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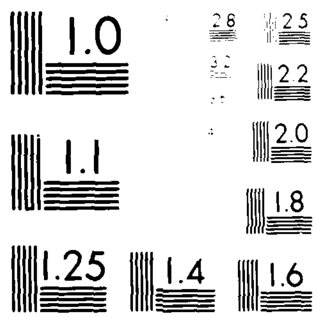
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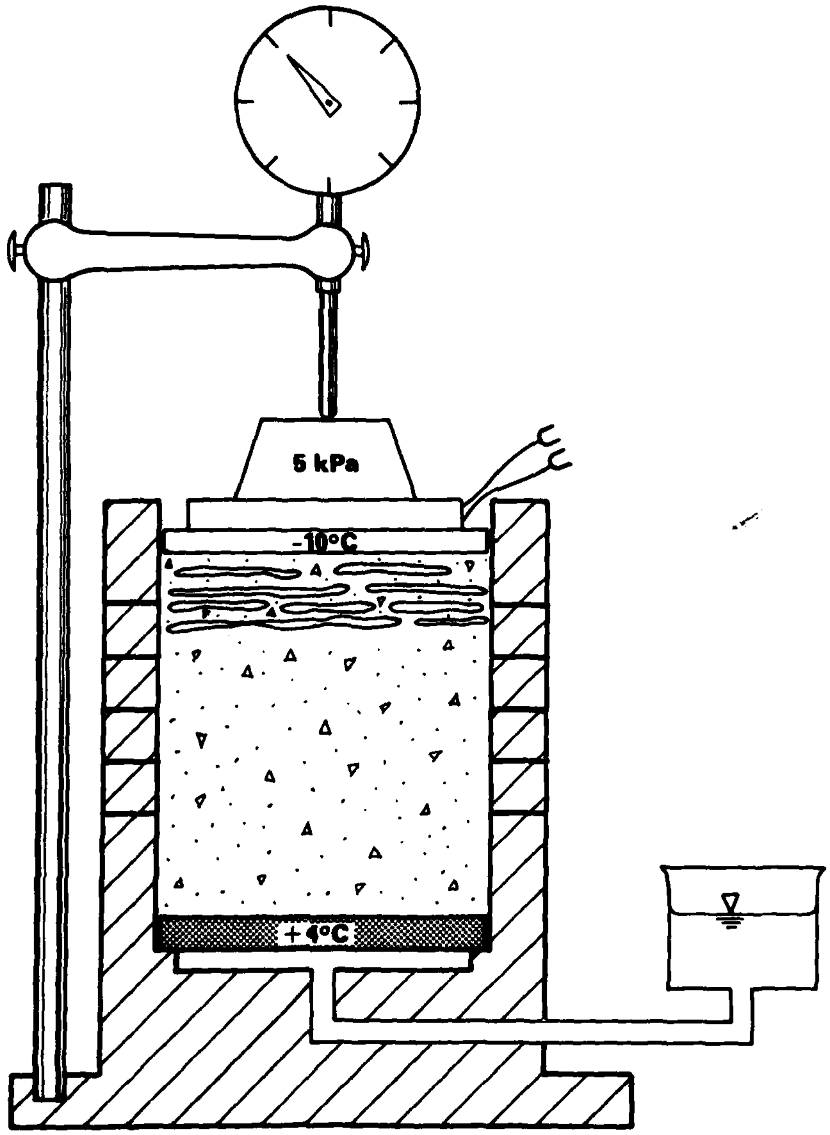
Frost susceptibility of soil

Review of index tests



CRREL Monograph 81-2

Frost susceptibility of soil

Review of index tests

Edwin J. Chamberlain

December 1981

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Methods of determining the frost susceptibility of soils are identified and presented in this report. More than one hundred criteria were found, the most common based on particle size characteristics. These particle size criteria are frequently augmented by information such as grain size distribution, uniformity coefficients and Atterberg limits. Information on permeability, mineralogy and soil classification has also been used. More complex methods requiring pore size distribution, moisture-tension, hydraulic-conductivity, heave-stress, and frost-heave tests have also been proposed. However, none has proven to be the universal test for determining the frost susceptibility of soils. Based on this survey, four methods are proposed for further study. They are the U.S. Army Corps of Engineers Frost Susceptibility Classification System, the moisture-tension hydraulic-conductivity test, a new frost-heave test, and the CBR-after-thaw test.

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PREFACE

The *CRREL Monograph* series is a continuation of the *Cold Regions Science and Engineering* series published by CRREL between 1961 and 1975.

This review was conducted by Edwin J. Chamberlain, Jr., Research Civil Engineer, Applied Research Branch, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory. Funding was provided by the Federal Highway Administration and the Federal Aviation Administration under Order No. FHWA 8-3-0187, *Full-Scale Field Tests to Evaluate Frost Action Predictive Techniques*, and by DA Project 4A762730AT42, *Design, Construction and Operations Technology for Cold Regions*, Task A2, *Soils and Foundations Technology in Cold Regions*, Work Unit 002, *Seasonal Changes in Strength and Stiffness of Soils and Base Courses*, and Task A3, *Facilities Technology for Cold Regions*, Work Unit 006, *Volume Change Induced by Freezing and Thawing of Pavement Systems*.

Members of the Board of Consultants reviewing this study are Dr. B.J. Dempsey, University of Illinois; Dr. D.G. Fredlund, University of Saskatchewan; Dr. M.E. Harr, Purdue University; Edward Penner, National Research Council of Canada; and Dr. M.W. Witczak, University of Maryland. The author thanks Richard McGaw for his early contributions to this review and Thaddeus Johnson and Dr. Richard Berg for reviewing the monograph.

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METRIC CONVERSION FACTORS

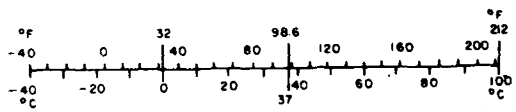
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



FROST SUSCEPTIBILITY OF SOIL
Review of Index Tests

Edwin J. Chamberlain

INTRODUCTION

The search for a reliable method to evaluate the frost susceptibility of soils has gone on for at least the past 50 years. More than 100 methods have been proposed since Taber's treatise (1929) on the mechanism of ice segregation in soils and Casagrande's conclusions (1931) that "under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than three percent of grains smaller than 0.02 mm, and in very uniform soils containing more than 10 percent smaller than 0.02 mm." Even though there has been almost continuous research on frost heave since then, Casagrande's criteria are still the most successful for predicting the frost susceptibility of soils, in spite of the probability that he never intended that they be universally applied.

The abundance of methods for determining the frost susceptibility of soils is evidence of the lack of success in developing a comprehensive method. Obviously each has been developed because others have proven to be unsatisfactory. In many cases the new criteria have been successful for specific but limited purposes. In most cases, however, there is little evidence as to the degree of success, i.e. most new criteria receive little scientific field validation.

The sponsors of this study seek a relatively simple index test for frost susceptibility (in contrast to a more comprehensive frost heave test or mathematical model of the frost heave process). It is important, though, that all methods for evaluating frost action in soil be considered in the same context so that comparative judgments can be made of their utility. Accordingly this report will cover any method that holds promise for indicating the frost susceptibility of soils.

It may be that no single method can be comprehensive enough. However, it is the purpose of this report to evaluate the available methods of determining the frost susceptibility of soil and then to select for further analysis a few that appear to be the most reliable. The survey ranges from the early work of Taber (1929), Casagrande (1931), Beskow (1935) and Ducker (1939) to methods reported up to January 1981. Although an attempt was made to identify all the index test methods developed during this period, some may have been missed. The most serious omissions may be from the eastern European and Asian nations because of the difficulty in gaining access to their literature.

It is important to explain frost susceptibility before discussing the index tests. From this basis the various tests may be assessed according to how they address the basic elements affecting the frost susceptibility of soils.

FROST SUSCEPTIBILITY AND ITS RELATION TO FROST HEAVING AND THAW WEAKENING

The freezing of frost-susceptible soil (with water available) normally involves opposing actions: the downward advance of the freezing front and upward frost heave. Heaving is the result of ice segregation during the freezing process. The advance of the freezing front causes alternating bands of soil and ice to form. The external manifestation is frost heave. This structure may or may not be visible to the unaided eye. When the ice melts, the aggregates of soil particles usually can not reabsorb all the water immediately after thawing. Consequently soils are frequently weaker after thawing than before freezing. With time and proper drainage the initial strength usually returns.

Frost heave is not necessary for thaw weakening. For instance, it is known that some clay soils develop segregated ice (and hence thaw weakening) while exhibiting little or no heave (Cook 1963, Titov 1965). The shrinkage of compressible soil aggregates cancels the heave normally associated with ice segregation, particularly where the water supply is restricted and the permeability is low.

It is apparent, then, that two major phenomena result from freezing and thawing: frost heaving and thaw weakening. Both can cause considerable damage to engineering structures, the former during freezing and the latter during thawing. Both seem to be major indicators of frost-susceptible soils. However, for decades there has been an almost universal tendency to define frost susceptibility in terms of frost heaving alone, i.e. a frost-susceptible soil was one which heaved when frozen.

The definition given by the Highway Research Board Committee on Frost Heave and Frost Action in Soil (1955) focuses more on processes within the soil than on external effects. It states, "A frost-susceptible soil is one in which significant ice segregation will occur when the requisite moisture and freezing conditions are present." This has remained one of the most widely accepted definitions. Here the basis of frost susceptibility is seen to be "significant ice segregation," a process occurring within the soil. This is a step ahead of previous definitions, which had relied on the external effects of freezing.

However, this statement is only partially complete, as ice segregation and frost susceptibility were associated solely with detrimental heaving until very recently. Today the effects of thaw weakening can in many cases be of greater practical significance than frost heaving. Thaw weakening continues to gain importance, as the lack of clean, granular material makes it necessary to use marginal soils or recycle existing materials. Even so, the assumption persists in many quarters that heaving must occur before thaw weakening can take place.

It is important, then, that both kinds of frost damage (heaving and weakening) be addressed in any frost susceptibility criteria. Both are important in evaluating soil materials for use in road and runway foundations, as are bearing capacity and settlement in the design of foundations. Like bearing capacity and settlement, frost heaving and thaw weakening have been treated as though they were unrelated. Some link should be developed between these two damaging results of frost action. Realistically, until we are successful in reliably determining the susceptibility of soil to frost heave and thaw weakening separately, it is fruitless to attempt to combine the two in a single scheme.

For the purpose of this discussion, then, frost heave susceptibility is equated with heave during freezing, and thaw weakening susceptibility with the loss of strength after thawing. It follows that frost susceptibility (FS) simply reflects the combined effects of frost heave susceptibility and thaw weakening susceptibility.

To select index tests for FS we first need to know the material properties and freezing conditions involved. Any index test must then be related to one or more of these factors.

REQUISITE CONDITIONS FOR FROST HEAVE

Frost heave is generally attributed to the formation of ice lenses during freezing. For this to happen, it is generally agreed that 1) subfreezing temperatures, 2) water and 3) a frost-susceptible soil must be present. With all of these factors present the degree of FS may vary with the rate of heat removal, the temperature gradient, the mobility of the water, the depth to the water table, the overburden stress, the soil density and texture and so on.

To understand the effect of these factors on frost heave, it is helpful to understand the mechanics of frost heave and to review some experimental observations of frost heave.

MECHANICS OF FROST HEAVE

The classic works of Taber (1929) and Beskow (1935) on the migration of water to a growing ice lens stood until the 1950's as the most serious attempts to identify the mechanism of frost heaving. Taber attributed the migration to "molecular cohesion" and identified the factors controlling ice segregation as soil particle size, amount of water available, size of voids and void ratio, and rate of cooling. Beskow related the suction pressure to "capillary rise" and showed the relationships of the height of capillary rise to grain size and depth to the water table. Neither of these explanations provided a rigorous theory for frost heave.

However, in the past two decades three fundamentally different explanations for ice segregation and frost heave have received considerable attention. They are the so-called capillary theory, secondary heaving theory and segregation freezing theory. Until recently the first two appeared to be in harmony, the capillary rise theory being applied to granular soils and the secondary heaving theory to clay soils. The segregation freezing theory, however, has always been at odds with the others. Although the theories disagree about the mechanism of frost heave, they are in general agreement on the factors affecting frost heave. A brief examination of these theories should help to demonstrate their differences and determine the material properties and freezing conditions important to frost heave. No attempt will be made to judge the merits of these theories.

Capillary theory

Frost heave occurs as a result of ice segregation. The capillary theory says that the heave pressure and the suction pressures that develop during the formation of ice lenses are related to the porous matrix of the soil.

Penner (1957) and Gold (1957) observed that the magnitude of the suction was related to the geometry of the porous soil matrix in which ice lenses develop. Penner concluded that moisture tensions develop as a result of freezing point depressions and that higher tensions develop in soils with small pores than in soils with large pores because the freezing point decreases with the radius of curvature of the ice/water interface.

Miller et al. (1960) concluded that when the radius of curvature of the ice/water interface is taken into account, equilibrium thermodynamics could be used to predict the relationship between the freezing point and the suction pressure.

Penner (1959) also tried to understand ice segregation in this way. These studies led to the work of Everett (1961) and Everett and Haynes (1965), who finally developed a rigorous equilibrium thermodynamics formula for ice growth in porous materials.

The resulting relationship, which has often been referred to as the capillary rise model for ice segregation, takes the form

$$p_{\mathrm{i}} = p_{\mathrm{u}} + \frac{2\sigma_{\mathrm{i,w}}}{r_{\mathrm{i,w}}} \tag{1}$$

where p_{i} = steady state heaving pressure at the base of the ice lens (pore ice pressure)

p_{u} = pore water pressure (pore water tension)

$\sigma_{\mathrm{i,w}}$ = surface tension at an ice/water interface

$r_{\mathrm{i,w}}$ = radius of the ice/water interface.

It is assumed that adsorption forces are negligible and that the soil is an ideal granular material. For determining the maximum heaving pressure, $r_{\mathrm{i,w}}$ becomes the radius of the pore necks through which the ice must grow (Fig. 1).

This relationship has been verified experimentally by Penner (1966) for uniform glass spheres in a close-pack array. However, for soils which commonly have a range of particle sizes, choosing a representative value of $r_{\mathrm{i,w}}$ can be a problem. Although Penner (1973) found that heaving pressures calculated from eq 1 were too large when the average value of $r_{\mathrm{i,w}}$ was used, they agreed well with the measured values when the size of the smallest particles was used.

The rate of heaving for a given soil is a function of the rate of heat extraction at the freezing front, the stress borne by the ice lens, the suction in the pore water, and the hydraulic conductivity in the zone beneath the ice lens. In compressible soils such as clays the rate of heave is also a function of the compressibility of the unfrozen soil beneath the ice lens and the magnitude of the suction pressure generated at the freezing front. The compressibility becomes a factor because of the increase in the effective stress beneath the ice lens.

According to Terzaghi (1936), the effective stress between soil particles can be represented by the following equation:

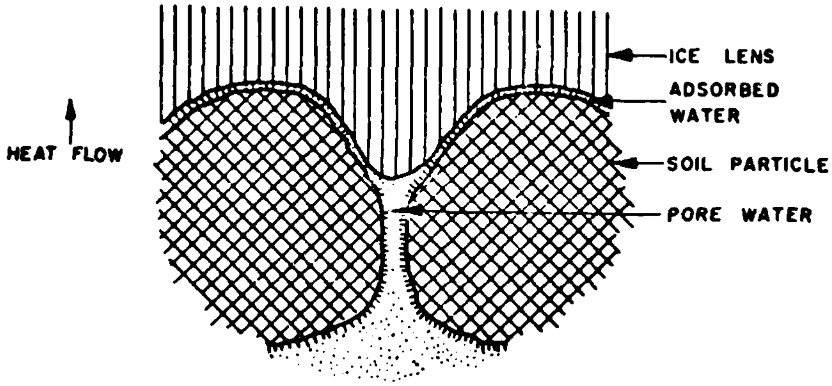


Figure 1. Section of an ice lens with a soil particle and soil pore. (From Penner 1959.)

$$\bar{\sigma} = \sigma - \sigma_n \qquad (2)$$

where $\bar{\sigma}$ = effective intergranular stress
σ = total stress
σ_n = pressure supported by the pore contents (sometimes called the neutral stress).

In saturated soils σ_n is equal to the pore water pressure. In partially saturated soils σ_n is a function of both the pore water pressure and the pore air pressure p_a, usually represented in the form

$$\sigma_n = \chi p_w + (1-\chi)p_a \qquad 0 < \chi < 1. \qquad (3)$$

The partition factor χ rises with increasing degrees of saturation ($\chi = 1$ when the soil is 100% saturated).

Because σ_n is always negative beneath a growing ice lens, the effective stress on the soil beneath is always higher than before freezing. If the soil is practically incompressible under this stress (as are most dense sands), then σ_n has little effect on the soil structure. If, however, the soil is compressible (as are clay soils, for example), then the void ratio decreases as the effective stress increases and the soil becomes more dense. This has two important influences on frost heave. First, a surface manifestation of frost heave may not be apparent, as the increased volume of the segregated ice will be at least partially compensated for by the decrease in volume occupied by the soil beneath the ice lens. The effect is to overconsolidate the soil by freezing. Nixon and Morgenstern (1973), Chamberlain and Blouin (1978) and many others have observed this process.

The second effect of the increase in effective stress on compressible soils is to decrease the pore size and thus increase the maximum values of pore water suction and frost heave stress and change the hydraulic conductivity.

In summary, the capillary theory attributes frost heaving to 1) the rate of heat removal, 2) the pore size, 3) the hydraulic conductivity of the unfrozen soil, 4) the compressibility of unfrozen soil, and 5) the weight of material supported by the ice lens.

Secondary heave theory

Miller (1972) disagreed with the simple capillary theory and introduced the concept of secondary heaving. He was bothered by the discrepancy that was frequently found between the measured and calculated values of heaving pressure using the simple capillary model, and he was not satisfied with Penner's explanations. In 1977 Miller came to the conclusion that the only kind of ice segregation that could occur, according to the simple capillary model (which he termed the primary heaving model), was the formation of needle ice at the soil surface.

Miller has continued to revise his thoughts on secondary heaving. The following is a brief review of his secondary heave theory for saturated, salt-free, non-clay soils taken from papers published at the Frost Action in Soils Symposium in 1977 and the Third International Permafrost Conference in 1978.

Miller contended that secondary frost heave involves the growth of ice into some of the pores formed by stationary soil particles below the ice lens itself. He called this region where the ice front propagates beyond the ice lens the "frozen fringe" (Fig. 2). In this region both ice and liquid water are transported.

The concept of the frozen fringe has also been

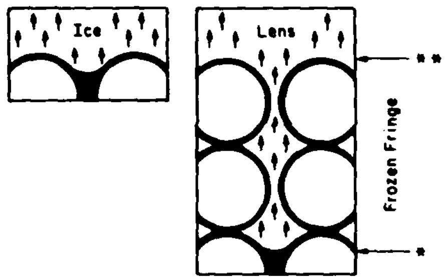


*Figure 2. Primary frost heaving (left) and secondary heaving (right). The shaded area represents liquid water (**leading edge of the ice lens, *leading edge of the frozen fringe). (After Miller 1977.)*

reported by others. For instance, the Soviet scientist Fel'dman (1967) reported experiments that established that moisture migration during freezing takes place not only in the unfrozen soil but also in a "certain zone of freezing soil." Hoekstra (1969) observed a layer adjacent to the ice lens where "ice crystals are present... but the ice phase is discontinuous and does not constitute an ice lens." Miller (1978) reported that E.D. Ershov, another Soviet scientist, agreed with this concept. Penner (1977) and Penner and Walton (1978) also seemed convinced of the frozen fringe concept. Penner (1977), however, appeared to apply this concept only to clay soils.

Loch (197Ja) observed that the frozen fringe was 4–4.5 mm thick in silty clay and clayey silt soils. Phukan-Morgenstern-Shannon (1979) reported that the thickness of the frozen fringe can range from less than a millimeter to several centimeters, depending on soil type, composition, temperature gradients and applied pressure. They also suggest that the moisture migration to the freezing front is controlled entirely by processes that develop in the frozen fringe.

According to Miller, the driving force for frost heave in saturated granular soil is the interaction of pore ice, pore water and temperature and the swelling properties of adsorbed films within the frozen fringe. The Clausius-Clapeyron equation is used to relate the pore water tension p_u and the pore ice pressure p_i to the freezing temperature T of the pore water:

$$p_u/\varrho = p_i/\varrho_i + (L/K)T \qquad (4)$$

where ϱ and ϱ_i = densities of water and ice, respectively
L = latent heat of fusion
K = absolute freezing point of water.

Miller used eq 1 to describe the relationship between the radius of curvature of the ice/water interface in a pore and the pore ice pressure and pore water tension.

Miller related the stresses by the effective stress equation

$$\bar{\sigma} = p + \chi p_u + (1-\chi)p_i \qquad 0 \leq \chi \leq 1 \qquad (5)$$

where p is the total stress on the ice lens. The partition factor χ, relating the contributions of the ice pressure and water tension to the effective stress, equals one at the leading edge of the frozen fringe, where the soil is ice-free, and zero at the base of the growing ice lens, where all non-adsorbed water is frozen.

Pore ice pressure and pore water tension thus vary within the frozen fringe during ice lens growth (Fig. 3). The hydraulic conductivity also varies within the frozen fringe, possibly as illustrated in Figure 4. The thickness of the frozen fringe is governed by the temperature gradient. Increasing the temperature gradient reduces the thickness of the fringe and its impedance to the flow of water. According to Miller (1972), the limiting process in secondary frost heaving is the *transmission of water through the frozen fringe* to the growing ice lens.

For unsaturated granular soils the process is complicated by the air in the voids. Miller has not yet attempted to solve for the case where the maximum pore water tension that can be sustained at the leading edge of the frozen fringe becomes a factor. According to Miller, the temperature gradient in the unfrozen soil controls the pore water tension at this boundary. Obviously the hydraulic conductivity and moisture content of the unfrozen soil are also factors.

Although Miller does not specifically mention it, the principal difference in the analysis of frost heave in granular and clayey soils is compressibility. As in the capillary theory the compressibility of clayey soils complicates the treatment.

Thus, according to the secondary heave theory, frost heaving depends on 1) the rate of heat extraction, 2) the size of the soil pores, 3) the freezing point of the water at the base of the growing ice lens, 4) the hydraulic conductivity of the frozen fringe, 5) the temperature gradient within the frozen fringe, 6) the thickness of the frozen fringe, 7) the in situ moisture tension in the unfrozen soil, 8) the hydraulic conductivity of the unfrozen soil, 9) the compressibility of the unfrozen soil, and 10) the magnitude of the overburden pressure.

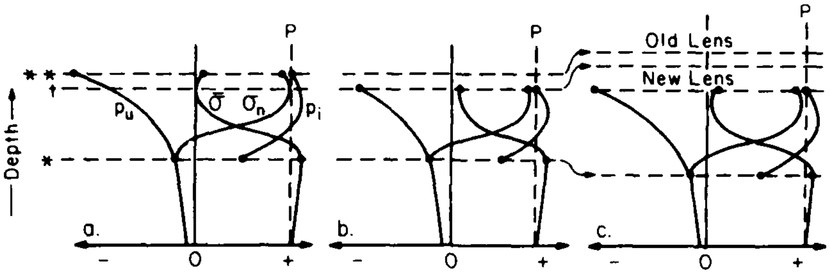


Figure 3. The nature of profiles of pore water pressure p_u, pore ice pressure p_i, neutral stress σ_n and effective stress $\bar{\sigma}$ in a heaving column; a) profiles a moment before a new ice lens is initiated; b) profiles immediately after initiation of a new lens; c) profiles just before initiation of another lens. Stresses and pressures are positive to right of the vertical lines at O and are equal to the overburden pressure at the vertical lines at P. (After Miller 1977.)

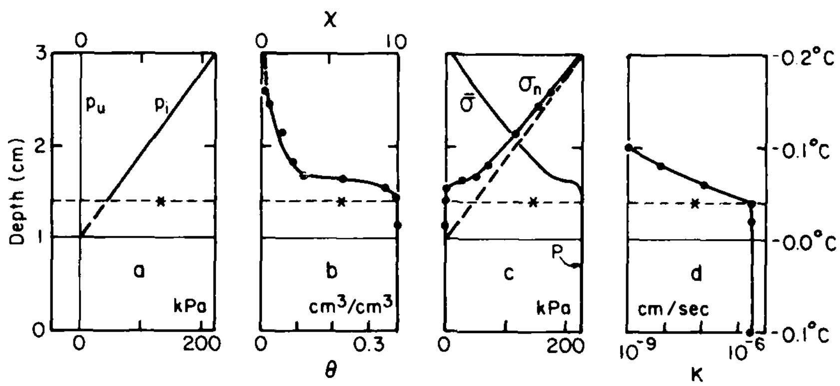


Figure 4. Profiles for a static column; a) pore water pressure p_u, pore ice pressure p_i; b) liquid water content θ and approximate values of χ; c) effective stress $\bar{\sigma}$ and neutral stress σ_n; d) tentative data for k. The base of the frozen fringe is marked with an asterisk. All data are for a 4- to 8-μm silt fraction. (After Koopmans and Miller 1965.)

