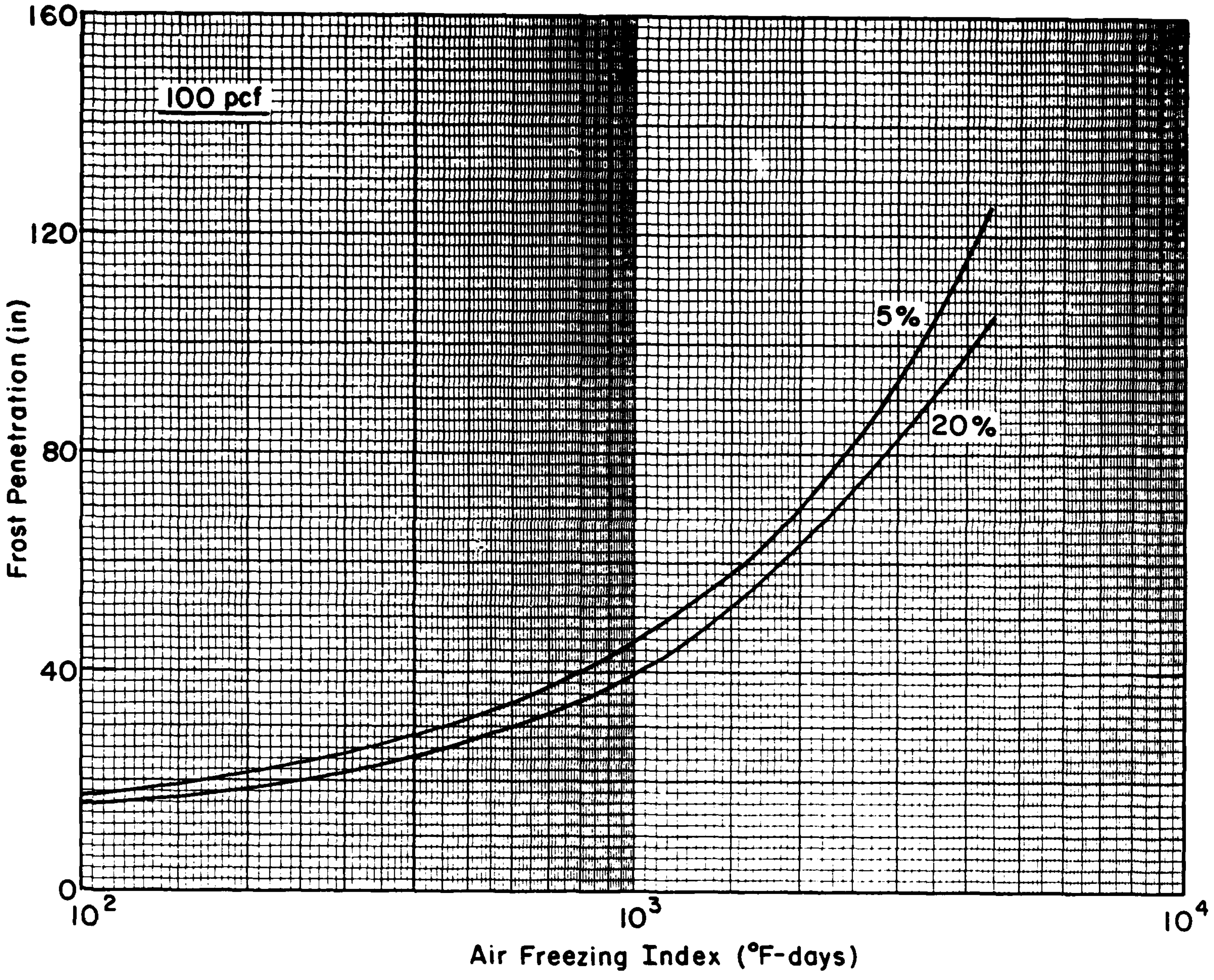


Figure 3-5 (cont'd).

Figures 3-1 through 3-3 are not sufficiently accurate for use in designing pavements and are included only to illustrate geographic differences in the air freezing indices. For designing pavements, the design air freezing index should be calculated from air temperatures, as explained in paragraph 3-3,a. and shown in figure 1-1.

3-4. Depth of frost penetration. The depth to 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 3-5 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°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; frost penetration beneath rigid pavements more than 12 inches thick is discussed in paragraph 4-3,b.

a. Where individual analysis is desired or unusual conditions make special computation desirable, the modified Berggren equation may be applied (see figure 3-5). The digital solution of the modified Berggren equation is useful for this purpose (see USACRREL Special Report 122). Neither this equation nor the curves in figure 3-5 are 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 3-5 should be verified whenever possible by observations in the area under consideration. Methods of estimating frost penetration depths beneath surfaces, other than pavements kept free of snow and ice, are discussed in TM 5-852-6 (AFM 88-19, Chap. 6).

3-5. Water. A potentially troublesome water supply for ice segregation is present if the highest groundwater 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 with associated problems of severe frost heaving and thaw weakening. If the depth to the water table is between 5 and 10 feet, the potential severity of frost heaving and thaw weakening will be between that with the water table 5 feet deep and that with the water table more than 10 feet deep, as described below. 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 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 TM 5-822-5 (AFM 88-7, Chap. 3), TM 5-822-6 (AFM 88-7, Chap. 1), TM 5-825-2 (AFM 88-6, Chap. 2), TM 5-824-3 (AFM 88-6, Chap. 3) and TM 5-823-3, 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 (chap. 5) 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, except when approved by HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-). For pavements where layers of synthetic thermal insulation are permitted, full protection of the subgrade against freezing may be feasible. Guidance for the use of insulation is provided in appendix C.

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 (chap. 5) 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 (chap. 7) 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 might dictate unusually severe restrictions on tolerable pavement roughness, requiring that subgrade frost penetration be strictly limited or even prevented.

a. 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 TM 5-822-5 (AFM 88-7, Chap. 3), TM 5-822-6 (AFM 88-7, Chap. 1), TM 5-825-2 (AFM 88-6, Chap. 2), TM 5-824-3 (AFM 88-6, Chap. 3) or TM 5-823-3, since the thickness requirements under non-frost-criteria must be met in addition to the frost design requirements.

b. In accordance with anticipated traffic patterns, airfield pavements are normally divided into four traffic areas (A, B, C and D) as defined in TM 5-824-1 (AFM 88-6, Chap. 1). Where the limited subgrade frost penetration method is used, the traffic area concept is not applicable in determining the required combined thickness of pavement and base, the latter being a fixed value for all traffic areas. When the reduced subgrade strength design method is used for flexible pavements, the combined thicknesses of pavement and base required for each traffic area differ. Thus, the total thickness required may change abruptly in the longitudinal direction or across the transverse section of a feature because two types of traffic areas are included. Transitions in the combined thickness of pavement and base should be provided as described in paragraph 7-3. All such thickness transitions should be made by increasing the thickness of the less costly materials used in the subbase.

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. The design freezing index is more explicity defined in paragraph 1-2,b(5).

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 start of 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 (see chap. 5), the average moisture content and dry unit weight should be weighted in proportion to the thicknesses of the various layers. Alternatively, if layers of bound base course (para. 1-2,g(2)) and granular unbound base course (para. 1-2,g(10)) are used in the pavement, the average may be assumed to be equal to the moisture content and dry unit weight of the material in the granular unbound base course.

(2) From figure 3-5, 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 3-5 to determine frost penetration *a*. Then add the extra concrete pavement thickness to the determined frost penetration.

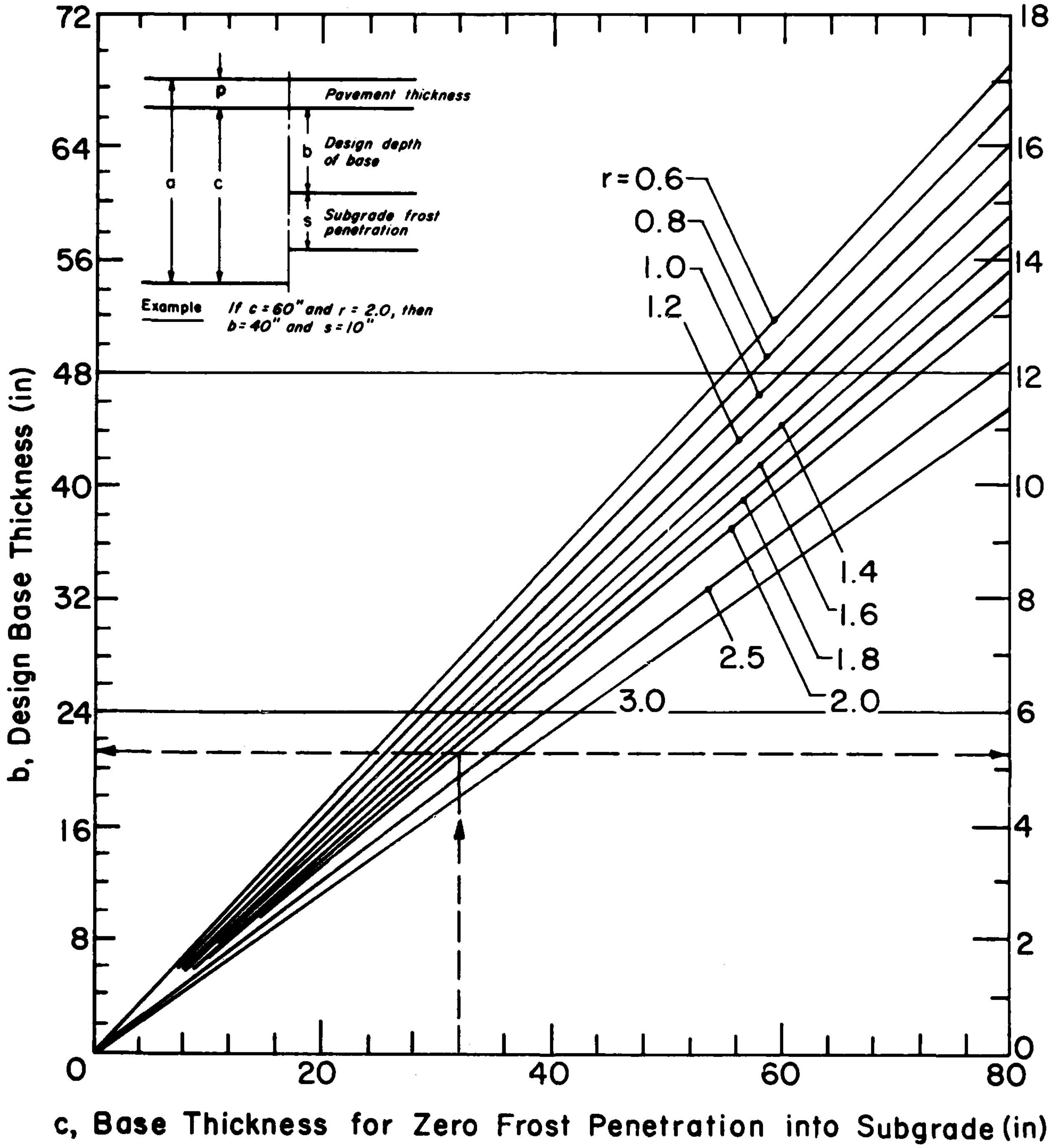
(3) Compute base thickness *c* (fig. 4-1) required for zero frost penetration into the subgrade as follows:

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

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

(5) Enter figure 4-1 with *c* as the abscissa and, at the applicable value of *r*, find on the left scale the design base thickness *b* that will result in the allowable subgrade frost penetration *s* shown on the right scale. *If r computed in subparagraph (4) above is equal to or exceeds 2.0, use 2.0 in figure 4-1 for type A and B traffic areas on airfield pavements. If r is equal to or exceeds 3.0, use 3.0 for all pavements other than those in type A or B traffic areas at airfields.*

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. By limiting *r* to 2.0 for type A and B traffic areas on airfields, greater thickness will result, thereby causing differential frost heave to be less than on other pavements.



NOTES

$\underline{a}$ = Combined thickness of pavement and non-frost-susceptible base for zero frost penetration into subgrade.

$\underline{c} = \underline{a} - \underline{p}$

$\underline{w}_b$ = Water content of base.

$\underline{w}_s$ = Water content of subgrade.

$\underline{r} = \frac{\underline{w}_s}{\underline{w}_b}$ Not to exceed 2.0 for type A and B areas on airfields and 3.0 for the other pavements.

Figure 4-1. Design depth of non-frost-susceptible base for limited subgrade frost penetration.

d. When the maximum combined thickness of pavement and base required by this design procedure exceeds 60 inches, consideration shall 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.

(1) 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 TM 5-820-2 (AFM 88-5, Chap. 2) must still be met. If steel reinforcement, reduced slab dimensions, high-moisture-retention base course or combined thickness over 60 inches is selected for frost design purposes, specific approval of HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-) shall be obtained.

(2) Less total thickness of pavement and base than indicated by the basic design method may also be used if definite justification, based on local experience or on special conditions of the design, is provided; again this is subject to approval of HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-).

e. If the combined thickness of pavement and base required by the non-frost-criteria of TM 5-822-5 (AFM 88-7, Chap. 3), TM 5-822-6 (AFM 88-7, Chap. 1), TM 5-825-2 (AFM 88-6, Chap. 2), TM 5-824-3 (AFM 88-6, Chap. 3) or TM 5-823-3 exceeds the thickness given by the limited subgrade 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 of chapter 5 should be rigorously followed. The design base thickness determined in paragraph 4-3,b(5) 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 TM 5-825-2 (AFM 88-6, Chap. 2) and TM 5-822-5 (AFM 88-7, Chap. 3). The thickness of rigid pavement slab will be determined from TM 5-824-3 (AFM 88-6, Chap. 3), TM 5-823-3, and TM 5-822-6 (AFM 88-7, Chap. 1).

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 heavy-load 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 shall 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 design curves in TM 5-825-2 (AFM 88-6, Chap. 2) 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 TM 5-822-5 (AFM 88-7, Chap. 3) 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. The soil support index S1 and S2 material meeting current specifications for base or subbase will be determined by conventional CBR tests in the unfrozen state.

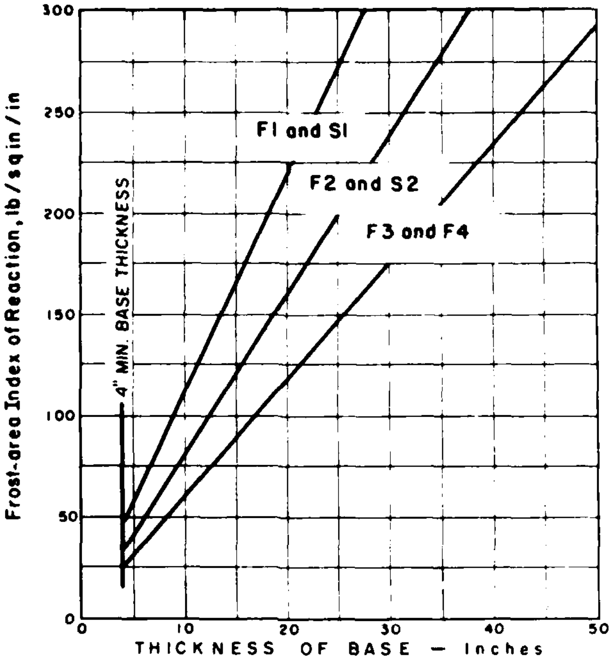
Table 4-1. Frost-area soil support indices for subgrade soils for flexible pavement design.

Frost group of subgrade soil	F1 and S1	F2 and S2	F3 and F4
Frost-area soil support index	9.0	6.5	3.5

(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 for frost conditions of up to 10 percent may be permitted for highways on substantial fills.

(2) TM 5-825-2 (AFM 88-6, Chap. 2) and TM 5-822-5 (AFM 88-7, Chap. 3) 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. The base course composition requirements set forth in chapter 5 must be followed rigorously.

b. Thickness 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 may be reduced to a minimum of 4 inches. The material shall meet the requirements set forth for free-draining material in paragraph 5-1, as well as the criteria for filter under pavement slab stated in paragraph 5-5. If it does not also meet the criteria for filter over subgrade as stated in paragraph 5-4, a second 4-inch layer meeting that criterion shall be provided.



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

Figure 4-2. Frost-area index of reaction for design of rigid airfield and highway pavements.

(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. TM 5-824-3 (AFM 88-6, Chap. 3), TM 5-823-3, and TM 5-822-6 (AFM 88-7, Chap. 1) establish criteria for determination of the required thickness of rigid pavement slabs in combination with a bound base course. The provisions of chapter 5, referenced above, 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 TM 5-824-3 (AFM 88-6, Chap. 3) or TM 5-823-3 for airfields and TM 5-822-6 (AFM 88-7, Chap. 1) for roads and parking areas, using the frost-area index of reaction determined from figure 4-2 of this report. 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 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 shall 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 such as 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 TM 5-825-2 (AFM 88-6, Chap. 2); enter the curves with the applicable frost-area soil support index given in table 4-1 of this report. The thickness established by this procedure shall have the following limitations:

- It shall not be less than required for non-frost-condition design in overrun areas, as determined from TM 5-825-2 (AFM 88-6, Chap. 2).

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

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

The subgrade preparation techniques and transition details outlined in chapter 7 of this report are required for overrun pavements. The composition of the layered pavement structure shall conform with the applicable requirements of TM 5-825-2 (AFM 88-6, Chap. 2), except that the composition of base courses shall also conform with the requirements of chapter 5 of this report.

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 TM 5-825-2 (AFM 88-6, Chap. 2) 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 (chap. 7), 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 in subparagraphs 4-5,c(1) and 4-5,c(2) apply only if they require a combined pavement and base thickness in excess of that required in subparagraph 4-5,b, which is the minimum thickness 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 heavy-, medium- and light-load design airfields, the flexible pavement and base shall have the combined thickness given by the design curve in TM 5-825-2 (AFM 88-6, Chap. 2); enter the curve with the applicable frost-area soil support index shown in table 4-1. Subgrade preparation as set forth in chapter 7 is required. 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. The composition of base courses for shoulder pavements will be as provided in chapter 5.

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 in accordance with the provisions of paragraph 7-3. 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 TM 5-822-5 (AFM 88-7, Chap. 3) and TM 5-822-6 (AFM 88-7, Chap. 1). 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 Army 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 unless approved by HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-). 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 shall be placed directly beneath the lower layer of bound base or, if there is no bound base, directly beneath the pavement slab or surface course. The free-draining material shall 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 the filter requirements prescribed in paragraphs 5-4 and 5-5. 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 of paragraphs 5-4 and 5-5. 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 shall 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 the filter requirements in paragraph 5-4, 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 TM 5-820-2 (AFM 88-5, Chap. 2) 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 ensure that the materials meet these design criteria. When placed and compacted, subbase and base materials must meet the applicable compaction requirements in TM 5-822-5 (AFM 88-7, Chap. 3), TM 5-822-6 (AFM 88-7, Chap. 1), TM 5-824-3 (AFM 88-6, Chap. 3) or TM 5-825-2 (AFM 88-6, Chap. 2).

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. The thickness of F1 and F2 materials and the thickness of pavement and base above the F1 and F2 materials must meet the non-frost-criteria in TM 5-822-5 (AFM 88-7, Chap. 3) or TM 5-822-6 (AFM 88-7, Chap. 1).

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 shall 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 TM 5-820-2 (AFM 88-5, Chap. 2), 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 thickness 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. HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-) should be contacted for guidance and approval of the materials proposed for a specific project. Gradations of materials to be located above and below the fabric should also be furnished.

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 Lime-Cement-Flyash (LCF) are the most common additives used in stabilized soils. Other stabilizers may be used for pavement construction in frost areas only with the express approval of HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-), as applicable. The limitations of use, the basic requirements for mixture design and the stabilization procedures using bituminous and chemical stabilizers are set forth in TM 5-822-4 (AFM 88-7, Chap. 4). Pertinent information also is presented in TM 5-825-2 (AFM 88-6, Chap. 2) and TM 5-824-3 (AFM 88-6, Chap. 3). Special or supplemental requirements 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. Appendix D contains additional guidance on the use of MESL in pavement systems in cold regions. 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°F, for example, than at 70° or 80°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 construction cut-off dates that will allow time for development of frost-resistant bonds are presented in Transportation Research Board 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 TM 5-822-4 (AFM 88-7, Chap. 4), with the additional requirement that the mixture, after freeze-thaw testing as set forth below, should meet the weight-loss criteria specified in TM 5-822-4 (AFM 88-7, Chap. 4) 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 TM 5-822-4 (AFM 88-7, Chap. 4) 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 TM 5-822-4 (AFM 88-7, Chap. 4), cured specimens should be subjected to the 12 freeze-thaw cycles of ASTM D-560 (but omitting wire-brushing) or other applicable freeze-thaw procedures. This should be followed by determination of frost-design soil classification by means of standard laboratory freezing tests. These tests should be conducted by USACRREL in Hanover, New

Hampshire. For lime-stabilized or lime-modified soil used in lower layers of the base course, the frost-susceptibility, determined after freeze-thaw cycling, should meet the requirements set forth for base course in chapter 5 of this report. If lime-stabilized or lime-modified soil is used as subgrade, its frost-susceptibility, determined after freeze-thaw cycling, should be used as the basis of the pavement thickness design if the reduced subgrade strength design method is applied.

6-3. Stabilization with portland cement. Cement-stabilized soil meeting the requirements set forth in TM 5-822-4 (AFM 88-7, Chap. 4), 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 TM 5-822-4 (AFM 88-7, Chap. 4) also may be used in frost areas. However, in addition to the procedures for mixture design specified in the TM, cured specimens of cement-modified soil should be subjected to the 12 freeze-thaw cycles of ASTM D-560 (but omitting wire-brushing) or other applicable freeze-thaw procedures. This should be followed by determination of frost design soil classification by means of standard laboratory freezing tests. These tests should be conducted by USACRREL in Hanover, New Hampshire. For cement-modified soil used in the base course, the frost-susceptibility, determined after freeze-thaw cycling, should meet the requirements set forth for base course in chapter 5 of this report. If cement-modified soil is used as subgrade, its frost-susceptibility, determined after freeze-thaw cycling, should be used as the basis of the pavement thickness design if the reduced subgrade design method is applied.