Adsorption force theory

Another explanation of frost heaving has been proposed by Takagi, the most recent versions being published in 1978 and 1980. He suggested that the primary cause of frost heaving is the creation of a "solid-like stress" in the unfrozen film of water between the ice and soil surfaces. The weight of the ice lens is supported by the film and the soil particle. The heaving stress is determined by the the solid-like stress in the film; it cannot exceed the pressure imposed by the material overlying the growing surface of the ice lens. The heaving stress is also limited by the segregation freezing temperature, which cannot be lower than the freezing point of the film water. Takagi (1980) stated that the decisive factor for determining the freezing point depression, and thus the limit of the heaving pressure, is the specific surface area of the soil particles, as suggested by Anderson and Tice (1972). Takagi has not yet formulated a method for determining this limiting value.

According to Takagi's adsorption force theory, the tension in the pore water is independent of the heaving stress. The origin of the tension is in the film water. The freezing film, in response to the loss of its thickness to the growing ice lens, generates the tension that draws pore water to the region of freezing (Fig. 5). If the uppermost part of the film water separating the soil particles and the ice lens freezes, water must be sucked in from neighboring areas to maintain the thickness of the film. If the soil particles remain stationary and the ice lens continues to grow, then frost heaving occurs. Takagi calls this process "segregation freezing." He has not yet

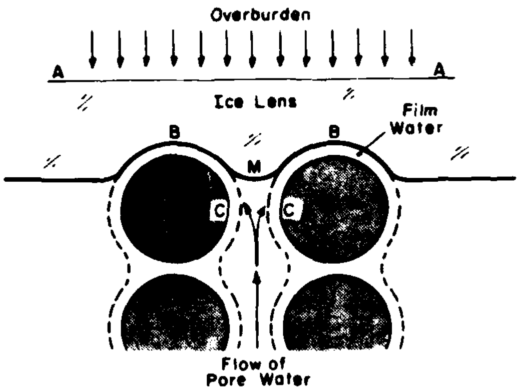


Figure 5. Ice lens forming on the film water. (From Takagi 1979.)

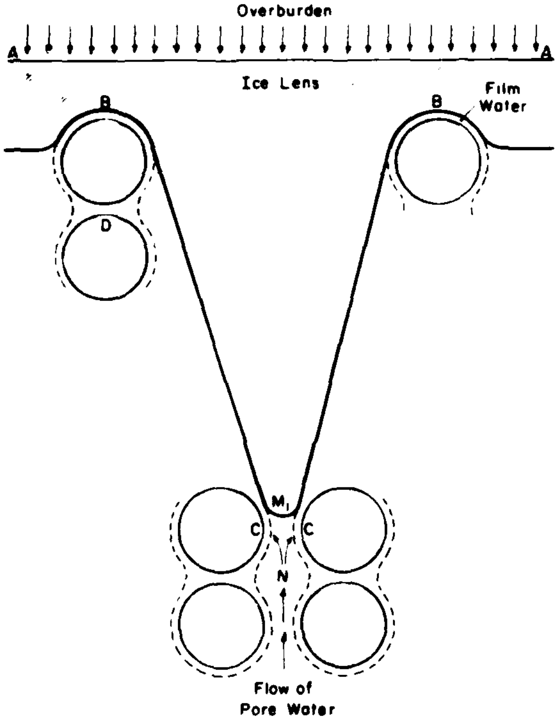


Figure 6. Structure of the diffused freezing zone front. B is the leading edge of the growing ice lens, M is the diffused freezing zone front, CC is the pore restricting pore water flow, D is the location of the next ice lens, and N is the corresponding location of the next diffused freezing zone front. (From Takagi 1979.)

formulated a method for calculating the limit of the stress in the film water; however, he says that the suction results from the tension gradient in the film water (near points B and C in Figure 5).

Takagi believes that there is another zone of freezing, which he calls the "zone of diffused freezing" (Fig. 6). The lower boundary of this zone is the site of in situ freezing, which, according to Takagi, does not contribute to frost heave but does govern the availability of water to the freezing zone. The upper boundary of this region is where the ice lens grows, causing frost heave. Just as in Miller's frozen fringe theory the temperature gradient in the zone of diffused freezing has a significant effect on the rate of water flow.

As in the other theories the rate of heave depends on the rate of heat extraction, the rate of water flow to the growing ice lens, and the compressibility of the unfrozen soil.

Thus, according to the adsorption force theory, frost heaving is affected by 1) the rate of heat removal, 2) the freezing point of the film water, 3) the specific surface area of the soil particles, 4) the hydraulic conductivity of the film water, 5) the thickness of the zone of freezing, 6) the temperature gradient in the zone of freezing, 7) the hydraulic conductivity of the unfrozen soil, 8) the compressibility of the unfrozen soil, and 9) the weight of the material supported by the ice lens.

EXPERIMENTAL OBSERVATIONS OF FACTORS AFFECTING FROST HEAVE

Considerable study has been made of the factors affecting frost heave. A review of the literature revealed that the most important factors are 1) soil texture, 2) pore size, 3) rate of heat removal, 4) temperature gradient, 5) moisture conditions, 6) overburden stress or surcharge, and 7) freeze–thaw cycling.

Soil texture

The most important soil factor affecting frost heave appears to be grain size. Grain size is used as the basis for most FS criteria because it is the most easily measured soil property that has been correlated with frost heave. Soils with no particles smaller than 74 μ simply do not heave under natural conditions. Taber (1929, 1930a) recognized this long ago, and Casagrande (1931) suggested that grain size be used to define the limits of frost-susceptible soils. Lambe (1953) reported that mineralogy is an important factor, particularly for clay particles, as the nature of the exchangeable ion has a pronounced effect on FS. Lambe et al. (1969) reported that clay minerals can both enhance and inhibit frost heave. Concentrations of only 0.1% to 1.0% of montmorillonite fines in a silt caused an increase in frost heave; higher concentrations caused a decrease.

Linell and Kaplar (1959) recognized that the soil texture and material type are the most important factors affecting frost heave and also that they are the most feasible elements to control in highway pavement design for frost regions.

Leary et al. (1968) concluded that the grain size effect is very complex, that only a certain fraction of particle sizes in a soil influences frost heave behavior, and that the amount and activity of the clay-size particles and the uniformity of the gradation of soil particle sizes less than 74 μ are controlling factors. More recently Penner (1976) concluded that soil texture, a measure of particle size gradation, is the single most important physical characteristic of soil for purposes of identifying its FS.

Obviously grain size, mineralogy, uniformity, and texture are only indicators of FS. The question is, what in the frost heaving process do these soil factors affect? From the discussion of the various frost heave theories, it is apparent that the soil factors influence 1) the pore size distribution, 2) the pore water tension, 3) the frost heaving pressures, 4) the hydraulic conductivity in the unfrozen soil, 5) the hydraulic conductivity in the frozen fringe, and 6) the compressibility of the unfrozen soil.

Pore size

The influence of pore size on frost heave was originally suggested by Taber (1930b). Considerable time passed before Penner (1957, 1959) resurrected the idea that pore size was important in interpreting pore water tensions during soil freezing.

Later Csathy and Townsend (1962) reported that "every essential factor in the mechanism of frost action is intimately related to pore size." Jessberger (1969) concluded that "all of the frost-favoring potentials, such as capillarity, suction, and the thermal, electrical and osmotic potentials," depend on pore size. Hoekstra (1969) also showed that a good correlation between pore size and frost heaving pressure may exist.

Rate of heat removal

The effect of the rate of heat removal on frost heave has long been studied. Beskow (1935) concluded from field observations that the rate of heave is independent of the rate of freezing. The U.S. Army Corps of Engineers (USACRREL 1968) arrived at the same conclusion from coldroom studies. Penner (1960), however, came to a different conclusion; he found that "there is a strong influence of net heat flow on heaving rate." Penner (1972) further concluded that "the rate of heat extraction is the basic variable in the frost heave process." Kaplar (1970) concluded that the heaving rate is directly proportional to the heat extraction rate, while Loch (1977) found that the rate of heave did not depend on the rate of heat extraction.

The confusion on this issue began because the early research concentrated on a narrow band of heat extraction rates. Penner's work (1972) revealed that there is a limiting rate of heat extraction below which the rate of heave increases and above which the rate of heave decreases (Fig. 7). More recently this observation has also been made by several other researchers, including Hill

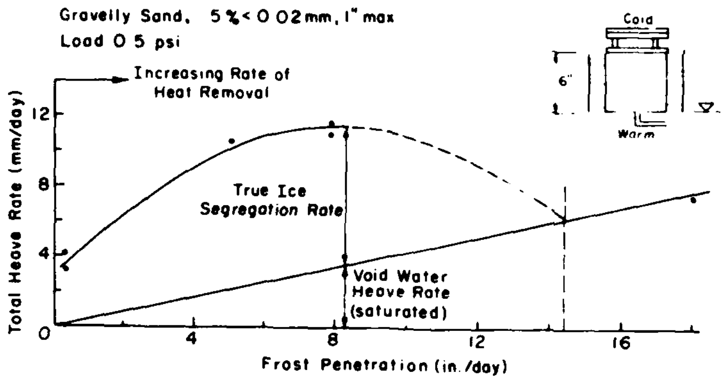


Figure 7. Heave rate versus rate of frost penetration. (After Penner 1972.)

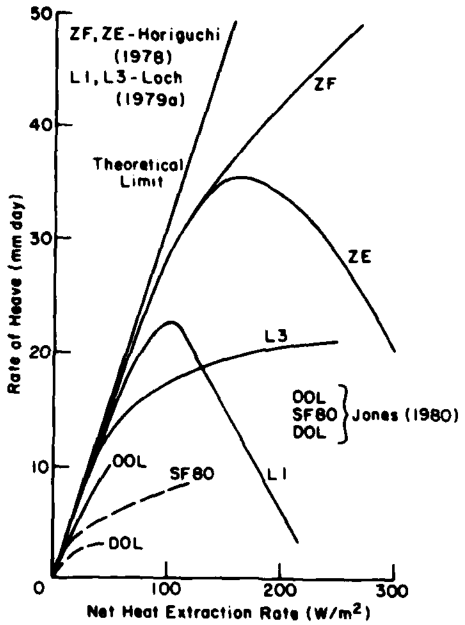


Figure 8. Influence of heat extraction on heaving rate. (After Jones 1980.)

and Morgenstern (1977), Horiguchi (1978), Takashi et al. (1978), Loch (1979a), and Jones (1980). Examples of Horiguchi's, Loch's and Jones's observations are shown in Figure 8. Hill and Morgenstern have referred to the rate of heat flow at which the maximum rate of heave occurs as the "limiting value." Both Penner and Loch have recognized that this limiting value is different for different soils and, therefore, that it is misleading to compare the frost heaves of different soils when the tests are carried out at the same frost penetration rate (i.e. different rates of heat removal).

Because of this, both Penner and Loch concluded that frost heave tests should be conducted at a constant rate of heat removal. Furthermore, they both advised that the rate of heat extraction should be similar to that in the field.

Temperature gradient

The temperature gradient has only recently been recognized as a factor affecting frost heave. Williams (1966), Loch and Kay (1978), and Phukan-Morgenstern-Shannon (1979) have shown that the temperature gradient affects the thickness and hydraulic conductivity of the

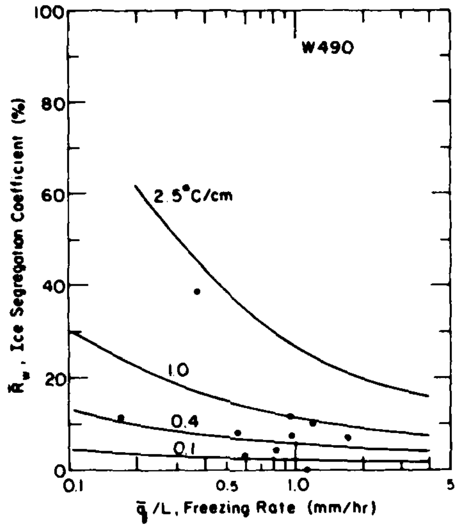


Figure 9. Effect of temperature gradient and freezing rate on frost heave in a sand. (After Gorlé 1980.)

frozen fringe. Most recently Gorlé (1980) showed that frost heave is strongly dependent on the temperature gradient under the ice front, as well as on the rate of heat extraction (Fig. 9). He observed that the temperature gradient had the greatest effect on sands, while it had no significant influence on silts. He concluded that the reproducibility of direct frost heave tests can be improved by expressing the results as functions of both the temperature gradient and the rate of heat removal.

Moisture conditions

It has long been assumed that the moisture condition most likely to produce frost heaving is one where the soil voids are filled with water. That this is a logical and correct assumption can be readily understood if one views the frost heave process as an interaction between the driving forces in the freezing zone and the gravitational and interparticle forces restricting the flow of water. As can be seen in a typical moisture-tension curve for a soil (Fig. 10), the moisture tension is zero at saturation, and as the moisture content decreases, tension increases at a rate that depends on the soil characteristics. For frost heave to occur, the tension generated in the freezing zone must exceed the tension in the unfrozen material (Miller 1977). Furthermore, as the moisture tension increases, the hydraulic conductivity decreases (Ingersoll and Berg 1981), and thus the potential rate of frost heave is lowered. The depth to the water table is important in determining the moisture tension before freezing (and the hydraulic conductivity), and thus it is a major factor in determining the rate and magnitude of frost heave. McGaw (1972), Burns (1977), Kinosita (1978), Loch (1979b), Jones and Berry (1979), Gorlé (1980) and many others have observed that the heave rate decreases as the distance to the water table increases. Burns's observations (1977), for instance, are shown in Figure 11.

Thus, the condition most conducive to heave occurs when the soil is saturated and the water table is at the frost front. If the pore water pressure becomes positive prior to freezing because

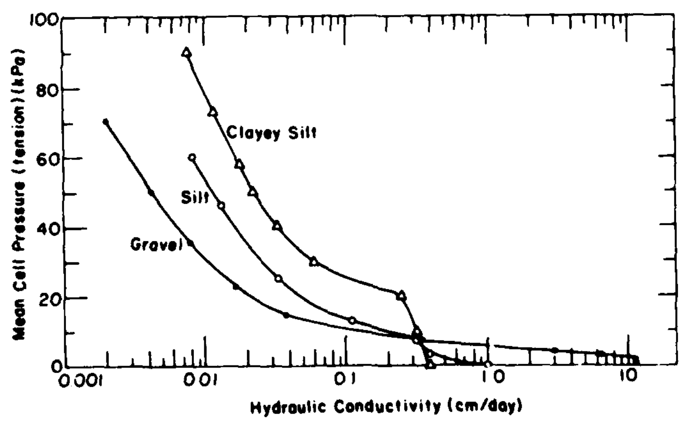


Figure 10. Hydraulic conductivity of three soils. (After Ingersoll and Berg 1981.)

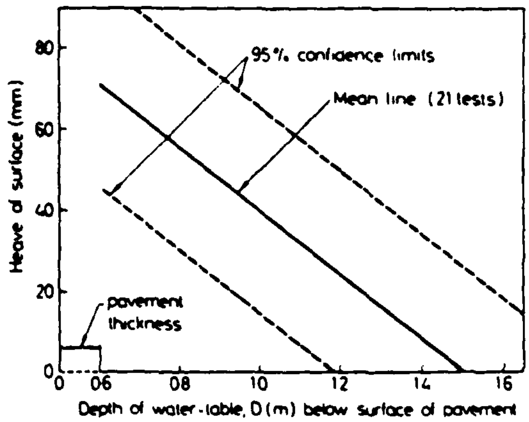


Figure 11. Effect of the depth to the water table on frost heave. (From Burns [1977], reproduced by permission of the Transport and Road Research Laboratory, Crown copyright.)

of a confined seep or aquifier, frost heave is even more severe. This is, however, an unlikely design condition. Thus, it can be concluded that void saturation with a high water table is the most dangerous condition for frost heave.

Overburden stress or surcharge

Long ago Taber (1929) and Beskow (1935) recognized that increasing the applied stress on a freezing soil decreases the heave rate. Linell and Kaplar (1959) found in laboratory tests that the rate of heave for a range of soil types was reduced one order of magnitude by the application of an approximately 40-kPa surcharge. Similar observations were made by Penner and Ueda (1978) (Fig. 12). Aitken (1963, 1974) observed at field test sites that the same surcharge reduced the heave by a factor of only three or four; he attributed the differences from the earlier results

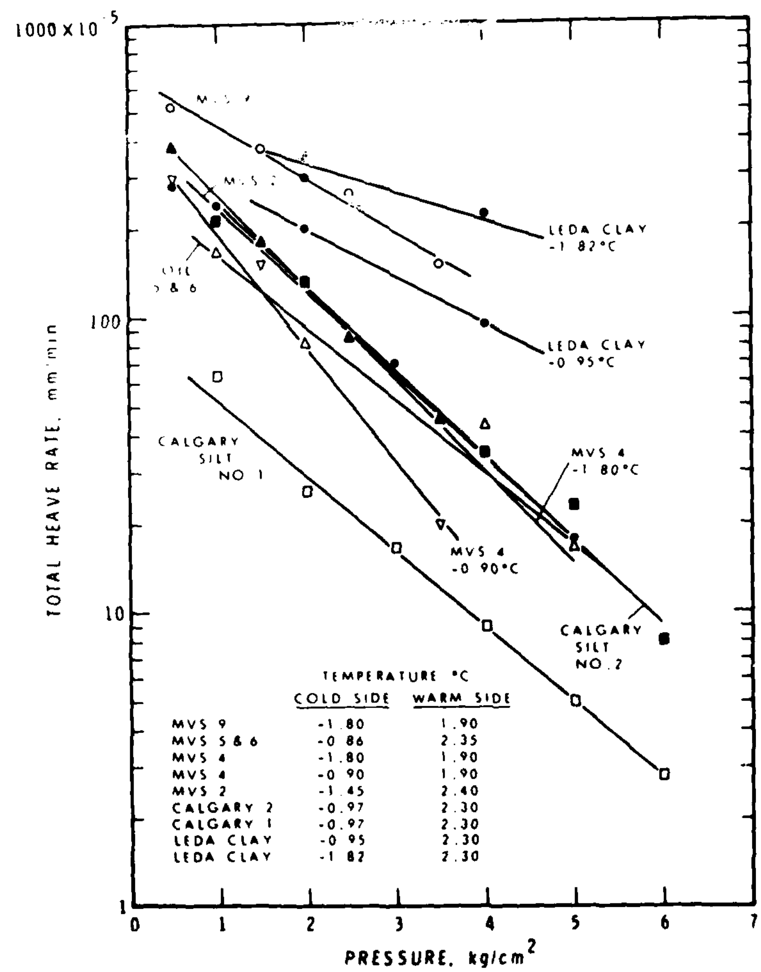


Figure 12. Total frost heave rate vs pressure. (From Penner and Ueda [1978], reproduced by permission of the National Research Council of Canada.)

of Kaplar to the unlimited supply of water in the laboratory tests.

Penner (1958) theorized that there should be a critical pressure for any given pore size at which frost heaving would cease. This theme has been taken up by numerous researchers (e.g. Penner 1960, 1967, 1972; Koopmans and Miller 1966; Hoekstra 1969; Miller 1972; McRoberts and Nixon 1975; Loch and Miller 1975; Osler 1967). Hill and Morgenstern (1977) determined that there is a critical "shut-off pressure" at which moisture transfer to the freezing zone ceases. Penner and Ueda (1977), however, found that no shut-off pressure exists below 465 kPa for sand, silt and clay soils, although marked reductions in frost heave rate were observed.

Repeated freeze-thaw cycling

The occurrence of several freeze-thaw cycles in soil and granular base material during a winter has been widely observed. The effects of freeze-thaw cycling on the FS of soils and granular base materials have, however, been generally ignored. Jessberger and Carbee (1970) recognized this problem and demonstrated in a series of laboratory tests that freeze-thaw cycling caused progressively smaller thaw-CBR values, particularly for clay soils (Fig. 13).

Few observations of the effects of freezing and thawing on frost heave, however, have been reported. At CRREL several unpublished studies have shown that freezing and thawing can greatly affect frost heave. For instance, freeze-thaw cycling was reported (USACRREL 1974) to have increased by a factor of four the frost heave of a till frozen under a surcharge of 14 kPa, most of the increase occurring during the second freeze-thaw cycle (Fig. 14). Under higher surcharges (21 and 100 kPa) little or no effect of freezing and thawing was observed (USACRREL 1974, 1978).

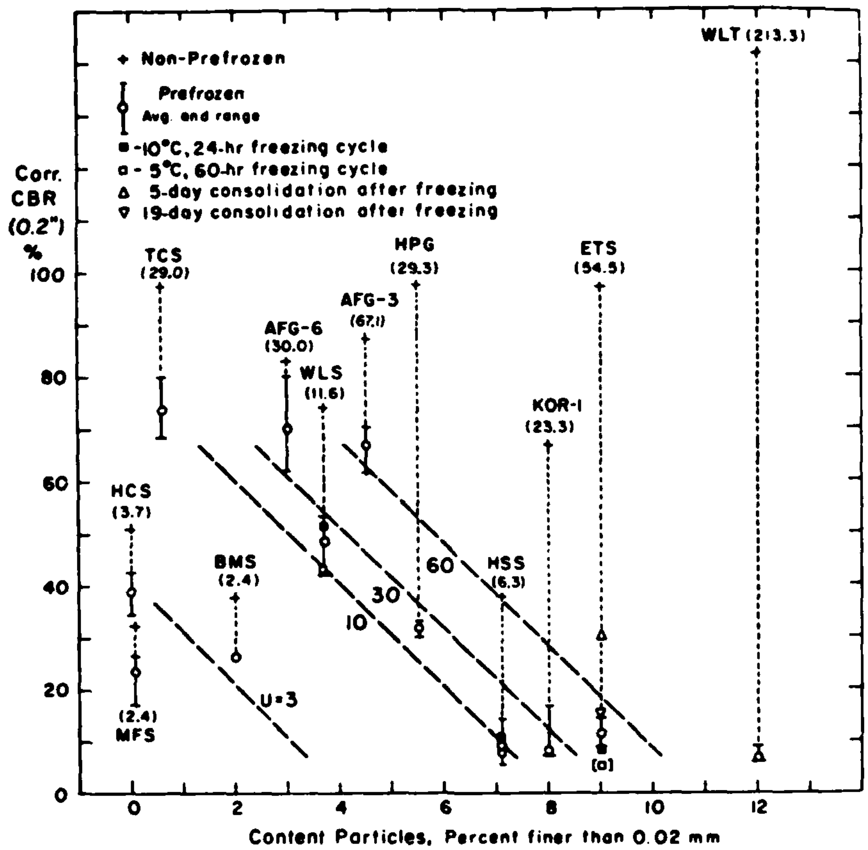


Figure 13. Average CBR values of soils with less than 15% of their particles smaller than 0.02 mm versus content of particles smaller than 0.02 mm. Numbers in parentheses are uniformity coefficients. (After Jessberger and Carbee 1970.)