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 (see app. B). Excepting these special conditions affecting the suitability of particular types of bitumen, the procedures for mixture design set forth in TM 5-822-4 (AFM 88-7, Chap. 4) and TM 5-822-8 (AFM 88-6, Chap. 9) usually will ensure 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 shall be especially prepared to achieve uniformity of soil conditions. In fill sections the least frost-susceptible soils shall 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 shall be subjected to the subgrade preparation procedures required for cut sections. In cut sections the subgrade shall be scarified and excavated to a prescribed depth, and the excavated material shall 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, shall be the lesser of 24 inches, or two-thirds of the frost penetration given by figure 3-5 (except one-half of the frost penetration for airfield shoulder pavements and for roads, streets and open storage areas of class D, E and F) 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. The prepared subgrade must meet the compaction requirements in TM 5-822-5 (AFM 88-7, Chap. 3), TM 5-822-6 (AFM 88-7, Chap. 1), TM 5-824-3 (AFM 88-6, Chap. 3) or TM 5-825-2 (AFM 88-6, Chap. 2). At transitions from cut to fill, the subgrade in the cut section shall be undercut and back-filled with the

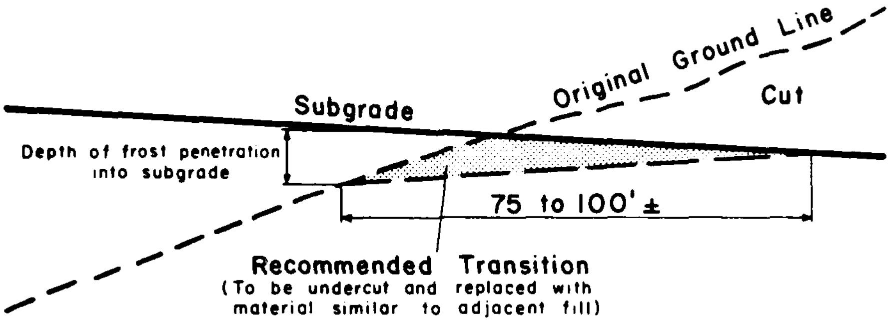


Figure 7-1. Tapered transition used where embankment material differs from natural subgrade in cut.

same material as the adjacent fill (fig. 7-1). Refer to appendix A 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.

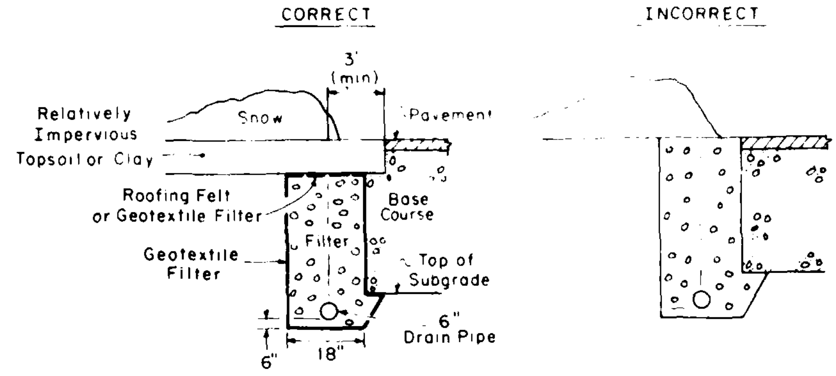
(2) Fine-grained subgrades containing moisture well in excess of the optimum for compaction, with no feasible means of drainage nor of otherwise reducing the moisture content, and which consequently it is not feasible to scarify and recompact, are also exceptions.

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 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.

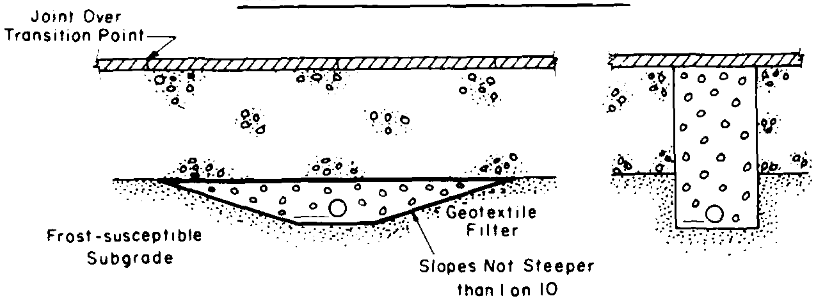
c. Boulder removal. It is essential that all stones more than about 6 inches in diameter be removed from frost-susceptible subgrades to prevent boulder heaves from damaging the pavement. In the process of constructing fills, all large stones should be removed from subgrade materials that will experience 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



SUBDRAINS ALONG PAVEMENT EDGES

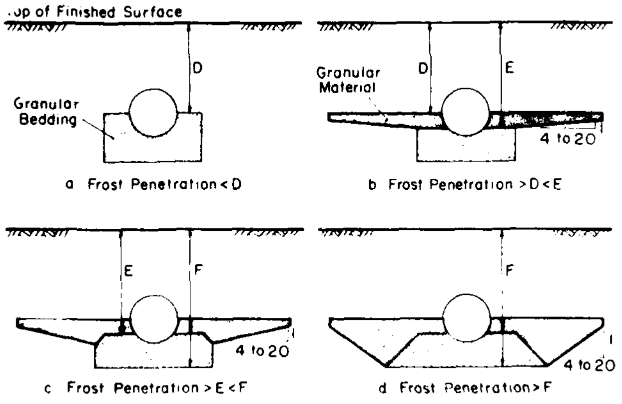


SUBDRAINS UNDER PAVED SURFACES

NOTES

1. For additional details on design and depth of subdrains and filter courses see TM 5-820-2 (AFM 88-5, Chap. 2).
2. Granular or geotextile fabric 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).

Figure 7-2. Subgrade details for cold regions.



a Frost Penetration < D

b Frost Penetration > D < E

c Frost Penetration > E < F

d Frost Penetration > F

TREATMENT OF CENTERLINE CULVERTS IN PLASTIC SOILS

Figure 7-3. Transitions for culverts beneath pavements.

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 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 (see para. 4-5,c).

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 between traffic areas C and D (see para. 4-2,b) should be located entirely within the limits of traffic area D and 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 (see app. A).

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. Light-load airfield pavements. Design flexible and rigid pavements on Air Force airfields for the following conditions:

- Design aircraft: single wheel, tricycle gear, contact area 100 square inches.
- Gross weight: 60,000 pounds
- Number of passes: 300,000
- Traffic area: B
- Design freezing index: 700 degree-days
- Highest groundwater: about 3 feet below surface of subgrade
- Concrete flexural strength: 650 psi
- Subgrade material:

    Lean clay, CL

    Plasticity index, 18

    Frost group, F3

    Water content, 25 percent (average)

    Normal-period CBR, 8

    Subgrade modulus (normal period) $\underline{k}$ = 150 psi/inch on subgrade and 250 psi/inch on top 24-inch base course.

    Local experience indicates that subgrade materials, if scarified, blended and recompacted, do not produce excessive nonuniform heave.

- Base course materials:

    High quality base material -- graded crushed aggregate, normal-period CBR=100, 30 percent passing no. 10 sieve, 1 percent passing no. 200 sieve.

Good quality base course material -- non-frost-susceptible sandy gravel (GW), normal-period CBR=50, 35 percent passing no. 10 sieve, 4 percent passing no. 200 sieve, does not meet filter criteria for material in contact with CL subgrade.

Subbase material — coarse 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 CL subgrade.

- Average dry unit weight (assumed equivalent to that of good quality base): 135 pounds per cubic foot.

- Average water content after drainage (assumed equivalent to that of good quality base): 5 percent.

a. Flexible pavement design by limited subgrade frost penetration method. From figure 3-5, the combined thickness of pavement and base a to prevent any freezing of the subgrade in the design index year (complete protection) is 45 inches. From TM 5-825-2 (AFM 88-6, Chap. 2), the minimum required flexible pavement thickness p is 3 inches. Thickness of base c to prevent frost penetration into subgrade, then, is 42 inches. The ratio of subgrade to base water content $r = 25/5 = 5$. From figure 4-1, required total base thickness b is 28 inches, using the maximum allowable value of r for the type B traffic area of 2.0. This base thickness will allow 7 inches of frost penetration s into the subgrade 1 year in 10 and would limit to tolerable amounts pavement frost heaving and cracking, and loss of subgrade strength. Required combined thickness of pavement and base is 31 inches.

b. Flexible pavement design by reduced subgrade strength method. From paragraph 4-4,a the frost-area soil support index is 3.5, which is less than the normal-period CBR and consequently will be used to enter the appropriate design curve of TM 5-825-2 (AFM 88-6, Chap. 2). The design curve gives a required combined thickness of pavement and base of 34 inches. This is more than the 31 inches required under design for limited subgrade frost penetration, and therefore the latter design is more economical. Since the 31-inch thickness is also greater than the 21 inches required by TM 5-825-2 (AFM 88-6, Chap. 2) for non-frost-design, 31 inches will be selected as the combined thickness of pavement and base for the flexible pavement design. This could be made up of 3 inches of flexible pavement, 6 inches of high quality base (since the high quality base contains only 1 percent passing the no. 200 sieve, it can also be used as the 4-inch free-draining layer [see para. 5-1]), 8 inches of good quality base, and 14 inches of S2 subbase material. In accordance with paragraph 7-1, no subgrade preparation is required because the combined thickness of pavement and base exceeds two-thirds of the design frost penetration depth.

c. Rigid pavement design by limited subgrade frost penetration method. The required slab thickness, from TM 5-824-3 (AFM 88-6, Chap. 3), with no subgrade weakening is 12 inches. By the same computation procedure as just described for flexible pavement, but using a 12-inch instead of a 3-inch pavement, minimum thickness of base required is 22 inches. The resultant combined thickness of pavement and base, then, is 11 + 22 = 33 inches. No subgrade preparation would be required.

d. Rigid pavement design by reduced subgrade strength method. Since subgrade conditions are suitable to achieve uniform heave, only a minimum base course of 4 inches is required as a free-draining layer.

Only the high quality base course meets the gradation requirements for material directly beneath the slab, or alternatively the good quality base may be washed and processed to reduce the material that passes the no. 200 sieve to 2 percent or less. Neither of these materials meets the filter criteria for material in contact with the clay subgrade. Therefore, a 4-inch layer of subbase material is also required. From figure 4-2, the frost-area index of reaction is 50 psi per inch. The slab thickness required, from TM 5-824-3 (AFM 88-6, Chap. 3), is 13 inches. The combined thickness of 13 + 8 = 21 inches is possibly more economical than that obtained by the limited subgrade frost penetration method, even though subgrade preparation to a depth of 2/3 x 45 - 21 = 9 inches would be required. Comparative cost estimates would indicate which design should be adopted.

8-2. Example 2. Heavy-load airfield pavements. Design heavy load flexible and rigid pavements on Air Force airfields for the following conditions:

- Design aircraft: twin-twin assembly, bicycle gear, spacing 37-62-37 inches, contact area 267 square inches each wheel.
- Gross weight: 480,000 pounds
- Number of passes: 12,000
- Traffic area: B
- Design freezing index: 3000 degree-days
- Subgrade material:

    Lean clay, CL

    Plasticity index, 18

    Frost group, F3

    Water content, 25 percent (average)

    Normal-period CBR, 5

Normal-period modulus of reaction <u>k</u> = 125 psi/inch on the subgrade and 400 psi/inch on top of a 42-inch thick base.

Subgrade shows relatively uniform heave characteristics in existing pavements, which in general have performed well.

- Base course materials:

High quality base material -- graded crushed aggregate, normal-period CBR=100, 30 percent passing no. 10 sieve, 1 percent passing no. 200 sieve.

Good quality base material -- non-frost-susceptible sandy gravel (GW), normal-period CBR=50, 35 percent passing no. 10 sieve, 4 percent passing no. 200 sieve, does not meet filter criteria for material in contact with CL subgrade.

Subbase material -- coarse 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 pounds per cubic foot.

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

- Highest groundwater: approximately 3 feet below surface of subgrade.

- Concrete flexural strength: 650 psi.

<u>a</u>. <u>Flexible pavement design by limited subgrade frost penetration method</u>. From figure 3-5, the combined thickness <u>a</u> of pavement and base to prevent freezing of subgrade in the design freezing index year (complete protection) is 128 inches. From TM 5-825-2 (AFM 88-6, Chap. 2) required flexible pavement thickness <u>p</u> is 4 inches. Thickness of base

to prevent frost penetration into subgrade, then, is 124 inches. The ratio of subgrade to base water content r is over 2.0. Therefore, 2.0 is used in figure 4-1, which yields a required base thickness b of 83 inches. The required combined thickness of pavement and base to limit subgrade frost penetration is 83 + 4 = 87 inches. As shown in figure 4-1, this will allow about 21 inches of frost penetration into the relatively uniform F3 subgrade on an average of 1 year in 10. (Note: Since this is limited subgrade frost penetration design, the same total thickness would apply for types A, C and D traffic areas. However, the thicknesses of bituminous surfacing and high quality base would vary between the traffic areas as required by TM 5-825-2 [AFM 88-6, Chap. 2]). Whereas the local experience with existing pavements indicates that heave has been relatively uniform, a limiting thickness of 60 inches will be adopted for the limited subgrade frost penetration method of design. This design will limit pavement heaving and cracking and loss of subgrade strength to tolerable amounts, provided all other requirements are met, such as use of base material meeting the prescribed composition requirements, uniformity of the base course as placed, subsurface drainage meeting the criteria of TM 5-820-2 (AFM 88-5, Chap. 2), use of procedures of subgrade preparation meeting the prescribed requirements, and use of appropriate transitions at any substantial and abrupt changes in the subgrade characteristics. The 60-inch thickness also is in excess of the thickness required by TM 5-825-2 (AFM 88-6, Chap. 2) for non-frost-design.

b. Flexible pavement design by reduced subgrade strength method. From paragraph 4-4,a the frost-area soil support index is 3.5. That value, used with the appropriate design curve of TM 5-825-2 (AFM 88-6, Chap. 2), yields a required combined thickness of pavement and base of 68 inches. This would not be adopted because it is more than the 60 inches required for limited subgrade frost penetration design. It is possible, however, that a pavement section that incorporates a bound base might be developed which, based on reduced subgrade strength, would reduce the 68-inch requirement to a section thinner and less costly than the 60-inch section. If not, the 60-inch section would be adopted. Its composition could be: 4 inches of asphalt concrete, 9 inches of high quality base (since the high quality base course contains only 1 percent passing the no. 200 sieve, it can also be used as the free-draining layer), 19 inches of good quality base, and 28 inches of S2 subbase. Subgrade preparation would be required to a depth of 24 inches, since this is less than 2/3 x 128 - 60 = 25 inches (para. 7-1).

c. *Rigid pavement design by limited subgrade frost penetration method*. The required pavement thickness *p*, based on the normal-period *k* = 400 psi per inch, is 18 inches. Each inch of concrete pavement in excess of 12 inches reduces the design freezing index by 10 degree-days. In this example the reduction = 10 x (18 - 12) = 60 degree-days. Therefore, the modified freezing index is 3000 - 60 = 2940. From figure 3-5, the combined thickness *a* of 12-inch pavement and base required to prevent freezing of the subgrade is 125 inches. Adding the originally deducted 6-inch thickness of pavement results in a combined thickness of pavement and base of 131 inches. Therefore, the thickness of base *c* required for zero frost penetration into the subgrade is 113 inches. From figure 4-1, the required design base thickness *b* is 75 inches, which permits a corresponding subgrade frost penetration *s* of 19 inches in the design year. The combined thickness of (75 + 18) = 93 inches would be reduced to the maximum limiting value of 60 inches since existing pavements show satisfactory performance. The 60 inches could comprise 18 inches of portland cement concrete, 4 inches of high quality base, 17 inches of good quality base and 21 inches of S2 subbase. Subgrade preparation would be required to a depth of 23 inches. A study should be made to determine whether a thinner slab with a bound base over various layers of granular unbound material would be more economical.

d. Rigid pavement design by reduced subgrade strength method. Since the experience with heaving of existing pavements has been favorable, a minimum of 4 inches of free-draining material could be used, plus 4 inches of filter material on the subgrade. For this case the frost-area index of reaction would be 50 psi/inch (fig. 4-2), requiring a pavement slab 27 inches thick, according to the criteria established in TM 5-824-3 (AFM 88-6, Chap. 3). Preferred practice for high-speed pavements, however, would be to use a base of total thickness equal to the slab thickness. Accordingly, the modulus would be increased, and by a trial and error process it can be determined that, with a 24-inch base (giving a modulus of 145 psi/inch), a 24-inch portland cement concrete slab would be required. Subgrade preparation would be specified to a depth of 24 inches. Cost comparisons of either of the two latter pavement designs with that developed under the method for limiting subgrade frost penetration, which would essentially involve trade-off costs of concrete versus base course, would indicate the choice of design. At equal cost the design that includes the greater combined thickness of pavement and base is preferred because it would provide greater protection against frost action in the subgrade.

8-3. Example 3. Heavy-load overrun pavement. Design a heavy-load overrun (non-blast area) pavement at an Air Force airfield for the following conditions:

- Design aircraft: 360,000 pounds gross weight, twin-twin assembly, bicycle gear, spacing 37-62-37 inches, contact area 267 square inches each wheel.

- Design freezing index: 600 degree-days
- Subgrade material:

  Uniform sandy clay, CL

  Plasticity index, 18

  Frost group, F3

  Water content, 20 percent (average)

  Normal-period CBR, 10
- Base course materials:

  Good quality base material -- crushed gravel (GW), normal-period CBR=80, 30 percent passing no. 10 sieve, 1 percent passing no. 200 sieve.

  Subbase material -- coarse 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 pounds per cubic foot.
- Average water content after drainage (good quality base and subbase): 5 percent.
- Highest groundwater: approximately 4 feet below surface of subgrade.

a. Alternative designs. From the design curves of TM 5-825-2 (AFM 88-6, Chap. 2), the required combined thickness of pavement and base for the normal-period subgrade CBR is 18 inches. According to the reduced subgrade strength method of design, the required combined thickness for

F3 subgrade is 37 inches (from para. 4-5,b of this report, and appropriate design curve of TM 5-825-2 [AFM 88-6, Chap. 2]).

b. Limited subgrade frost penetration design method. The combined thickness of pavement and base a to prevent any freezing of the subgrade in the design year is 40 inches. With the thickness of the double bituminous surface treatment neglected, the thickness of base c required to prevent freezing into the subgrade is also 40 inches. The ratio of subgrade to base water content is r = 20/5 = 4. Since this is an overrun pavement, the maximum allowable r of 3.0 is used in figure 4-1 to obtain the required thickness of base b of 23 inches, which would allow about 6 inches of frost penetration into the subgrade 1 year in 10. From comparison of the alternative frost designs, the 23-inch thickness would be selected. The layered structure of the pavement could comprise the following: double bituminous surface treatment, 12 inches of good quality base (since the good quality base contains less than 2 percent passing the no. 200 sieve, it can also be used as the free-draining layer), and 11 inches of subbase. Subgrade preparation would be required to a depth equal to 2/3 x 44 - 23 = 4 inches.

8-4. Example 4. Shoulder pavement. Design a flexible shoulder pavement at an Air Force facility for the following conditions:

- Design air freezing index: 2800 degree-days
- Mean annual air temperature: 39°F
- Subgrade material: silty clay (CL), F4, known locally as highly frost-susceptible material subject to marked differential heave.

Water content, 29 percent

Plasticity index, 10

Normal-period CBR, 7

- Base course materials:

Good quality base -- stabilized aggregate, normal-period CBR=80, 30 percent passing no. 10 sieve, 1 percent passing no. 200 sieve. Average dry unit weight 135 pounds per cubic foot, average water content 5 percent.

Subbase -- coarse 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: 115 pounds per cubic foot, average water content 12 percent.

a. Conventional frost designs. According to paragraph 4-6,a in this report, and the appropriate design curve of TM 5-825-2 (AFM 88-6, Chap. 2), the required combined thickness of pavement and base is 17 inches. Since local experience indicates frost action in the subgrade produces excessive differential heave, additional protection is necessary. For the conditions summarized, the depth of frost penetration into granular soil having thermal properties equal to those of the good quality bas would be given by figure 3-5 as about 122 inches. Pavement thickness required by TM 5-825-2 (AFM 88-6, Chap. 2) is 2 inches. According to figure 4-1 a combined thickness of pavement and base of 71 inches would be needed under the method of design for limited subgrade frost penetration, allowing subgrade freezing to a depth of about 17 inches. Since the cost of such a shoulder pavement would be intolerably high, an alternative design incorporating polystyrene insulation would be considered.

b. Insulated pavement designs for prevention of subgrade freezing. (See app. C.) A readily available extruded polystyrene that has been used for pavement insulation has a compressive strength of 35 psi. For this facility, the minimum cover will be estimated as that necessary to limit to 11.5 psi the vertical stress on the insulation caused by the overburden and a single-axle truck with a load on dual tires of 12,000 pounds. Using the Boussinesq equations of stress distribution in a semi-infinite elastic solid, we find that the cover required under this criterion would be about 24 inches of pavement and base. The mean annual soil temperature is estimated as 39° + 7° = 46°F. Figure C-1 gives a surface temperature amplitude A = 38°F, and the initial temperature differential v_o is 14°F. With $v_o/A = 14/38 = 0.37$, figure C-2 indicates that about 3.2 inches of insulation is required to prevent frost penetration through the insulation. Accordingly, it probably will be more economical to use a lesser thickness of insulation and a layer of subbase material beneath the insulation. Figure C-3 shows that with total cover above the insulation of 24 inches (2 inches asphalt pavement and 22 inches base), the following combinations of insulation and underlying granular material, with thermal properties equal to those of the subbase, would fully contain the freezing zone:

Insulation thickness (inches)	Total depth of frost (inches)	Thickness of granular material beneath insulation (inches)
1	58	33
2	45	19
3	40	13

Since pavement sections that include these thicknesses of subbase still appear excessively thick, consideration should be given to permitting limited frost penetration into the subgrade.

c. Insulated pavement designs permitting limited subgrade freezing. Taking the total depth of frost tabulated above as the value a in figure 4-1, deducting the 2-inch thickness of surface course to obtain c, and averaging the water contents of good and intermediate quality base materials to establish r as 18/8.5 = 2.1, we see that the following thicknesses of base plus insulation are required to meet the criteria for limited subgrade frost penetration:

For no subgrade freezing			For limited subgrade frost penetration		
Insul. thickness (inches)	Total depth of frost (inches)	Total base plus insulation (inches)	Total base plus insulation[a] (inches)	Total base below insulation[b] (inches)	Depth of subgrade freezing[a] (inches)
1	58	56	37	14	9
2	45	43	28	4	7
3	40	38	25	0 (4)[c]	6 (3)

[a] From figure 4-1, with r = 2.1.

[b] For example, 37-22-1 = 14.

[c] If frost will penetrate through the insulation, a minimum of 4 inches of granular material must be provided beneath the insulation.

d. Summary of alternative designs. The trial design with 3 inches of insulation over 4 inches of base, which permits 3 inches of subgrade freezing, does not appear advantageous because with only 0.5 inches (rounded upward from 3.2 inches) of additional insulation, the base can be dispensed with and frost penetration into the subgrade can be prevented. Accordingly, the following alternative pavement designs should be considered and compared on a functional and economic basis:

	Thickness for various alternatives (inches)						
	I	II	III	IV	V	VI	VII
Asphalt concrete	2	2	2	2	2	2	2
Good quality base	8	35	11	11	11	11	11
Subbase	7	34	11	11	11	11	11
Insulation	-	--	3.5	2	1	2	1
Subbase	-	--	----	19	33	4	14
Total	17	70	27.5	45	58	30	39
Depth of subgrade frost penetration, inches	a	18	0	0	0	7	9

[a] Not determined but judged to be excessive.