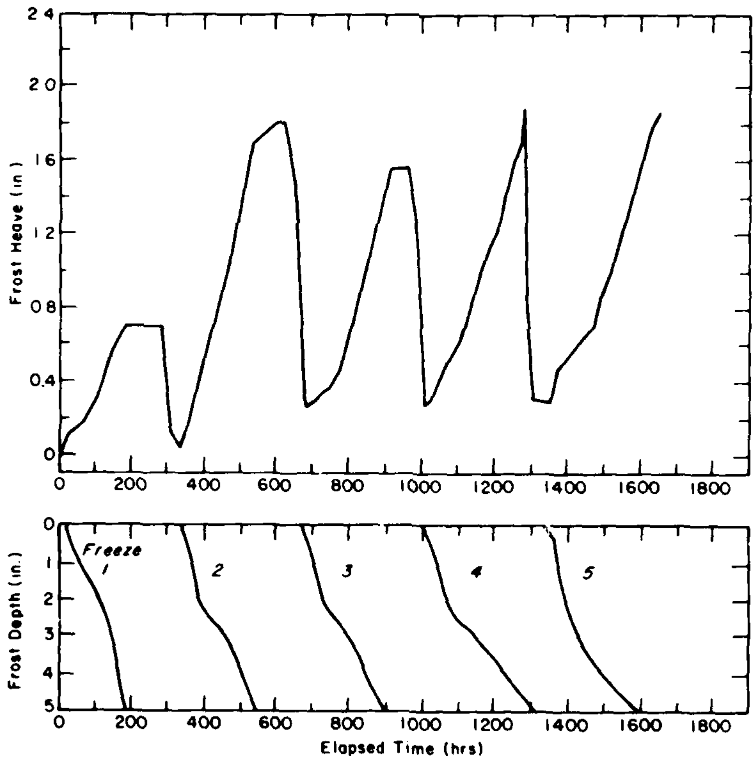


Figure 14. Frost heave and frost penetration vs time for several freeze-thaw cycles on James Bay glacial till (2.0 psi surcharge, 135.9 pcf dry density, 8.5% molding water content, 8.5% testing water content, 97% saturation). (After USACRREL 1974.)

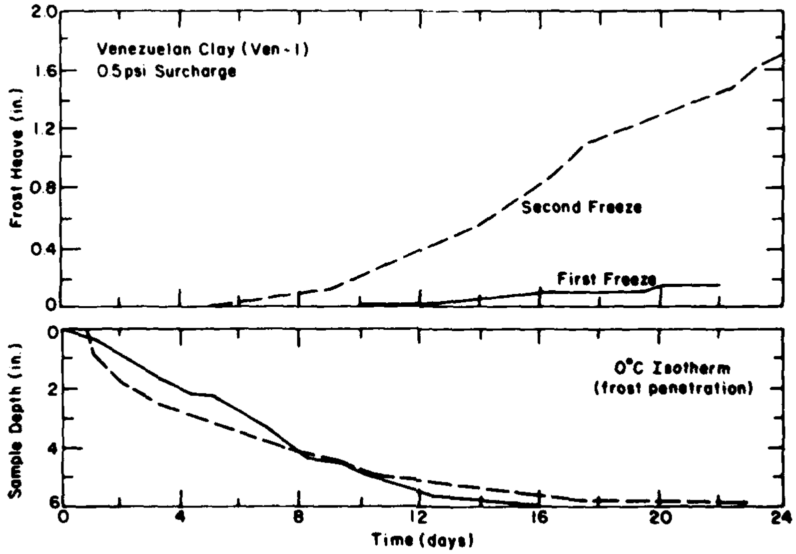


Figure 15. Influence of freezing and thawing on the frost heave of a clay soil. (After USACRREL 1977.)

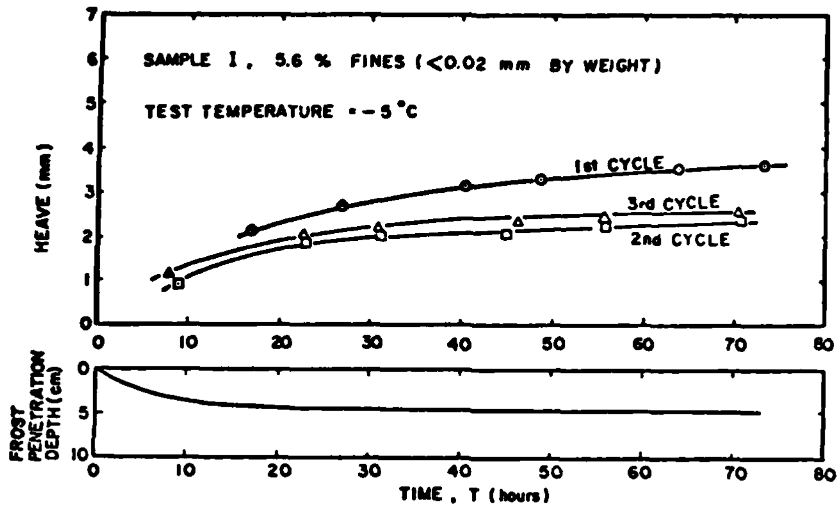


Figure 16. Effect of freezing and thawing on frost heave. (From Sherif et al. [1977], courtesy of Cold Regions Engineers Professional Association.)

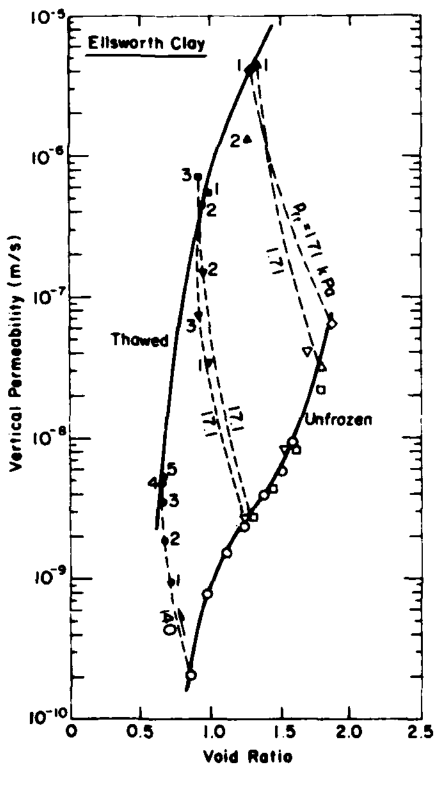


Figure 17. Vertical permeability for Ellsworth clay after freeze-thaw cycling. (From Chamberlain and Gow 1978.)

For a clay soil a second freeze was reported (USACRREL 1977) to have increased the amount of frost heave by a factor of eight (Fig. 15) when the surcharge was 3.5 kPa.

Sherif et al. (1977) reported that the amount of frost heave for a silty sand decreased with freeze-thaw cycling (Fig. 16). They attributed the decrease to the reduction of heave potential and the poorer continuity of the adsorbed water films caused by the loosening and rearranging of particles that occur with successive freeze-thaw cycling.

Chamberlain and Gow (1978) have shown that the freezing and thawing of silt and clay slurries cause an increase in both density and permeability (Fig. 17). This rather incongruous behavior is attributed to particle rearrangement and shrinkage cracking occurring beneath a freezing front because of the increase in effective stress. If the permeability increases because of freezing and thawing, then the rate of frost heave would be expected to increase if other factors remain constant. However, any change in structure that increases permeability would also be expected to change the frost heave potential. This complex interrelated process obviously is not well understood but must be considered, especially when developing a direct frost heave test or relating laboratory tests to field conditions.

TYPES OF FROST SUSCEPTIBILITY TESTS

This survey of FS criteria has covered more than 100 methods in use or proposed for use. Out of these, five fundamentally different methods of determining FS have been identified. They are based on 1) particle size characteristics, 2) pore size characteristics, 3) soil/water interaction, 4) soil/water/ice interaction, and 5) frost heave.

Several reports proved to be particularly valuable in reviewing the literature on FS criteria, including those of Johnson (1952), von Moos (1956), Armstrong and Csathy (1963), Erickson (1963), Sutherland and Gaskin (1963), Townsend and Csathy (1963a, b), Jessberger (1969, 1973, 1976), Cominsky et al. (1972), Gorlé (1973), Obermeier (1973), Johnson et al. (1975), and Christensen and Palmquist (1976).

The more recent review by Jessberger (1976) was especially helpful in identifying a large number of methods, particularly from Europe. This very comprehensive report contains reviews of 31 studies that classify soils as to their degree of FS. An earlier and even more comprehensive report by Jessberger (1969) proved to be nearly as valuable, as did the reports by Townsend and Csathy. The extensive report by Christensen and Palmquist, although not yet translated from Danish, provided information on several European methods of determining FS.

The report by Armstrong and Csathy provided information on methods used in Canada, and the report by Johnson et al. reviewed methods used by the various states in the U.S., as well as some of the more recent methods under development. Obermeier also reviewed some of the more recent developments.

Particle size tests

Classification methods based on particle size are by far the most extensively used tests for determining the FS of soils. The simplest of these tests includes only grain size as the determining factor. The most widely used, the Casagrande (1931) criteria, requires the determination of the percentage of grains finer than 0.02 mm and the uniformity coefficient ($C_u = D_{60}/D_{10}$, where D_{60} and D_{10} equal the particle diameters corresponding to 60% and 10% finer on the grain size distribution curve, respectively).

More complex classification systems, such as the U.S. Army Corps of Engineers (1965) criteria, are related to the Unified Soil Classification System (U.S. Army Engineer Waterways Experiment Station 1957), which requires information about the entire grain size distribution curve and the Atterberg limits (a soil/water interaction test). Others require information on capillary rise and hydroscopicity (Beskow 1935), permeability (Freiberger [in Jessberger 1976], Scheidig 1934, and Koegler et al. 1936), or mineralogy (Brandl 1976, 1979).

A tabulation of soil classification tests for determining frost susceptibility is given in Appendix A. Details on each are listed below by country. Each listing is followed by the reference source and a brief description of the criteria. Where appropriate, the classification is discussed.

Austria

Brandl (1976) developed criteria for determining the FS of coarse-grained base materials in Austria. These criteria are based on the 0.02-mm grain size and the mineral type. The classification is given in Table 1. Brandl (1979, 1980) reported the revised mineral criteria for FS shown in Table 2.

Table 1. Frost susceptibility criteria of Brandl (1976).

Maximum percentage by weight of particles <0.02 mm	*Allowable mineral composition of non-frost-susceptible soils*
3	≥50% chlorite <10% iron hydroxide (crystalline) <5% iron hydroxide (amorphous)
5	1) Non-active Ca-montmorillonite minerals 2) Combinations of (1) and a maximum of a) 10% kaolinite b) 20% chlorite c) 30% biotite mica d) 40% Na-montmorillonite e) 50% muscovite mica f) 70% illite 3) 80-90% kaolinite or chlorite and 10-20% Na-montmorillonite
8	1) Non-active minerals with a maximum of 1% <0.002 mm 2) Quartz and feldspar in dolomite and calcite obtained from quarries and rock slides, for the rock slides, the fine chlorite and muscovite fractions must not exceed 5-8% <0.02 mm, if 10% chlorite, only 5% <0.02 mm

Table 2. Frost susceptibility criteria for gravel of Brandl (1979, 1980).

Maximum percentage of grains < 0.02 mm	*Allowable mineral composition of non-frost-susceptible soils*
3	Non-frost-susceptible, no mineral type determination necessary
5	Normally, if heave properties are known from field or laboratory observations, no mineral type determination is necessary. If frost heave properties are not known, the gravel is non-frost-susceptible if 1) the minerals are inactive or 2) there is a mixture of the inactive minerals and a maximum of a) 10% kaolinite b) 30% chlorite c) 30% vermiculite d) 40% montmorillonite, and/or e) 50% mica, with boundary conditions of a) 60% mica and chlorite b) 50% mica, chlorite and kaolinite c) 50% mica and kaolinite d) 40% mica, chlorite, kaolinite and montmorillonite In addition, up to 40% complex silicate is allowable 3) If evidence of iron hydroxide, frost heave tests are required
8	*Inactive minerals with 1% ≤ 0.002 mm*

Brandl (1980) suggested that because a hydrometer analysis must be conducted to determine the percentage of particles finer than 0.02 mm, the percentage passing the 0.06-mm sieve should be correlated with the percentage finer than 0.02 mm for certain classes of soils. Then determinations of the percentage finer than 0.02 mm can be made from the percentage finer than 0.06 mm, which can be more easily determined by sieve analysis. Brandl also suggested that a modified Proctor compaction test be conducted to determine the amount of particle breakdown during compaction.

Canada

Alberta. In Alberta (Johnson et al. 1975) the U.S. Army Corps of Engineers (1965) grain size distribution criteria are used for subgrade soils with a plasticity index (PI) less than 12. Clays with a PI between 12 and 25 are considered to have medium FS, and clays with PIs greater than 25 have low FS. Base and subbase materials are non-frost-susceptible if less than 10% is finer than 0.074 mm and the PI < 5-6%.

Canadian Department of Transport. When actual measurements are not available, the Canadian Department of Transport (Armstrong and Csathy 1963) uses a zoned particle-size distribution diagram (Fig. 18) in conjunction with information on the pavement and ground water conditions to estimate the probable spring loss in

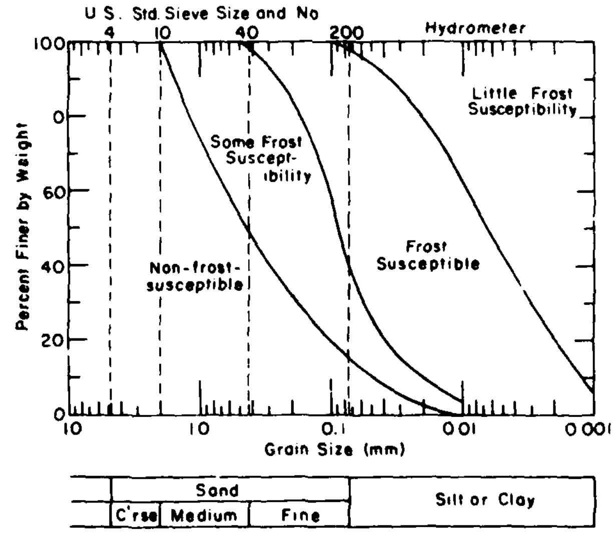


Figure 18. Limits of frost susceptibility according to the Canadian Department of Transport. (After Armstrong and Csathy 1963.)

bearing capacity. The percentages between the curves in Figure 18 are load reduction factors used in their pavement design method.

Canadian National Parks. In the National Parks (Armstrong and Csathy 1963), the Canadian Department of Public Works applies a combination of the criteria of Beskow and Casagrande. They have determined that all silt and clay soils with 36% or more of the particles finer than 0.074 mm are frost susceptible and are not allowable within 3 ft of the pavement. Clay soils with plasticity indexes greater than 11 are also frost susceptible if they lie within 5 ft of the pavement.

Manitoba. Armstrong and Csathy (1963) reported that the province of Manitoba uses a grain size method. Soils with less than 20% clay and greater than 60% silt and sand are classified as frost susceptible. Soils with 20–30% clay may be frost susceptible. No details were given.

New Brunswick. Armstrong and Csathy (1963) reported that in New Brunswick, soils with greater than 50% silt, gravels with 6–8% silt, and clay loams and loam tills with mica in small sizes (>0.074 mm) are classified as frost susceptible.

Newfoundland. Armstrong and Csathy (1963) also reported that Newfoundland uses grain size to determine the FS of granular base courses. The classification is given in Table 3.

Table 3. Newfoundland frost susceptibility criteria.

Frost susceptibility	*Grains >0.074 mm (%)*
None	0-6
Moderate	6-12
High	>12

Nova Scotia. Armstrong and Csathy (1963) reported that Nova Scotia uses the FS classification system given in Table 4.

Table 4. Nova Scotia frost susceptibility criteria.

Frost susceptibility	*Grains >0.074 mm (%)*
None	0-10
Moderate	10-30
High	>30

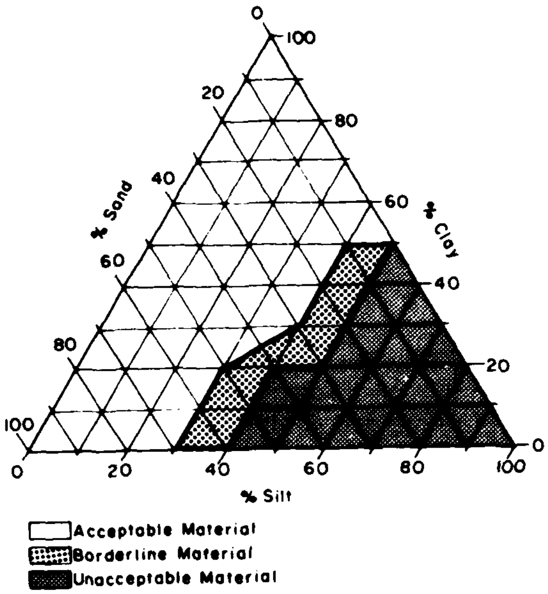


Figure 19. Guide to the frost susceptibility of soils according to the Ontario Department of Highways (1957).

Ontario. Townsend and Csathy (1963a) reported that the Ontario Department of Highways (1957) assesses the FS of soils using a classification based primarily on frost heaving (Table 5). Figure 19 shows this classification on a textural classification chart. More recently Johnson et al. (1975) reported that the Ontario Department of Highways states that soils with 0–8% of the particles smaller than 0.074 mm and a PI of zero are non-frost-susceptible.

Table 5 Ontario frost susceptibility criteria.

Frost susceptibility	*Amount of silt (%)*	*Amount of very fine sand and silt (%)*
None	0-40	0-45
Slight-medium	40-50	45-60
High	50-100	60-100

Quebec. Armstrong and Csathy (1963) reported that the FS criteria in Table 6 are used in the province of Quebec. More recently Johnson et al. (1975) reported that Quebec classifies subgrade soils as frost susceptible when more than 10% of the particles are smaller than 0.074 mm and more than 3% are smaller than 0.053 mm.

Table 6. Quebec frost susceptibility criteria.

Frost susceptibility	*Grains <0.074 mm (%)*	*Amount of silt and fine sand (%)*
None	0-10	0-20
Moderate	10-30	20-40
High	>30	>40

Saskatchewan. According to Johnson et al. (1975), Saskatchewan determines the FS of subgrade soils principally by experience. Base materials with 7-10% of the particles smaller than 0.074 mm are usually considered non-frost-susceptible, as are subbase materials with 0-20% smaller than 0.074 mm.

Denmark

Riis (1948) reported that the Danish State Road Laboratory adopted frost susceptibility criteria (Fig. 20) based on the criteria of Beskow (1935) and Casagrande (1931). Homogeneous soils (moraines) and heterogeneous soils (sediments) are treated separately, the criteria being more severe for heterogeneous soils. Homogeneous and heterogeneous soils are not defined for the Beskow criteria. However, reference is made to a Swedish soil classification system for details. Under Casagrande's method Riis defines homogeneous soils as those having a uniformity coefficient C_u less than 5 and heterogeneous soils as those having a C_u greater than 5. Riis reported that the methods of Beskow and Casagrande are in general agreement, however, for borderline materials the Casagrande method is more stringent. Beskow's capillarity test is also used to augment the grain size criteria. The limitations imposed are given in Table 7. The classification of soils with capillarities between 2 and 10 m is not specified. However, it is believed that Beskow would have classified these materials as highly frost susceptible. Riis reported that in many cases, the capillarity rule alone is sufficient to decide if a given soil is frost susceptible.

Table 7. Capillarity frost susceptibility criteria used by the Danish State Road Laboratory (Riis 1948).

Frost susceptibility	*Capillarity (m)*
None	<1
High	1-2
High	10-20
Slight	>20

More recently Christensen and Palmquist (1976) reported that the Danish State Road Laboratory specifies that soils with more than 10%

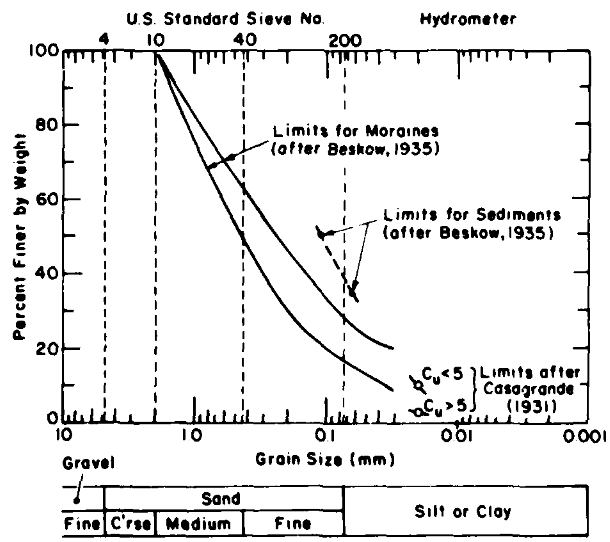


Figure 20. Grain size frost susceptibility criteria according to Riis (1948).

Table 8. East German frost susceptibility criteria according to Klengel (1970).

Gravel type	*Particles < 0.1 mm diameter (%)*	*Adsorbed water capacity*	*Mineral chemical activity*	*Frost heave susceptibility*	*Bearing capacity reduction during thawing*
Coarse-grained aggregate	<10	<0.25	Low to high	None	None
	10-30	<0.30	Low	Variable	Slight
	30-50	>0.30	Low	Slight	Slight to moderate
Fine-grained aggregate	50-75	0.30-0.50	Low	Slight to moderate	Moderate to high
	>75	0.50-0.80	Low	Slight to very high	Slight to moderate
		>0.80	High	Slight	Slight

Table 9. Factors which influence frost susceptibility (Klengel 1970).

	←short	Duration of frost period	long→	
	←high	Freezing temperature level	low→	
	←low	Water table	high→	
	←much	Quartz in sand-grain domain	little→	
Decrease	←little	+ clay minerals	much→	Increase
in	←little	Quartz in silt-grain domain	much→	in
frost	←much	+ clay minerals	little→	frost
susceptibility	←high	Degree of compaction in gravel rich in silt	low→	susceptibility
	←high	Water content in gravel rich in silt	low→	
	←low	Degree of compaction for gravel rich in clay	high→	
	←low	Water content for gravel rich in clay	high→	
	←high	*Load*	low→	

of the particles finer than 0.075 mm in diameter are frost susceptible.

East Germany

Klengel (1970) proposed the FS classification system given in Table 8 for use for gravels and crushed stone in the German Democratic Republic (East Germany). This classification method has been developed from field and laboratory measurements of frost heave and reduction in bearing capacity. Few details were given of these observations. Klengel concluded that FS is a "variable quantity" that changes value in response to changing environmental factors. Table 9 shows the various influences Klengel has identified and how they affect the FS of crushed stone or gravel.

According to Klengel's classification system, soils with less than 10% of the particles smaller than 0.1 mm are not affected by frost, and those with more than 10% smaller than 0.1 mm have variable responses to frost, depending on grain size, adsorbed water, mineral type, availability of water, compaction, load, and freeze-thaw history. Klengel reported that bearing capacity reduction is generally affected to a greater degree than is frost heave for the same conditions.

England

According to Townsend and Csathy (1963a, b), Croney (1949) suggested that the gradation limits shown in Figure 21 should be used to identify frost-susceptible soils. These limits are based on experience in Britain, where "frost rarely penetrates more than 12 to 18 inches below the road surface." The criteria are apparently for the most severe conditions: a high water table and a cold winter. According to this classification system, all soils with less than 20% of the grains smaller than 0.02 mm are not frost susceptible. This limitation appears to be unreasonably high and inappropriate for conditions in the United States. Indeed, Townsend and Csathy (1963b)

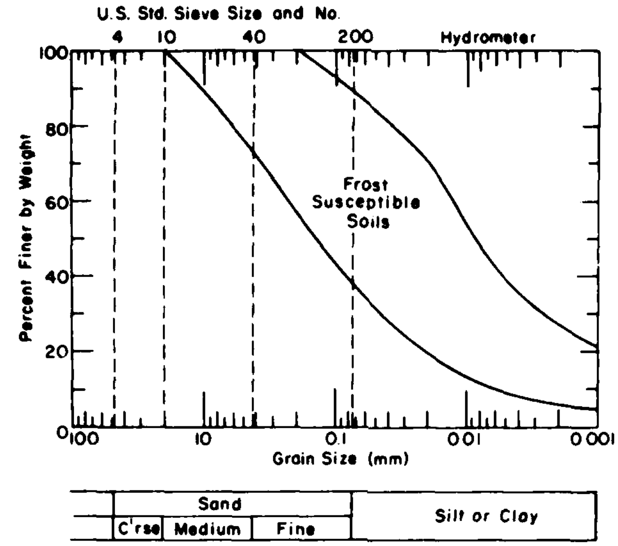


Figure 21. Limits of frost-susceptible soils according to Croney (1949).

found this criterion to be the least reliable in rejecting frost-susceptible soils.

Finland

Jessberger (1976) reviewed Orama's report (1970) on the determination of FS of soils in Finland. The basis of the classification system is Casagrande's criteria (1931).

Figure 22 shows that the grain size plot is divided into four critical regions. The boundary between Regions 3 and 4 is determined by Casagrande's criterion where 3% of the particles are smaller than 0.02 mm and the uniformity coefficient is 15; the boundary between Regions 1 and 2 is where 10% of the particles are smaller than 0.02 mm and the uniformity coefficient is 5.

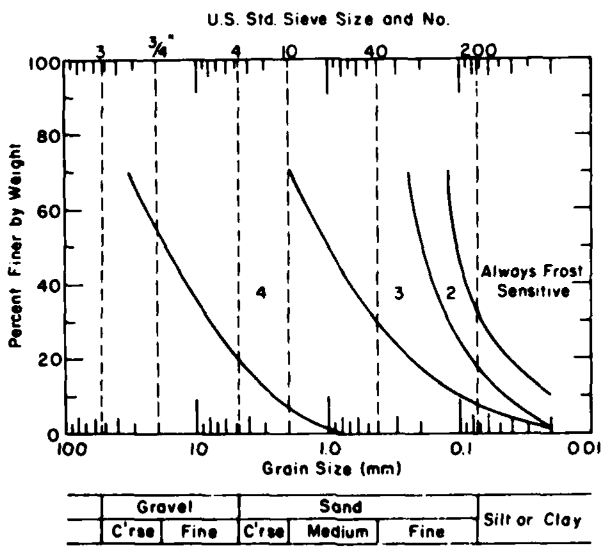


Figure 22. Frost susceptibility classification of soils according to Orama (1970). The soil is non-frost-susceptible if all of its grain size distribution curve lies within Regions 2, 3 or 4.

All soils with grain size distribution curves that lie entirely within Region 1 are always frost susceptible. Soils with grain size distribution curves that lie wholly within Regions 2, 3 or 4 are non-frost-susceptible. These soil types with curves whose lower portions fall to the left of Regions 2, 3 or 4 are frost susceptible. Soils for which the lower portion of the grain size curve passes through a region to the right are non-frost-susceptible, as are soils where the upper portion of the curve is only partially in a finer-particle region. For borderline cases the capillarity of the soil is used (no details were given by Jessberger).

Greenland

Nielsen and Rauschenberger (1957) reported the following FS criteria based on an evaluation of soil particles smaller than 2 mm:

1 All the soil types containing less than 5% of particles less than 0 075 mm in diameter (Fig 23) (i e soil types in which the grain-size curve drops below Point A [Fig 23] are non-frost-susceptible)

2 The other soil types are divided as follows

a Sediments are not frost susceptible when less than 50% is smaller than 0 125 mm and at the same time not more than 35% is smaller than 0 074 mm (i e when the grain-size curve lies below Points B and C) Sediments with grain-size curves which lie above Points B and C are frost susceptible

b Ungraded soil types are not frost susceptible when the grain-size curve lies below Curve D Ungraded soil types with grain-size curves that lie above Curve E are frost susceptible

3 If less than 20% of the sample passes through a 2-mm sieve, the soil is non-frost-susceptible

This classification was developed for use in Greenland and is based on the susceptibility to frost heave.

Japan

According to Jessberger (1969), the Japanese (Japan 1960) classify all sands, gravels, crushed rocks and volcanic ash with less than 6% of the particles smaller than 0.075 mm as non-frost-susceptible.

Netherlands

According to von Moos (1956), the Netherlands classifies soils with less than 5% of the particles smaller than 0.05 mm and less than 3% organic humus as non-frost-susceptible.

Norway

According to Christensen and Palmquist (1976), Brudal classified soils with less than 20% of the total sample less than 0.125 mm as non-frost-susceptible. No details on these criteria are available as neither the original nor a translation

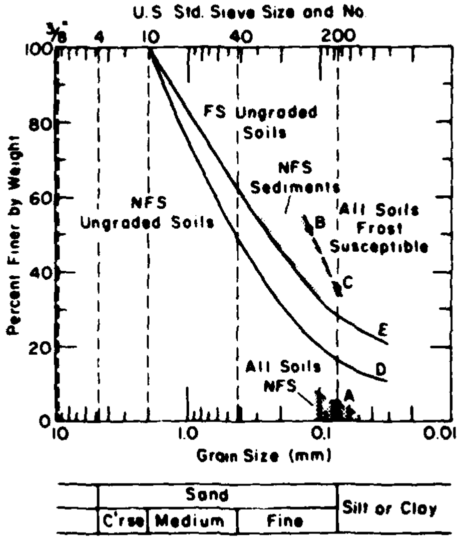


Figure 23. Frost susceptibility classification of soils according to Nielsen and Rauschenberger (1962). (NFS = non-frost-susceptible.)

of the Christensen and Palmquist report is available.

According to von Moos (1956), Norway has been classifying soils with less than 25% of the particles smaller than 0.25 mm and 20% smaller than 1.00 mm as non-frost-susceptible.

Poland

Pietrzyk (1980) developed the FS classification scheme shown in Figure 24 for a temperature of -5°C. It appears that this classification is the result of laboratory direct frost heave tests. The author apparently has also developed similar graphs for other temperatures; he admits, however, that application to field problems is uncertain because of the almost continuous variability in air temperature. It should be noted that the criteria in Figure 24 are for the worst hydrologic conditions, where water is freely available. A unique feature of these criteria is the dependence on overburden stress.

Romania

The Romanian FS standards are based on grain size and Atterberg limits. Vlad (1980) reported the Romanian standards shown in Table 10 and Figure 25. This standard is based on Schaible's most recent proposal (1957), with the plastic limit introduced by the Romanians as a refinement.

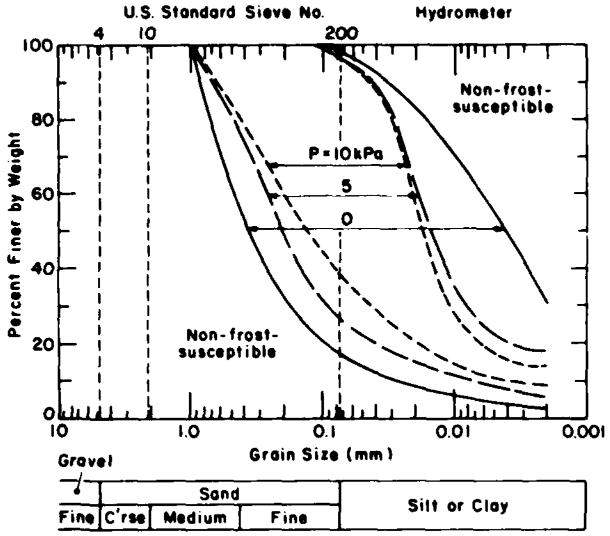


Figure 24. Frost susceptibility classification according to Pietrzyk (1980).

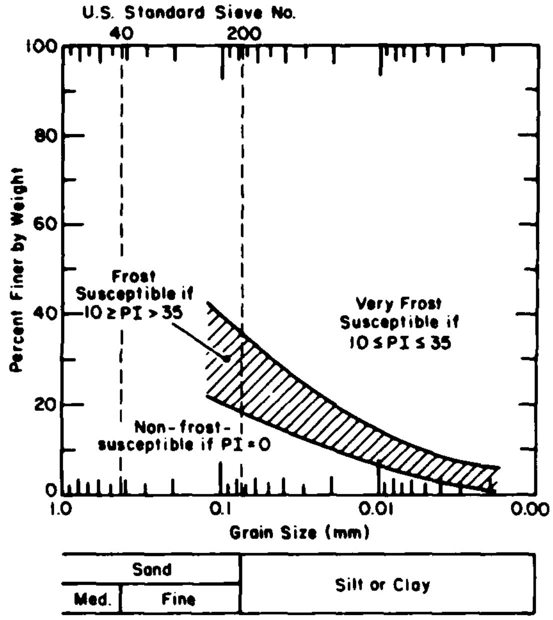


Figure 25. Frost susceptibility criteria according to Vlad (1980).

Table 10. Frost susceptibility criteria according to Vlad (1980).

		Criteria		
			Grading	
Frost susceptibility	*Type of soil*	*Plasticity*	*Particle diameter (mm)*	*Percentage of the total specimen mass*
None	Non-cohesive soil without clay	PI = 0	<0.002	≤1
			<0.02	≤10
			<0.1	≤20
Low-high	Non-cohesive soil with clay	PI ≤ 10	<0.002	≤6
			<0.02	≤20
			<0.1	≤40
	Cohesive soil	PI >35		
Very high		10≤PI≤35	<0.002	>6
			<0.02	>20
			<0.1	>40

Sweden

Beskow (1935) determined from numerous laboratory experiments and field observations in Sweden that "non-frost-heaving" soils exhibit less than 3-4 cm of heave during one winter. He concluded that it is practically impossible to fix a definite grain-size boundary between frost-heaving and non-frost-heaving soil because of the effects of grain size distribution, surcharge, and distance to the water table. However, he decided that the degree of variation of these factors is so strongly marked that for practical purposes, limits were appropriate. He suggested that limits be based on the soil type (sediment or moraine), the average diameter, the amounts finer than 0.062 mm and 0.125 mm, the capillarity parameters K_F and K_M, and the hygroscopicity. Beskow did not define moraine or sediment,

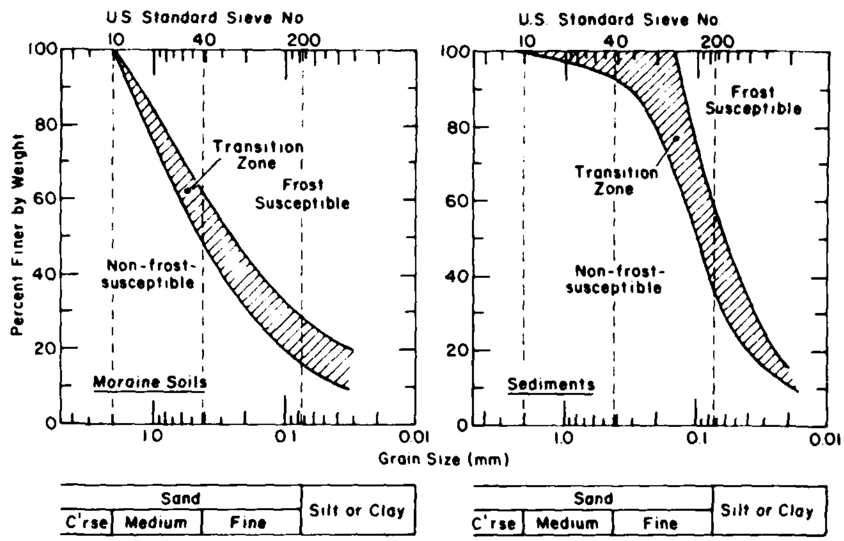


Figure 26. Frost susceptibility limits according to Beskow (1935). (After Townsend and Csathy 1963a.)

Table 11. Frost susceptibility criteria according to Beskow (1935).

Frost susceptibility	*Soil group*	*Average diameter (mm)*	*Amount passing sieve* 0.062 mm*	*Amount passing sieve* 0.125 mm*	*Capillarity K_f (m)*	*Hygroscopicity W_h (%)*
Non-frost-heaving under any circumstances	Sediment	0.1	<30	<55	<1	—
	Moraine	—	<15	<22	<1	
Causing frost heave only at surface and for very high ground water	Sediment	0.1-0.07	30-50	—	1 to 1¼	—
	Moraine	—	30-50	—	1 to 1¼	—
Same, except affects whole road base for very high ground water	Sediment	0.8-0.05	15-25	22-36	—	—
	Moraine	—	15-25	22-36	1¼ to 2½	—
Normally frost heaving and liable to frost boils for ground water depths ≤1.5 m (≤1 m for moraines)	Sediment	<0.05	>50	—	2-20	<5
	Moraine	—	>25	>36	2-20	1-4
Frost-heaving clays but not liable to boils	(Sediment)	—	—	—	20-?†	5-(10?)
Non-frost-heaving stiff clays	(Sediment)	—	—	—	?	(>10?)

* In percent of material finer than 2 mm

† Original unclear

but according to Townsend and Csathy (1963b), the uniformity coefficient for soils that Beskow labeled moraines is greater than 50, and for sediments, C_u is less than 20. By capillarity, Beskow meant the suction required to break down capillary saturation, K_F being for loose packing and K_M for dense packing. However, it is uncertain what he meant by hygroscopicity, except that it is the water content of the soil particle surface adsorbed from water vapor. The method used to determine this was not reported.

As a result Beskow proposed the following (Table 11 and Fig. 26):

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

"2. For small loads (and high ground water), soils with a capillarity of K_f = 1-2½ meters and K_M = 1¼-4 meters may be dangerous (silt sediments: 30-50% less than 0.062 mm). Such soils may cause bank slides even if they don't have any heave in roadways. For an extremely high ground water and slow freezing they may even be dangerous in the roadbed.

"3. Soils with a capillarity of K_f greater than 2 meters and K_M greater than 3 meters (fine silts and finer sediments of which more than 50% is less than 0.062 mm) are under all circumstances frost-heaving. These soils usually have a hygroscopic value of W_h greater than 1.

"These values are for the upper limit of grain size which are critical. For the lower grain size limit the following data may be given:

"1. Sediments. The soils which are essentially frost-heaving and cause frost boils have a hygroscopic value up to W_h = 4, which is the division between lean clay and medium clay. However, even the leaner of the medium clays (W_h = 4-5) may become dangerous under very variable hydrographic conditions and under a very small load pressure. The extreme limit may then be put at W_h = 5 for soils which may form frost boils. But stiffer clays may still be frost-heaving, and from a practical standpoint the entire range of medium clays may be considered frost-heaving. Therefore, the ultimate limit for any danger at all must be put at W_h = 10.

"2. Moraines. The limit is here quite difficult to fix definitely. Only the silt and fine silt sediments are really dangerous to form frost boils. For a considerable clay content, and especially when there is a very even distribution of grain size causing a small pore volume, the permeability and therefore the possibility of frost-heave become very small."

Beskow (1938) later discussed the criteria actually used in Sweden. The original of this paper was not available for review. The details in Table 12 have been taken from Townsend and Csathy (1963a). This classification differs from the Beskow (1935) classification principally in the sieve size, apparently as a concession to the Unified Soil Classification System, where the 0.074-mm particle size is used to differentiate between sands and silts.

Table 12. Frost susceptibility criteria according to Beskow (1938).

Soil type	Allowable amount finer than 0.074 mm (%)	Allowable capillarity (m)
Well-sorted sediments	<40	<1
Well-graded moraines	<19	<1

Rengmark (1963) presented the FS classification system used by the National Road Research Institute in Sweden. These criteria are based on both frost heave and thaw-weakening susceptibility. However, no details for developing these standards were reported.

Soils are classified according to their FS as follows:

1. Non-frost-susceptible soils are those inorganic soils that are not prone to frost heaving and are not softened during the thawing process.

2. Moderately frost-susceptible soils are those inorganic soils that are normally subject to frost heaving only when the rate of freezing is low or when the depth to the ground water table is small. During thawing, these soils undergo small to moderate reductions in bearing capacity.

3. Highly frost-susceptible soils are those inorganic soils where frost heave is considerable under normal freezing conditions or if the ground water table is high. Large reductions in bearing capacity occur during thaw.

The soil types in each of these categories are shown in Table 13 and the grain sizes for each soil type are shown in Table 14.

Table 13. Frost susceptibility for different soil types according to Rengmark (1963).

Frost susceptibility	Soil type
1 None	Gravel
	Sand
	Coarse mo (sandy silt)
	Gravelly moraine
2 Moderate (possibly none)	Sandy moraine
2 Moderate	Normal moraine
	Sandy moraine
	Moraine clay
	Heavy medium clay
	Heavy clay
	Very heavy clay
2 Moderate (possibly high)	Clayey moraine
3 High	Moey moraine (sandy, silty)
	Silty moraine
	Fine mo (sandy silt)
	Silt
	Light clay
	Light medium clay

Table 14. Grain sizes of different soil types according to Rengmark (1963).

Soil type	Grain size (mm)
Boulders	>200
Large stone	200-60
Small stone	60-20
Coarse gravel	20-6
Fine gravel	6-2
Sand	2-0.6
Medium sand	0.6-0.2
Coarse mo (sandy silt)	0.2-0.06
Fine mo (sandy silt)	0.06-0.02
Coarse silt	0.02-0.006
Fine silt	0.006-0.002
Clay	<0.002

Fredén and Stenberg (1980) reported that sedimentary soils in Sweden are now classified according to capillarity and the portion finer than 0.074 mm. The Swedish FS classification system for sediments is shown in Table 15. This system appears to have evolved from the early work of Beskow.

Table 15. Swedish frost susceptibility criteria.

Frost susceptibility	Amount finer than 0.074 mm (%)	Capillarity (m)
None	<16	<1
Low-high	16-43	1.0-1.5
High	>43	>1.5

Switzerland

Ruckli (1950) proposed criteria for Switzerland based principally on Beskow's work (1935). These are basically frost heave criteria and do not appear to consider thaw weakening. The classification in Table 16 is taken from Jessberger's review (1976).

These criteria must be modified to fit the situation. Laboratory studies, such as those discussed by Beskow (1935) for determining hygroscopicity, capillarity and frost heave, may also be necessary. The effect of the ground water level in soils with relatively high permeabilities must also be taken into consideration. According to Jessberger (1976), Ruckli essentially agrees with Beskow (1938), Taber (1930a) and Ducker (1939) with regard to determining FS.

Table 16. Frost susceptibility criteria according to Ruckli (1950). (After Jessberger 1976.)

Frost susceptibility	Soil type
I None (22% <0.125 mm or 17% <0.075 mm)	Peaty and swampy soils Gravel Sand if >50% <0.125 mm
II Moderate (22% >0.125 mm or 17% >0.075 mm)	Mud Loam Compacted ballast Normal moraine
III Considerable	Rock flour (silt) Light loam Moraine with high loam or silt content Fine sand if >50% <0.125 mm

Bonnard and Recordon (1958) presented the early Swiss standards for determining the FS of soils (Association of Swiss Road Engineers 1957). They considered soils to be non-frost-susceptible if less than 3% of the soil particles are less than 0.02 mm in diameter. This appears to be based on the Casagrande (1931) criteria. A more detailed classification based on gradation characteristics is given in Table 17.

Table 17. Frost susceptibility criteria according to Bonnard and Recordon (1958).

This standard was adopted by the Swiss Federal Government (norm 40325).

Frost susceptibility	Soil type	Unified Soil Classification*
None	Clean gravel and clean sand	GW, GP SW, SP
Slight	Silty or claylike gravel	GM, GC
Average	Silty or clayey sand, highly plastic clay, organic clay	SM, SC CH OH
High	Low or highly plastic silt, clay of low plasticity, organic silt	ML, MH CL OL

* G = gravel, S = sand, M = silt, C = clay, O = organic, W = well-graded, P = poorly graded, H = high plasticity, L = low plasticity.

Bonnard and Recordon (1969) discussed more recent developments in the Swiss FS standards for gravel base course materials. The existing standards and those under development are

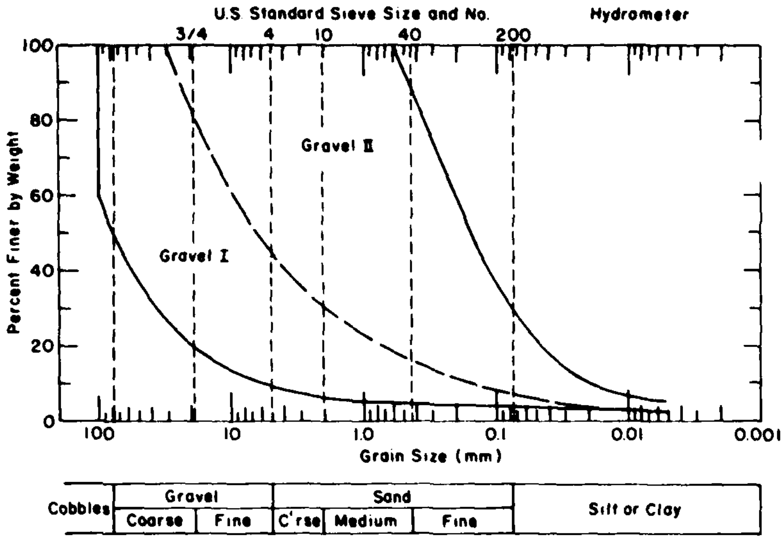


Figure 27. Limits for Gravel I and Gravel II. Gravel I is non-frost-susceptible. Gravel II requires a frost heave test and a loss of bearing capacity test. (After Bonnard and Recordon 1969.)

given in Table 18. Gravels with properties failing the criteria in the table are considered to be frost susceptible and cannot be used as road base material. Grain size distribution criteria for gravel classifications I and II are given in Figure 27.

Recordon and Rechsteiner (1971) presented a standard for determining the FS of gravel base and subbase materials in Switzerland. This standard was developed to permit the use of marginal gravels because clean gravels are becoming scarce in Switzerland. Both frost heave and thaw weakening are considered in the standard. The essential factors include grain size characteristics before and after compaction, Atterberg limits, compaction characteristics and CBR after soaking or freezing and thawing. Two categories of gravels have been established (Gravel I and Gravel II); *the sensitivity to freezing must be determined in the laboratory*. Materials passing the standards given in Table 19 are non-frost-susceptible. These FS criteria are among the most thorough of the methods reviewed. The principal limitation appears to be in adequately sampling heterogeneous gravels in their natural state.

Table 18. Frost susceptibility criteria according to Bonnard and Recordon (1969).

	Existing standards	*Standards in preparation*	
	*I**	*I**	*II**
Grain size characteristics			
Maximum particle size	100 mm	30-100 mm	10-100 mm
Amount less than 0.02 mm	< 3%	< 3%	3-10%
Uniformity coefficient	> 4	15-100	—
Curvature coefficient	1-3	1-3	—
Atterberg limits			
Plasticity index	none	none	< 6%
Liquid limit	none	none	< 25%
Laboratory tests required for frost resistance	none	none	a) frost heave b) loss of bearing capacity

* Gravel quality class

Table 19. Frost susceptibility criteria according to Recordon and Rechsteiner (1971).

	Gravel I		*Gravel II*
	Undisturbed material	*Compacted* material*	*Crushed and undisturbed material*
Max particle diam (mm)	200	30-100	10-100
Amount finer than 0.02 mm (%)	≤3	≤3	≤10
Uniformity coefficient	3-15	10-50	—
Coefficient of curvature	1-3	1-3	—
Optimum water content (%)	—	<5	≤10
Liquid limit (%)	—	—	≤25
Plasticity index (%)	—	—	≤6
CBR† (undisturbed)	—	—	≥30
CBR† (crushed)	—	—	≥80
CBR† reduction (%)	—	—	<50
Increase in amount of 0.02-mm size after compaction	—	<2**	<2

* American Association of State Highway Officials compaction standards.
† After soaking for four days or after one freeze-thaw test.
** If fraction exceeds 3%, then the material is usable only as Gravel II

The Association of Swiss Road Engineers (1976) reported the most recent developments in the Swiss standards. According to Jessberger (1976), this FS classification standard considers both frost heaving and thaw weakening.

It makes the distinction between frost-safe and frost-endangered materials. Frost-safe materials are those in which no ice lenses form during ground freezing and which undergo little or no reduction of load capacity during thawing, even when subjected to the worst hydrological and climatic conditions. Frost-endangered materials are those which do not meet the above definition. Frost heave damage can result from ice lenses or the loss of load-carrying capacity upon thawing.

Three levels of determining the FS of soils are specified. The first of these is based on the Casagrande criteria, with some consideration of the U.S. Army Corps of Engineers criteria. This classification is given in Table 20.

The second level used by the Swiss is based on soil classification tests. This standard, which is essentially the same as the U.S. Army Corps of Engineers criteria, is given in Table 21.

The third level in the Swiss standards is for determining the FS of granular base and subbase materials. It requires laboratory CBR tests before and after soaking with water or after freezing and thawing. It is not clear from the description of this standard which of the two conditioning tests are preferable or what differences in CBR response will result. This test requires that the CBR is not reduced by more than 50%. As in the earlier Swiss standard described by Recordon and Rechsteiner (1971), the granular materials are separated into two groups, Gravel I and Gravel II. Gravel I is the clean base or subbase material that would not be affected by frost action, and Gravel II is the base or subbase material that has a higher percentage of fines and for which some small but acceptable effects of freezing are expected. Materials passing the requirements given in Table 22 are determined to be non-frost-susceptible.

Table 20. First level of the Swiss frost susceptibility criteria.

Frost susceptibility	*Amount finer than 0.02 mm* (%)*
None†	≤1.5
Borderline	1.5-3
High	>3

*Applied only to the fraction smaller than 60 mm.
†Homogeneous sands with C_u >5 are practically non-frost-susceptible if they contain less than 10% finer than 0.02 mm.

Table 21. Second level of the Swiss frost susceptibility criteria.