8-5. Example 5. 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: 5 (from TM 5-822-5 [AFM 88-7, Chap. 3] for flexible pavements), 4 (from TM 5-822-6 [AFM 88-7, Chap. 1] 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 <u>k</u> = 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:

  Coarse 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 pounds per cubic feet
- Average water content after drainage (good quality base and subbase): 5 percent
- Highest groundwater: about 4 feet below surface of subgrade.

- Concrete flexural strength: 650 psi

Since this pavement has a design index greater than 4, criteria in TM 5-822-5 (AFM 88-7, Chap. 3) and TM 5-822-6 (AFM 88-7, Chap. 1) must be used rather than local highway department requirements. 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 3-5, 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 TM 5-822-5 (AFM 88-7, Chap. 3), the minimum pavement thickness is 2-1/2 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 (para. 7-1).

b. Flexible pavement design by reduced subgrade strength method. From paragraph 4-4,a the frost-area soil support index is 3.5, which, from the design curve in TM 5-822-5 (AFM 88-7, Chap. 3), yields a required combined thickness of pavement and base of 21 inches. Since this is less than the (2-1/2 + 24) 26-1/2-inch thickness required by the limited subgrade frost penetration method, the 21-inch thickness would be used. The pavement structure could be composed of the following: 2-1/2 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 10 inches of the silty sand subbase material. Subgrade preparation would be required to a depth of 1/2 x 45 - 21 = 1-1/2 inches.

c. Rigid pavement design by limited subgrade frost penetration method. From TM 5-822-6 (AFM 88-7, Chap. 1) the required pavement thickness p, based on the normal-period k = 400 psi per inch, the concrete flexural strength of 650 psi and the design index of 4, is 5.5 inches. From figure 3-5, 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. Criteria from chapter 6 and TM 5-822-4 (AFM 88-7, Chap. 4) must be followed when using stabilized layers.

8-6. Example 6. Lightly trafficked road. Design flexible pavements for the following conditions:

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

a. Limited subgrade frost penetration method. By use of the procedure outlined in example 5, paragraph 8-5, 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 pounds per cubic foot. From TM 5-822-5 (AFM 88-7, Chap. 3), 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 paragraph 4-4,a the frost area soil support index is 3.5, which, from the design curve in TM 5-822-5 (AFM 88-7, Chap. 3), 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, 7 inches of base course and 6-1/2 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 pavement. The pavement structure from paragraph 8-6,b can be used to obtain the thickness required through the use of equivalency factors listed in TM 5-822-5 (AFM 88-7, Chap. 3). 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

- Design index: 2 (from TM 5-822-5 [AFM 88-7, Chap. 3])
- Design air freezing index: 1500 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 pounds per cubic foot.
- 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 groundwater: approximately 3 feet below surface of subgrade.

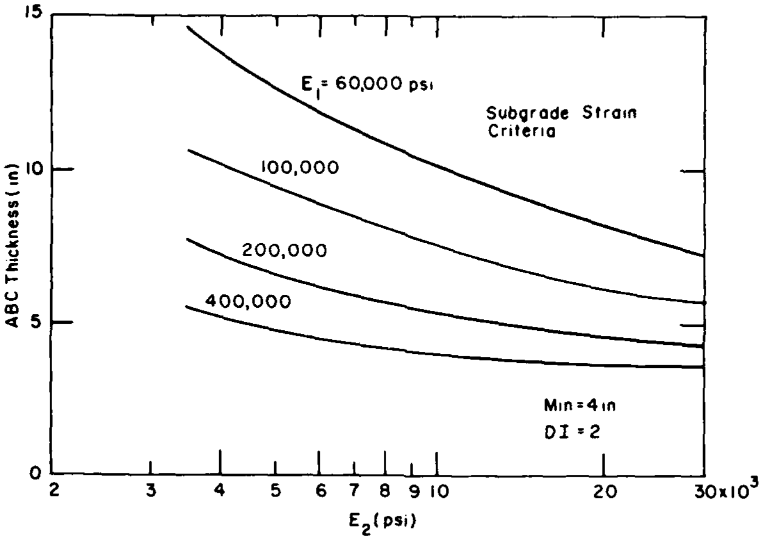


Figure 8-1. Design curves for ABC road pavements.

course. The equivalency factor for the subbase is 2.30, and 7.5 inches ÷ 2.30 = 3.3 inches of bituminous concrete that could be substituted for the subbase. The all-bituminous concrete pavement would be 1.5 + 7.0 + 3.3 = 11.8 inches or 12 inches thick. A filter course a minimum of 4 inches thick is required beneath the pavement (para. 5-5). Subgrade preparation would be required to a depth of 1/2 x 70 - 16 = 19 inches. TM 5-822-5 (AFM 88-7, Chap. 3) also states that the design of all-bituminous concrete pavements may be made by the procedure outlined in Waterways Experiment Station TR 5-75-10. Designs using the above procedure should be coordinated with HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-). The required thickness of the pavement is determined from elastic modulus values for the pavement and subgrade. The procedure for obtaining the modulus values is too lengthy to describe here, but figure 8-1 is used to obtain the pavement thickness when the modulus values have been obtained. For this example, a subgrade modulus, $\underline{E}_2$, of 4000 psi and a pavement modulus, $\underline{E}_1$, 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 subbgrade. 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. The design curve in TM 5-822-5 (AFM 88-7, Chap. 3) yields a required thickness of pavement and base of 11 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 6 inches of F2 soil above the subgrade to comply with the minimum of 15 inches of cover required over the F4 subgrade, (para. 8-6,b). The pavement structure outlined above would still require processing and preparation of the upper 20 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 12 inches would be necessary in the F4 soil.

e. Use of local highway design criteria. As stated in paragraph 4-7, the local state highway design criteria and standards could be used for this project. If the state criteria are used, however, they must be completely adopted. Portions of the state criteria and portions of the Corps of Engineers criteria should not be mixed.

APPENDIX A

FIELD CONTROL OF SUBGRADE AND BASE COURSE CONSTRUCTION FOR FROST CONDITIONS

A-1. General. Personnel responsible for 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 paragraph 7-1 of this report 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.

A-2. Subgrade preparation. The basic requirements of subgrade preparation are set forth in paragraph 7-1 of this report. 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 4-2 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 most careful 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 groundwater 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.

A-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 report, 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 ensure 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 ensure uniformity. Complete surface stripping of pits should be enforced to prevent mixing of detrimental fine soil particles or lumps in the base material.

AD-A134 480 REVISED PROCEDURE FOR PAVEMENT DESIGN UNDER SEASONAL FROST CONDITIONS(U) COLD REGIONS RESEARCH AND ENGINEERING LAB HANOVER NH R L BERG ET AL. SEP 83 2/2

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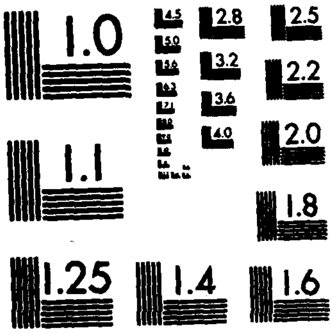


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

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 ensuring 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 ensure 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 report, 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.

A-4. Compaction. Subgrade, subbase and base course materials must meet the applicable compaction requirements in TM 5-822-5 (AFM 88-7, Chap. 3), TM 5-822-6 (AFM 88-7, Chap. 1), TM 5-823-2, TM 5-824-3 (AFM 88-6, Chap. 3) or TM 5-825-2 (AFM 88-6, Chap. 2) when placed and compacted.

APPENDIX B

MINIMIZING LOW-TEMPERATURE CONTRACTION CRACKING OF BITUMINOUS PAVEMENTS

B-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 movement 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.

B-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°F and the kinematic viscosity at 275°F (fig. B-1). Current Corps of Engineers specifications

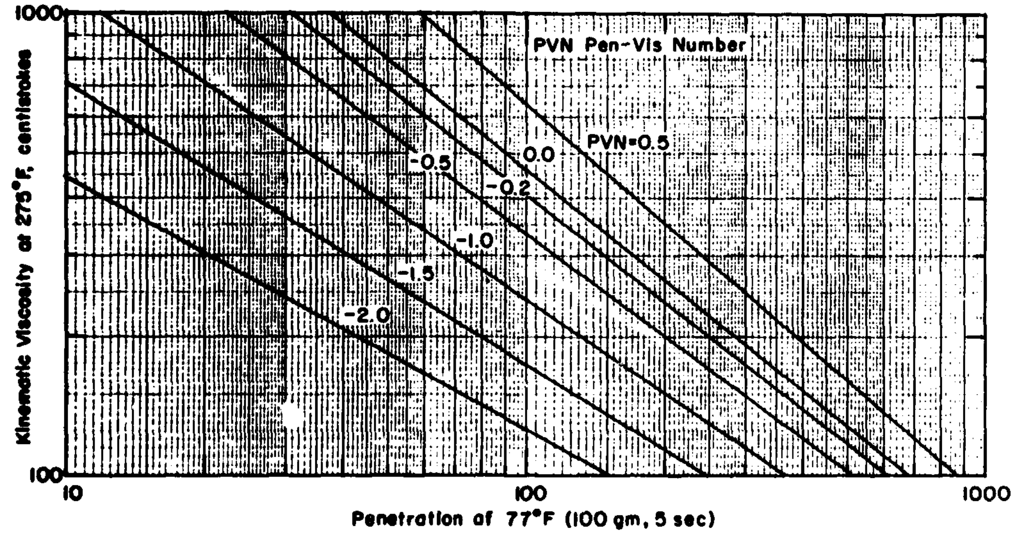


Figure B-1. Pen-vis numbers of asphalt cement.

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 selected and specified in accordance with the requirements for special grades having a minimum PVN of -0.2.

B-3. Selection of asphalt. Figure B-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 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°F. It can be seen from figure B-2 that if asphalt of relatively high PVN can be obtained, selection of extremely soft grades of asphalt will be

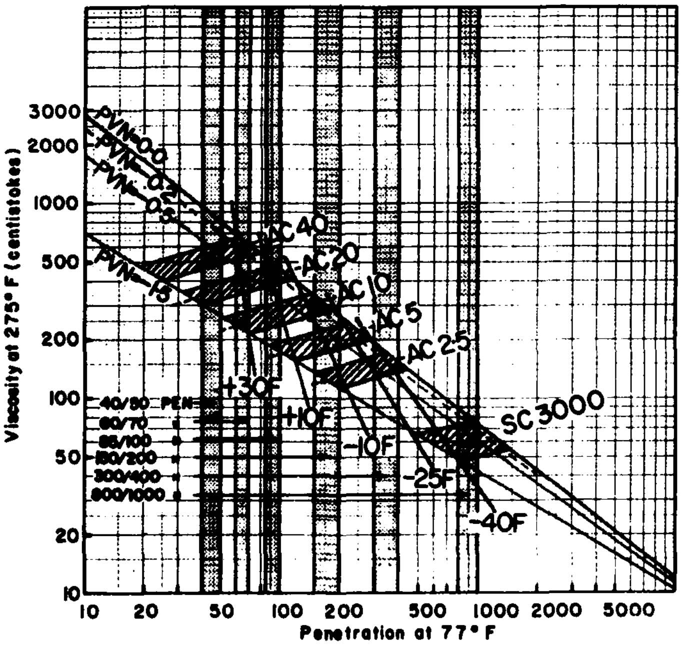


Figure B-2. Guide to selection of asphalt for pavements in cold regions.

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 SC-3000 road oil also is acceptable.

B-4. Effect of mix design variables. It may not always be possible to use the extremely soft grades indicated by figure B-2 for very low temperatures and still produce mixtures meeting the requirements of TM 5-822-8 (AFM 88-6, Chap. 9). In that event the softest grade that will still meet those requirements should always be selected. In designing asphalt-aggregate mixtures in accordance with TM 5-822-8 (AFM 88-6, Chap. 9), 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. The latest version of TM 5-822-8 (AFM 88-6, Chap. 9) and special criteria issued by HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-) should be followed rigorously.

APPENDIX C

USE OF INSULATION MATERIALS IN PAVEMENTS

C-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. Approval from HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-) is required for use of insulating materials other than extruded polystyrene.

a. The use of a synthetic insulating material within a pavement cross section is permissible for airfield shoulder pavements, including small structures inserted in shoulder pavements. With the written approval of HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-), insulation may also be used for other pavements. Experience has shown that surface icing may occur on insulated pavements at times when uninsulated pavements near-by 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 stop 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. The determination of the thickness of insulation and of additional granular material is predicated on the placement of the latter beneath the insulation.

C-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. If a major project incorporating insulation is planned, advice and assistance in regard to the structural analysis should be sought from HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-).

C-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 indices and the mean annual soil temperature at the site. If the latter is unknown, it may be approximated by adding 7°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 1.2,*b*(5) and 3.3, by the appropriate *n*-factor from TM 5-852-6 (AFM 88-19, Chap. 6). For paved surfaces kept free from snow and ice, an *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 C-1 to determine the surface temperature amplitude A. The initial temperature differential v_o is obtained by subtracting 32°F from the mean annual soil temperature, or it also may be read directly from figure C-1. The ratio v_o/A is then determined.

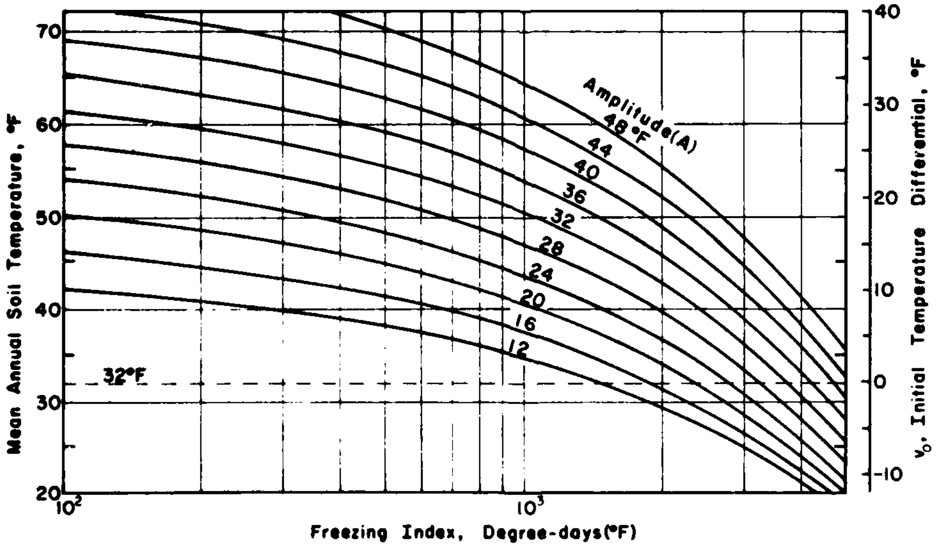
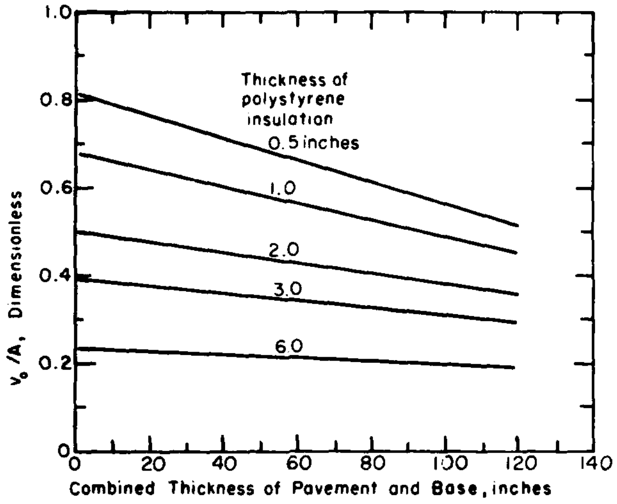


Figure C-1. Equivalent sinusoidal surface temperature amplitude A and initial temperature difference v_o.



NOTES

Design curves based on the following material properties:

Pavement: same thermal properties as upper base

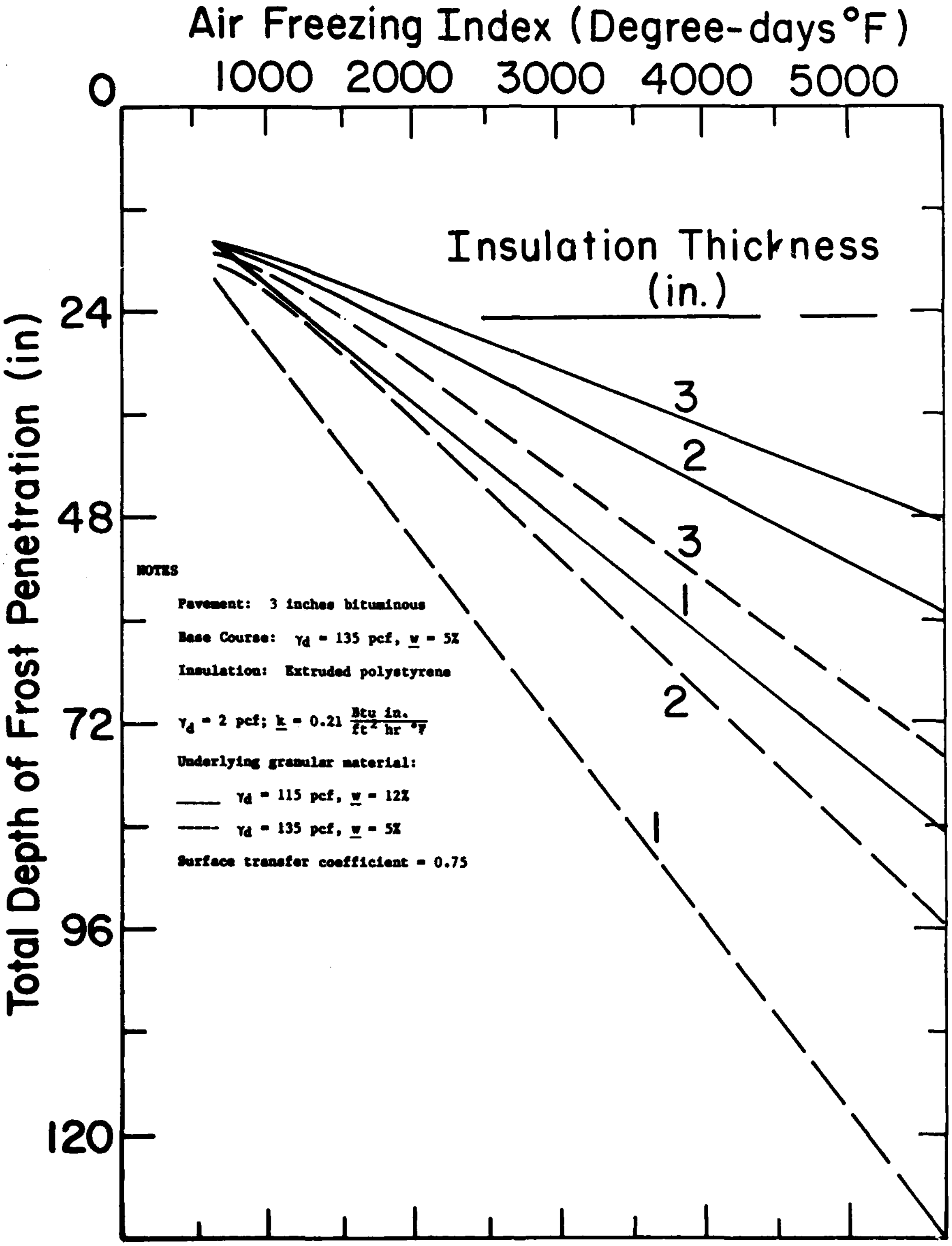
Base: γ_d = 135 pcf, $\underline{w}$ = 7 percent

Extruded polystyrene insulation

γ_d = 2.0 pcf, $\underline{k}$ = 0.21 $\frac{\text{Btu in.}}{\text{ft}^2\ \text{hr}\ °\text{F}}$

Figure C-2. Thickness of extruded polystyrene insulation to prevent subgrade freezing.

Figure C-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 C-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 C-3, which shows the total depth of frost for various freezing indices, thicknesses of



NOTES

Pavement: 3 inches bituminous

Base Course: $\gamma_d = 135$ pcf, $\underline{w} = 5\%$

Insulation: Extruded polystyrene

$\gamma_d = 2$ pcf; $\underline{k} = 0.21 \frac{\text{Btu in.}}{\text{ft}^2 \text{ hr } °\text{F}}$

Underlying granular material:

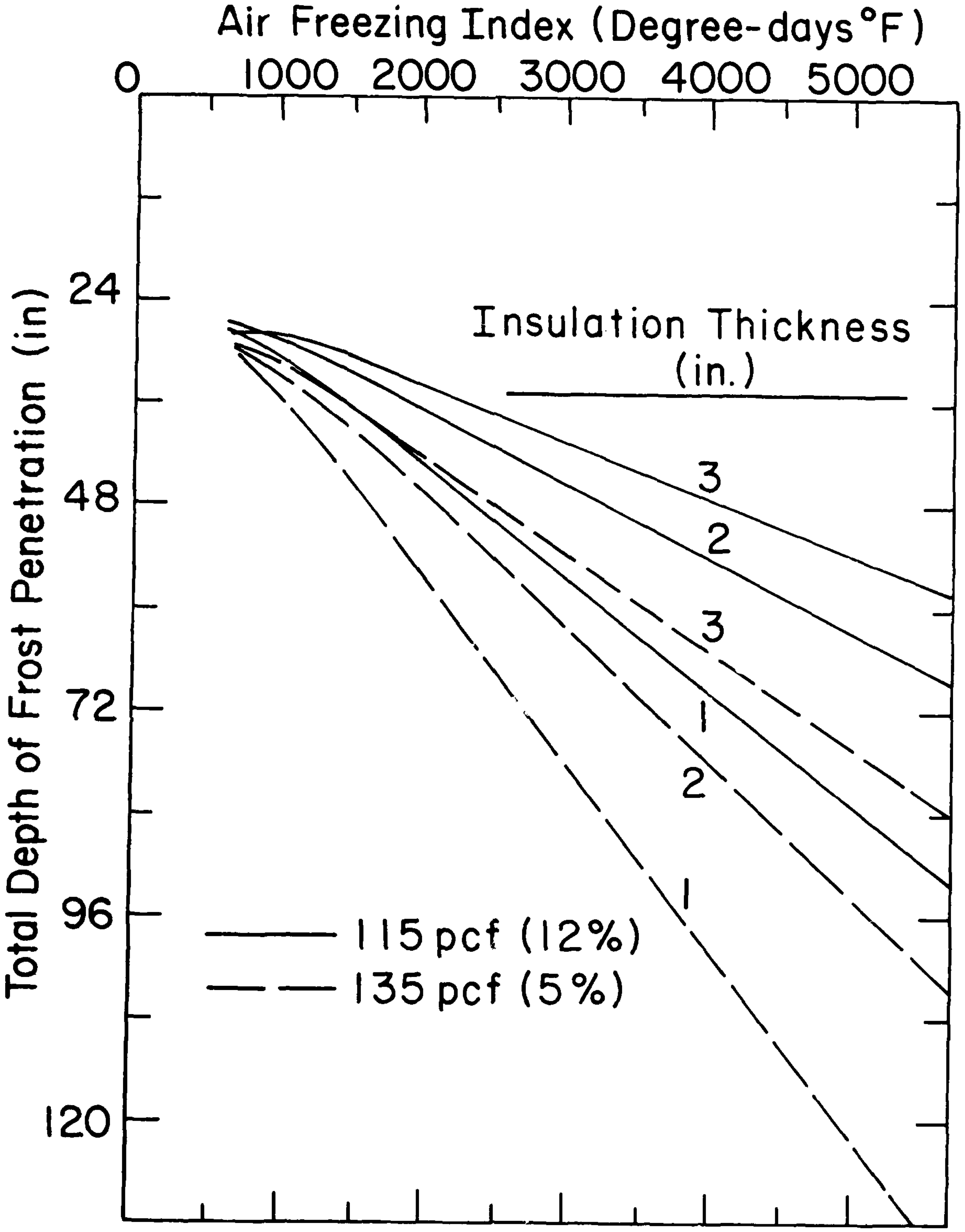
—— $\gamma_d = 115$ pcf, $\underline{w} = 12\%$

– – – $\gamma_d = 135$ pcf, $\underline{w} = 5\%$

Surface transfer coefficient = 0.75

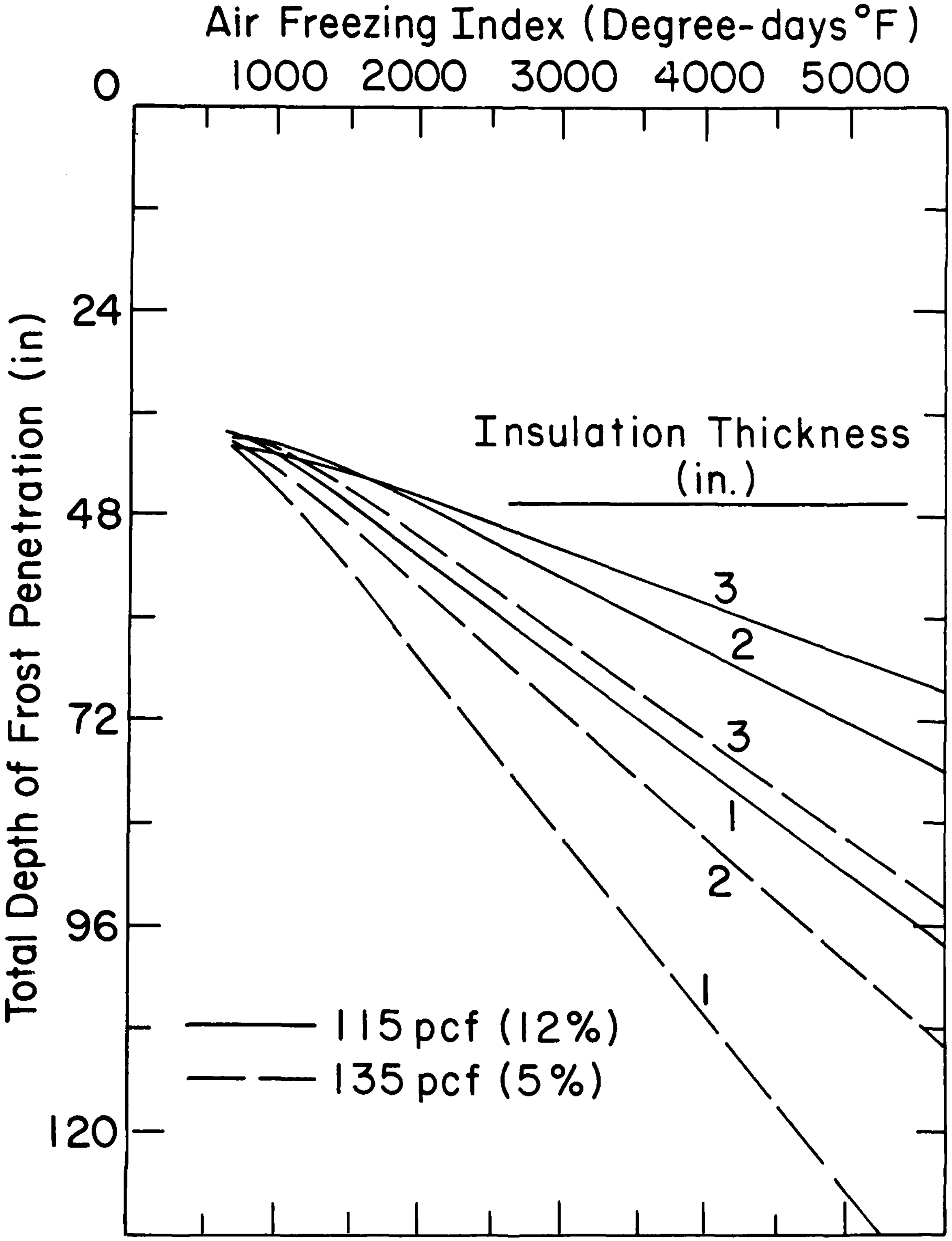
a. 9-in. base course.

Figure C-3. Effect of thickness of insulation and base on frost penetration.



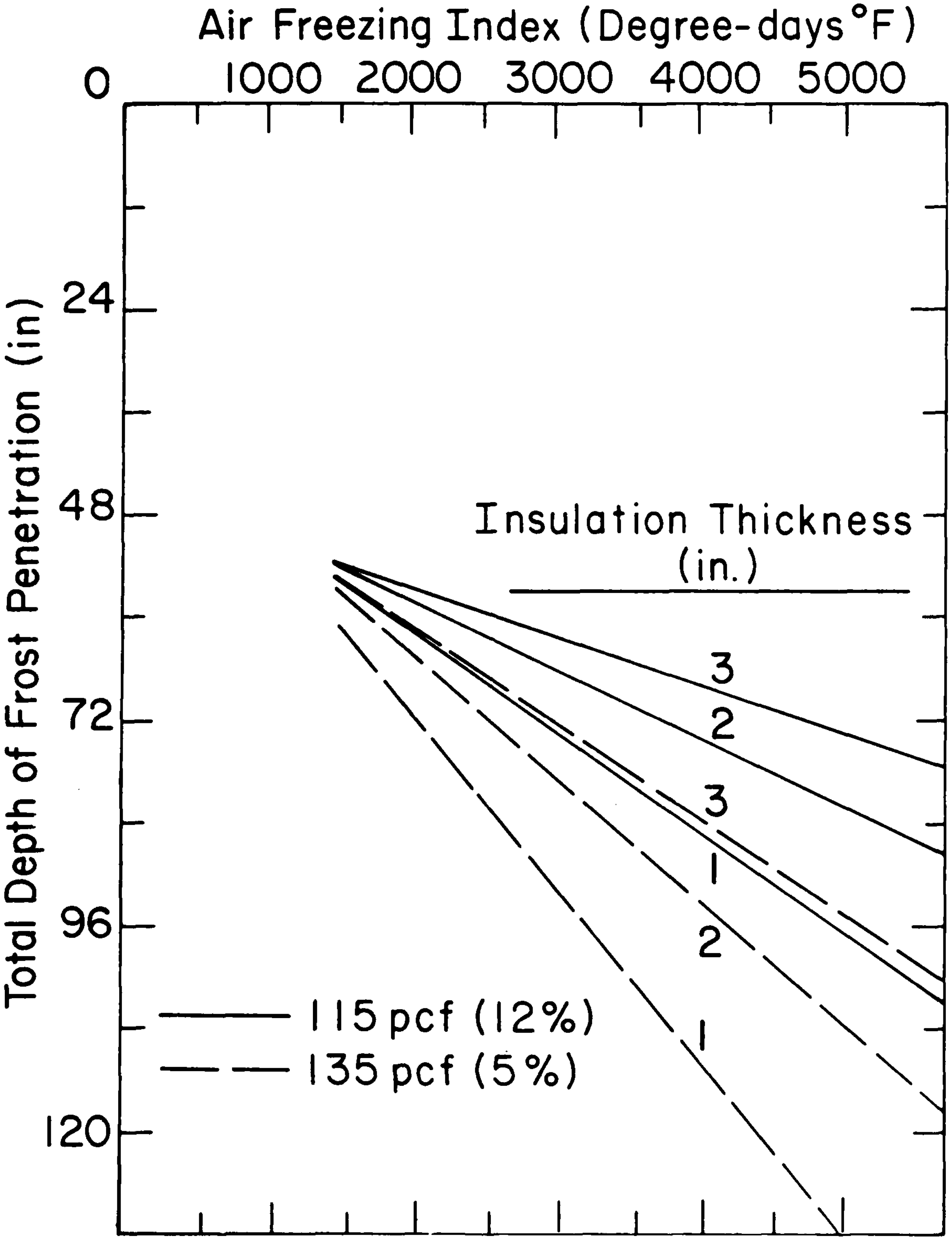
b. 21-in. base course.

Figure C-3 (cont'd). Effect of thickness of insulation and base on frost penetration.



c. 33-in. base course.

Figure C-3 (cont'd).



d. 45-in. base course.

Figure C-3 (cont'd). Effect of thickness and insulation and base on frost penetration.

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.

C-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 C-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 C-3. Not less than 4 inches of subbase material meeting the requirements of paragraph 5-5 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.

C-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 groundwater 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. HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-) may be contacted for more detailed construction procedures.

APPENDIX D

MEMBRANE-ENCAPSULATED SOIL LAYERS (MESL)

D-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 that is meant 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.

D-2. Testing requirements. If a MESL is proposed for use in a pavement system in a frost area, any soil that is intended to be encapsulated should be thoroughly tested to determine classification index properties and CBR-moisture-density relationships. Representative samples, together with the test properties, should be sent to the USACRREL in Hanover, New

Hampshire, for further testing 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 tests by USACRREL, together with pavement design criteria in TM 5-822-5 (AFM 88-7, Chap. 3) and TM 5-825-2 (AFM 88-6, Chap. 2), will also serve to indicate at what levels in the layered pavement system the MESL may be used.

D-3. Permissible uses of MESL. If the results of freezing tests are favorable, the use of MESL is permissible as supporting layers in pavements for roads, streets, walks and storage areas of classes D, E and F; for airfield shoulders, and for airfield overruns. With the approval of HQDA (DAEN-ECE-G) or HQ(USAF/LEEE-), MESL incorporating soil of demonstrated low susceptibility to closed-system freezing may be used as supporting layers for other areas.

D-4. 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.

D-5. 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 D-2.

APPENDIX E

SUMMARY OF RESULTS OF FREEZING TESTS ON NATURAL SOILS

E-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 road 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.

E-2. Presentation of test data and results. Table E-1a 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 E-1a 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

Table E-1. Summary of frost-susceptibility tests on natural soils (1).

a. Open system, nominal surcharge pressure 0.5 psi.

Specimen Number	Material Source	Soil Gradation Data (As Frozen): Unified Soil Classification Symbol (2)	Maximum Size in.	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	Compaction Data (5): Maximum Dry Unit Weight pcf	Optimum Moisture Content %	Specimen Data (As Molded): Dry Unit Weight pcf	Degree of Compaction %	Void Ratio %	S, at Start of Test (6) %	Permeability k (7) cm/sec ×10⁻⁴	Freezing Test Data: Avg. Water Content: Before Test %	After Test %	Total Heave (8) %	Rate of Heave mm/day (9): Avg	Max	Heave Rate Var. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
														GRAVELS AND SANDY GRAVELS															
BPR-5	B.P.R. Alaska	GW	1	40	5.0	1.5	0.7	0.4	0.2	14	1.0			2.77	126.7 (b)	-	124	98	0.395	90	-	13.4	9.8	4.1	0.3	0.8	2.67	M-VL	SC
KA-4	Keflavik		3/4	49	10	3.0	0.8	0.8	0.5	17	1.4			2.81	112.0 (b)	-	109	98	0.589	100	-	21.3	17.7	5.9	0.1	0.3	3.00	M	SC
FC-4	Fairchild		1½	30	6.0	2.9	1.1	0.7	0.4	8.2	1.7			2.96	128.8 (b)	-	126	98	0.462	100	-	11.7	10.7	1.3	0.1	0.3	3.00	M	TL
PBJ-6	Project Blue Jay		3/4	38	18	4.0	1.7	1.3	0.9	53	2.4			2.72	148.2 (b)	-	140	95	0.212	100	-	7.8	28.4	51.8	3.4	5.8	1.70	M-H	SC
DFB-2	Dow Field		3/4	49	12	4.7	2.4	1.7	0.9	20	1.1			2.72	142.9 (b)	-	138	97	0.231	100	-	8.5	24.3	52.5	2.6	4.3	1.65	M-H	SC
DFB-3	Dow Field		3/4	42	13	4.9	2.4	-	-	33	2.4			2.73	137.6 (b)	-	131	95	0.296	95	-	10.3	13.6	13.8	1.0	1.6	1.60	L	SC
DFB-4	Dow Field		3/4	42	13	4.9	2.4	-	-	33	2.4			2.73	137.6 (b)	-	131	95	0.300	[illegible]	-	10.9	14.8	15.7	1.1	1.8	1.64	L	SC
HN-1	Hancock		3/4	35	7.0	4.8	2.6	1.5	1.0	8.2	1.8	18	3.0	2.76	136.0 (b)	-	130	96	0.322	[illegible]	-	11.6	14.8	12.8	0.7	1.3	1.86	VL-L	SC
HN-2	Hancock		3/4	35	7.0	4.8	2.6	1.5	1.0	8.2	1.8	18	3.0	2.76	136.0 (b)	-	132	97	0.309	100	-	11.2	12.3	12.8	0.4	1.5	3.75	VL-L	SC
LSG-7	Loring		3/4	39	11	4.9	3.2	2.6	2.0	24	1.4			2.71	143.8 (d)	6.1	137	95	0.237	100	2.05	8.6	14.8	18.3	2.3	3.2	1.39	M	SC
LSG-36	Loring		2	40	8.0	4.6	3.7	3.3	2.7	17	1.0			2.71	139.3 (b)	-	135	97	0.255	100	-	9.4	17.7	24.6	1.9	3.2	1.68	L-M	SC
PBJ-11	Project Blue Jay	GP	3/4	46	17	1.4	0.4	0.3	0.2	57	0.4			2.74	148.2 (b)	-	144	97	0.188	100	0.11	6.9	12.4	16.0	1.9	3.3	1.74	L-M	SC
PBJ-12	Project Blue Jay		3/4	46	17	1.4	0.4	0.3	0.2	57	0.4			2.74	148.2 (b)	-	140	95	0.218	91	0.21	7.3	25.5	43.0	3.1	5.7	[illegible]	M-H	SC
														SILTY SANDY GRAVELS															
CDB-1	Cape Dyer	GW-GM	2	42	19	5.7	2.0	1.3	1.0	87	1.1			2.67	140.8 (b)	-	139	99	0.200	100	-	7.5	10.8	9.8	0.5	1.0	2.00	VL	T
KA-8	Keflavik		3/4	42	14	5.3	2.1	1.2	0.7	38	2.2			2.77	123.0 (b)	-	120	97	0.446	91	-	14.6	15.0	1.3	0.1	0.2	2.00	M	SC
KA-9	Keflavik		3/4	42	14	5.3	2.1	1.2	0.7	38	2.2			2.77	123.0 (b)	-	121	98	0.435	85	-	13.3	14.8	2.1	0.1	0.3	3.00	M	SC
TAFB-1	Thule		3/4	42	18	7.0	2.5	1.9	1.3	59	1.7	17.8	2.4	2.76	143.6 (b)	-	140	98	0.228	100	-	8.3	13.0	13.5	0.7	1.5	2.14	L-M	SC
TAFB-3	Thule		3/4	44	18	7.0	2.9	2.1	1.5	57	2.0	17.8	2.4	2.77	143.6 (b)	-	140	98	0.230	100	-	8.4	16.2	21.4	1.2	2.5	2.08	L-M	SC
DFSB-2	Dow Field		3/4	49	17	8.0	3.2	-	-	57	2.1			2.73	138.8 (b)	-	134	96	0.274	100	4.6	10.0	16.8	20.5	1.1	1.4	1.27	L	SC
DFSB-3	Dow Field		3/4	49	17	8.0	3.2	-	-	57	2.1			2.73	138.8 (b)	-	132	95	0.288	99	6.2	10.4	15.9	18.4	1.2	1.6	1.33	L	SC
SA-1	Stewart		2	53	20	7.4	3.5	2.5	1.3	48	1.0			2.69	143.3 (b)	-	139	97	0.231	100	2.3	8.4	13.7	16.2	3.1	3.7	1.19	H	T
SA-5	Stewart		2	53	20	7.4	3.5	2.5	1.3	48	1.0			2.69	143.3 (b)	-	141	98	0.222	100	2.0	8.1	19.1	29.6	2.5	4.0	1.60	M	T
LSG-8	Loring		3/4	51	12	5.5	4.0	3.3	2.3	22	1.3			2.71	139.1 (b)	-	137	98	0.237	98	1.1	8.4	13.2	14.6	2.1	2.7	1.28	M	SC
AFG-1A	Afghanistan		1	52	24	9.2	4.0	3.0	2.2	-	-			2.73	143.2 (b)	-	141	-	0.202	99	-	7.4	-	25.0	2.3	3.7	1.60	M	T
BPG-1	Bowley Pit		3	47	13	7.5	4.3	3.2	1.8	47	2.2			2.69	138.0 (b)	-	132	96	0.267	100	-	9.4	23.4	36.1	2.5	3.5	1.40	M	T
PI-1	Presque Isle		3/4	44	14	7.0	4.5	3.1	2.5	32	1.3	16.8	4.7	2.74	142.9 (b)	-	140	98	0.220	100	-	8.1	16.8	22.6	2.0	2.2	1.10	M	SC
LSG-37	Loring		1	48	9.0	5.6	4.6	4.1	3.1	16	1.0			2.71	138.4 (b)	-	134	97	0.259	100	-	9.6	21.1	34.3	3.1	5.0	1.61	M-H	SC
LSG-14	Loring		1	50	12	8.0	6.3	5.4	4.0	26	1.9	24	6	2.71	-	-	134	>95	0.263	96	14.0	9.3	19.1	32.8	3.4	4.3	1.26	M-H	SC
LSG-1A	Loring		3/4	47	17	9.5	6.8	-	-	-	-	17	17	2.71	-	-	137	-	0.250	100	-	9.1	30.0	61.1	2.9	4.5	1.55	M-H	SC
CDB-2	Cape Dyer	GP-GM	2	47	23	9.1	3.2	2.1	1.5	120	0.6			2.69	141.3 (b)	-	136	96	0.233	97	-	8.4	15.9	23.0	1.4	2.7	1.92	L-M	T
SA-3	Stewart		2	51	12	5.8	3.3	2.5	1.8	23	0.8			2.70	144.0 (b)	-	141	98	0.218	100	1.9	7.9	15.4	21.3	3.3	4.0	1.21	M	T
SA-7	Stewart		2	51	12	5.8	3.3	2.5	1.8	23	0.8			2.70	144.0 (b)	-	141	98	0.221	100	2.0	8.0	19.1	30.5	2.2	3.2	1.45	M	T
MP-3	Marble Point		2	56	32	11	3.7	3.0	2.0	101	0.3			2.74	150.0 (b)	-	142	95	0.199	99	-	7.2	9.3	7.9	1.0	2.0	2.00	L	T
PBJ-13	Project Blue Jay		3/4	54	32	10	4.0	2.2	1.5	81	0.4			2.73	143.2 (b)	-	143	100	0.194	100	0.14	7.1	15.2	19.6	1.8	2.3	1.28	L-M	SC
AFG-3	Afghanistan		1	57	24	10	4.5	3.3	2.4	75	1.5			2.70	146.1 (b)	-	142	97	0.184	96	-	6.8	14.4	[illegible]	2.1	2.4	1.14	M	T
AFG-4	Afghanistan		1	49	18	9.8	6.5	-	-	71	1.7			2.72	146.1 (b)	-	144	98	0.175	95	-	6.3	12.4	19.3	4.2	4.4	1.04	H	T
MP-2	Marble Point		2	45	25	11	6.8	6.0	4.0	258	0.7			2.72	139.3 (b)	-	135	97	0.262	100	-	9.6	12.0	7.0	1.2	2.3	1.92	L-M	T
MP-6	Marble Point		2	45	25	11	6.8	6.0	4.0	258	0.7			2.72	139.3 (b)	-	135	97	0.260	99	-	9.5	14.2	14.6	1.2	2.3	1.92	L-M	T
AFG-6	Afghanistan		1½	55	20	11	6.9	4.7	3.4	125	3.1			2.71	-	-	144	>95	0.171	99	-	9.2	[illegible]	24.7	4.6	7.6	1.65	H	T
														SILTY GRAVELS															
AFG-2	Afghanistan	GM	1	48	21	13	6.3	4.4	3.0	193	3.6			2.72	144.3 (b)	-	142	98	0.191	96	-	6.8	11.2	-	2.2	3.0	1.36	M	T
CRG-1	Cold Brook Pit		2½	55	28	15	6.3	4.1	2.0	167	0.9			2.72	144.7 (b)	-	139	96	0.218	96	-	7.6	20.9	40.0	[illegible]	[illegible]	1.60	M-H	T
BM-7	Ball Mountain Till		2	91	35	18	7.0	-	-	250	0.3			2.81	-	-	145	-	0.210	100	-	7.5	8.7	10.7	0.7	2.2	3.14	VL-M	T
BPR-4	B.P.R. Alaska		1	58	38	27	10	5.0	2.2	270	0.1			2.72	132.4 (b)	-	127	96	0.338	92	-	11.4	26.9	38.7	[illegible]	4.5	1.60	M-H	SC

(1) General Note: See last sheet of these tables for notes referred to by numbers in parentheses.

Table E-1a (cont'd).