Frost susceptibility	*Soil type*	*Amount (%) finer than 0.02 mm*	*U.S.C.S. soil classification**
Slight	Gravel	3-10	GW, GP GM, GC
Slight to moderate	a) Gravel	10-20	GM, GC-CL GM-GC, GM-ML
	b) Sand	3-15	SW, SP, SM, SC
Moderate	a) Gravel	> 20	GC-CL, GM-GC, GM-ML
	b) Sand (except very fine silty sand)	> 15	SC-CL, SM-SC, SM-ML
	c) Clays, PI > 12		CL, CH
High	a) Silt		ML, MH
	b) Very fine silty sand	> 15	SM-ML
	c) Clayey silt, PI < 12		CL, CL-ML
	d) Banded clays and other banded fine soils		In alternate layers CL, ML CL, ML, SM CL, CH, ML CL, CH, ML, SM

* G = gravel, S = sand, M = silt, C = clay, W = well-graded, P = poorly graded, H = high plasticity, L = low plasticity.

Table 22. Third level of the Swiss frost susceptibility criteria.

Materials characteristics	*Gravel I*		*Gravel II*	
	Round	*Broken*	*Round*	*Broken*
Amount (%) < 0.02 mm	< 3		< 10	
Uniformity coefficient (C_u)	12-100	10-50	—	—
Coefficient of curvature (C_c)	1-31		—	—
Maximum particle size (mm)	30-100		10-100	
Optimum water content (%)	< 5		< 10	
Plastic limit (%)	—	—	< 25	
Plasticity index (%)	—	—	< 6	
CBR_2 or CBR_3* (%)	— †	—	> 30	> 80
CBR_2/CBR_1 or CBR_3/CBR_1	— †	—	> 0.5	

* CBR_1 = CBR as compacted, CBR_2 = CBR after soaking with water for four days; CBR_3 = CBR after one freeze-thaw cycle

† Gravel I is not subject to the CBR reduction test

Table 23. Arizona frost susceptibility criteria.

Elevation above sea level (ft)	*Maximum amount greater than 0.075 mm (%)*
<2500	12
2500–3500	10
>3500	8

United States

Alaska. Johnson et al. (1975) reported that Alaska specifies that soils with less than 3% of the particles finer than 0.074 mm are non-frost-susceptible. The FS criteria based on the U.S. Army Corps of Engineers (1965) criteria are also used.

More recently, Esch et al. (1981) reported that base and subbase materials with 0–6% of the particles finer than 0.074 mm are considered to be non-frost-susceptible.

Arizona. According to the method used in Arizona (Erickson 1963), FS depends on the elevation above sea level (Table 23). This effect is probably related to climatic differences.

Asphalt Institute of North America. Johnson et al. (1975) reported that the Asphalt Institute uses 7% finer than 0.074 mm as the dividing point between non-frost-susceptible and frost-susceptible soils.

Bureau of Public Roads. Morton (1936) established subdivisions within the Bureau of Public Roads soil classification system. According to Townsend and Csathy (1963a), the basis for Morton's FS classification system (Table 24) was his experience in New Hampshire.

California. Johnson et al. (1975) reported that California classifies subgrade soils with less than 5% finer than 0.074 mm as non-frost-susceptible. No limits were reported for base and subbase materials.

Colorado. Johnson et al. (1975) reported that Colorado calls base and subbase materials non-frost-susceptible if 5–10% of the particles are smaller than 0.074 mm.

Connecticut. Haley (1963) and Johnson et al. (1975) reported that the FS of soils in Connecticut is determined with the Casagrande (1931) criteria, with the special restrictions that less than 10% must be smaller than 0.074 mm and the fines must be non-plastic.

Delaware. Haley (1963) reported that Delaware allows non-frost-susceptible soils to contain up to 35% of their particles smaller than 0.074 mm.

Idaho. According to Erickson (1963), all silty and organic clayey soils (with 36% smaller than 0.074 mm and PIs less than 10%) have been considered to be frost susceptible in Idaho. A more recent survey (Johnson et al. 1975) found that base and subbase materials with more than 5% less than 0.074 mm are frost susceptible if the sand equivalent is less than 30% of the total.

Illinois. Johnson et al. (1975) reported that all silty soils with more than 36% of the particles smaller than 0.074 mm, a PI of less than 10%, and a LL of less than 40% are considered to be frost susceptible in Illinois, as are all other soil with 70% or more smaller than 0.074 mm.

Iowa. Johnson et al. (1975) reported that soils with more than 15% of the particles smaller than 0.074 mm are considered to be frost susceptible.

Kansas. Johnson et al. (1975) reported that all

Table 24. Frost susceptibility criteria according to the Bureau of Public Roads.

Frost susceptibility	*Potential frost heave (cm)*	*Soil classification**	*Soil type*	*Allowable amount (%) finer than 0.05 mm*
None	<0.8	A-3	Cohesionless sand & gravels	–
Low	0.8–1.6	A-2G	Sand & gravel hard pans	<10
Medium	1.6–2.4	A-2F	Silt hard pans	10–25
High	2.4–3.5	A-2P	Clay hard pans or boulder clays	>25
Very high	>3.5	A-4	Fine-grained silts	–

*Bureau of Public Roads classification system.

silty subgrade soils are classified as frost susceptible in Kansas, as are base and subbase materials with more than 15% smaller than 0.074 mm.

Maine. Johnson et al. (1975) reported that Maine has used the U.S. Army Corps of Engineers grain size distribution criteria for subgrade soils. Base materials with 0–5% less than 0.074 mm and subbase materials with 0–7% less than 0.074 mm are classified as non-frost-susceptible.

Maryland. Johnson et al. (1975) reported that Maryland has used the U.S. Army Corps of Engineers grain size distribution FS classification system for subgrade soils, but usually classifies base and subbase materials with as much as 12% smaller than 0.074 mm as non-frost-susceptible.

Massachusetts. Haley (1963) reported that Massachusetts has classified soils with more than 15% smaller than 0.074 mm as frost susceptible. Johnson et al. (1975) more recently reported that Massachusetts classifies subgrade soils with more than 12% of the particles smaller than 0.074 mm and base and subbase materials with more than 10% smaller than 0.074 mm as frost susceptible.

According to Johnson et al. (1975), the Massachusetts Turnpike Authority uses the U.S. Army Corps of Engineers criteria for subgrade soils, and like the Commonwealth of Massachusetts, it classifies base and subbase materials with more than 10% smaller than 0.074 mm as frost susceptible.

Massachusetts Institute of Technology. Casagrande (1931), while studying the frost heave problem at the Massachusetts Institute of Technology, concluded that "under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than 3% of grains smaller than 0.02 mm., and in very uniform soils containing more than 10% smaller than 0.02 mm." This conclusion was based principally on a study of a test road at MIT and on field observations in New Hampshire.

Later, Casagrande (1934) stated that in soils with less than 3% smaller than 0.02 mm little or no ice is formed and that no ice segregation would occur if less than 1% of the soil particles was smaller than 0.02 mm in diameter.

Jessberger (1976) criticized Casagrande's criteria, saying that they were based on insufficient evidence; he stated that they fail to take into account the depth to the water table, the variations in climate and material type, and the loss of bearing capacity during thaws. However, Jessberger conceded that Casagrande's criteria are a significant contribution, as they seldom lead to adverse experiences.

Casagrande (1947) presented one of the earliest frost susceptibility criteria based on a soil classification system. The original report was unavailable for review. According to Townsend and Csathy (1963a), this is Casagrande's so-called Airfield Classification System, the forerunner of

Table 25. Frost susceptibility classification system according to Casagrande (1947).

Soil type	*Unified Soil Classification**	*Frost susceptibility*
Well-graded gravel-sand, no fines	GW	None to very slight
Well-graded gravel-sand with clay	GC	Medium
Poorly graded gravel	GP	None to very slight
Gravel with fines, silty gravel	GF	Slight to medium
Well-graded sands, no fines	SW	None to very slight
Well-graded sands, clay binder	SC	Medium
Poorly graded sands, few fines	SP	None to very slight
Sands with fines	SF	Slight to high
Silts and very fine sands	ML	Medium to very high
Silty clays of low plasticity	CL	Medium to high
Organic silts, organic silt-clays	OL	Medium to high
Fine sandy, silty, micaceous silts	MH	Medium to very high
Inorganic clays of high plasticity	CH	Medium
Organic clays of medium plasticity	OH	Medium

*G = gravel, S = sand, M = silt, C = clay, W = well-graded, P = poorly graded, H = highly plasticity, L = low plasticity the U.S. Army Corps of Engineer classification system (U.S. Army Engineer Waterways Experiment Station 1957). This classification system is given in Table 25.

Michigan. Johnson et al. (1975) reported that FS in Michigan is determined from a visual inspection of subgrade soils, base and subbase materials are classified as frost susceptible when the loss of fines by washing is greater than 7%.

Minnesota. According to Johnson et al. (1975), Minnesota classifies all fine-grained soils and base and subbase materials with more than 10% of the particles smaller than 0.074 mm as frost susceptible.

Montana. Erickson (1963) reported that Montana classifies A-1-a, A-1-b and A-2-4 granular materials (AASHO soil classification, Table 26) as least frost susceptible.

Nebraska. Johnson et al. (1975) reported that Nebraska classifies all subgrade materials except clean and coarse sands as frost susceptible. Base and subbase materials with plasticity indexes of less than 6 are classified as follows:

Base materials 8-12% < 0.074 mm, non-frost-susceptible.

Subbase materials 5-13% < 0.074 mm, non-frost-susceptible.

New Hampshire. Haley (1963) reported that the FS classification system in Table 27 is used in New Hampshire. Johnson et al. (1975), however, reported that officials in New Hampshire had later adopted the Casagrande criteria (if less than 3% is finer than 0.02 mm, then the soil is non-frost-susceptible) for subgrade materials. For non-frost-susceptible base and subbase materials, 0-8% less than 0.074 mm is allowable for crushed stone, and 0-12% of the fraction finer than 5.2 mm can be less than 0.074 mm for sand, gravel, and crushed gravel. The New Hampshire rapid freezing test, which will be discussed later, is required when materials are borderline.

New Jersey. Turner and Jumikis (1956) evaluated the behavior of 30 New Jersey soils in terms of frost heave and thaw weakening (Table 28).

Table 26. Materials considered least frost susceptible in Montana.

Material type	*Soil classification**	*Amount (%) finer than* 0.074 mm	0.42 mm	2 mm	*Liquid limit (%)*	*Plasticity index (%)*
—	A-1-a	<15	<30	<50	—	<6
—	A-1-a	<25	<50	—	—	<6
—	A-2-4	<35	—	—	<40	<10
Subbase & base sands & gravels	—	<12	—	—	<35	<6

*According to the American Association of State Highway Officials

Table 27. New Hampshire frost susceptibility criteria.

Frost susceptibility	*Soil classification**	*Amount (%) finer than 0.074 mm*
None-low	A2	<10
Medium	A2	10-20
High	A2	25-35
Very high	A4	>35

*According to the American Association of State Highway Officials

Table 28. New Jersey frost susceptibility criteria.

Frost susceptibility	*Soil type*	*Amount (%) finer than 0.074 mm*	*Plasticity index (%)*	*Liquid limit (%)*
None	Gravel, sand	<25	<6	—
Uncertain	Gravel, sand	<35	<10	<40
Medium	Silt	>35	<10	>40
High	Clay	>35	>10	<40
Very high	Silt	>35	<10	<40
Very high	Clay	>35	>10	>40

New York. According to Haley (1963), New York has required that the Casagrande criteria be used for both subgrade and base/subbase materials. An additional stipulation that the plasticity index be less than or equal to three has also been made. Johnson et al. (1975) confirmed all but the plasticity index requirement.

Ohio. Johnson et al. (1975) reported that Ohio has classified AASHO A-4 subgrade materials with more than 50% silt and a PI of less than 10 as especially frost susceptible. Base/subbase materials with more than 15% smaller than 0.074 mm are also considered to be frost susceptible.

Oregon. Erickson (1963) reported that Oregon has classified all soils with more than 10% smaller than 0.074 mm as frost susceptible. More recently, Johnson et al. (1975) reported that officials in Oregon determine subgrade materials to be frost susceptible if more than 8% of the particles are smaller than 0.074 mm. Base materials with more than 8% smaller than 0.074 mm and a sand equivalent of less than 25% and subbase materials with a sand equivalent of less than 30%, a liquid limit greater than 33%, and a plasticity index greater than 6% are also considered to be frost susceptible.

Texas. Details of the Texas method for determining FS were reported by Carothers (1948) and were taken from Townsend and Csathy (1963a). The gradation limits shown in Figure 28 were suggested for non-frost-susceptible base materials.

U.S. Civil Aeronautics Administration. This standard was contained in the CAA (1948) specifications for the construction of airports. According to Townsend and Csathy (1963b), the CAA specified requirements for subbase materials where the frost penetration is 10 inches or more. These requirements are primarily based on general strength considerations, but they consider frost effects as well. No special considerations for frost are made for base materials. The requirements for non-frost-susceptible subbase materials are given in Table 29.

Table 29. U.S. Civil Aeronautics Administration frost susceptibility criteria.

Particle size (mm)	*Allowable amount (%)*
<76	100*
<0.42	70*
<2.0	100†
<0.42	25-75†
<0.074	0-15†

* Percentage of total sample
† Percentage of portion smaller than 2.0 mm

The liquid limit can be no more than 25% and the plasticity index, 6%. If more than 45% of the entire sample is larger than 2.0 mm, the amount

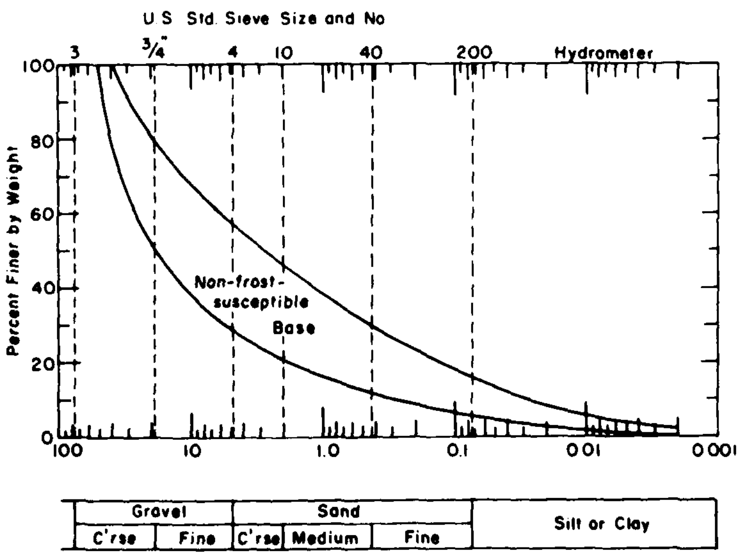


Figure 28. Limits of non-frost-susceptible base materials in Texas according to Carothers (1948). (After Townsend and Csathy 1963a.)

smaller than 0.074 mm may be increased to 25% if no increase in the liquid limit or the plasticity index occurs.

U.S. Army Engineer Waterways Experiment Station. In the Unified Soil Classification System, USAE WES (1957) has identified the potential effects of frost action on soils (Table 30). This FS classification is based on both frost heave and thaw weakening.

Table 30. U.S. Army Engineer Waterways Experiment Station frost susceptibility criteria.

Frost susceptibility	*Soil type*	*Unified Soil Classification**
None to very slight	Gravels	GW, GP
	Sands	SW, SP
Slight to medium	Gravels	GM, GC
Slight to high	Sands	SM, SC
Medium to very high	Silts	ML, MH
Medium	Clays, LL >50	CH, OH
Medium to high	Clays, LL < 50	CL, OL
Slight	Peat	PT

*G = gravel, S = sand, M = silt, C = clay, O = organic, PT = peat, W = well-graded, P = poorly graded, H = high plasticity, L = low plasticity

U.S. Army Corps of Engineers. Linell and Kaplar (1959) and Linell et al. (1963) reported on an early version of the frost design criteria (Table 31) used by the U.S. Army Corps of Engineers. These criteria are primarily used to select a pavement design method for given material characteristics. The frost classifications F1, F2, F3, and F4 are used to determine the thickness of base courses for various levels of road and airfield service requirements. Details of this design procedure are given in the Technical Manual "Soils and Geology—Pavement Design for Frost Conditions" (U.S. Army Corps of Engineers 1965).

This FS classification system, with some modifications, is essentially what is used today by the Corps of Engineers. It is based on Casagrande's system (the amount finer than 0.02 mm), extensive laboratory frost heave tests in which severe moisture and freezing conditions were imposed, and field observations of reduced bearing capacity after thaw.

The FS system (Table 32, Fig. 29) presently used by the Corps of Engineers (U.S. Army Corps of Engineers 1965) classifies most inorganic materials with 3% or more of their grains finer than 0.02 mm in diameter as frost susceptible for pavement design purposes. Gravels, well-graded sands and silty sands, especially those with densities near the theoretical maximum density curve, are considered to be possibly frost susceptible if they contain 1.5–3% finer than 0.02 mm; they must be subjected to a standard FS test to evaluate their behavior during freezing. Uniform sandy soils may have as much as 10% of their grains finer than 0.02 mm without being frost susceptible.

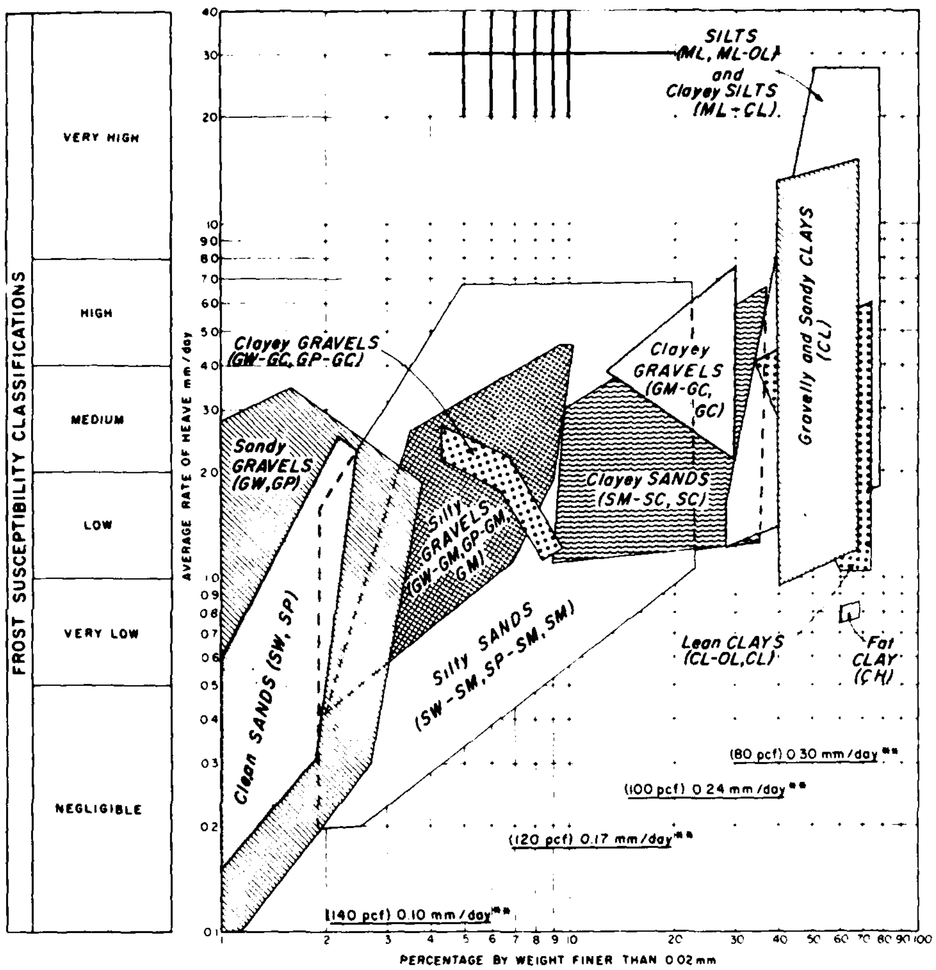
Soils classified as non-frost-susceptible may heave measurably under field conditions. However, few detrimental effects of frost heaving or thaw weakening would be expected.

Table 32 and Figure 29 show that there is a considerable range in the degree of FS within frost groups. This variability probably reflects the effects of differences in grain size distribution characteristics, dry density, mineralogy, etc., which are not included in the basic FS classification system. The variability is not necessarily a problem, since the Corps of Engineers

Table 31. U.S. Army Corps of Engineers frost design criteria.

*Frost design group**	*Soil type*	*Amount (%) finer than 0.02 mm*	*Plasticity index*
NFS	All soils	<3	—
F1	Gravelly soils	3-20	—
F2	Sands	3-15	—
F3	Gravelly soils	>20	—
	Sands	>15	—
	Clays	—	>12
	Varved clays/uniform subgrade	—	—
F4	All silts	—	—
	Very fine silty sands	>15	—
	Clays	—	<12
	Varved clays/non-uniform subgrade	—	—

*NFS = non-frost-susceptible, the degree of frost susceptibility generally increases from F1 to F4



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

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

* Undisturbed specimen

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

Figure 29. Range in the degree of frost susceptibility of soils according to the U.S. Army Corps of Engineers (1965).

Table 32. U.S. Army Corps of Engineers (1965) frost design soil classification system.

*Frost susceptibility**	*Frost group*	*Kind of soil*	*Amount finer than 0.02 mm (% by weight)*	*Typical soil type under Unified Soil Classification System†*
NFS**	None	(a) Gravels	0-1.5	GW, GP
		(b) Sands	0-3	SW, SP
Possibly‡	?	(a) Gravels	1.5-3	GW, GP
		(b) Sands	3-10	SW, SP
Very low to high	F1	Gravels	3-10	GW, GP, GW-GM, GP-GM
Medium to high	F2	(a) Gravels	10-20	GM, GM-GC, GW-GM, GP-GM
Negligible to high		(b) Sands	10-15	SW, SP, SM, SW-SM, SP-SM
Medium to high	F3	(a) Gravels	>20	GM,GC
Low to high		(b) Sands, except very fine silty sands	>15	SM, SC
Very low to very high		(c) Clays, PI > 12	—	CL, CH
Low to very high	F4	(a) All silts	—	ML, MH
Very low to high		(b) Very fine silty sands	>15	SM
Low to very high		(c) clays, PI < 12	—	CL, CL-ML
Very low to very high		(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

*Based on laboratory frost heave tests

†G = gravel, S = sand, M = silt, C = clay, W = well-graded, P = poorly graded, H = high plasticity, L = low plasticity

**Non-frost-susceptible.

‡Requires laboratory frost heave test to determine frost susceptibility

lists all the soil properties and frost heave test results used to develop these criteria (Appendix B). This tabulation contains the Unified Soil Classification, detailed grain size distribution data, coefficients of uniformity and curvatures, initial dry densities and void ratios, Atterberg limits, average rates of heave per day and frost susceptibility classifications. This list includes 79 classifications of gravels, 157 of sands, 52 of silts, and 89 of clays for a total of 377 classification tests with detailed information on material properties. By comparing the properties of a soil in question with those of the most similar soil in Appendix B, one can determine the relative frost susceptibility without conducting the frost heave test.

Utah. Erickson (1963) reported that Utah classifies all permeable fine sands and silts with more than 25% of the particles larger than 0.074 mm as frost susceptible.

Vermont. According to Haley (1963), Vermont considers soils to be frost susceptible if 10% of the particles are larger than 0.074 mm or 3% are larger than 0.02 mm. More recently, Johnson et al. (1975) reported that Vermont considers all silt-clay subgrade materials with more than 36% finer than 0.074 mm as potentially frost susceptible.

Washington. Both Erickson (1963) and Johnson et al. (1975) reported that Washington determines all soil with 10% or more of the particles smaller than 0.074 mm to be frost susceptible.

Wisconsin. Johnson et al. (1975) reported that Wisconsin uses the U.S. Army Corps of Engineers FS criteria for subgrade materials and generally determines base and subbase materials to be frost susceptible if 5% or more of the particles are smaller than 0.074 mm.

Wyoming. Erickson (1963) reported that Wyoming has classified base and subbase materials as frost susceptible if 20% or more of the particles are smaller than 0.074 mm, the liquid limit is greater than 25%, and the plasticity index is greater than 6%.

West Germany

According to Jessberger (1973), the system of Koegler et al. (1936) is a modification of the Casagrande (1931) criteria, where non-uniform soils with 3% or less of the particles smaller than 0.02 mm or uniform soils with 10% or less smaller than 0.02 mm are non-frost-susceptible. Soils failing this test are rated as to their degree of FS according to their permeability, as shown in Table 33.