Specimen Number	Material Source	SOIL GRADATION DATA (As Frozen)										PHYSICAL PROPERTIES OF BASIC SOIL					SPECIMEN DATA (As Molded)					FREEZING TEST DATA							Type of Cyl. (12)
		Unified Soil Classification Symbol (2)	Maximum Size	Percent finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity	Compaction Data (5)		Dry Unit Weight	Degree of Compaction	Void Ratio	S, at Start of Test (6)	Permeability k (7) cm/sec x10⁻⁴	Avg. Water Content		Total Heave (8)	Rate of Heave mm/day (9)		Heave Rate VBE Index (10)	Frost Susc. Class (11)	
			in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI		Maximum Dry Unit Weight pcf	Optimum Moisture Content %	pcf	%	%	%		Before Test %	After Test %	%	Avg.	Max.			
												CLAYEY SANDY GRAVELS																	
WDO-1	Washington, D.C.	GW-GC	1½	37	16	6.4	4.2	-	-	57	2.5	26	9	2.65	133.9 (d)	4.7	135	101	0.220	97	-	8.0	12.7	15.6	2.1	3.0	1.42	M	T
WDO-2	Washington, D.C.		1½	37	16	6.4	4.2	-	-	57	2.5	26	9	2.65	133.9 (d)	4.7	136	101	0.218	96	-	7.7	12.0	15.5	2.6	3.3	1.26	M	T
PI-2	Presque Isle	GP-GC	3/4	37	24	11	6.6	5.0	3.2	145	16	24.9	8.7	2.72	136.8 (b)	-	134	98	0.265	97	-	9.7	22.3	42.5	2.9	3.7	1.28	M	SC
PI-4	Presque Isle		3/4	33	15	18	8.7	6.9	-	315	38	22.3	8.1	2.75	-	-	131	96	0.250	98	-	8.8	16.2	19.9	1.5	2.0	1.31	L	SC
												CLAYEY SILTY GRAVELS																	
CL-1	Clinton County	GM-GC	1½	54	30	20	15	9.0	5.0	485	1.9	[illegible]	6.8	2.74	130.2 (a)	9.0	129	99	0.320	100	0.1	11.7	30.3	65.6	4.6	5.7	1.24	H	SC
												CLAYEY GRAVELS																	
GF-1	Great Falls	GC	1½	48	36	22	17	15	12	4000	1.2	42.6	24.6	2.66	140.0 (d)	5.6	133	95	0.252	100	.00003	9.5	21.0	28.0	2.4	5.0	2.08	M-H	SC
LST-18	Loring		3/4	68	52	41	30	25	18	945	0.1	22.1	7.8	2.73	135.8 (d)	7.5	129	95	0.320	100	-	11.6	34.8	84.4	4.0	6.8	1.70	H	SC
LST-19	Loring		3/4	68	52	41	30	25	18	945	0.1	22.1	7.8	2.73	135.8 (d)	7.5	132	97	0.290	100	-	10.3	19.0	30.2	2.3	3.7	1.60	M	SC
LST-20	Loring		3/4	68	52	41	30	25	18	945	0.1	22.1	7.8	2.73	135.8 (d)	7.5	136	100	0.250	100	-	9.0	17.6	28.9	1.5	2.7	1.80	L-M	SC
LST-21	Loring		3/4	68	52	41	30	25	18	945	0.1	22.1	7.8	2.73	135.8 (d)	7.5	134	99	0.270	100	-	9.7	24.3	42.5	2.6	4.0	1.54	M	SC
LST-34	Loring		3/4	68	52	41	30	25	18	945	0.1	22.1	7.8	2.73	135.8 (d)	7.5	132	97	0.290	95	-	10.0	32.0	81.9	4.9	13.2	2.69	H-VH	SC
												SANDS AND GRAVELLY SANDS																	
SA-4	Stewart	SW	2	58	15	4.9	2.3	1.5	1.1	23	1.3			2.72	139.9(b)	-	136	97	0.254	100	3.1	9.7	18.1	20.6	2.9	4.0	1.38	M	T
SA-8	Stewart		2	58	15	4.9	2.3	1.5	1.1	23	1.3			2.72	139.9(b)	-	136	97	0.250	100	3.0	9.3	21.4	32.3	2.4	3.8	1.58	M	T
PAF-3	Plattsburg	SP	1½	59	20	2.1	1.0	0.8	0.5	24	0.3			2.67	132.8(b)	-	130	98	0.281	100	-	10.5	11.2	6.0	0.6	0.7	1.16	VL	SC
PAF-4	Plattsburg		1½	59	20	2.1	1.0	0.8	0.5	24	0.3			2.67	132.8(b)	-	130	98	0.283	100	-	10.6	12.8	9.6	0.3	0.4	1.33	V	SC
PAF-7	Plattsburg		1	72	7.0	3.0	1.3	0.9	0.5	5.3	2.0			3.20	139.1(b)	-	139	100	0.440	86	-	11.7	11.7	7.5	0.3	0.4	1.33	M	SC
FC-1	Fairchild		2	85	8.6	3.6	1.3	1.2	-	3.4	0.2			2.74	119.2(b)	-	116	98	0.469	100	-	17.0	19.0	10.4	0.8	1.6	2.00	VL-L	-
FC-3	Fairchild		2	70	6.9	3.4	1.4	1.3	-	4.7	1.3			2.74	132.1(b)	-	125	95	0.368	100	-	13.4	15.4	10.8	0.7	1.1	1.57	VL-L	T
PAF-5	Plattsburg		1½	72	36	4.5	1.8	1.4	1.0	5.1	0.7			2.67	125.2(b)	-	124	99	0.338	95	-	12.0	12.0	5.3	0.6	0.8	1.33	VL	SC
PAF-6	Plattsburg		1½	72	36	4.5	1.8	1.4	1.0	5.1	0.7			2.67	125.2(b)	-	125	100	0.329	90	-	12.3	13.9	9.8	0.7	0.9	1.28	VL	SC

Table E-1a (cont'd). Open system, nominal surcharge pressure 0.5 psi.

Specimen Number	Material Source	SOIL GRADATION DATA (As Frozen): Unified Soil Classification Symbol (2)	Maximum Size in.	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 -	Compaction Data (5): Maximum Dry Unit Weight pcf	Optimum Moisture Content %	SPECIMEN DATA (As Molded): Dry Unit Weight pcf	Degree of Compaction %	Void Ratio %	S, at Start of Test (6) %	Permeability k (7) cm/sec $\times 10^{-4}$	FREEZING TEST DATA: Avg. Water Content: Before Test %	After Test %	Total Heave (8) %	Rate of Heave mm/day (9): Avg.	Max.	[illegible] Ratio Vol. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
												SILTY GRAVELLY SANDS																	
KA-5	Keflavik	SW-SM	3/4	57	15	5.0	1.4	-	-	27	1.1			2.81	112.0(b)	-	111	99	0.532	100	-	19.5	19.3	2.0	0.3	0.5	1.66	N	SC
BPR-2	B. P. R., Alaska		1	68	12	5.6	2.9	2.3	1.8	10	1.0			2.75	123.1(b)	-	117	95	0.467	93	-	15.8	21.6	15.7	1.2	1.8	1.50	L	SC
SPK-1	Spokane		1½	58	11	7.0	3.5	2.3	1.2	6.7	1.4			2.80	-	-	128	>95	0.365	100	-	13.0	15.8	13.6	1.1	2.0	1.82	L	SC
MIN-1	Minnesota		2	69	20	9.6	3.8	-	-	28	1.8			2.73	135.6(d)	6.5	135	100	0.268	97	-	9.4	22.0	37.0	2.8	4.3	1.54	M-H	-
SA-2	Stewart		2	68	26	9.1	4.0	2.9	1.8	31	1.1	19.3	4.3	2.70	141.7(b)	-	139	98	0.214	100	3.5	8.5	18.2	27.9	4.4	5.0	1.36	H	-
SA-6	Stewart		2	68	26	9.1	4.0	2.9	1.8	31	1.1	19.3	4.3	2.70	141.7(b)	-	138	98	0.224	100	4.0	8.5	20.4	32.2	2.7	4.3	[illegible]	M-H	-
MIT-4	M. I. T.		1½	70	23	9.7	4.4	3.2	2.5	24	1.2			2.70	137.9(b)	-	131	95	0.285	97	-	10.2	20.7	21.9	1.2	2.0	1.66	L	-
HDG-6	Hutchinson's Pit		1	57	20	8.7	5.0	3.5	2.0	43	1.1			2.75	143.3(e)	5.3	144	101	0.179	99	0.02	6.7	24.7	49.7	6.1	[illegible]	1.26	[illegible]	-
HDG-12	Hutchinson's Pit		1	57	20	8.7	5.0	3.5	2.0	43	1.1			2.75	143.3(e)	5.3	141	98	0.221	87	0.13	7.0	37.0	81.3	4.8	5.8	1.20	H	-
HDG-13	Hutchinson's Pit		1	57	20	8.7	5.0	3.5	2.0	43	1.1			2.75	143.3(e)	5.3	138	96	0.242	99	0.29	8.7	23.3	43.8	3.5	4.8	1.37	M-H	-
LSG-38	Loring		½	62	13	7.2	5.7	5.0	4.0	15	1.3			2.71	139.1(b)	-	135	97	0.256	98	-	9.2	24.7	37.4	3.3	4.7	1.42	M-H	SC
RC-1	Rapid City		1½	57	30	12	8.7	7.1	5.8	183	1.1	19.0	2.0	2.75	-	-	137	>95	0.253	98	0.048	8.9	13.9	16.4	1.7	2.5	1.47	L-M	SC
AFG-7	Afghanistan		2	58	23	8.2	3.7	2.3	1.8	48	1.2			2.72	146.7(b)	-	147	>100	0.150	100	-	5.4	16.4	31.3	3.7	5.3	1.43	M-H	-
GR-4	Greenland	SP-SM	3/4	60	33	9.7	1.8	0.8	-	62	0.2			2.73	138.0(d)	5.3	137	99	0.246	100	0.48	9.0	16.9	20.4	1.3	1.8	1.38	L	-
FC-2	Fairchild		2	84	11	5.3	1.9	1.7	-	4.0	1.6			2.75	123.5(b)	-	121	98	0.421	100	-	15.3	17.9	10.8	0.9	1.5	1.66	VL-L	-
MIN-4	Minnesota		-	100	71	8.8	2.2	1.3	-	4.3	1.5			2.70	114.4(b)	-	114	100	0.473	100	-	16.8	16.0	2.4	0.2	0.5	2.50	N	-
VF-6	Volk Field		-	100	86	5.0	2.6	2.4	1.8	2.0	0.9			2.66	115.6(b)	-	115	100	0.450	100	-	15.3	16.3	2.8	0.1	0.5	5.00	N	-
DU-1	Indiana		-	100	100	6.3	2.6	2.2	1.7	1.9	1.0			2.65	107.1(b)	-	109	102	0.516	100	26.0	19.3	16.7	0.7	0.1	0.5	5.00	N	SC
DU-2	Indiana		-	100	100	6.3	2.6	2.2	1.7	1.9	1.0			2.65	107.1(b)	-	105	98	0.576	100	38.0	21.8	19.8	1.3	0.1	0.5	[illegible]	N	SC
DU-3	Indiana		-	100	100	6.3	2.6	2.2	1.7	1.9	1.0			2.65	107.1(b)	-	109	102	0.514	100	26.0	19.4	20.5	2.3	0.4	0.5	1.25	N	SC
MI-1	Minot		1½	73	11	5.2	2.7	2.2	1.6	8.1	0.9			2.73	130.5(b)	-	129	99	0.316	100	-	11.5	14.3	8.8	0.5	1.0	2.00	VL	
DFB-1	Dow Field		3/4	66	18	6.0	2.8	1.7	1.0	15	0.9			2.72	137.6(b)	-	133	97	0.278	100	0.2	10.7	21.8	27.6	1.8	3.3	1.83	L-M	SC
SLF-1	Selfridge		1½	74	25	6.9	3.2	2.7	1.8	15	0.6			2.70	126.8(b)	-	127	100	0.329	100	-	12.2	19.9	18.3	1.0	1.7	1.70	L	
SLF-2	Selfridge		1½	77	27	7.1	3.3	3.0	2.6	13	0.7			2.70	126.8(b)	-	127	100	0.329	100	-	12.2	20.7	18.0	1.2	2.2	1.83	L-M	-
SCA-1	Schenectady		3/4	99	84	10	3.3	3.0	2.0	3.4	1.8			2.68	113.0(b)	-	113	100	0.484	98	-	17.7	25.5	16.5	1.1	2.2	2.00	L-M	SC
SCA-2	Schenectady		3/4	99	84	10	3.3	3.0	2.0	3.4	1.8			2.68	113.0(b)	-	112	99	0.487	100	-	18.2	26.2	17.5	1.0	2.0	2.00	L	SC
KIS-6	Kinross		1½	98	80	8.8	3.3	2.0	0.9	2.8	1.4			2.62	109.0(b)	-	108	99	0.518	100	-	19.8	21.5	6.2	0.7	1.2	1.71	VL-L	-
KIS-4	Kinross		-	100	82	9.0	3.4	2.0	0.9	2.8	1.4			2.62	109.0(b)	-	106	98	0.542	100	-	20.6	20.9	3.3	0.4	0.7	1.75	N-VL	-
KIS-3	Kinross		-	100	82	9.0	3.4	2.0	0.9	2.8	1.4			2.63	109.0(b)	-	105	97	0.552	100	-	20.4	21.1	4.0	0.4	0.8	2.00	N-VL	-
HDG-1	Hutchinson's Pit		2	56	17	6.0	3.5	2.4	-	28	0.7			2.74	141.0(b)	-	140	99	0.222	100	0.1	8.1	18.2	28.1	3.7	5.5	1.48	H	-
K-2A	Korea		2½	58	28	9.4	3.5	2.1	1.4	111	0.3			2.61	127.0(b)	-	128	100	0.268	96	-	9.6	13.0	46.2	2.2	3.5	1.59	M	-
K-2B	Korea		2½	56	28	9.4	3.5	2.1	1.4	111	0.3			2.61	127.0(b)	-	124	98	0.310	99	-	11.9	17.3	20.8	3.8	5.0	1.32	M-H	-
LIN-2	Lincoln		1	66	22	6.5	3.9	2.7	2.0	17	0.9			2.65	134.0(d)	-	134	100	0.238	100	-	9.0	13.7	14.0	0.8	1.4	1.75	VL-L	SC
SPK-3	Spokane		3/4	79	13	8.1	4.1	2.7	1.5	6.4	3.2			2.80	-	-	128	<95	0.361	100	-	12.6	17.7	13.5	1.1	1.4	1.27	L	SC
SPK-4	Spokane		3/4	79	13	8.1	4.1	2.7	1.5	6.4	3.2			2.80	-	-	128	<95	0.351	90	-	11.3	18.5	16.3	1.4	2.3	1.64	L-M	SC
KIS-1	Kinross		3/4	92	67	9.0	4.5	2.9	1.8	4.2	1.2			2.65	120.4 (b)	-	115	95	0.438	100	-	16.5	19.3	8.2	0.8	1.7	2.12	VL-L	T
KIS-4	Kinross		3/4	92	67	9.0	4.5	2.9	1.8	4.2	1.2			2.65	120.4 (b)	-	119	98	0.396	100	-	14.9	31.1	36.5	2.7	3.7	1.37	M	T
KIS-5	Kinross		1½	92	67	9.0	4.5	2.9	1.8	4.2	1.2			2.65	120.4 (b)	-	120	100	0.367	99	-	13.9	32.9	44.4	5.4	7.8	1.44	H	T
PBJ-3	Proj. Blue Jay		3/4	71	46	10	4.5	4.0	1.8	20	0.3			2.70	142.6 (b)	-	138	97	0.215	100	-	8.0	25.8	29.0	3.1	4.5	1.45	MH	SC
PBJ-4	Proj. Blue Jay		3/4	71	46	10	4.5	4.0	1.8	20	0.3			2.70	142.6 (b)	-	137	96	0.230	100	-	8.5	37.8	69.4	3.2	5.8	1.81	MH	SC
T-3	Tobyhanna		1½	59	39	8.5	4.5	2.5	1.6	6.0	0.2			2.72	140.4 (b)	-	134	96	0.280	100	-	9.9	20.2	21.8	1.5	2.3	1.53	L-M	SC
LIN-3	Lincoln		1	80	24	5.5	4.9	3.8	3.0	15	0.8			2.65	133.1 (d)	-	135	101	0.228	98	-	8.6	13.5	14.9	1.0	1.4	1.40	L	SC
LIN-1	Lincoln		1	63	30	7.0	5.0	3.0	2.0	28	0.4			2.65	133.1 (d)	-	137	103	0.212	100	-	8.0	12.7	15.8	1.0	1.4	1.40	L	SC
LIN-4	Lincoln		1	71	27	7.8	5.0	4.0	3.2	16	0.6			2.65	133.1 (d)	-	132	99	0.250	98	-	9.3	15.5	19.6	1.2	1.7	1.42	L	SC
CDB-3	Cape Dyer		2	61	27	9.7	5.1	4.2	3.1	52	0.7			2.68	134.8 (b)	-	130	97	0.289	94	-	10.1	24.1	37.3	2.1	3.3	1.57	M	-
AFG-1	Afghanistan		1	71	32	11.9	5.5	3.9	2.8	-	-			2.73	143.2 (b)	-	141	-	0.205	96		7.2	19.8	35.5	3.9	7.1	1.82	MH	-
WVS-3	West Virginia		1½	57	33	10	5.6	4.5	-	81	0.3			2.70	129.1 (b)	-	125	97	0.349	87	-	11.3	23.4	27.6	1.8	3.3	1.83	L-M	-
VF-7	Volk Field		2	94	83	20	5.6	5.0	3.5	3.0	1.5			2.62	121.6 (b)	-	120	97	0.364	100	-	13.9	31.2	39.2	2.3	3.2	1.34	M	-
LSG-19	Loring		1	65	14	[illegible]	[illegible]	5.2	4.6	260	34	24	6	2.71	139.1 (b)	-	135	97	0.254	93	2.3	8.5	14.6	17.4	2.0	3.8	1.70	M	SC
LSG-39	Loring		1	88	17	10	8.2	7.2	[illegible]	[illegible]	[illegible]			2.72	139.1 (b)	-	135	97	0.259	99	-	8.5	31.6	59.8	2.0	4.2	2.10	MH	SC

Table E-1a (cont'd).

Specimen Number	Material Source	SOIL GRADATION DATA (As Frozen): Unified Soil Classification Symbol (2)	Maximum Size in.	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	Compaction Data (5): Maximum Dry Unit Weight pcf	Optimum Moisture Content %	SPECIMEN DATA (As Molded): Dry Unit Weight pcf	Degree of Compaction %	Void Ratio %	G, at Start of Test (6) %	Permeability k (7) cm/sec x10⁻⁴	FREEZING TEST DATA: Avg. Water Content: Before Test %	After Test %	Total Heave (8) %	Rate of Heave mm/day (9): Avg.	Max.	Heave Ratio Var. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
													SILTY SANDS																
PAF-1	Plattsburg	SM	-	100	95	28	1.5	1.2	0.9	2.5	0.9			2.68	110.3 (b)	-	107	97	0.567	85	-	18.6	18.7	4.4	0.2	0.5	2.50	M	SC
PAF-2	Plattsburg		-	100	95	28	1.5	1.2	0.9	2.5	0.9			2.68	110.3 (b)	-	109	99	0.540	96	-	19.2	19.2	4.4	0.1	0.2	2.00	M	SC
AFS-4	Alaska		-	100	100	33	2.5	-	-	1.6	1.0			2.79	106.4 (b)	-	108	101	0.605	100	-	21.7	24.6	9.4	0.7	1.5	2.14	VL-L	SC
WO-2	Westover		-	100	86	20	2.5	-	-	4.1	1.2			2.69	119.9 (b)	-	115	96	0.458	99	-	16.5	17.6	4.3	0.2	0.3	1.50	M	SC
MIN-5	Minnesota		-	100	95	20	3.8	2.2	-	3.7	1.3			2.68	114.2 (d)	-	114	100	0.434	99	-	16.0	42.8	49.8	2.3	9.3	4.04	H-VH	T
BRF-2	Bradley		3/4	79	27	14	4.2	2.6	-	47	1.9			2.76	133.6 (b)	-	133	100	0.300	100	-	10.9	20.6	20.0	1.2	1.6	1.33	L	T
BRF-1	Bradley		3/4	67	31	14	4.4	2.6	-	62	0.9			2.76	143.1 (b)	-	143	100	0.202	100	-	7.3	21.9	21.9	2.4	3.3	1.38	M	T
BAFS-1	Bethel		-	100	100	21	4.5	2.5	1.0	3.0	1.1			2.68	106.4 (d)	-	106	100	0.578	96	-	20.7	21.6	21.6	0.5	1.0	2.00	VL	T
BAFS-2	Bethel		-	100	100	21	4.5	2.5	1.0	3.0	1.1			2.68	106.4 (d)	-	105	99	0.593	88	-	19.4	21.6	21.6	0.6	1.0	1.66	VL	T
WO-3	Westover		-	100	86	26	5.1	-	-	27	1.3			2.69	114.3(b)	-	114	100	0.467	100	-	17.7	23.9	14.2	0.7	1.0	1.42	VL	SC
GR-1	Greenland		3/4	66	45	17	5.2	3.7	2.4	47	0.4			2.73	137.9(b)	-	135	98	0.258	100	-	9.5	22.6	35.3	2.2	2.7	1.22	M	T
GR-2	Greenland		3/4	66	45	17	5.2	3.7	2.4	47	0.4			2.73	137.9(b)	-	137	99	0.244	97	-	8.6	31.6	60.2	3.8	5.5	1.44	M-H	T
GR-3	Greenland		3/4	66	45	17	5.2	3.7	2.4	47	0.4			2.73	137.9(b)	-	136	99	0.252	100	-	9.2	22.9	38.4	2.0	2.9	1.45	M	T
PBJ-10	Proj. Blue Jay		3/4	82	53	21	6.0	5.2	2.8	25	0.6			2.71	136.0(e)	7.0	129	95	0.312	88	-	10.1	28.5	36.9	2.9	3.8	1.31	M	SC
AFG-5	Afghanistan		1	57	23	12	6.2	3.9	2.7	111	3.1			2.69	144.6(b)	-	144	99	0.155	98	-	6.0	13.9	24.7	2.2	3.7	1.68	M	T
K-1	Korea		2	88	43	18	6.5	4.0	2.5	28.6	1.3			2.58	120.0(b)	-	120	99	0.398	96	-	12.5	27.2	39.5	5.5	8.0	1.45	H	T
WO-4	Westover		-	100	85	27	7.0	-	-	6.9	1.2			2.71	116.4(b)	-	117	100	0.450	100	-	16.5	26.5	19.6	1.2	1.8	1.50	L	SC
WO-6	Westover		-	100	85	27	7.0	-	-	6.9	1.2			2.71	116.4(b)	-	111	95	0.521	100	-	15.2	22.4	10.8	0.6	1.3	2.16	VL-L	SL
MIT-3	M. I. T.		2	84	47	13	7.5	5.3	3.6	17	1.9			2.70	123.0(d)	13.2	123	100	0.374	96	-	13.2	21.9	22.4	2.3	3.2	1.39	M	T
MIT-2	M. I. T.		2	76	49	17	7.8	4.5	3.0	28	1.4			2.70	122.1(d)	14.2	122	100	0.384	100	-	14.2	25.3	28.3	1.9	2.7	1.42	L-M	T
PAFB-7	Portsmouth		3/4	98	94	29	8.2	5.4	3.7	4.0	1.8			2.73	111.2(d)	-	109	98	0.560	96	-	19.8	26.2	13.5	0.8	1.5	1.86	VL-M	SC
MIN-8	Minnesota		-	100	97	48	8.8	4.5	-	4.4	0.8			2.72	126.0(b)	-	120	95	0.419	99	-	15.3	22.0	18.3	1.4	3.3	2.36	L-M	T
WN-1	Wendover		2	58	27	14	8.9	7.5	6.0	250	2.2	21.9	3.0	2.70	129.4(b)	-	128	>99	0.312	100	0.6	11.5	12.4	8.5	0.9	1.2	1.33	VL-L	SC
VF-5	Volk Field		-	100	88	13	11	9.5	7.7	20	7.5			2.72	119.5(b)	-	114	95	0.375	100	-	16.7	33.5	37.2	2.2	3.0	1.36	M	T
MH-1	Mansfield Hollow		3/4	78	53	23	11	7.5	4.5	38	1.3			2.70	136.0(d)	-	131	96	0.290	98	0.097	10.5	30.1	45.0	3.3	4.0	1.21	M	SC
MH-2	Mansfield Hollow		3/4	78	53	23	11	7.5	4.5	38	1.3			2.70	136.0(d)	-	131	96	0.291	88	0.098	9.5	24.2	35.3	4.6	6.5	1.41	M-H	SC
WWC-2	Fairchild		3/4	71	34	23	11	6.3	4.0	95	2.2	21.6	2.0	2.79	142.9(b)	-	136	95	0.280	100	-	10.0	22.2	27.1	2.8	4.8	1.71	M-H	SC
TAFB-5	Thule		3/4	73	47	20	12	9.0	6.9	71	1.8	14.1	2.2	2.88	150.9(b)	-	145	96	0.243	100	0.092	8.4	30.2	58.0	2.6	5.3	2.04	M-H	SC
TAFB-6	Thule		3/4	73	47	20	12	9.0	6.9	71	1.8	14.1	2:2	2.88	150.9(b)	-	144	95	0.248	98	0.10	8.4	34.1	66.4	3.0	6.5	2.16	M-H	SC
PAFB-8	Portsmouth		3/4	68	45	23	14	9.1	1.2	14	1.2			2.71	128.6(b)	-	127	99	0.333	100	-	12.3	46.1	81.8	5.6	8.8	1.57	H-VH	SC
WO-8	Westover		3/4	97	75	38	14	7.0	-	17	0.8			2.65	-	-	112	>95	0.483	97	-	17.6	73.1	116.9	4.9	7.4	1.51	H	T
TD-5	Truax		3/4	90	79	28	15	12	9.0	36	4.2			2.70	137.3(d)	5.6	130	95	0.300	93	-	10.2	19.5	23.2	1.5	2.7	1.80	L-M	SC
MIN-2	Minnesota		3/4	97	73	31	17	14	13	280	18	18.3	2.8	2.73	130.2(b)		124	95	0.374	100	-	13.7	53.3	118.0	6.3	10.2	1.62	H-VH	T
TD-13	Truax		1/2	82	71	32	19	13	9.4	50	4.6	14	2	2.72	-	-	129	>95	0.311	95	0.056	10.8	14.8	10.8	2.2	3.0	1.36	M	SC
MIT-1	M. I. T.		1½	81	58	33	19	12	6.5	56	0.9	20.7	0.9	2.70	119.0(d)	15.0	119	100	0.404	100	-	15.0	30.2	35.4	2.1	2.8	1.33	M	T
TD-36	Truax		3/4	82	71	32	19	13	9.5	50	4.6	14.4	1.6	2.72	139.0(c)	5.3	136	98	0.246	100	0.0085	9.1	17.2	21.0	1.8	3.5	1.94	L-M	S-TR
TD-9	Truax		3/4	98	79	35	22	15	12	55	1.9	14.4	1.6	2.72	137.3(d)	5.6	130	95	0.303	100	-	11.1	18.2	22.0	2.5	[illegible]	1.12	M	SC
TD-10	Truax		3/4	92	79	35	22	15	12	55	1.9	14.4	1.6	2.72	137.3(d)	5.6	134	98	0.265	98	-	9.6	14.9	14.7	2.5	2.8	1.12	[illegible]	SC
TD-31	Truax		3/4	92	79	35	22	15	12	55	1.9	14.4	1.6	2.72	137.3(d)	5.6	139	102	0.216	100	-	8.0	16.3	15.5	1.3	1.7	1.30	L	SC
TD-32	Truax		3/4	92	79	35	22	15	12	55	1.9	14.4	1.6	2.72	137.3(d)	5.6	132	96	0.280	100	-	10.3	23.3	37.1	2.9	3.3	1.34	M	SC
													CLAYEY SILTY SANDS																
SF-1	Sioux Falls	SM-SC	1	71	28	16	9.0	6.0	4.3	108	3.7	24.1	5.9	2.72	137.0(d)	7.2	131	96	0.292	100	0.16	10.6	15.7	16.6	1.7	2.7	1.58	L-M	SC
LSG-16	Loring		1	87	22	15	13	11	8.0	260	34	24	6.0	2.72	139.1(b)	-	134	96	0.265	99	0.12	9.7	29.1	56.7	5.5	7.5	1.36	H	SC
TAFB-7	Thule		3/4	65	39	22	14	10	7.0	310	0.9	16.1	4.3	2.87	152.5(b)	-	148	97	0.215	100	0.021	7.5	31.0	61.4	2.6	4.7	1.80	M-H	SC
TAFB-8	Thule		3/4	65	39	22	14	10	7.0	310	0.9	16.1	4.3	2.87	152.5(b)	-	146	96	0.223	100	0.029	7.8	35.9	68.8	3.3	6.5	1.96	M-H	SC
CA-3	Casper		1½	91	48	23	15	13	11	225	13	22.0	4.6	2.64	120.8(d)	7.2	120	99	0.378	100	0.66	14.2	19.6	17.1	1.5	2.7	1.80	L-M	SC
P-1	Patterson		1½	62	33	22	15	10	5.5	400	2.7	22.0	6.1	2.74	-	-	135	95	0.267	100	0.001	9.7	26.0	44.4	3.3	4.2	1.27	M-H	SC
CA-1	Casper		1½	98	62	21	16	14	12	137	14	21.8	6.0	2.65	120.8(d)	7.2	118	98	0.403	100	1.3	15.2	21.7	17.7	1.6	2.3	1.44	L-M	SC
CA-2	Casper		3/4	98	68	29	18	16	14	195	11	22.0	4.6	2.66	120.8(d)	7.2	119	99	0.393	95	0.53	14.0	22.0	20.0	2.2	3.2	1.45	M	SC
BOSG-1	Bong		1½	94	75	44	21	15	10	33	1.3	16.8	5.1	2.76	139.5(c)	-	135	97	0.290	100	0.0003	10.8	13.1	13.1	1.3	2.0	1.54	L	T
BOSG-2	Bong		1½	94	75	44	21	15	10	33	1.3	16.8	5.1	2.76	139.5(c)	-	136	97	0.267	100	0.0001	9.7	12.8	12.8	1.3	1.7	1.30	L	T
BOSG-5	Bong		1½	94	75	44	21	15	10	33	1.3	16.8	5.1	2.76	139.5(c)	-	134	96	0.282	100	0.0002	10.2	16.2	21.8	1.7	3.7	2.18	L-M	T
LST-5	Loring		1½	83	63	46	30	25	18	188	0.8	21.1	6.0	2.71	-	-	127	>95	0.334	100	0.0034	12.3	44.0	77.6	6.7	8.7	1.30	H-VH	SC
LST-3	Loring		1½	87	62	48	32	24	15	100	0.2	21.1	6.0	2.71	-	-	127	>95	0.334	100	0.0034	12.3	50.1	56.9	2.8	3.3	1.18	M	SC

Table E-1a (cont'd). Open system, nominal surcharge pressure 0.5 psi.

Specimen Number	Material Source	SOIL GRADATION DATA (As Frozen): Unified Soil Classification Symbol* (2)	Maximum Size in.	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	Compaction Data (5): Maximum Dry Unit Weight pcf	Optimum Moisture Content %	SPECIMEN DATA (As Molded): Dry Unit Weight pcf	Degree of Compaction %	Void Ratio %	S, at Start of Test (6) %	Permeability k (7) cm/sec $\times10^{-4}$	FREEZING TEST DATA: Avg. Water Content: Before Test %	After Test %	Total Heave (8) %	Rate of Heave mm/day (9): Avg.	Max.	Heave Rate Var. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
														CLAYEY SANDS															
FA-1	Fargo	SC	3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70	127.2(d)	9.0	123	97	0.374	100	0.09	13.9	21.5	18.7	1.5	2.7	1.80	L-M	SC
FA-4	Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70	127.2(d)	9.0	118	93	0.424	100	0.17	15.7	32.8	42.4	3.3	4.5	1.36	M-H	SC
PBJ-1	Proj. Blue Jay		3/4	73	55	35	23	20	15	500	1.7	24.7	8.1	2.70	133.1(e)	9.8	134	101	0.272	100	0.033	8.0	17.9	25.3	2.2	2.8	1.27	M	SC
BH-1A	Breed's Hill (EBT)		3/4	76	60	41	24	-	-	191	1.1	24.0	11.0	2.75	138.7(e)	7.2	139	100	0.237	94	-	8.0	10.5	7.3	0.6	1.0	1.66	VL	SL
WO-9	Westover		3	82	66	48	30	23	17	115	0.9	20.7	7.2	2.71	-	-	130	>95	0.297	100		10.9	22.3	31.5	3.1	4.6	1.48	M-H	T
MIN-7	Minnesota		3/4	97	78	48	31	-	-	-	-	28.7	10.7	2.70	-	-	114	>95	0.478	91	-	16.2	32.0	38.6	1.8	2.5	1.38	L-M	T
PBJ-15	Proj. Blue Jay		3/4	80	58	44	35	31	22	310	0.1	18.6	9.2	2.75	139.6(e)	7.0	139	100	0.234	100	0.0027	8.5	17.3	26.3	2.2	3.8	1.72	M	SC
PBJ-16	Proj. Blue Jay		3/4	80	58	44	35	31	22	310	0.1	18.6	9.2	2.75	139.6(e)	7.0	132	95	0.301	100	0.0042	10.9	34.7	83.0	4.6	8.3	1.80	H-VH	SC
														SILTS AND SANDY SILTS															
GB-3	Gonee Bay	ML	-	100	99	55	6.0	-	-	-	-			2.74	102.0(e)	7.9	102	100	0.688	100	3.6	24.4	25.6	7.0	0.3	1.0	3.33	M-VL	SC
WO-5	Westover		-	100	91	53	13	-	-	-	-			2.69	113.6(d)	11.0	112	99	0.484	100	-	18.0	26.0	17.3	1.0	1.6	1.60	L	SC
L-1	Labrador		-	100	100	95	27	10	-	-	-	26.0	3.0	2.77	102.0(d)	18.1	106	104	0.626	100	0.4	22.7	27.3	11.4	1.2	1.5	1.25	L	SC
L-1B	Labrador		-	100	100	95	27	10	-	-	-	25.0	3.0	2.77	102.0(d)	18.1	103	102	0.668	94	0.5	22.4	30.0	16.3	1.5	2.3	1.53	L-M	SC
VIS-1	Valparaiso		-	100	100	99	54	25	15	-	-	23.7	4.0	2.72	115.8(d)	13.5	113	98	0.501	98	0.024	18.4	62.1	95.3	9.8	11.5	1.17	VH	SC
VIS-2	Valparaiso		-	100	100	99	54	25	15	-	-	23.7	4.0	2.72	115.8(d)	13.5	113	98	0.501	100	0.024	18.0	65.6	100.0	10.0	13.3	1.33	VH	SC
HAN-1	Hanover		-	100	100	98	35	18	8	-	-	39.5	12.7	2.75	[illegible]	16.9	101	96	0.696	100	-	24.8	79.0	105.6	14.1	16.8	1.19	VH	T
DFC-14	Dow Field		3/4	95	91	87	54	40	28	-	-	32.8	8.1	2.66	107.1(e)	-	104	98	0.590	100	0.000004	22.2	136.8	104.0	13.3	23.3	1.68	VH	SC
MIN-3	Minnesota		3/4	97	92	83	63	44	28	-	-	36.0	5.1	2.62	-	-	101	>95	0.611	99	-	23.0	50.6	84.6	3.5	5.8	1.66	M-H	-
NH-4	New Hampshire		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7(e)	16.5	106	99	0.589	100	0.13	21.7	79.6	126.2	15.3	20.7	1.30	VH	SC
NH-11	New Hampshire		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7(e)	16.5	104	97	0.618	100	0.16	22.8	120.7	130.1	15.9	18.0	1.13	VH	SC
NH-12	New Hampshire		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7(e)	16.5	108	101	0.567	100	0.11	20.9	99.9	190.6	26.0	28.3	1.08	VH	SC
NH-20	New Hampshire		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7(e)	16.5	105	98	0.611	100	0.15	22.6	116.7	239.2	12.8	19.7	1.54	VH	SC
														CLAYEY SILTS															
YS-1	Yukon	ML-CL	-	100	100	98	60	37	22	-	-	25.3	5.8	2.73	124.5(d)	11.5	123	99	0.389	100	0.0031	14.2	28.9	37.0	2.2	3.5	1.59	M	SC
NH-79A	New Hampshire		-	100	100	86	61	34	14	-	-	24.1	5.9	2.76	106.7(e)	16.5	105	98	0.643	88	0.054	20.5	78.1	150.2	7.9	15.8	2.00	H-VH	SC
NH-31	New Hampshire		-	100	96	90	67	36	16	-	-	25.0	6.0	2.70	106.7(e)	16.5	101	95	0.662	100	0.033	24.5	84.6	117.6	14.0	18.3	1.30	VH	SC
NH-29A	New Hampshire		-	100	93	85	73	47	23	-	-	26.0	5.0	2.70	106.7(e)	16.5	101	95	0.674	100	0.036	25.0	86.8	235.3	14.0	15.5	1.10	VH	SC
NH-44A	New Hampshire		-	100	100	99	73	37	13	-	-	23.7	6.0	2.70	110.1(e)	14.7	107	97	0.577	99	-	21.2	42.4	50.8	3.7	4.5	1.22	M-H	SC
NH-43A	New Hampshire		-	100	100	99	73	37	13	-	-	23.7	6.0	2.70	110.1(e)	14.7	106	96	0.596	100	-	22.0	36.4	29.8	4.0	5.3	1.32	M	SM
														SILTS WITH ORGANICS															
LF-4	Ladd Field	ML-OL	-	100	100	91	38	13	6.0	-	-	31.6	0	2.75	101.6(d)	18.1	98	97	0.737	100	0.64	26.8	45.7	36.5	3.1	4.0	1.23	M	SC
LFT-13	Fairbanks		-	100	100	97	42	22	12	-	-	32.6	6.2	2.67	107.4(e)	17.1	101	95	0.646	100	0.20	24.2	112.6	229.6	11.3	14.0	1.24	VH	SC
LFT-14	Fairbanks		-	100	100	97	42	22	12	-	-	32.6	6.2	2.67	107.4(e)	17.1	111	103	0.505	100	0.09	18.9	105.7	281.2	11.5	13.7	1.19	VH	SC
														GRAVELLY AND SANDY CLAYS															
DF[illegible]-5	Dow Field	CL	3/4	82	70	62	40	31	23	-	-	25.6	7.9	2.73	-	-	133	>95	0.352	100		12.8	42.7	73.0	4.8	10.3	2.14	M-VH	SC
FB-20A	Fort Belvoir		3/4	95	87	64	43	36	30	-	-	41.0	18.0	2.70	114.9(e)	15.6	115	100	0.468	94	-	16.3	25.1	25.0	1.3	2.0	1.54	L	SL
EBT-3**	East Boston		3/4	84	72	56	43	35	25	-	-	23.0	7.0	2.76	130.8(d)		126	96	0.371	100	0.0009	13.4	46.5	95.3	6.5	10.5	1.62	H-VH	SC
EBT-4*	East Boston		3/4	84	72	56	43	35	25	-	-	23.0	7.0	2.76	130.8(d)		130	99	0.324	100	0.0003	11.7	30.2	47.7	4.0	5.5	1.38	H	SC
EBT-21	East Boston		3/4	84	72	56	43	35	25	-	-	23.0	7.0	2.76	130.8(d)		125	96	0.374	100	0.0010	13.6	22.9	122.9	7.0	11.1	1.58	H-VH	SC
EBT-22	East Boston		3/4	84	72	56	43	35	25	-	-	23.0	7.0	2.75	130.8(d)		130	99	0.328	100	0.0002	11.9	34.9	81.1	6.5	7.5	1.15	H	SC
F[illegible]A	Fort Belvoir		1/4	98	90	61	49	41	34	-	-	43.8	20.3	2.73	114.9(e)	15.6	110	96	0.536	100	0.148	19.8	81.6	188.4	7.7	12.0	1.56	VH	SC
F[illegible]	Fort Belvoir		1/4	98	90	61	49	41	34	-	-	43.8	20.3	2.73	114.9(e)	15.6	117	102	0.456	90	0.098	15.0	22.3	18.2	1.5	3.7	2.45	L-H	SC
F[illegible]	Fort Belvoir		1/4	98	90	61	49	41	34	-	-	43.8	20.3	2.73	114.9(e)	15.6	113	98	0.504	100	0.130	18.5	27.4	22.1	1.3	3.0	2.30	L-M	SC
F[illegible]-14A	Fort Belvoir		1/4	98	98	61	49	41	34	-	-	43.8	20.3	2.73	114.9(e)	15.6	118	103	0.441	100	0.08	16.2	27.0	27.6	2.2	3.2	1.45	M	SC
[illegible]	Portsmouth		-	100	100	96	49	38	30	-	-	30.0	11.7	2.73	110.3(e)	17.7	109	99	0.569	95	-	13.8	60.7	112.7	4.5	12.8	2.84	H-VH	SC
[illegible]	East Boston		3/4	86	73	57	49	42	30	-	-	21.0	7.0	2.75	130.8(d)		129	98	0.336	100	0.012	12.2	37.4	72.5	7.8	12.7	1.62	H-VH	SC
E[illegible]-29	East Boston		3/4	86	73	57	49	42	30	-	-	21.0	7.0	2.75	130.8(d)		130	99	0.328	100	0.011	11.9	29.9	48.3	7.3	9.2	1.2[illegible]	H-VH	SC
EBT-30	East Boston		3/4	86	73	57	49	42	30	-	-	21.0	7.0	2.76	130.8(d)		129	98	0.336	100	0.012	12.2	32.0	51.2	8.0	10.5	1.31	VH	SC
EBT-31	East Boston		3/4	86	73	57	49	42	30	-	-	21.0	7.0	2.76	130.8(d)		131	100	0.317	100	0.009	11.5	18.3	22.8	4.5	5.7	1.24	M	SC
DFC-12	Dow Field		3/4	96	93	86	51	38	27	-	-	26.4	8.4	2.6[illegible]	119.8 (d)	14.0	118	98	0.424	100	0.0028	15.7	49.1	67.0	6.2	10.7	1.72	H-VH	SC
DFC-13	Dow Field		3/4	85	82	78	53	40	30	-	-	27.6	9.5	2.73	119.8 (d)	14.0	119	99	0.429	100	0.00007	15.8	66.0	125.0	6.6	11.0	1.5[illegible]	H-VH	SC
ART-2	AASHO		1¼	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	117	97	0.467	100	0.0047	17.2	25.4	28.4	2.3	4.3	1.86	M-H	-
ART-3	AASHO		1¼	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	120	99	0.420	100	0.0020	15.5	20.2	17.1	1.3	2.0	1.54	L	-
ART-4	AASHO		1¼	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	125	103	0.367	100	0.0008	13.5	18.6	13.9	1.1	2.3	2.0[illegible]	L-M	-
ART-15	AASHO		1¼	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	119	98	0.442	100	0.0030	16.3	26.2	27.0	2.8	3.8	1.35	M	-
ART-16	AASHO		1¼	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	126	104	0.360	100	0.0007	13.3	18.0	10.1	1.2	1.3	1.08	L	-
BOSG-4	Bong		3/4	97	90	80	60	48	36	-	-	28.6	12.6	2.80	128.8 (c)		125	98	0.395	96	0.00002	13.5	16.1	16.1	1.5	1.7	1.13	L	-
BOSG-6	Bong		3/4	97	90	80	60	48	36	-	-	28.6	12.6	2.80	128.8 (c)		125	98	0.403	100	0.00005	14.5	17.7	17.4	1.2	1.7	1.42	L	
BOSG-3	Bong		3/4	97	91	81	61	50	35	-	-	29.6	13.6	2.80	128.8 (c)		126	98	0.389	97	0.00001	13.6	16.7	16.7	1.4	1.5	1.07	L	-
DFT-1	Dow Field		1½	94	88	80	64	52	37	-	-	30.0	12.0	2.71	119.8 (d)	14.0	117	98	0.448	100	0.0032	16.4	69.7	124.3	10.1	12.0	1.20	VH	SC
DFT-3	Dow Field		1½	94	88	80	64	52	37	-	-	30.0	12.0	2.71	119.8 (d)	14.0	118	98	0.431	100	0.0026	16.1	42.4	79.7	3.3	3.8	1.15	M	SC

* 4-inch diameter

Table E-1a (cont'd).