Table 33. Frost susceptibility criteria according to Koegler et al. (1936).

Frost susceptibility	*Permeability (m/s)*
None	$>1\times10^{-6}$
Moderate	1×10^{-7} to 1×10^{-6}
High	1×10^{-8} to 1×10^{-7}

This modification apparently takes into account the amount of water that can be supplied to the freezing front. According to Jessberger (1976), these criteria are based principally on frost heave theory and have not been verified in the field.

Jessberger (1969, 1976) also reviewed Ducker's (1939) FS criteria and reported that Ducker defended the Casagrande (1931) criteria that all cohesionless soils with more than 3% of the particles smaller than 0.02 mm are frost susceptible. Ducker added that soils with no more than 10% of the particles larger than 0.1 mm and at least 25% between 0.05 and 0.02 mm are frost susceptible, even if 0% is less than 0.02 mm.

Schaible (1950) defined frost-susceptible soils as those having greater than 20% of the particles smaller than 0.02 mm and permeabilities in the range of 10^{-5} to 10^{-7} cm/sec. He later suggested (Schaible 1953) the criteria shown in Table 34. This classification is based on an analysis of 193 soil samples in the field and in the laboratory.

Table 34. Frost susceptibility criteria according to Schaible (1953).

Frost susceptibility	*Amount* (%) finer than* 0.02 mm	0.1 mm
None-low	<10	<20
Medium	10-15	20-30
Medium-high	15-20	30-40
Very high	>20	>40

*All percentages are expressed in terms of the fraction finer than 2 mm.

Still later, Schaible (1957) modified his classification system to one that divides soil types into non-frost-susceptible, frost susceptible and very frost susceptible groups on the basis of two grain size distribution curves determined from the 0.1-, 0.02-, and 0.002-mm-diameter particles. These FS criteria are shown in Figure 30 and Table 35. It appears that these criteria are based

Table 35. Frost susceptibility criteria according to Schaible (1957).

Frost	*Amount (%) finer than*		
susceptibility	*0.002 mm*	*0.02 mm*	*0.01 mm*
None-low	<1	<10	<20
Medium-high	*1-6*	*10-20*	*20-40*
Very high	>6	>20	>40

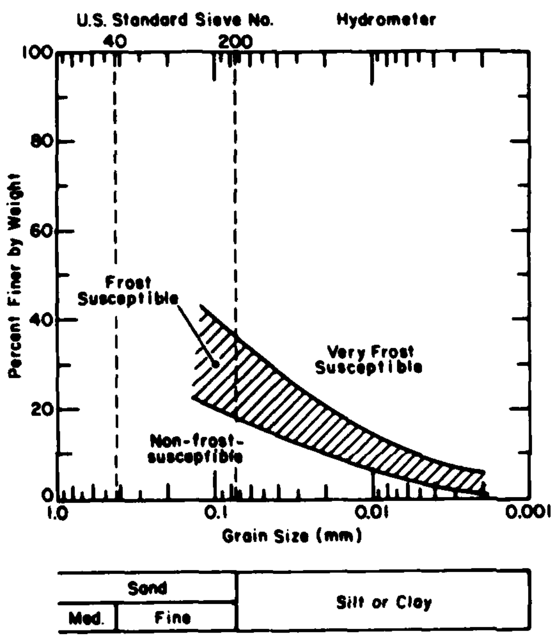


Figure 30. Frost susceptibility classification according to Schaible (1957).

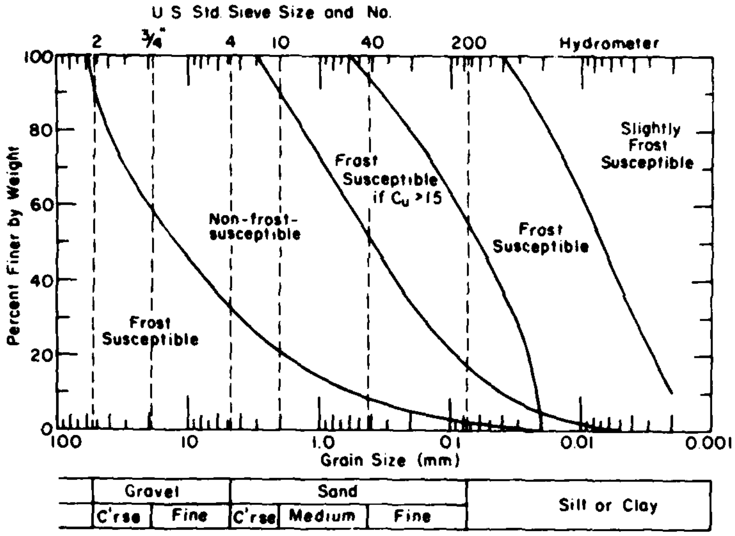


Figure 31. Limits of frost susceptibility of soils according to Jessberger and Hartel (1967).

on both frost heave and thaw weakening. Factors such as the water table level, the drainage conditions, the overburden stress and the freezing conditions were not considered. It seems likely that these criteria are for the worst conditions.

Maag (1966) suggested FS criteria based on the 0.06-mm particle size, but Jessberger (1976) reported that these criteria are questionable. They consider soil to be *non-frost-susceptible* if less than 15% of its particles are smaller than 0.06 mm and definitely frost susceptible when more than 30% of its particles are smaller than 0.06 mm. The classification of the soil in the intermediate range between 15% and 30% is not clear.

According to Jessberger (1976), Maag stated that no frost damage will occur in frost-susceptible soils if the water supply is limited and that no danger from frost heave will occur if the maximum depth of the freezing front is farther from the ground water table than the height of capillary rise. Other statements such as this led Jessberger to conclude that these criteria are based *on an insufficient understanding* of the frost heave process and should not be seriously considered.

A year later, Jessberger and Hartel (1967) reported a FS classification system based on grain size distribution curves (Fig. 31). This report was unavailable, so the basis for this classification is uncertain. However, it appears to be the result of frost heave tests.

In the early 1970's Floss (1973) reported on the first FS classification system developed in West Germany based on a soil classification system.

Table 36. Frost susceptibility criteria according to Floss (1973).

Frost susceptibility	*Soil classification* (West German Standards)*	*Allowable amount (%) finer than 0.063 mm*
None	S, G, TA, HN, F	–
	SU, GU, ST, GT	8
Low-medium	OT, TM, TL, UL, UM	–
	SU, GU, ST, GT	20
High	OU	–
	SU, GU, ST, GT	40

*G = gravel, S = sand, U = silt, T = clay, O = organic, HN = peat, F = mud, A = high plasticity, M = medium plasticity, L = low plasticity

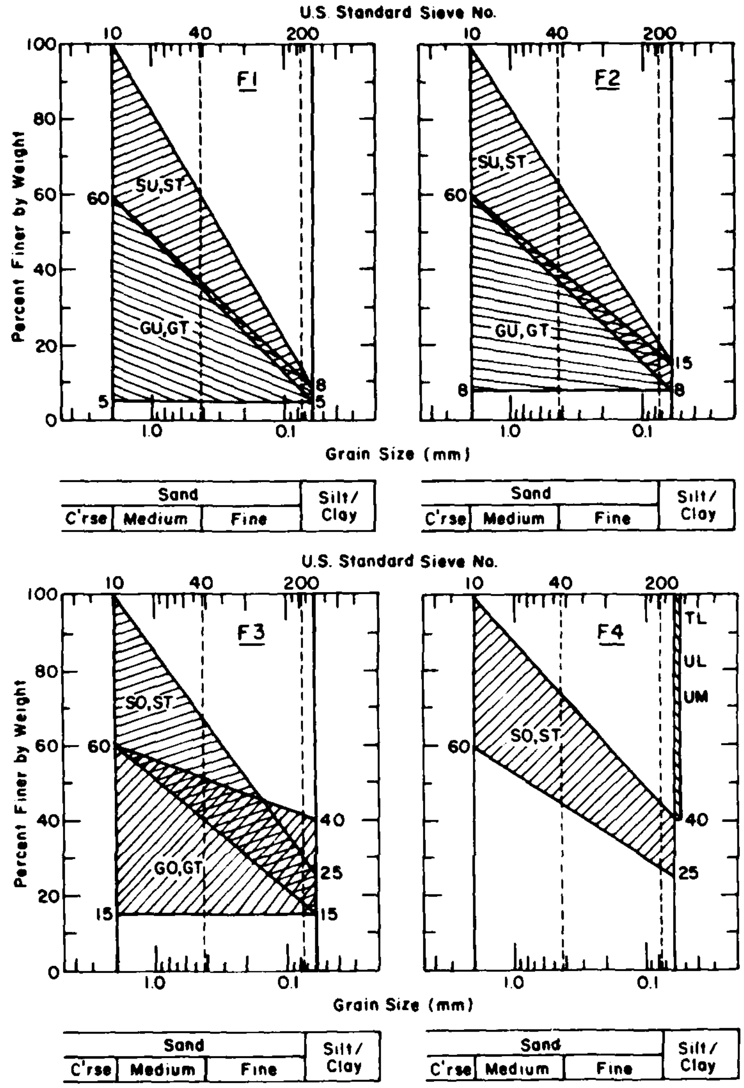


Figure 32. Ruhr University, Bochum, frost susceptibility criteria. F1 = non-frost-susceptible, F2 = slightly frost susceptible, F3 = moderately frost susceptible, F4 = highly frost susceptible; other abbreviations are defined in Table 37. (After Jessberger 1976.)

The Floss criteria were reported by Jessberger (1976) and are shown in Table 36. According to Jessberger, load-carrying capacity during thaw is considered in this classification. However, no details were given.

In the same report, Jessberger (1976) presented what appears to be a modification of the Floss (1973) FS criteria (Table 37, Fig. 32). This classification was developed at Ruhr University at Bochum and is referred to as the RUB system. It was developed from thaw-CBR values, not from frost heave as are most of the classification systems. The classification is broken into four groups of increasing FS according to the soil type, the percentage that is smaller than 0.06 mm, and the plasticity index. It appears that this FS classification system is a predecessor of the FS criteria currently being considered for adoption as a standard in West Germany. According to Jessberger, these criteria are less strict than the Schaible, Casagrande and U.S. Corps of Engineers criteria.

Table 37. Ruhr University at Bochum frost susceptibility criteria according to Jessberger (1976).

Frost susceptibility	*Soil classification* (West German Standards)*	*Amount (%) finer than 0.06 mm*
None	G, S	—
	SU, GU, ST, GT	<8
Low	TA	—
	SU, GU, ST, GT	8-15
Medium	TM, TL (PI >12)	—
	S$\bar{U}$, ST	15-25
	G$\bar{U}$, G$\bar{T}$	15-40
High	UL, UM, TL (PI <12)	—
	S$\bar{U}$, S$\bar{T}$	25-40

*G = gravel, S = sand, U = silt, T = clay, A = high plasticity, M = medium plasticity, L = low plasticity, $\bar{U}$ = very silty, $\bar{T}$ = very clayey

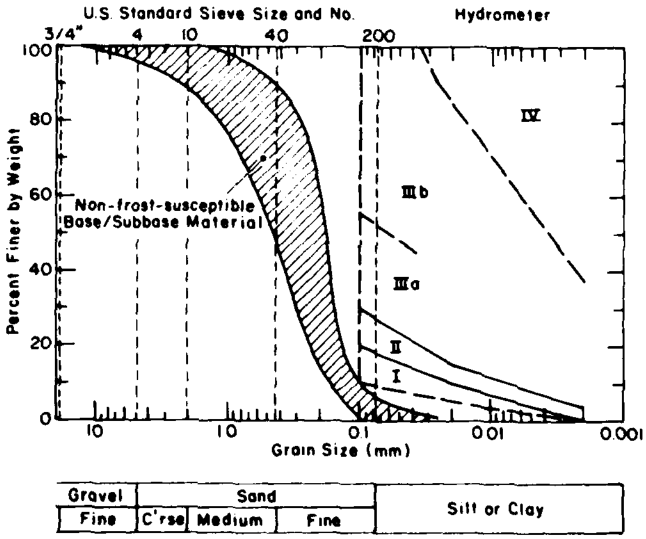


Figure 33. Limits of non-frost-susceptible base/subbase materials in W. Germany. (After Jessberger 1969.)

According to Jessberger (1969), the West Germans have been using a slightly modified form of Schaible's criteria (1957). Gravels are considered to be frost susceptible if 10% or more of their particles are smaller than 0.1 mm and the grain size distribution curve falls within the designated area for frost-susceptible soils in Figure 33. Sands are considered to be frost susceptible if the organic content is greater than 1%. The regions marked I, II, IIIa, IIIb, and IV are apparently regions of increasing frost susceptibility. However, no explanation was given by Jessberger.

The present stage of the West German FS criteria, which are now being considered for adoption as a standard, are shown in Table 38. The source of this table is an untranslated draft report (Germany 1979) provided by Jessberger; it has apparently evolved from Jessberger's work, the thaw-CBR value being an important factor.

Table 38. West German frost susceptibility criteria (Germany 1979).

Frost susceptibility	*Thaw CBR*	*Soil classification* (West German Standards)*	*Allowable amount (%) finer than 0.063 mm*
None	>20	GW, GI, GE, SW, SI, SE	5
		TA	—
		OT, OH	—
Low-medium	4-20	TM	—
		ST, GT	5 if $C_u > 15$, 15 if $C_u < 6$†
		SU, GU	5 if $C_u > 15$†, 15 if $C_u < 6$†
		TL	—
		UL, UM	—
High	<4	$S\bar{T}$, $G\bar{T}$	—
		$S\bar{U}$, $G\bar{U}$	—

*Listed in order of increasing frost susceptibility. G = gravel, S = sand, U = silt, T = clay, O = organic, H = peat, A = high plasticity, M = medium plasticity, W = well-graded, I = intermediate gradation, E = poorly graded, $\bar{T}$ = very clayey, $\bar{U}$ = very silty.

† If $6 < C_u < 15$, then the allowable amount finer than 0.063 mm should be linearly interpreted between 5 and 15%.

Pore size tests

The importance of pore size to frost action was recognized long ago by Taber (1929). Penner (1959) also recognized that pore size strongly affects the FS of soils. However, Csathy and Townsend (1962) and Townsend and Csathy (1963b) *were the first to express this soil property quantitatively* and to include it in a FS criterion. Since then, Guillot (1963), Gaskin and Raymond (1973), Reed (1977) and Reed et al. (1979) have also suggested using pore size as an index of FS. Each of these proposals is examined in the next paragraphs.

Csathy and Townsend determined pore size distribution in the laboratory using a capillary method. Their technique involved allowing water to rise by capillarity in a soil column until it reached 160 cm or until 35 days passed. The water content is determined at various heights above the water table. The degree of saturation versus the height above the water table is then calculated, and the maximum pore diameter d that is still filled with water at any particular height h is determined from the surface tension equation:

$$d = 4\sigma_{a,w}/h \tag{6}$$

where $\sigma_{a,w}$ is the surface tension at an air/water interface. A plot of the pore size distribution can then be made. Figure 34 illustrates this process.

Csathy and Townsend compared the pore size distribution data with field frost performance for 39 soil samples taken from 30 locations. They found that the slope of the pore size distribution curve between the 90% (P_{90}) and 70% (P_{70}) limits generally became steeper with increasing FS. Using the notation $P_u = P_{90}/P_{70}$, they established that when $P_u < 6$, the soil was non-frost-susceptible.

Csathy and Townsend compared the reliability of this method with numerous grain size distribution methods and found that it was significantly more reliable in determining the FS of soils.

According to Jessberger (1969), Guillot (1963) has also proposed a pore size distribution criterion. However, no details were provided and Guillot's report was not available for review.

Because of the time required for the Csathy and Townsend capillary rise test (up to 35 days), Gaskin and Raymond (1973) evaluated two other methods: the pressure-plate suction test and the mercury-intrusion test. They compared the effectiveness of all three methods with actual field observations of frost heave for 36 soil samples.

The pressure-plate suction device (Fig. 35) was obtained from a commercial source. The method uses successively increasing and decreasing pressure differentials (up to 100 cm of water) across wafers of soil to determine the relationships of drying and wetting moisture content ver-

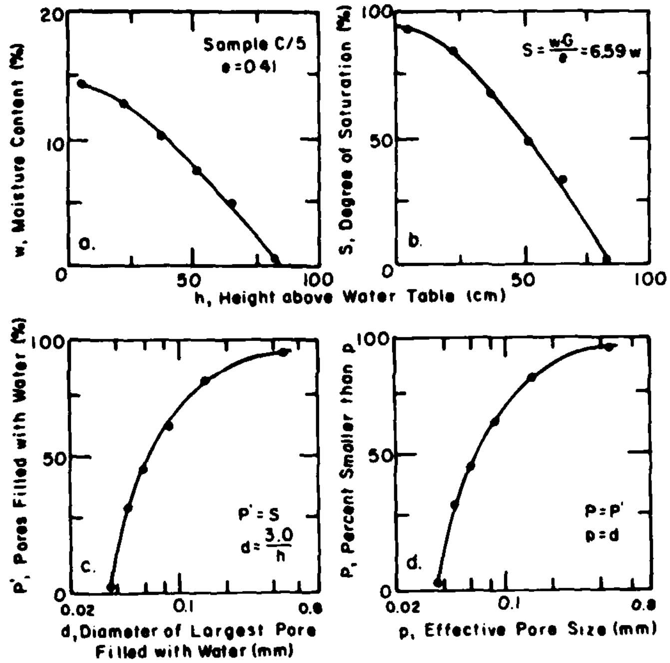


Figure 34. Determination of pore size distribution curve. (After Csathy and Townsend 1962.)

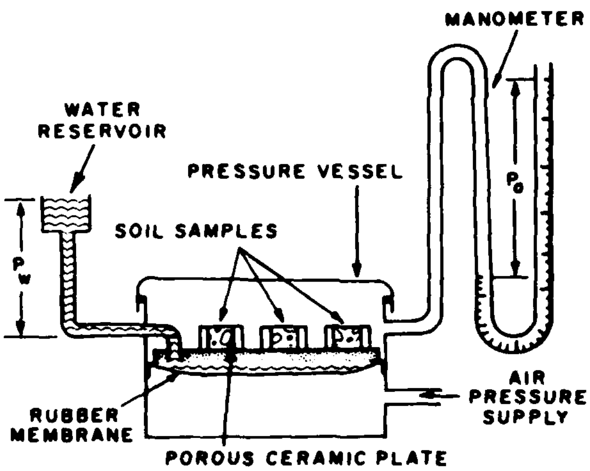


Figure 35. Pressure-plate suction test apparatus. (From Gaskin and Raymond [1973], courtesy of the Organization for Economic Cooperation and Development.)

sus pressure. At each pressure differential 2-5 days are required for the moisture to reach equilibrium. A specific pore diameter is calculated for each pressure differential using the surface tension equation, and the pore-size distribution curve is constructed.

A mercury-intrusion test device (Fig. 36) was also obtained commercially. However, it was modified to increase its capacity from 0.3 to 20 cm³ of soil. This method requires dry soil. The volume of mercury that is intruded into the sample is measured at successively increasing pressure increments. Pore size is calculated using the surface tension equation; however, in this case

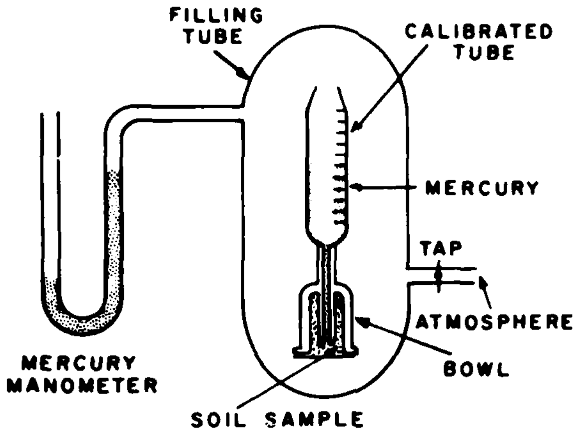


Figure 36. Mercury-intrusion test apparatus. (From Gaskin and Raymond [1973], courtesy of the Organization for Economic Cooperation and Development.)

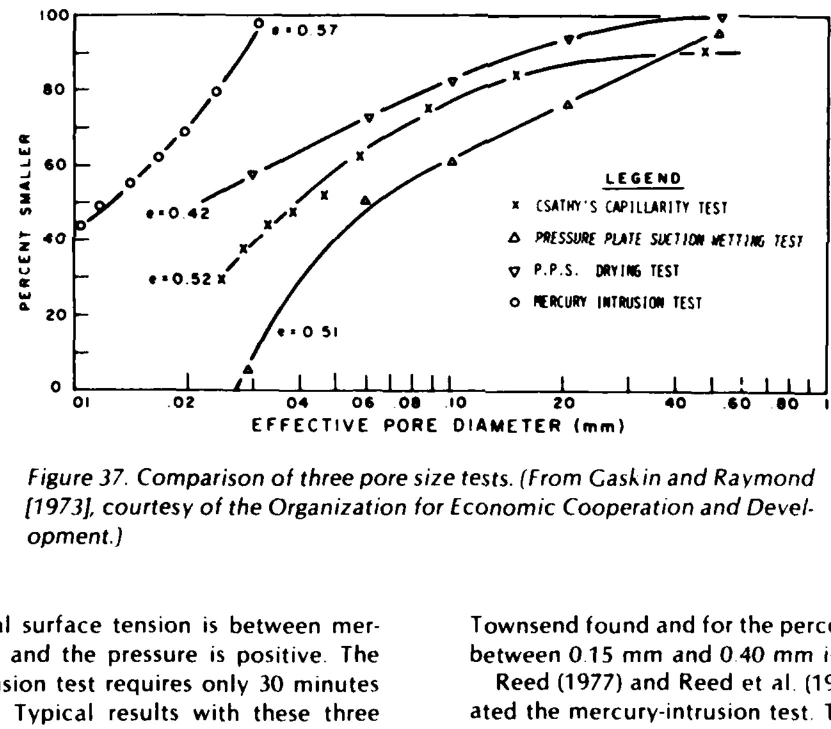


Figure 37. Comparison of three pore size tests. (From Gaskin and Raymond [1973], courtesy of the Organization for Economic Cooperation and Development.)

the interfacial surface tension is between mercury and air, and the pressure is positive. The mercury-intrusion test requires only 30 minutes to complete. Typical results with these three methods are shown in Figure 37.

For each test Gaskin and Raymond determined ratios of the percentage of pores less than a given diameter to the percentage of pores between certain sizes. They compared these ratios with field frost heave performance and found a high degree of correlation for only the capillary rise test. Correlations with frost heave were obtained for the same P_{90}/P_{70} ratio that Csathy and Townsend found and for the percentage of pores between 0.15 mm and 0.40 mm in diameter.

Reed (1977) and Reed et al. (1979) also evaluated the mercury-intrusion test. They compared their pore size distribution curves with heave rate data obtained from rapid frost heave tests conducted on saturated compacted samples. Fixed top and bottom temperatures of -6° and +4°C, respectively, were applied to samples with a 3.3-kPa surcharge, and heave was observed for two days. To obtain the dry specimens required for the mercury-intrusion test, samples were cut from freeze-dried, compacted samples. This process took ten hours.

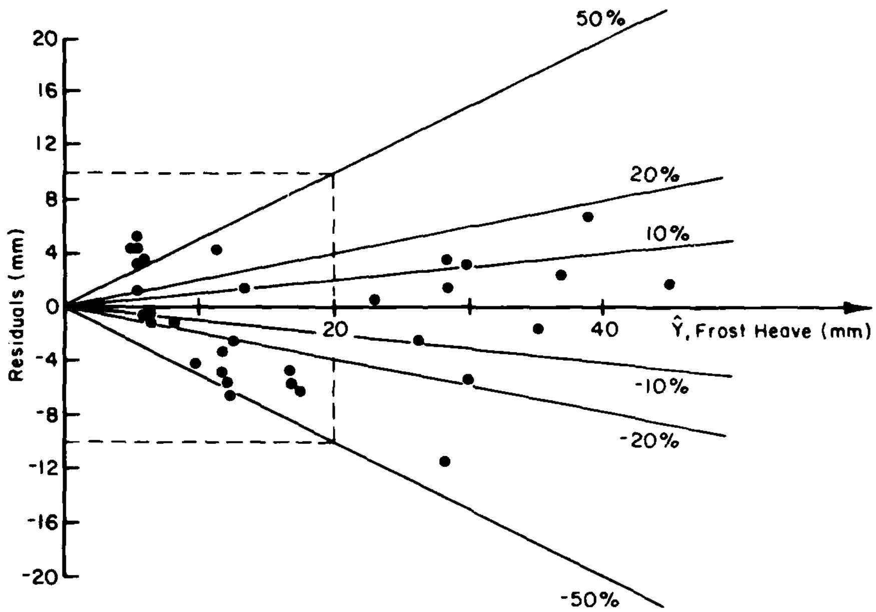


Figure 38. Differences (residuals) between measured and predicted frost heaves $\hat{Y}$. (After Reed et al. 1979.)