Specimen Number	Material Source	SOIL GRADATION DATA (As Frozen)										PHYSICAL PROPERTIES OF BASIC SOIL					SPECIMEN DATA (As Molded)					FREEZING TEST DATA							Type of Cyl (12)
		Unified Soil Classification Symbol (2)	Maximum Size	Percent finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity	Compaction Data (5)		Dry Unit Weight	Degree of Compaction	Void Ratio	S, at Start of Test (6)	Permeability k (7)	Avg. Water Content		Total Heave (8)	Rate of Heave mm/day (9)		Heave Rate Var. Index (10)	Frost Susc. Class (11)	
			in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI		Maximum Dry Unit Weight pcf	Optimum Moisture Content %	pcf	%	%	%	cm/sec ×10⁻⁴	Before Test %	After Test %	%	Avg.	Max.			
													LEAN CLAYS																
PAFB-1A	Portsmouth	CL	-	100	98	91	33	24	19	-	-	28.0	12.0	2.71	113.4 (d)	-	113	100	0.474	92	-	16.3	38.0	47.1	4.0	4.8	1.20	H	SL
CCL-1	Crosby		-	100	98	91	58	41	31	-	-	36.5	16.8	2.78	119.3 (e)	13.5	117	98	0.485	100	-	17.5	24.6	17.7	1.4	2.3	1.64	L-M	T
FBW-1	Greenland		-	100	100	97	60	43	34	-	-	31.3	15.2	2.79	119.4 (d)	15.0	116	97	0.518	100	-	18.3	30.1	26.8	2.2	5.3	2.40	M-H	T
YS-7	Yukon		-	100	100	100	67	37	29	-	-	28.0	8.6	2.74	121.4 (d)	12.8	117	96	0.460	89	0.000003	15.0	22.0	24.0	1.1	2.5	2.27	L-M	SL
YS-8	Yukon		-	100	100	100	67	37	29	-	-	28.0	8.6	2.74	121.4 (d)	12.8	118	97	0.448	94	0.000002	15.4	33.0	45.7	3.8	5.3	1.37	M-H	SC
YS-14	Yukon		-	100	100	100	67	37	29	-	-	28.0	8.6	2.74	121.4 (d)	12.8	123	101	0.385	100	0.0000006	14.1	29.5	38.5	2.1	4.0	1.70	H	SC
YS-15	Yukon		-	100	100	100	67	37	29	-	-	28.0	8.6	2.74	121.4 (d)	12.8	120	98	0.424	100	0.000001	15.5	29.1	34.3	1.8	3.7	2.06	L-M	SC
YS-16	Yukon		-	100	100	100	67	37	29	-	-	28.0	8.6	2.74	121.4 (d)	12.8	115	95	0.476	94	0.000005	16.5	36.6	46.2	2.5	4.2	1.68	M-H	SC
													LEAN CLAYS WITH ORGANICS																
WASHO-1	Malad, Idaho	CL-OL	-	100	99	96	65	48	35	-	-	37.0	13.0	2.58	99.6	21.0 (e)	99	99	0.630	100	-	24.4	31.4	20.9	3.4	4.0	1.18	H	SC
WASHO-5	Malad, Idaho		-	100	99	96	65	48	35	-	-	37.0	13.0	2.58	99.6	21.0 (e)	96	96	0.678	100	-	26.3	60.8	61.0	4.6	7.3	1.58	H	SC
WASHO-6	Malad, Idaho		-	100	99	96	65	48	35	-	-	37.0	13.0	2.58	99.6	21.0 (e)	98	98	0.644	100	-	25.0	42.5	42.3	4.1	5.2	1.26	H	SC
WASHO-7	Malad, Idaho		-	100	99	96	65	48	35	-	-	37.0	13.0	2.58	99.6	21.0 (e)	99	99	0.627	100	-	24.3	45.0	45.0	4.2	5.0	1.19	H	SC
													FAT CLAYS																
FCH-1	Frederick	CH	-	100	99	74	61	52	43	-	-	55.0	37.0	2.88	106.7	19.5 (e)	105	98	0.715	86	-	21.2	38.4	39.0	0.8	1.7	2.12	VL-L	T

85 percent of full saturation. The test results listed in table E-1b (average rate of heave versus percentage by weight of grains finer than the 0.02-millimeter size) are plotted on figure 2-2, in envelopes according to soil type. Table E-1b contains data grouped similarly in every respect to those in Table E-1a, except that they do not meet the compaction criterion of 95 percent or greater and do not have the required initial degree of saturation. Table E-1c contains heave rate data on specimens tested under a lower load pressure than specimens in tables E-1a and E-1b. Data from tables E-1b and E-1c have not been plotted on figure 2-2.

E-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, both 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

Table E-1 (cont'd). Summarv of frost-susceptibility tests on natural soils (1).

b. Open system, nominal surcharge pressure 0.5 psi (soils do not meet compaction criterion of 95% or greater and do not have the 85% or greater initial degree of saturation).

Specimen Number	Material Source	SOIL GRADATION DATA (As Frozen)										PHYSICAL PROPERTIES OF BASIC SOIL					SPECIMEN DATA (As Molded)					FREEZING TEST DATA							Type of Cyl (12)
		Unified Soil Classification Symbol (2)	Maximum Size	Percent finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity	Compaction Data (5)		Dry Unit Weight	Degree of Compaction	Void Ratio	S, at Start of Test (6)	Permeability k (7) cm/sec ×10⁻⁴	Avg. Water Content		Total Heave (8)	Rate of Heave mm/day (9)		Heave Rate Vs. Index (10)	Frost Susc. Class (11)	
			in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI		Maximum Dry Unit Weight pcf	Optimum Moisture Content %	pcf	%	%	%		Before Test %	After Test %	%	Avg	Max			
														GRAVELS AND SANDY GRAVELS															
BPR-1	Alaska Highway	GW	1	41	11	2.0	1.0	0.6	0.4	24	1.1			2.75	143.9 (b)	-	136	95	0.261	80	-	7.6	11.6	11.3	0.7	1.0	1.42	VL	SC
LSO-5	Loring		3/4	45	9.0	3.9	1.8	1.5	1.2	16	1.3			2.71	143.8 (d)	6.1	123	86	0.374	91	-	12.5	14.4	13.8	1.3	1.8	1.38	L	SC
LSO-6	Loring		3/4	46	10	4.4	3.4	2.9	2.1	18	1.4			2.71	143.8 (d)	6.1	130	90	0.390	98	-	10.8	11.1	8.3	1.1	1.8	1.64	L	SC
KA-3	Keflavik	GP	2	37	9.0	3.0	1.0	-	-	38	0.8			2.81	112.0 (b)	-	112	100	0.562	82	-	16.6	16.6	9.3	0.1	0.1	1.00	N	T
BPR-3	Alaska Highway		1	44	11	2.6	1.2	0.7	0.5	26	0.9			2.73	143.3 (b)	-	127	95	0.341	83	-	10.4	13.9	9.6	0.7	1.3	1.86	VL-L	SC
KA-1	Keflavik		3	38	12	4.1	1.6	0.9	0.5	91	0.5			2.64	145.5 (b)	-	137	94	0.390	81	-	11.4	11.4	7.8	0.5	1.4	2.80	VL-L	SC
														SILTY SANDY GRAVELS															
KA-2	Keflavik	GW-GM	3	32	14	6.0	2.1	1.1	0.1	159	2.7			2.65	138.6 (b)	-	137	98	0.380	70	-	8.7	8.7	2.7	0.3	0.7	2.33	N-VL	SC
LG-27	Loring		3/4	53	10	6.2	4.9	4.4	3.4	15	1.0			2.71	139.1 (b)	-	139	100	0.210	74	-	6.0	11.1	24.6	1.5	3.0	2.00	L-M	SC
LSO-28	Loring		3/4	53	10	6.2	4.9	4.4	3.4	15	1.0			2.71	139.1 (b)	-	133	96	0.273	72	-	7.4	14.9	23.9	1.8	3.5	1.94	L-M	SC
LSO-29	Loring		3/4	53	10	6.2	4.9	4.4	3.4	15	1.0			2.71	139.1 (b)	-	126	91	0.342	75	-	9.5	13.1	20.0	1.5	2.8	1.86	L-M	SC
LSO-30	Loring		3/4	53	10	6.2	4.9	4.4	3.4	15	1.0			2.71	139.1 (b)	-	120	86	0.409	73	-	11.0	13.3	11.4	1.1	1.7	1.54	L	SC
LSO-13	Loring		2	41	9.0	6.4	5.3	4.4	3.4	22	1.3	24.0	5.5	2.71	139.1 (b)	-	135	94	0.256	100	10	9.5	17.7	33.1	2.9	3.8	1.31	M	SC
MP-7	Marble Point	GP-GM	2	56	32	11	3.7	3.0	2.0	101	0.3			2.74	150.8 (b)	-	141	93	0.213	100	-	7.8	12.8	17.0	1.4	2.2	1.57	L-M	T
MP-1	Marble Point		2	38	21	10	3.9	-	-	185	5.7			2.75	145.6 (b)	-	137	94	0.252	100	-	9.2	9.6	3.5	0.3	0.8	2.66	VL	T
MP-5	Marble Point		2	38	21	10	3.9	-	-	185	5.7			2.75	145.6 (b)	-	137	94	0.252	100	-	8.6	11.0	7.7	0.6	1.0	1.66	VL	T
FLJ-14	Project Blue Jay		3/4	54	32	10	4.0	2.2	1.5	139	0.2			2.73	143.4 (b)	-	138	96	0.238	79	-	6.9	24.6	47.4	3.3	5.2	1.58	M-H	SC
														CLAYEY GRAVELS															
LST-31	Loring	GC	3/4	68	52	41	30	25	18	945	0.1	22.1	7.8	2.73	135.8 (d)	7.5	120	88	0.420	97	-	15.1	69.7	134.3	8.0	13.8	1.72	VH	SC
LST-32	Loring		3/4	68	52	41	30	25	18	945	0.1	22.1	7.8	2.73	135.8 (d)	7.5	122	90	0.394	94	-	13.5	58.8	106.5	6.5	10.3	1.58	H-VH	SC
LST-33	Loring		3/4	68	52	41	30	25	18	945	0.1	22.1	7.8	2.73	135.8 (d)	7.5	127	93	0.290	98	-	12.1	56.6	111.3	6.6	10.8	1.64	H-VH	SC
														SANDS AND GRAVELLY SANDS															
PAF-8	Plattsburg	SP	3/8	60	1.0	0.1	<0.1	<0.1	<0.1	3.8	0.9			2.96	126.7 (b)	-	127	100	0.455	80	-	12.3	12.3	1.4	0.1	0.1	1.00	N	SC
														SILTY GRAVELLY SANDS															
HDO-10	Hutchinson's Pit	SW-SM	2	57	20	8.7	5.0	3.5	2.0	43	1.1			2.75	143.3 (c)	5.3	141	98	0.220	71	-	5.7	29.5	61.7	4.3	5.3	1.23	H	T
HDO-11	Hutchinson's Pit		2	57	20	8.7	5.0	3.5	2.0	43	1.1			2.75	143.3 (c)	5.3	140	98	0.231	78	-	6.5	34.1	73.8	4.8	5.8	1.20	H	T
TAFB-2	Thule	SP-SM	3/4	65	41	8.6	2.8	2.0	1.4	35	0.3			2.75	143.7 (b)	-	135	94	0.271	100	1.10	9.8	12.9	10.6	0.8	1.0	1.25	VL	SC
T-4	Tobyhanna		1½	59	39	8.5	4.5	2.5	1.6	6.0	0.2			2.72	140.6 (b)	-	132	94	0.280	100	-	10.0	20.5	24.0	1.4	2.8	2.00	L	SC
														SILTY SANDS															
AFS-3	Alaska Highway	SM	-	100	100	33	2.5	-	-	1.6	1.0			2.79	105.7 (b)	-	104	98	0.672	78	-	18.9	24.0	7.0	0.3	0.5	1.66	N	SC
T-5	Tobyhanna		1½	79	45	14	5.5	4.0	3.1	24	0.7			2.72	140.6 (b)	-	130	92	0.300	100	-	11.1	27.2	33.7	2.6	5.5	2.12	M-H	SC
DFSB-1	Dow		3/4	61	27	14	7.8	5.5	3.8	160	2.7	17.6	3.1	2.72	136.7 (b)	-	135	99	0.254	60	-	5.5	35.7	70.5	4.0	5.8	1.45	H	SC
WC-1	Fairchild		3/4	71	34	23	11	6.3	4.0	95	2.2	21.6	2.9	2.79	144.4 (b)	-	134	93	0.287	100	-	10.3	30.3	56.0	3.0	5.2	1.73	M-H	SC
BM-1	Ball Mountain		3/4	88	58	28	12	7.5	3.6	36	1.2			2.77	141.8 (d)	5.6	133	94	0.300	100	0.054	10.8	38.5	77.3	6.1	7.2	1.18	M	SC
BM-2	Ball Mountain		3/4	88	58	28	12	7.5	3.6	36	1.2			2.77	141.8 (d)	5.6	132	94	0.307	100	0.062	11.1	30.4	45.6	5.3	7.2	1.36	H	SC
HF-1	Hill Field		-	100	95	28	13	10	7.5	17	4.3			2.64	120.4 (d)	-	113	94	0.460	95	1.40	15.6	26.2	16.8	1.9	2.7	1.42	L-M	SC
FLJ-7	Project Blue Jay		3/4	70	54	31	19	12	8.5	147	0.4	16.0	3.7	2.70	137.3 (d)	7.5	136	99	0.238	73	0.0036	6.4	14.8	17.7	1.6	2.7	1.68	L-H	SC
PBJ-8	Project Blue Jay		3/4	70	54	31	19	12	8.5	147	0.4	16.0	3.7	2.70	137.3 (d)	7.5	132	96	0.275	68	-	6.9	26.9	37.4	3.0	5.8	1.93	M-H	SC
TD-6	Truax		3/4	92	79	35	22	15	1.9	55	1.9	14.4	1.6	2.72	137.3 (d)	5.6	129	94	0.315	94	0.0027	10.9	23.2	28.2	3.3	4.2	1.27	M-H	SC
TD-7	Truax		3/4	92	79	35	22	15	1.9	55	1.9	14.4	1.6	2.72	137.3 (d)	5.6	119	87	0.423	91	0.085	14.3	15.5	7.4	1.1	1.7	1.54	L	SC
TD-8	Truax		3/4	92	79	35	22	15	1.9	55	1.9	14.4	1.6	2.72	137.3 (d)	5.6	126	91	0.350	99	0.076	11.7	17.0	13.0	2.0	3.0	1.50	M	SC
TD-33	Truax		3/4	92	79	35	22	15	1.9	55	1.9	14.4	1.6	2.72	137.3 (d)	5.6	126	91	0.348	100	0.2	12.9	27.0	41.2	2.8	3.5	1.25	M	SC
TD-34	Truax		3/4	92	79	35	22	15	1.9	55	1.9	14.4	1.6	2.72	137.3 (d)	5.6	118	86	0.431	100	2.[illegible]	15.9	24.7	20.2	1.4	2.0	1.42	L	SC

Table E-1b (cont'd).

Specimen Number	Material Source	SOIL GRADATION DATA (As Frozen)										PHYSICAL PROPERTIES OF BASIC SOIL					SPECIMEN DATA (As Molded)					FREEZING TEST DATA							Type of Cyl. (12)
		Unified Soil Classification Symbol (2)	Maximum Size	Percent finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity	Compaction Data (5)		Dry Unit Weight	Degree of Compaction	Void Ratio	G, at Start of Test (6)	Permeability k (7)	Avg. Water Content		Total Heave (8)	Rate of Heave mm/day (9)		Heave Rate Ver. Index (10)	Frost Susc. Class (11)	
			in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI		Maximum Dry Unit Weight pcf	Optimum Moisture Content %	pcf	%	%	%	cm/sec $\times 10^{-4}$	Before Test %	After Test %	%	Avg.	Max.			
												GRAVELLY AND SANDY CLAYS																	
EBT-1	East Boston	CL	3/4	86	72	56	43	35	25	-		23	7	2.76	130.8(d)	-	110	84	0.565	100	0.13	20.5	76.9	109.1	7.7	10.7	1.38	H-VH	SC
EBT-2	East Boston		3/4	86	72	56	43	35	25	-	-	23	7	2.76	130.8(d)	-	120	91	0.435	100	0.005	15.8	122.2	145.0	9.8	12.7	1.30	VH	SC
EBT-19	East Boston		3/4	86	72	56	43	35	25	-	-	23	7	2.76	130.8(d)	-	110	84	0.565	100	0.13	20.5	65.8	42.2	6.8	10.2	1.50	H-VH	SC
EBT-20	East Boston		3/4	86	72	56	43	35	25	-	-	23	7	2.76	130.8(d)	-	120	91	0.435	100	0.005	15.8	84.1	101.8	8.2	12.2	1.48	VH	SC
EBT-23	East Boston		3/4	86	72	56	43	35	25	-	-	23	7	2.76	130.8(d)	-	120	91	0.430	87	0.004	13.6	49.5	45.7	2.4	3.2	1.33	M	SC
EBT-24	East Boston		3/4	86	72	56	43	35	25	-	-	23	7	2.76	130.8(d)	-	110	84	0.561	88	0.11	17.8	47.4	27.4	1.9	2.8	1.47	L-M	SC
PAFB-1*	Portsmouth		-	100	100	92	46	36	30	-	-	30.0	11.7	2.73	110.3(e)	17.3	96	-	0.772	98	-	27.7	62.2	85.3	4.2	7.0	1.66	H	SC
PAFB-2	Portsmouth		-	100	100	92	46	36	30	-	-	30.0	11.7	2.73	110.3(e)	17.3	95	-	0.798	95	-	27.7	73.3	114.9	4.1	8.0	1.95	H	SC
EBT-26	East Boston		3/4	86	72	56	49	42	30	-	-	21	7	2.76	130.8(d)	-	120	91	0.433	100	0.06	15.7	60.4	96.2	4.1	9.8	2.39	H-VH	SC
EBT-27	East Boston		3/4	86	72	56	49	42	30	-	-	21	7	2.76	130.8(d)	-	110	84	0.561	100	0.30	20.3	56.0	70.0	2.6	7.3	2.80	M-H	SC
ART-1	AASHO		1½	95	88	75	58	49	37	-	-	27.3	11.9	2.74	121.0(a)	13.5	110	92	0.553	100	0.0215	20.3	90.2	156.8	7.2	11.3	1.56	H-VH	T
												LEAN CLAYS																	
PBW-2	Greenland	CL	-	100	100	97	60	43	34	-	-	36.5	16.8	2.78	119.4(c)	15.0	92	77	0.930	99	-	31.3	52.8	41.3	2.9	5.3	1.82	M-H	T
WF-10	Volk Field		-	100	100	93	77	70	58	-	-	45.0	24.4	2.75	-	-	101	-	0.683	100	-	24.8	28.5	19.3	1.0	1.5	1.50	L	T
SC-1*	Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77	-	-	99	-	0.742	96	-	25.6	74.1	122.2	8.6	12.8	1.48	VH	SC
SC-8*	Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77	-	-	99	-	0.753	100	-	27.2	69.0	131.3	4.7	6.7	1.43	M	SC
SC-6	Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77	-	-	96	-	0.804	93	-	27.0	48.7	47.2	2.1	3.8	1.80	M	SC
SC-7*	Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77	-	-	96	-	0.808	94	0.0005	27.3	127.3	140.3	8.4	14.0	1.66	VH	SC
SC-9*	Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77	-	-	98	-	0.755	98	0.0004	27.3	88.5	155.2	6.2	7.7	1.24	M	SC
SC-10	Searsport		-	100	100	100	80	69	49	-	-	36.5	17.9	2.77	-	-	98	-	0.755	98	-	26.8	47.5	38.6	2.5	3.7	1.48	M	SC
BC-3*	Boston Blue Clay		-	100	100	100	84	74	63	-	-	43.3	21.6	2.72	106.2(e)	20.2	82	-	1.083	94	-	37.3	60.4	38.4	4.6	5.3	1.15	H	SC
BC-8*	Boston Blue Clay		-	100	100	100	84	74	63	-	-	43.3	21.6	2.72	106.2(e)	20.2	79	-	1.162	100	-	42.5	107.6	141.8	12.6	17.8	1.41	VH	SC
DFC-6*	Dow		-	100	100	100	89	75	57	-	-	33.8	16.4	2.79	117.0(d)	-	100	85	0.739	87	-	23.0	115.4	173.4	15.4	21.2	1.38	VH	SC
DFC-7*	Dow		-	100	100	100	89	75	57	-	-	33.8	16.4	2.79	117.0(d)	-	103	88	0.684	94	-	23.0	109.2	188.8	19.8	22.8	1.15	VH	SC
DFC-8*	Dow		-	100	100	100	89	75	57	-	-	33.6	16.4	2.79	117.0(d)	-	105	90	0.660	92	-	21.8	54.3	67.7	8.6	11.0	1.28	VH	SC
DFC-9*	Dow		-	100	100	100	89	75	57	-	-	33.8	16.4	2.79	117.0(d)	-	102	87	0.706	93	-	23.4	87.3	127.8	13.3	17.8	1.34	VH	SC
BC-10*	Boston Blue Clay		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72	106.2(e)	20.2	80	-	1.197	97	-	41.3	124.7	83.3	8.1	11.2	1.38	VH	SC
BC-11*	Boston Blue Clay		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72	106.2(e)	20.2	80	-	1.166	98	-	41.2	124.2	130.8	9.5	15.7	1.65	VH	SC
BC-12*	Boston Blue Clay		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72	106.2(e)	20.2	78	-	1.245	98	-	43.2	96.5	78.1	8.9	12.0	1.34	VH	SC
BC-13*	Boston Blue Clay		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72	106.2(e)	20.2	80	-	1.200	100	-	42.7	93.1	84.7	7.9	11.7	1.48	H-VH	SC
												LEAN CLAYS WITH ORGANICS																	
WASHO-8	Malad, Idaho	CL-OL	-	100	99	96	65	48	35	-	-	36.9	13.3	2.58	99.6(a)	21.0	92	92	0.745	100	-	28.9	53.3	63.3	5.4	6.8	1.26	H	SC
WASHO-9	Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58	99.6(a)	21.0	90	90	0.790	100	-	30.6	56.0	58.6	5.1	6.3	1.24	H	SC
WASHO-27	Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58	99.6(a)	21.0	80	80	1.012	100	-	39.7	94.4	110.7	6.0	9.5	1.58	H-VH	SC
WASHO-28	Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58	99.6(a)	21.0	84	84	0.913	99	-	35.7	78.6	90.5	5.2	8.7	1.67	H-VH	SC
WASHO-29	Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58	99.6(a)	21.0	88	88	0.828	100	-	32.4	99.1	116.1	5.8	9.2	1.58	H-VH	SC
WASHO-30	Malad, Idaho		-	100	99	96	65	48	35	-	-	36.9	13.3	2.58	99.6(a)	21.0	90	90	0.788	100	-	30.3	101.6	129.9	6.5	9.7	1.49	H-VH	SC
												FAT CLAYS																	
WF-9	Volk Field	CH	-	100	98	78	68	65	59	-	-	55.5	38.0	2.76	-	-	108	-	0.592	100	-	21.3	21.3	5.8	0.4	0.5	1.25	M	T
BC-20*	Boston Blue Clay		-	100	100	100	94	88	81	-	-	52.7	26.1	2.78	106.2 (e)	20.2	85	<95	1.031	97	-	36.1	101.8	111.8	4.1	8.3	2.02	H-VH	SC
BC-23	Boston Blue Clay		-	100	100	100	94	88	81	-	-	52.7	26.1	2.78	106.2 (e)	20.2	87	<95	0.989	100	-	35.3	61.2	58.9	2.4	4.8	2.00	M-H	SC
NF-5*	Niagara		-	100	100	100	94	92	86	-	-	59.8	37.0	2.79	-	-	95	88	0.835	95	-	29.8	43.7	43.9	2.4	3.0	1.25	M	SC
NF-3	Niagara		-	100	100	100	96	95	91	-	-	60.0	37.4	2.79	-	-	93	86	0.874	100	-	31.4	41.6	35.7	1.5	2.3	1.53	L-M	SC
NF-4	Niagara		-	100	100	100	96	95	91	-	-	60.0	37.4	2.79	-	-	94	87	0.845	100	-	30.4	-	36.8	1.5	2.8	1.86	L-M	SC
												FAT CLAYS WITH ORGANICS																	
FA(C)-7	Fargo	CH-OL	-	100	100	98	86	76	64	-	-	67.8	45.8	2.76	-	-	89	<95	0.988	100	-	35.7	44.5	18.4	1.0	2.0	2.00	L	SC
FA(C)-8	Fargo		-	100	100	98	86	76	64	-	-	67.8	45.8	2.76	-	-	89	<95	0.988	100	-	35.7	48.0	24.0	1.5	2.0	1.33	L	SC

* Undisturbed

Table E-1b (cont'd). Open system, nominal surcharge 0.5 psi (soils do not meet compaction criterion of 95% or greater and do not have the 85% or greater initial degree of saturation).