Many pore size parameters were tested for correlations. The best correlation was found between the cumulative porosity and the rate of frost heave $\hat{Y}$, which is given by the following equation:

$$\hat{Y} = -5.46 - [29.46(X_{3.0})]/(X_0 - X_{0.4}) + 581.1(X_{3.0}) \quad (7)$$

where $X_{3.0}$ = cumulative porosity for pores between 3.0 and 30 μm

X_0 = total cumulative porosity

$X_{0.4}$ = cumulative porosity for pores between 0.4 and 300 μm.

Figure 38 shows the differences between observed frost heaves and those determined by eq 7.

Soil/water interaction tests

Included in soil/water interaction tests are 1) moisture-tension tests, 2) capillary rise tests, 3) saturated hydraulic conductivity tests, 4) unsaturated hydraulic conductivity tests, and 5) centrifuge moisture content tests. These tests all rely on the interaction of soil and water; because they address both, they are one step closer than the pore size distribution tests to the factors affecting frost heave.

Moisture-tension tests

Air intrusion. Williams (1966) has proposed that air intrusion values obtained from moisture-tension curves can be used to determine the FS of soils. His apparatus is similar to a conventional pressure membrane device, but it has a much higher permeability (5×10^{-6} versus 2×10^{-10} cm/sec). Samples must be saturated and degassed. They are placed in a plexiglass ring on a membrane filter in the cell, the base of which is connected to a water column. The air pressure is raised in increments applied over several minutes; the drainage at each increment is recorded. At a certain pressure increment there is a sharp acceleration of drainage (Fig. 39). This pressure is defined as the air entry value. A typical test takes only one or two hours.

Williams suggested that the air intrusion value is related to the characteristic size of the largest continuous opening. He found that for four natural clay and silt soils and six graded fractions prepared from silt, the air intrusion value is directly related to the pore-water pressure at a penetrating frost line, i.e.

$$(p_a - p_u)/\sigma_{a,w} = (p_i - p_u)/\sigma_{i,w}. \quad (8)$$

The values for the variables on the left side of the equation are determined from a moisture tension test; those on the right are determined from a freezing test.

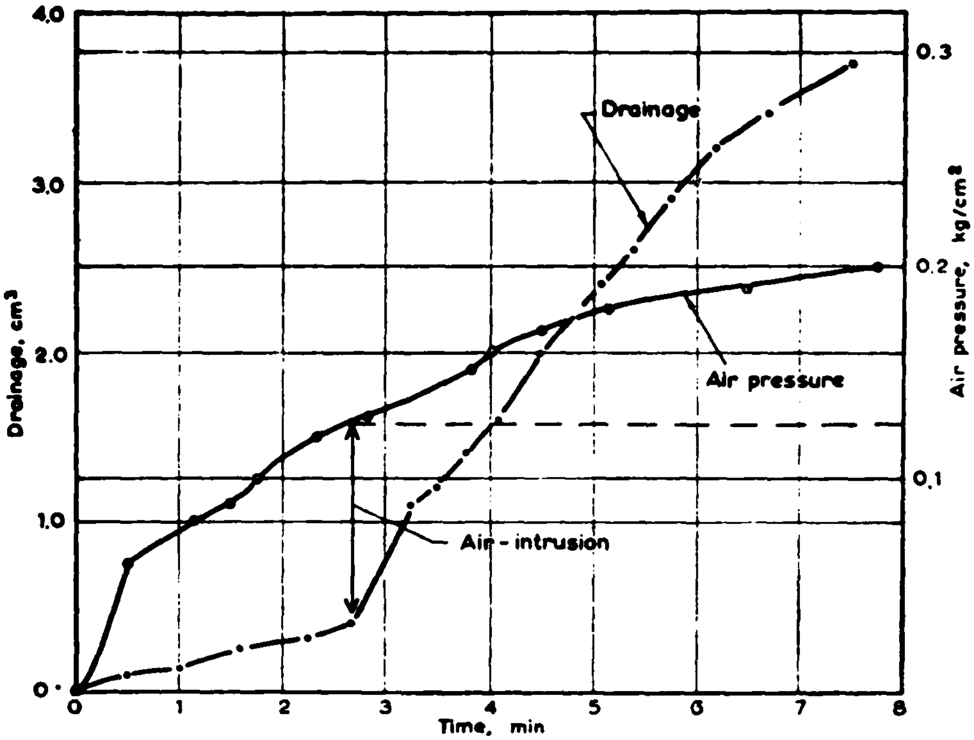


Figure 39. Test observations during measurement of the air intrusion value of silt. (From Williams [1966], courtesy of the Institution of Civil Engineers.)

Williams concluded that the air intrusion value can be used to determine the susceptibility of soils to frost heave. He did not propose FS criteria but suggested that for a particular problem, the maximum value of u_i (estimated from the air entry value $p_a - u_a$ using the above equation, assuming that p_i is the overburden pressure) be compared with the in situ value of the suction pressure near the frost line. If the suction pressure due to freezing is greater than the in situ value, then frost heave will occur. In situ values of suction u_x can be obtained directly from field measurements or laboratory tests. Williams also suggested that u_x can be estimated from the equation

$$u_x = (-d + x)/1000 \quad \text{kg/cm}^2 \qquad (9)$$

where d is the depth to the water table (cm) and x is the depth (cm) where the suction is measured.

Osmotic suction. Jones and Hurt (1978) suggested that an osmotic-suction technique can provide a simple and rapid methoo , ' determining the FS of coarse-grained materials from moisture-tension curves. Their apparatus is illustrated in Figure 40. Suction is applied to satu-

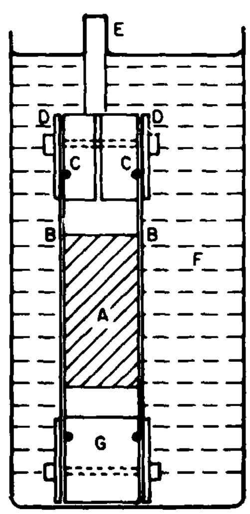


Figure 40. Osmotic suction apparatus. (After Jones and Hurt 1978.)

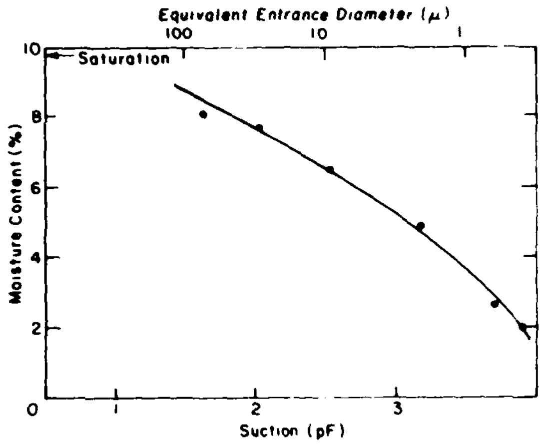


Figure 41. Drying moisture-tension curve for a compacted dolomite aggregate. (After Jones and Hurt 1978.)

rated degassed specimens through a semipermeable membrane by the osmotic pressure of an aqueous solution of polyethylene glycol. Osmotic suctions of up to 25 bars can be obtained by varying the concentration of the solution. The technique allows the aggregate suction characteristics to be measured at suctions up to 25 bars. Rock suction characteristics can also be determined with this apparatus. Typical results are shown in Figure 41 for a compacted dolomite aggregrate. As there is no well-defined air entry break in the curve, Jones and Hurt suggested that the aggregate's FS be ranked according to the suction value at 70% saturation. No classification method was given.

This moisture-tension method is the best for aggregates, as it avoids the problems of splitting membranes and long moisture equilibrium times that occur with the air-intrusion test.

Capillary rise tests

Maag (1966) has proposed a "physical frost criterion" based on capillary rise *H*, permeability, and height above the water table *h*. According to Jessberger (1969), Maag related the amount of water transported to the permeability and the *H*/*h* ratio. The effect of freezing was not considered. Maag's report was not available for review.

Saturated hydraulic conductivity tests

According to Johnson (1980), Onalp (1970) proposed that saturated hydraulic conductivity be used as an indicator of FS. The suggested classification is given in Table 39.

Table 39. Frost susceptibility classification according to Onalp (1970).

Frost susceptibility	Saturated hydraulic conductivity (cm/sec)
Borderline	$1.0 \times 10^{[illegible]} < k < 1.3 \times 10^{[illegible]}$
Frost susceptible	$1.3 \times 10^{[illegible]} < k < 1.7 \times 10^{-4}$
Borderline	$1.7 \times 10^{-4} < k < 1.0 \times 10^{-3}$
None	$1.0 \times 10^{-3} < k < 1.0 \times 10^{-2}$

Unsaturated hydraulic conductivity tests

Wissa et al. (1972) have proposed that both the unsaturated hydraulic conductivities and the air entry suction values can be used to characterize the FS of soils. Their apparatus is illustrated in Figure 42. Compacted specimens can be tested at suctions of up to 6 bars. Saturation can be ensured by back-pressuring up to 7 bars. Moisture-tension relationships are obtained by monitoring the volume of water flowing out of the cell at successively increasing pressure increments. After moisture-tension equilibrium is established for each pressure increment, the hydraulic conductivity values are determined by forcing water through the sample and monitoring the outflow and the pressure drop across two piezometers placed in the sample. A typical test can be completed in three days. Darcy's law is used to calculate the hydraulic conductivity. Permeabilities between 10^{-2} and 10^{-9} cm/sec can be measured. Figure 43 shows the results for a silt. After an evaluation of 33 soil tests and a comparison of the results with laboratory frost heave tests, it was determined that the product of the hydraulic conductivity at the air entry pressure K_c and the air entry pressure itself V_c characterized the degree of FS. The resulting classification system is given in Table 40.

Table 40. Frost susceptibility classification according to Wissa et al. (1972).

Frost susceptibility	$(K_c \times V_c) \times 10^7$ (kg/cm·s)
Severe	>20
High	4-20
Medium	1-4
Low	0.2-1
Very low	< 0.2

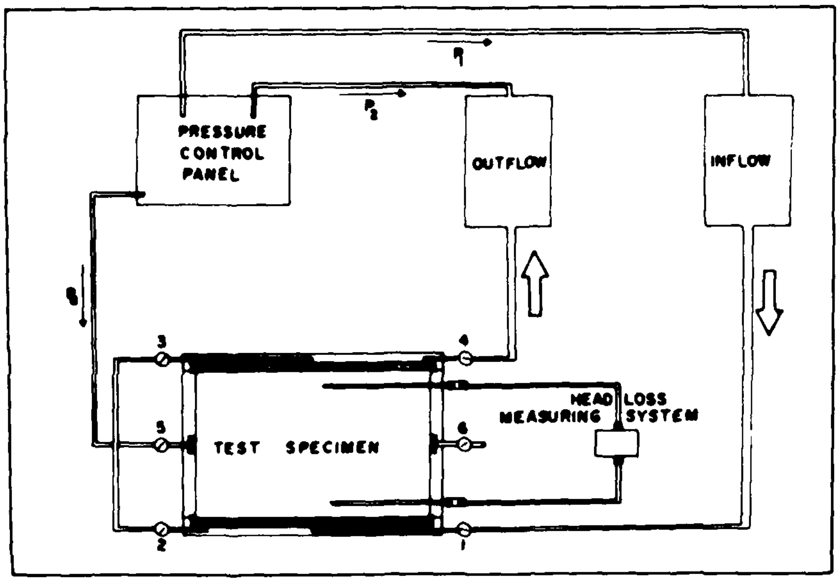


Figure 42. Schematic of permeability apparatus. (From Wissa et al. 1972.)

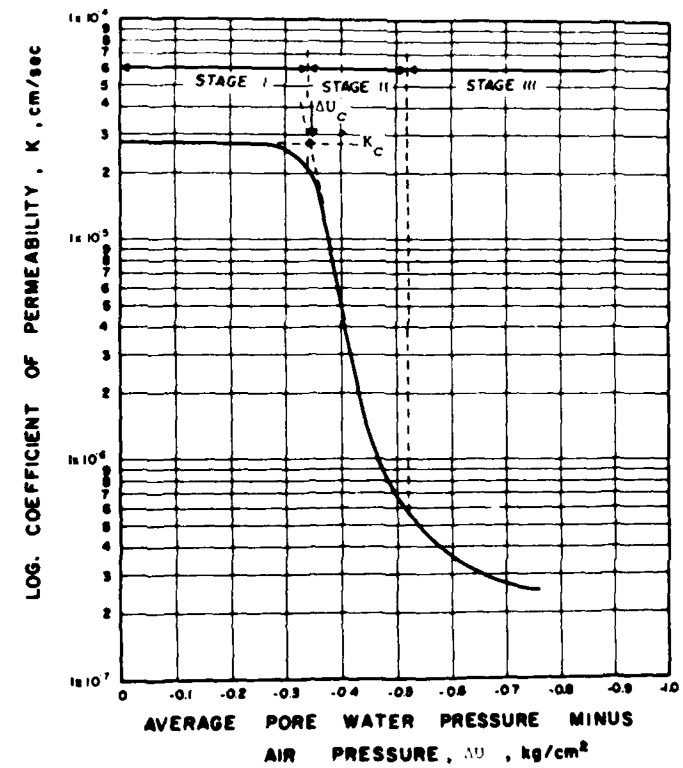


Figure 43. Typical relation between permeability and pore water pressure. (From Wissa et al. 1972.)

Centrifuge moisture content tests

Willis (1930) concluded that non-plastic sandy soils that have centrifuge moisture equivalents less than 12 or clay soils with liquid limits greater than 50%, plasticity indexes appreciably greater than the ratio (LL-14)/1.6, and shrinkage limits that do not greatly exceed 21.0-1.1[LL-(LL^2/800)] are not sensitive to frost heave. Unfortunately, the method for determining the centrifuge moisture equivalent is not known.

Soil/water/ice interaction tests

Tests that fall into the soil/water/ice interaction category are those that involve freezing soils but not measuring frost heave or thaw weakening. Some other quantity is measured to characterize FS. Tests of this type measure 1) frost heave stress or 2) pore-water suction.

Frost heave stress

Frost heave stress has been linked to FS for many years. Penner (1959) reported that frost heaving pressure is a function of dry density for a single material. Hoekstra et al. (1965) observed that the maximum pressure that develops during restrained freezing has a characteristic value for each soil. The apparatus for determining this value is illustrated in Figure 44. Saturated compacted soils are frozen from the top down, with free access to water at the base. Frost heave pressures are observed by means of a load cell placed on the upper cooling plate. Thermoelectric cooling devices are used to freeze the samples. Figure 45 shows the heave pressure results for several soils. Hoekstra and Chamberlain (1965) suggested FS criteria based on the maximum heave pressure (Fig. 46) developed at a stationary freezing front.

Penner (1966, 1967, 1968) concluded that the frost heaving pressures of soils can be directly related to the pore size of granular soils by eq 1. If the soil is saturated, the pore water pressure is zero when the freezing zone is just below the water table. Furthermore, if the soil is incompressible and is restrained from heaving, the ice pressure becomes the maximum heaving pressure. This is the same argument made by Hoekstra et al. (1965).

Researchers at the Massachusetts Institute of Technology (Wissa and Martin 1968, Shrestha 1971, Martin and Wissa 1973, and Olsen et al. 1974) were the first to make recommendations on how to conduct frost heave stress tests and how to use the heave stress data to predict FS. Their apparatus (Fig. 47) is essentially the same

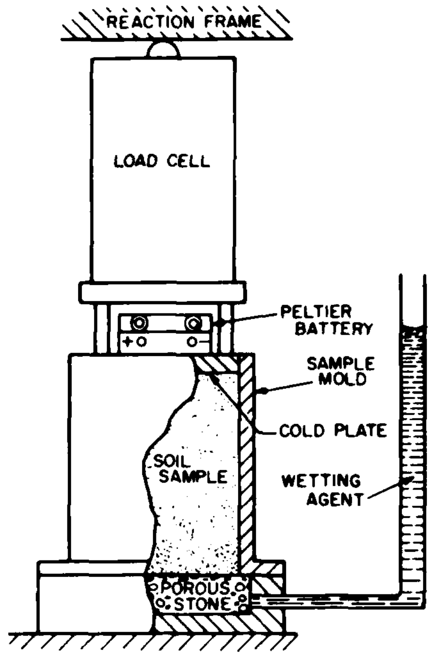


Figure 44. Schematic drawing of freezing chamber. (From Hoekstra et al. 1965.)

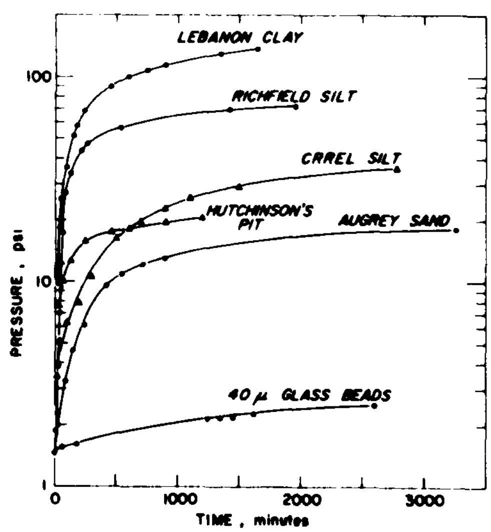


Figure 45. Pressure vs time for several soils. (From Hoekstra et al. 1965.)

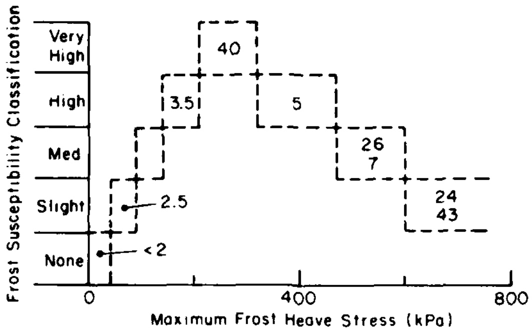


Figure 46. Frost susceptibility criteria based on frost heave stress. The numbers in the boxes refer to the percentage finer than 0.02 mm. (After Hoekstra and Chamberlain 1965.)

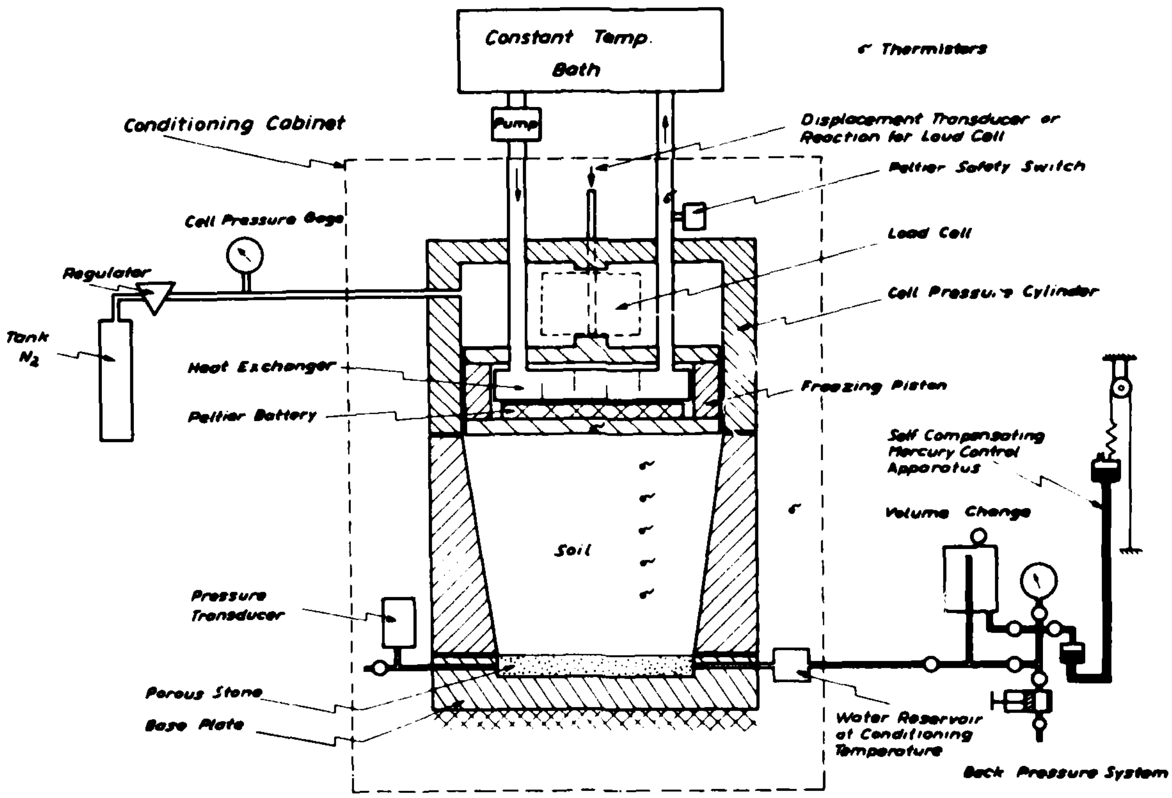


Figure 47. Schematic diagram of frost testing equipment developed at MIT. (From Wissa and Martin 1968.)

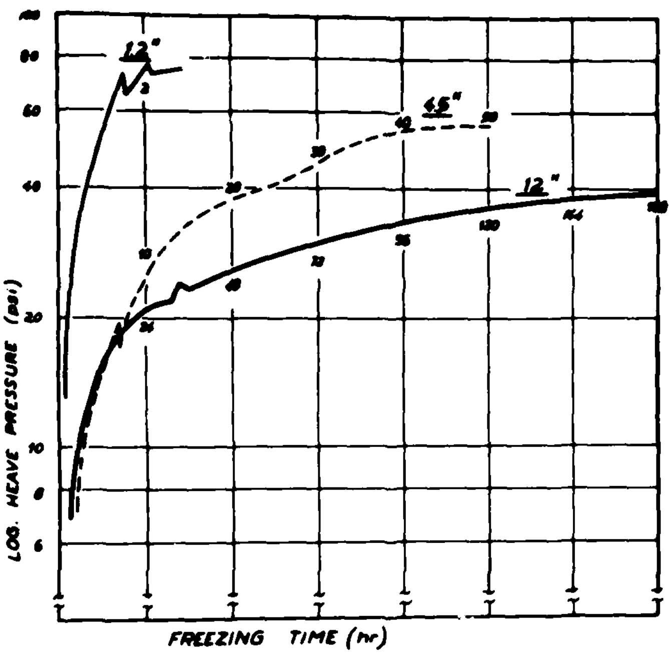


Figure 48. The development of heave pressure during freezing of New Hampshire silt samples of different lengths. (From Wissa and Martin 1968.)

as that employed by Hoekstra et al. (1965). This test is predicated on the concept that the maximum heave stress develops under steady state heat flow conditions at a stationary freezing front under a constant temperature gradient. The test is an open-system test (water is free to flow into and out of the soil) conducted at constant volume. Friction along the sample side is minimized by using a tapered mold, while friction between the upper cooling piston and the mold is minimized with a greased rubber membrane. The force required to keep the sample at constant volume is the heave stress.

Typical results illustrating the logarithm of the frost heave stress as a function of time are shown in Figure 48. Wissa and Martin proposed that the slopes R of the straight line portions of these curves are characteristic of FS. They later modified this statement (Olsen et al. 1974) to state that concave curves give only a lower bound to the correct R. They stated further that R values are not unique to the soil condition but are functions of the temperature. Similar observations have been made by Saetersdal (1973) for the maximum heave pressure. Wissa and Martin concluded that it is essential that the temperature be standardized and that this be done for the soil deemed most frost susceptible based on field performance.

Pore-water suction

The MIT researchers (Quinn 1968, Wissa and Martin 1968, Nussbaumer 1972, and Martin and Wissa 1973) also evaluated the use of the pore-water pressure change that occurs below the freezing front as an indicator of FS. Their equipment is similar to that of their heave stress test, except for a few accessories that monitor pore-water pressure. Water in the test specimen is back-pressured to prevent cavitation during freezing. The reduction in pore pressure and the heave stress are measured when the temperature gradient is constant and the freezing front stationary.