		SOIL GRADATION DATA (As Frozen)										PHYSICAL PROPERTIES OF BASIC SOIL					SPECIMEN DATA (As Molded)					FREEZING TEST DATA							
Specimen Number	Material Source	Unified Soil Classification Symbol (2)	Maximum Size	Percent finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity	Compaction Data (5)		Dry Unit Weight	Degree of Compaction	Void Ratio	G, at Start of Test (6)	Permeability k (7)	Avg Water Content		Total Heave (8)	Rate of Heave mm/day (9)		Heave Rate Vs. Index (10)	Frost Susc. Class (11)	Type of Cyl (12)
			in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI		Maximum Dry Unit Weight pcf	Optimum Moisture Content %	pcf	%	%	%	cm/sec $\times 10^{-4}$	Before Test %	After Test %	%	Avg	Max			
CLAYEY SILTY SANDS																													
VWB-1	Fairchild	SM-SC	3/4	76	29	17	9.5	7.0	4.5	55	7.2	24.6	6.3	2.77	142.1 (b)	-	131	92	0.314	94	-	10.7	22.7	29.0	3.2	5.7	1.78	M-H	SC
LST-1	Loring		1½	83	60	47	34	27	20	320	0.3	21.1	6.0	2.71	135.8 (d)	7.5	123	91	0.369	100	0.0022	13.5	78.8	159.4	15.4	21.3	1.38	H	SC
CLAYEY SANDS																													
PA-1	Pierre	SC	1½	67	31	17	8.7	7.0	4.3	100	3.0	25.3	7.3	2.72	134.5 (d)	6.9	123	91	0.381	100	1.30	14.0	16.5	9.7	0.6	0.7	1.16	VL	SC
FA-5	Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70	127.2 (d)	9.0	113	89	0.494	87	0.36	15.9	40.5	52.6	5.0	7.8	1.56	H	SC
FA-6	Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70	127.2 (d)	9.0	117	92	0.438	89	0.19	14.4	37.6	40.8	3.5	5.5	1.57	M-H	SC
FA-7	Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70	127.2 (d)	9.0	103	81	0.641	99	-	25.1	74.9	60.0	2.9	5.0	1.72	M-H	SC
FA-8	Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70	127.2 (d)	9.0	107	84	0.581	100	-	23.1	33.8	28.8	1.9	2.8	1.47	L-M	T
FA-9	Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70	127.2 (d)	9.0	108	85	0.560	100	-	20.8	60.2	65.0	3.9	6.2	1.58	M-H	T
FA-10	Fargo		3/4	98	33	17	9.5	7.5	5.5	50	5.2	30.7	10.5	2.70	127.2 (d)	9.0	112	88	0.507	100	-	18.8	41.9	49.0	3.0	4.3	1.43	M-H	T
PBJ-2	Project Blue Jay		3/4	73	55	35	23	20	15	500	1.7	24.7	8.1	2.73	133.1 (d)	9.4	128	96	0.334	83	-	10.0	44.4	77.1	5.1	9.2	1.30	H-VH	SC
LA-1	Lowry		-	100	86	39	25	21	17	150	6.9	24.5	7.8	2.64	121.0 (d)	-	112	92	0.468	100	0.44	17.7	34.3	37.8	2.7	4.3	1.59	M-H	SC
LA-5	Lowry		-	100	86	39	25	21	17	150	6.9	24.5	7.8	2.64	121.0 (d)	-	111	91	0.491	100	0.30	18.6	38.1	42.8	3.2	4.0	1.25	M	SC
LA-6	Lowry		-	100	86	39	25	21	17	150	6.9	24.5	7.8	2.64	121.0 (d)	-	112	92	0.467	98	0.22	17.4	27.4	32.1	2.9	3.8	1.31	M	SC
LA-2	Lowry		-	100	90	44	32	28	22	150	1.5	24.5	7.8	2.64	121.0 (d)	-	112	92	0.472	100	0.24	17.8	57.1	103.3	5.8	8.0	1.38	H	SC
SILTS AND SANDY SILTS																													
NH-1	New Hampshire	ML	-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7 (c)	16.5	90	85	0.872	100	0.78	32.3	72.0	60.4	8.3	12.8	1.54	VH	SC
NH-2	New Hampshire		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7 (c)	16.5	95	89	0.773	100	0.42	28.5	63.7	68.8	9.3	11.7	1.26	VH	SC
NH-3	New Hampshire		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7 (c)	16.5	98	92	0.712	100	0.29	26.0	123.2	72.7	4.2	12.7	2.04	H-VH	SC
NH-9	New Hampshire		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7 (c)	16.5	95	89	0.781	100	0.42	26.8	166.6	105.6	11.4	15.7	1.38	VH	SC
NH-10	New Hampshire		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7 (c)	16.5	97	91	0.742	100	0.35	27.4	185.4	144.4	15.9	19.0	1.19	VH	SC
CLAYEY SILTS																													
DFT-4	Dow Field	ML-CL	3/4	88	76	66	40	30	20	-	-	22	0.9	2.71	127.6 (d)	-	119	93	0.418	100	-	15.4	87.1	155.4	11.4	16.3	1.42	H	SC
LST-4	Loring		3/4	84	70	59	44	35	27	-	-	21.1	6.0	2.70	133.8 (d)	8.3	112	84	0.506	99	0.090	18.5	78.0	164.4	13.1	19.3	1.47	VH	SC
LST-2*	Loring		1	90	73	61	48	40	30	-	-	21.1	6.0	2.70	133.8 (d)	8.3	113	85	0.502	81	-	15.0	47.1	82.1	7.4	15.0	2.82	H-VH	SC
NH-32	New Hampshire		-	100	96	90	67	36	16	-	-	24.8	5.1	2.70	106.7 (c)	16.5	100	94	0.685	100	0.040	25.4	166.3	262.2	12.3	16.5	1.34	VH	SC
NH-35	New Hampshire		-	100	96	90	67	36	16	-	-	24.8	5.1	2.70	106.7 (c)	16.5	99	93	0.702	100	0.043	26.0	103.3	139.3	13.3	20.5	1.54	VH	SC
NH-36	New Hampshire		-	100	96	90	67	36	16	-	-	24.8	5.1	2.70	106.7 (c)	16.5	100	94	0.685	100	0.040	25.3	95.8	119.1	11.5	17.0	1.48	VH	SC
NH-98	New Hampshire		-	100	97	93	67	39	26	-	-	26.5	6.0	2.71	109.9 (d)	15.6	105	96	0.605	70	-	15.7	164.6	275.5	27.6	36.0	1.30	VH	T
NH-99	New Hampshire		-	100	97	93	67	39	26	-	-	26.5	6.0	2.71	109.9 (d)	15.6	105	96	0.605	82	-	18.2	138.9	221.7	22.7	28.8	1.26	VH	T
NH-100	New Hampshire		-	100	97	93	67	39	26	-	-	26.5	6.0	2.71	109.9 (d)	15.6	106	96	0.600	61	-	13.4	161.3	275.8	26.2	33.7	1.28	VH	T
NH-101	New Hampshire		-	100	97	93	67	39	26	-	-	26.5	6.0	2.71	109.9 (d)	15.6	104	94	0.631	100	-	23.3	142.1	226.4	24.7	31.3	1.26	VH	T
SILTS WITH ORGANICS																													
LFT-25	Fairbanks	ML-OL	-	100	100	95	32	16	10	-	-	28.4	4.4	2.72	112.5 (d)	15.7	85	75	1.000	100	-	36.6	34.4	2.9	0.5	1.0	2.00	VL	SL
LFT-26	Fairbanks		-	100	100	95	32	16	10	-	-	28.4	4.4	2.72	112.5 (d)	15.7	90	80	0.890	100	-	32.6	34.6	7.9	0.7	1.5	2.14	VL-L	SL
LFT-27	Fairbanks		-	100	100	95	32	16	10	-	-	28.4	4.4	2.72	112.5 (d)	15.7	98	87	0.740	100	-	26.9	29.2	12.4	0.5	1.7	3.40	VL-L	SL
LF-1	Ladd Field		-	100	100	91	38	13	6.0	-	-	31.6	0	2.75	101.6 (d)	18.1	84	83	1.040	98	2.1	37.1	38.4	7.8	0.6	1.5	2.50	VL-L	SC
LF-2	Ladd Field		-	100	100	91	38	13	6.0	-	-	31.6	0	2.75	101.6 (d)	18.1	90	89	0.829	97	1.2	31.6	35.8	11.2	0.6	1.0	1.66	VL	SC
LF-3	Ladd Field		-	100	100	91	38	13	6.0	-	-	31.6	0	2.75	101.6(d)	18.1	94	93	0.511	99	0.9	29.4	39.8	25.5	1.8	2.0	1.11	L	SC
LFT-10*	Fairbanks		-	100	100	94	40	23	13	-	-	25.8	3.8	2.67	107.4(d)	17.1	94	88	0.702	96	-	25.0	65.5	124.0	4.5	8.7	1.93	H-VH	SC
LFT-19	Fairbanks		-	100	100	94	40	23	13	-	-	25.8	3.8	2.67	107.4(d)	17.1	98	91	0.703	100	-	26.2	65.8	81.8	7.4	8.7	1.76	H-VH	SC
LFT-20	Fairbanks		-	100	100	94	40	23	13	-	-	25.8	3.8	2.67	107.4(d)	17.1	97	91	0.717	100	-	26.8	82.1	102.1	8.0	9.7	1.21	H	SC
LFT-4	Fairbanks		-	100	100	97	42	22	12	-	-	25.8	3.8	2.67	108.5(d)	14.8	99	91	0.695	86	-	22.4	30.1	10.4	0.7	1.2	1.71	VL-L	SC

*Undisturbed

Table E-1c. Open system, nominal surcharge pressure 0.073 psi.

Specimen Number	Material Source	SOIL GRADATION DATA (As Frozen)										PHYSICAL PROPERTIES OF BASIC SOIL					SPECIMEN DATA (As Molded)					FREEZING TEST DATA							Type of Cyl (12)
		Unified Soil Classification Symbol (2)	Maximum Size	Percent finer, mm						Coefficients (3)		Atterberg Limits (4)		Specific Gravity	Compaction Data (5)		Dry Unit Weight	Degree of Compaction	Void Ratio	G, at Start of Test (6)	Permeability k (7)	Avg Water Content		Total Heave (8)	Rate of Heave mm/day (9)		Heave Rate Var. Index (10)	Frost Susc. Class (11)	
			in.	4.76	0.42	0.074	0.02	0.01	0.005	C_u	C_c	LL	PI		Maximum Dry Unit Weight pcf	Optimum Moisture Content %	pcf	%	%	%	cm/sec x10⁻⁴	Before Test %	After Test %	%	Avg	Max			
GRAVELS and SANDY GRAVELS																													
ADG-1	Alaska Highway	GW	2	40	10	3.7	1.9	1.5	0.9	22	1.6			2.64	133.4 (b)	-	132	99	0.249	100	-	9.4	11.6	1.9	0.9	1.3	1.45	VL-L	SC
SILTY SANDY GRAVELS																													
AMS-1	Alaska Highway	GP-GM	2	27	10	5.2	3.1	2.0	1.2	40	4.7	38.6	2.7	2.73	123.6 (b)	-	121	98	0.401	100	-	14.7	18.3	17.6	1.1	2.5	2.27	L-M	SC
AMS-2	Alaska Highway	GW-GM	2	44	16	7.2	5.4	3.8	2.4	67	2.2	38.6	2.7	2.73	118.5 (b)	-	121	102	0.401	100	-	10.6	20.8	17.6	2.4	3.8	1.65	M	SC
ACR-1	Alaska Highway	GP-GM	2	34	18	11	6.2	4.2	2.7	440	6.6	25.7	3.6	2.72	127.0 (b)	-	126	99	0.336	77	-	9.5	20.8	30.5	1.9	3.7	1.95	L-M	SC
ACR-2	Alaska Highway	GP-GM	2	37	20	12	8.5	6.5	5.1	310	3.1	25.7	3.6	2.70	126.7 (b)	-	128	101	0.315	94	-	11.0	19.6	29.7	1.9	3.3	1.74	L-M	SC
SILTY GRAVELS																													
BM-6	Ball Mountain Till	GM	2	91	35	18	7	-	-	250	0.3			2.81	-	-	147	-	0.195	100	-	[illegible]	11.7	17.4	1.4	3.8	2.71	L-M	T
SANDS and GRAVELLY SANDS																													
ADG-2	Alaska Highway	SW	2	53	13	3.8	1.8	1.4	0.9	20	1.1			2.65	132.9 (b)	-	129	97	0.277	100	-	10.5	12.2	10.2	1.0	1.7	1.70	L	SC
SILTY SANDS																													
AFS-1	Alaska Highway	SM	-	100	100	33	2.5	-	-	1.6	1.0			2.79	106.4 (b)	-	112	105	0.551	92	-	18.2	32.8	20.0	2.0	3.0	1.50	M	SC
AFS-2	Alaska Highway		-	100	100	33	2.5	-	-	1.6	1.0			2.79	106.4 (b)	-	111	105	0.565	100	-	20.3	29.3	11.1	1.1	1.7	1.54	L	SC
CLAYEY SILTY SANDS																													
LST-6	Limestone Till	SM-SC	3/4	84	65	49.7	36	30	21	225	1.0	21.1	6.0	2.72	133.8 (d)	8.3	133	99	0.279	100	-	10.2	17.1	24.7	1.4	2.7	1.93	L-M	SC
SILTS and SANDY SILTS																													
VIS-3	Valparaiso, Indiana Silt	ML	-	100	100	99	54	25	15	-	-	23.7	4.0	2.72	115.8 (d)	13.5	112	96	-	72	.025	13.5	53.1	81.4	6.8	11.0	1.62	H-VH	SC
VIS-7			-	100	100	99	54	25	15	-	-	23.7	4.0	2.72	115.8 (d)	13.5	112	96	-	94	.026	17.7	45.2	142.3	5.8	11.5	1.98	H-VH	SC
NH-13	New Hampshire Silt		-	100	99	97	60	22	10	-	-	26.6	0.1	2.70	106.7 (c)	16.5	105	99	0.609	100	0.15	22.5	105.8	155.1	11.7	17.8	1.52	VH	SC
SILTS w/ORGANICS																													
LF-10	Ladd Field Silt	ML-OL	-	100	100	91	38	13	6.0	-	-	31.6	0.2	2.75	101.6 (d)	16.1	99	92	0.724	100	0.61	26.4	68.1	93.2	7.1	9.5	1.34	H-VH	SC
LFT-9	Fairbanks Silt		-	100	100	97	42	22	12	-	-	32.6	6.2	2.67	107.4 (c)	17.1	102	95	0.602	100	-	24.8	61.0	55.7	5.5	11.3	2.05	H-VH	SC
GRAVELLY and SANDY CLAYS																													
ELT-13	East Boston Till	CL	3/4	84	72	56	43	35	25	-	-	23.0	7.0	2.76	130.8 (d)	-	125	96	0.380	100	.0012	13.8	63.9	130.1	11.5	14.0	1.28	VH	SC
ART-7	AASHO Road Test		$1\frac{1}{2}$	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	116	96	0.481	100	.0063	17.6	31.2	34.9	3.1	3.3	1.06	M	T
ART-13	AASHO Road Test		$1\frac{1}{2}$	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	114	94	0.497	100	.0084	17.2	29.0	31.4	3.8	4.3	1.03	M-H	T
ART-19	AASHO Road Test		$1\frac{1}{2}$	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	122	105	0.414	100	.0020	15.3	43.4	72.7	2.5	3.7	1.48	M	T
YS-3	Yukon Silt		-	100	100	100	67	37	29	-	-	28.0	8.6	2.74	121.4 (d)	12.8	120	99	0.443	91	$8.7x10^{-7}$	15.3	26.2	33.1	1.6	2.9	1.75	L-M	SL
YS-10	Yukon Silt		-	100	100	100	67	37	29	-	-	28.0	8.6	2.74	121.4 (d)	12.8	118	97	0.775	99	$1.57x10^{-6}$	15.1	27.2	24.3	4.2	4.5	1.07	M	SL

Notes for Tables E-1a through c.

1. The data reported in these tables pertain to specimens frozen in the laboratory under conditions which include the following:

   a. Degree of saturation before freezing equal to or greater than 85 percent.

   b. Molded dry unit weight equal to or greater than 95 percent of the applicable maximum standard.

   c. Rate of penetration of the 32°F isotherm approximately 1/4 to 1/2 inch/day.

   d. Surcharge pressure:

      Table E1a - 0.5 psi
      Table E1b - 0.5 psi
      Table E1c - 0.073 psi (1/4-inch steel plate only)

   e. Height of molded specimen approximately 6 inches.

   f. Free water supply at base of specimen (water maintained at approximately 38°F).

   The specimens are listed in order of increasing percentage of grains finer than 0.02 millimeters within each soil classification group.

2. MS MIL-STD-619 B.

3. Gradation coefficients (for reference - see note 2):

$$C_u = \text{coefficient of uniformity} = \frac{D_{60}}{D_{10}}$$

$$C = \text{coefficient of curvature} = \frac{(D_{30})^2}{(D_{60})\ (D_{10})}$$

4. Atterberg limits tests performed on material passing the U.S. standard no. 40 sieve. If no limits are shown, material is non-plastic. LL = liquid limit, PI = plasticity index.

5. The maximum dry unit weight and the optimum moisture content are shown for the natural soil of each specimen. The type of compaction test used in each case is indicated by the letter in parentheses listed alongside the maximum dry unit weight.

   a. AASHTO T99-74 method A, "Moisture-Density of Soils Using a 5.5 lb Rammer and a 12 in Drop."

   b. Providence vibrated density test.

   c. AASHTO T180-57 method D, "Moisture-Density Relations of Soils Using a 10 lb Rammer and an 18 in Drop."

   d. AASHTO T180-57 method A, "Moisture-Density Relations of Soils Using a 10 lb Rammer and an 18 in Drop."

   e. Harvard miniature compaction test.

6. Degree of saturation in percent at start of freezing test. Remolded specimens allowed to drain for 24 hours just prior to freezing.

7. Permeability tested with de-aired water under falling head and corrected to 10°C. Values reported are for corresponding specimen void ratios.

8. Based on the original height of the frozen portion.

9. Rate of heave - the average rate of heave in millimeters per day, determined from a representative portion of the plot of heave versus time, in which the slope is relatively constant and during which the penetration of the 32°F isotherm is relatively linear and between 1/4 and 1/2 inch/day. Rate of heave is averaged over as much of the heave versus time plot as practicable, but the minimum number of consecutive days used for a determination is five. Maximum rate - the average of the three highest, not necessarily consecutive, daily heave rates.

10. Heave rate variability index - maximum heave rate/average heave rate.

11. The following tentative scales of average and maximum rates of heave have been adopted for rates of freezing between 1/4 and 1/2 inch/day.

Rate of heave millimeters/day	Relative frost-susceptibility classification	
0 - 0.5	Negligible	N
0.5 - 1.0	Very low	VL
1.0 - 2.0	Low	L
2.0 - 4.0	Medium	M
4.0 - 8.0	High	H
> 8.0	Very high	VH

12. Symbols indicate different types of specimen containers used during the studies:

    SC - Straight-wall, waxed cardboard
    SM - Straight-wall, Micarta
    SL - Straight-wall, acrylic
    S-TR - Straight-wall, Transite pipe
    T - Inside tapered, acrylic

13. The specimens listed in supplementary table E-1b do not fulfill requirements given under notes 1a and b above, otherwise all other notes apply.

14. The specimens listed in table E-1c have been tested under a surcharge pressure of 0.073 psi, and may or may not fulfill 1a and b, otherwise all other notes apply.

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. The types of containers used in these tests are indicated in the last column of tables E-1a and b.

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 E-1a through c, 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 2-2e presents a summary grouping of the individual envelopes shown in figures 2-2a-d. 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.

APPENDIX F

REFERENCES

Government publications

Department of Defense

Military Standards

MIL-STD-619 B	Unified Soil Classification System for Roads, Airfields, Embankments and Foundations

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TM 5-820-2/AFM 88-5, Chap. 2	Subsurface Drainage Facilities for Airfield Pavements
TM 5-822-4/AFM 88-7, Chap. 4	Soil Stabilization for Roads and Streets
TM 5-822-5/AFM 88-7, Chap. 3	Flexible Pavements for Roads, Streets, Walks and Open Storage Areas
TM 5-822-6/AFM 88-7, Chap. 1	Rigid Pavements for Roads, Streets, Walks and Open Storage Areas
TM 5-822-8/AFM 88-6, Chap. 9	Bituminous Pavements, Standard Practice
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TM 5-825-2/AFM 88-6, Chap. 2	Flexible Pavement Design for Airfields
TM 5-852-6/AFM 88-19, Chap. 6	Calculation Methods for Determination of Depths of Freeze and Thaw in Soils

Transportation Research Board, National Academy of Sciences

2101 Constitution Ave., N.W., Washington, D.C. 20418

Record 442	Determination of Realistic Cut-off Dates for Late-Season Construction with Lime-Flyash and Lime-Cement-Flyash Mixtures
Record 612	Evaluation of Freeze-Thaw Durability of Stabilized Materials
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P.O. Box 631, Vicksburg, MS 39180

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D-560-(R 1976)	Freezing and Thawing Tests of Compacted Soil-Cement Mixtures
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