Saetersdal (1973) also evaluated the use of the suction below the freezing front and found it to be greatly dependent on the rate of freezing.

Riddle (1973) also studied the use of the suction that develops during freezing as an indicator of FS. Figure 49 illustrates his apparatus. No details were given about the dimensions of the apparatus or about the temperature conditions imposed. Samples are frozen unidirectionally and very rapidly. Maximum pore-water suction

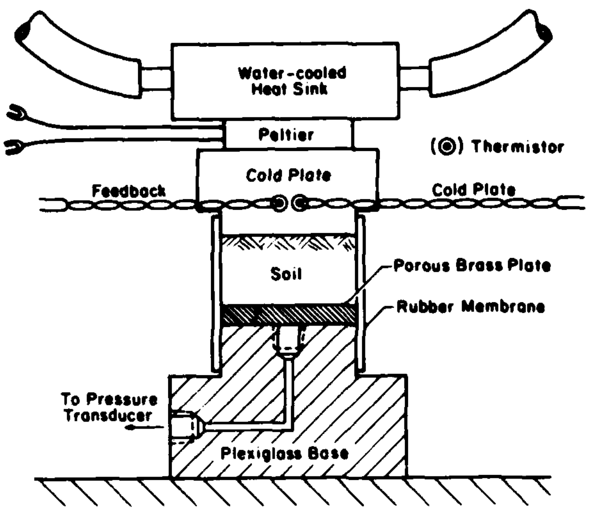


Figure 49. Pore water suction test apparatus. (After Riddle 1973.)

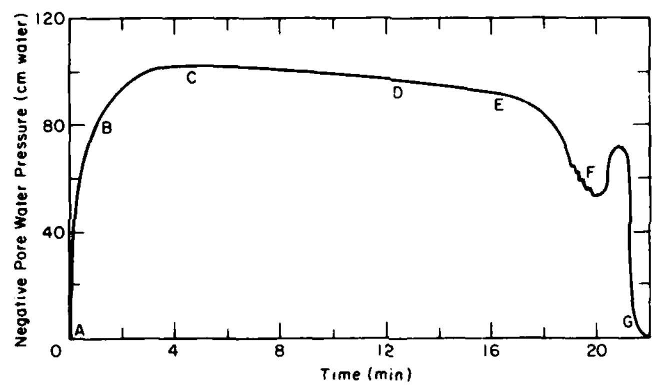


Figure 50. Typical suction vs time curve for a silty sand frozen at -5°C (cooling plate temp.). (After Riddle 1973.)

A—Mechanically induced nucleation results in rapid ice growth of supercooled water, which produces a small pressure increase and the liberation of heat (the latent heat of fusion).

B—Frost line is just entering soil sample. The greater the impermeability of the soil, the greater the lag time for the suction to be felt by the transducer.

C—Frost line has advanced about a quarter of the way through the soil sample. The maximum suction plateau has been reached.

D—Frost line had advanced about three-quarters of the way through the soil specimen with a slight loss of suction being recorded. The reason is not yet clear; however, it may be caused by migrating fines.

E—Sample completely frozen through and frost line now beginning to penetrate the large pores of the porous brass filter plate, which produces a marked loss in suction.

F—Heating and the resultant thawing of the soil specimen is achieved by reversing the leads to the thermo-electric unit from the Peltier module, which initially produces an increase in suction due to the volume decrease when ice changes to water.

G—Complete thawing of the ice soil sample results in rapid loss in suction back to atmospheric pressure.

data were usually obtained in less than 30 min with this method. Figure 50 shows a typical suction versus time curve for a silty sand and an explanation of specific features of this curve. The maximum suction occurred within 2 min of nucleation, making this the most rapid index test reviewed. Riddle gives no classification scheme; Table 41 is interpreted from his test results. More details of this test must be obtained before it can be adequately reviewed. However, from Riddata it appears that no back-pressuring system was used. If this is true, the test is limited to soils with pore-water suctions no greater than the cavitation pressure of water, 100 kPa. Perhaps Riddle assumed that soils that generate more than 100 kPa of suction will be non-frost-susceptible because of low permeability.

Table 41. Frost susceptibility classification interpreted from Riddle (1973).

*Frost susceptibility**	*Average soil suction (kPa)*
Negligible	0-10
Slight	10-20
Moderate	20-50
High	>50

*The basis for the frost heave classification is not known.

Frost heave tests

Frost heave tests are perhaps the most direct laboratory method of assessing the FS of soils. Three types of laboratory frost heave tests have been conducted. One involves one or more step changes in the cold-side temperature and observations of heave with time as thermal equilibrium is established, the second uses a steadily decreasing cold-plate temperature and a constant rate of frost penetration, and the third uses a constant rate of heat removal. Appendix C lists by country the tests found in the literature along with some of their features. Each of these tests is discussed briefly in the following paragraphs.

Austria

Brandl (1970) proposed a large-scale frost heave test to determine the FS of gravels. Compacted samples are contained in a multi-ring mold with an i.d. of 30 cm and a height of 50 cm. Figure 51 illustrates the test apparatus. A surcharge of 10 cm of asphalt concrete is placed on the upper surface of the test specimen. Samples are subjected to rigorous freezing tests under different moisture conditions. A typical test involves placing the specimen in a freezing cabinet, lowering the air temperature to -24°C for 24 hours, and raising the air temperature to +20°C; this process is repeated ten times and then the sample is kept at -24°C for 10 days. In some tests water was freely available at the sample base; in others water percolated from the top down. Figure 52 illustrates typical results. Brandl did not offer a FS classification system, as his work pertained only to individual gravels. He did conclude, though, that frost heave can be excessive in well-graded gravels if the amount less than 0.02 mm exceeds 5-6%.

More recently Brandl (1980) proposed a scaled-down frost heave test to serve as a standard for determining the FS of soils and granular materials when his mineral and grain size criteria are inconclusive. Samples are compacted in a 12.5-cm-diameter by 15-cm-high CBR mold. Details of the sample confinement during freezing were not provided; however, it is assumed that Brandl has continued to use the multi-ring mold. Samples are frozen in a freezing cabinet at -15°C and are thawed at +20°C. The base is maintained at +4°C and water is freely available during freezing and thawing. A surcharge of 5 kPa is

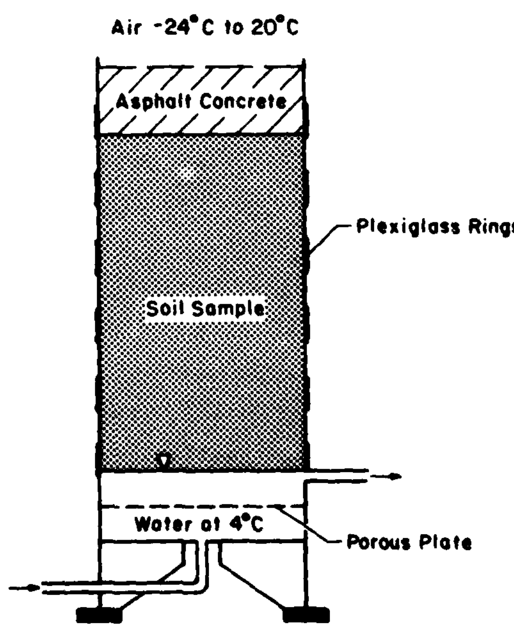


Figure 51. Frost heave test apparatus; a) 10 cm of asphalt concrete, b) sample. (After Brandl 1970.)

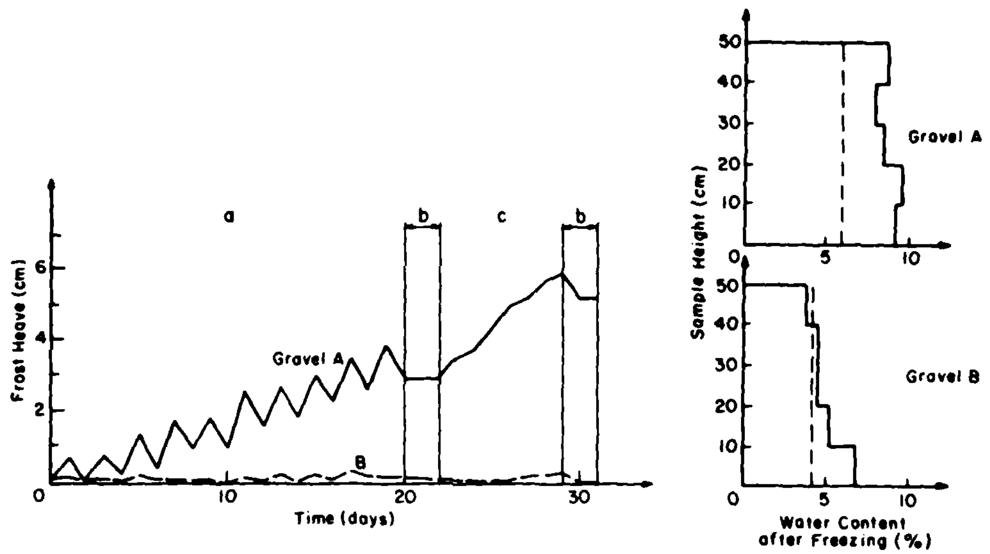


Figure 52. Example of results for two gravels; a) 10 freeze–thaw cycles, b) thaw, c) freeze. (After Brandl 1970.)

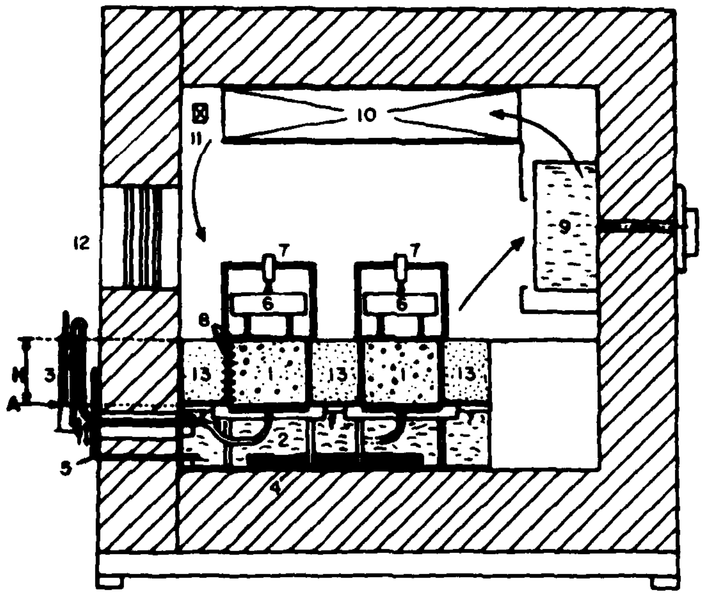


Figure 53. Schematic of Belgian Road Research Center frost susceptibility apparatus. (From Corlé 1980.)

1. sample
2. water reservoir
3. measuring cylinder
4. heater
5. thermometer
6. load (3.4-kPa surcharge)
7. displacement transducer
8. thermocouples
9. ventilator
10. refrigerator
11. heater
12. window
13. thermal insulation

maintained throughout the freeze-thaw cycling. Two freeze-thaw cycles are imposed for secondary road studies and four are imposed for main highways. The test takes 16 days for the former and 21 days for the latter. The maximum allowable frost heave is 1-2 cm for secondary roads and 2.5 cm for main highways. The minimum CBR value allowable is 20-25% for any type of road.

Belgium

Gorlé (1980) has developed a direct frost heave test at the Road Research Center in Belgium to evaluate the effects of the principal variables affecting frost heave. His apparatus (Fig. 53) includes a multi-ring freezing cell with an i.d. of 15.24 cm and a height of 12.7 cm. Each ring is 0.5 cm high. The sample can be saturated and wall friction kept to a minimum. No other details were given.

The samples are frozen from the top down, with water freely available at the base. The air temperature at the top and the water temperature at the base are kept constant throughout a test. The final temperatures are varied from test to test to evaluate the influence of freezing rate and temperature gradient. A surcharge of 3.4 kPa is placed on all samples.

No details on compaction were given; however, it is clear that the samples are saturated and stored at the base temperature for 48 hours before freezing. The freezing period lasts 24 hours, during which the heave, the heave rate, the water inflow rate, the temperature profile and the frost penetration rate are measured.

Gorlé did not report a FS classification system. However, he suggested that either the frost heave ratio or the ice segregatation ratio (the volume of ice to the volume of frozen soil) be used as an indicator of FS.

Canada

Penner and Ueda (1978) described a frost cell developed by the Northern Engineering Service Company, Limited, Calgary, Alberta, to determine shut-off heave pressures. A feature of this frost cell is that freezing is imposed from the bottom up to minimize heave restraint.

The test cell (Fig. 54) contains a sample 10.2 cm long by 10.2 cm in diameter. Water flows freely through a porous disk in the load piston. The sample is loaded by pressurizing the air chamber mounted on top of the freezing cell (the surcharge pressures were varied between 0.5 and 5.0 kg/cm²). Freezing is induced by circulating a methanol-water solution through a heat exchanger in the base of the cell. The piston temperature is determined by the air temperature in the cold chamber in which the tests are conducted.

Penner and Ueda (1978) observed that for a step change in the cold-plate temperature, the relation between frost heave and time is linear for periods up to three or four days; this relation is independent of frost penetration rate but dependent on overburden pressure. They did not propose a FS classification based on frost heave rate, but suggested that the scale of heave rates developed by Kaplar (1974) at the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) is acceptable.

Penner and Ueda concluded that the bottom-up freezing test is better than the CRREL test in that 1) there is no wall adfreeze problem, and thus there is no need for a tapered cell or movable rings, and 2) the test can be conducted in a much shorter time.

Penner and Ueda emphasized that the heat extraction rate and not the frost penetration rate is the fundamental parameter in the freezing process and that the rates of heat extraction used in the laboratory should be related to those in the field. Because of the variability in field conditions, he suggested that two tests be carried out at heat extraction rates bracketing the expected field values.

Penner (1978) also suggested that heave rates can be interpreted on the basis of cold-side temperature T_c and overburden pressure P with the equation

$$dh_{TOT}/dt = a \exp^{-bP/T_c} \tag{10}$$

where dh_{TOT}/dt is the total heave rate and a and b are coefficients determined by regression analysis. They observed that the warm-side temperature has little effect on the heave rate. They argued that under a constant surcharge pressure, the cold-side temperature alone determines the suction potential at the growing ice lens and thus controls the heave rate.

While the cold-side temperature may be a factor, it is only an indicator of something more fundamental. When Penner and Ueda's heave-rate data for constant pressure are compared to the temperature gradient in the frozen soil (Fig. 55), it does indeed appear that the cold-side temperature determines the heave rate. However, it

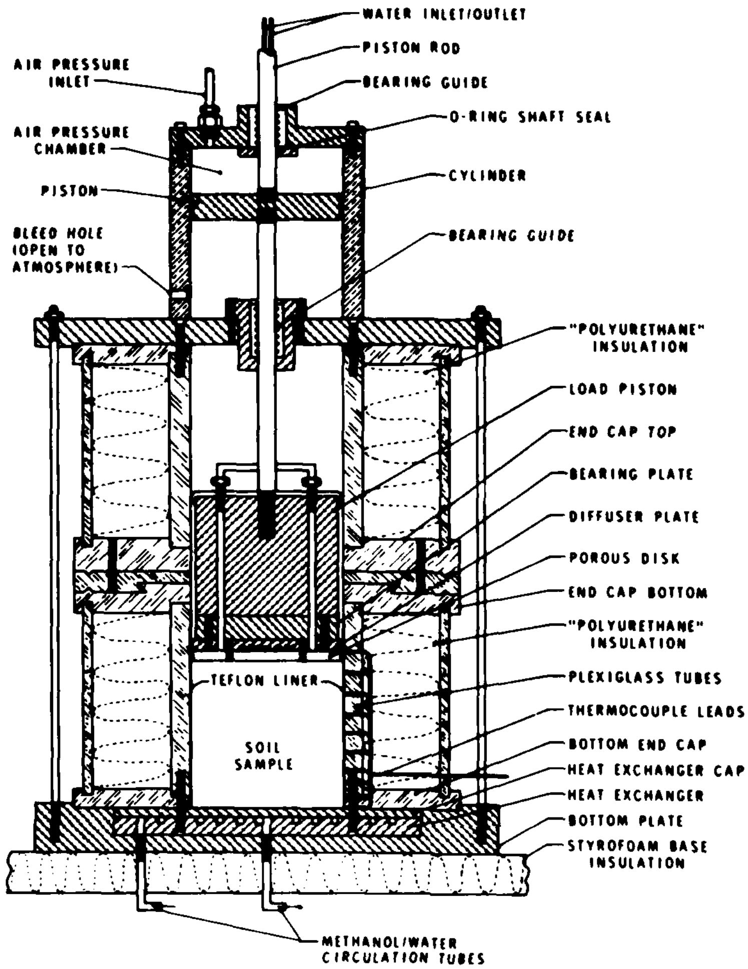


Figure 54. Frost heave test cell. (From Penner and Ueda 1977.)

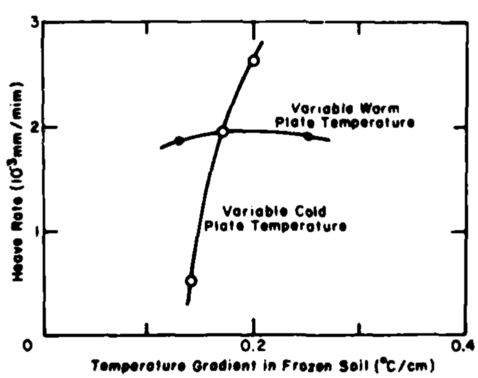


Figure 55. Penner's (1978) heave rate data vs the temperature gradient in frozen soil. ○—warm plate temperature ≈ 2.3°C. •—cold plate temperature ≈ 0.9°C.

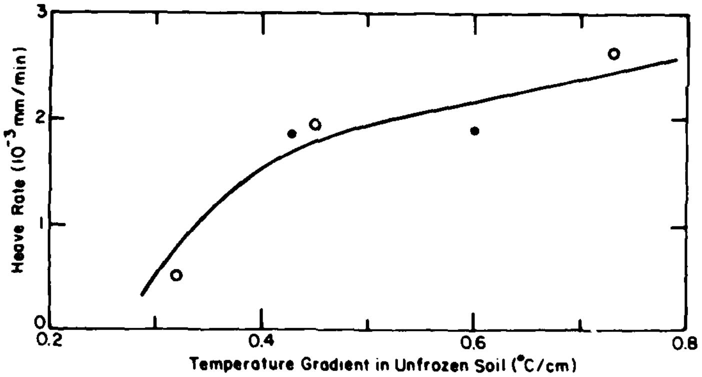


Figure 56. Penner's (1978) heave rate data vs the temperature gradient in unfrozen soil. ○—warm plate temperature ≈ 2.3°C. •—cold plate temperature ≈ -0.9°C.

seems doubtful that the temperature gradient in the frozen soil is a determining factor in frost heave. If the frozen fringe concept is valid, and Penner and Ueda concluded that it is, then it appears more likely that only the temperature gradient within the frozen fringe influences the heave rate. Figure 56, which is reconstructed from Penner and Ueda's data, is a plot of the heave rate versus the temperature gradient in the unfrozen soil. The assumption is that the temperature gradient in the frozen fringe is more like the temperature gradient in unfrozen soil than that in frozen soil. As can be seen in Figure 56, the heave rate is related to the temperature gradient in the unfrozen soil, assuming that the scatter is due to experimental error and errors in reconstructing Penner and Ueda's data. It is more appropriate, then, to relate the heaving rate to the temperature gradient in the unfrozen soil than to the cold-side temperature.

England

A laboratory frost heave test was developed at the Transport and Road Research Laboratory (TRRL) in the 1940's (Croney and Jacobs 1967) and has been used since 1969 as a compliance requirement for soils in British road construction (TRRL 1977). Compacted cylindrical samples (10.2 cm in diameter and 15.2 cm long) are frozen unidirectionally with one end in contact with water maintained at +4°C. The samples are contained by a stiff waxed paper sheet to minimize heave restraint. Nine samples are placed in a cabinet (Fig. 57) to soak at room temperature for 24 hours; the space between the samples is filled with a coarse dry sand. Little or no surcharge is applied during the test, as only a thin cardboard disk and a 0.5-cm-diameter brass push rod for measuring heave are placed on the top of the sample. After conditioning, the freezing cabinet is wheeled into a refrigerated room kept at -17°C; the base is kept in +4°C water. Frost heave is monitored for 10 days, and the total heave for this period is used as an index of FS. To establish the FS classification criteria (Table 42), subgrade soils from sites where frost failure occurred were tested together with soils that were not adversely affected by frost action.

Table 42. Frost susceptibility according to the TRRL test.

Frost susceptibility	*Frost heave in 10 days (in.)*
None	<0.5
Marginal	0.5-0.7
High	>0.7

Croney and Jacobs (1967) recognized that this test can only roughly estimate the actual performance of soils in a road structure, because other factors such as drainage also affect the results.

This test appears to minimize the problem of heave restraint. The variability in test results for cohesive soils can be explained by differences in dry density; the specimens with the highest compacted dry density heave the least and the specimens with the lowest compacted dry density heave the most. A complete freezing test takes a long time (240 hours); it could, however, be shortened to 100-150 hours (Jones 1980).

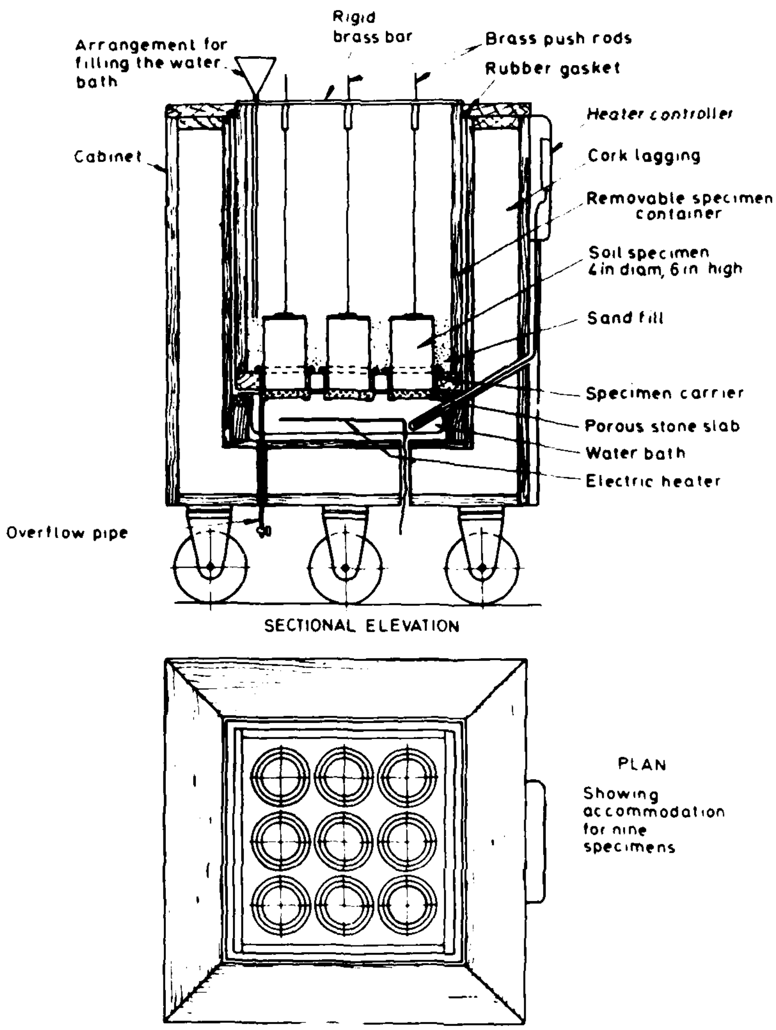


Figure 57. TRRL apparatus for testing frost susceptibility. (From Croney and Jacobs [1967], reproduced by permission of the Transport and Road Research Laboratory. Crown copyright.)

Field experience with the TRRL test (TRRL 1977) showed that the frost heave properties of granular materials were being misclassified. In particular, many materials which had been classified as being non-frost-susceptible were found from field experience to be frost susceptible. The erroneous results were attributed to variations in sample preparation, moisture and temperature. New test procedures (TRRL 1977) established rigorous standards that minimize the influence of human and procedural variations on the test results.

The sample diameter was increased to 15.2 cm to allow particle sizes up to 37.5 mm, and the sample was compacted according to the British standards to approximate more closely the field densities and water contents for granular materials. Changes were also made in the refrigeration facilities to control temperatures better.

Jones (1980) described other improvements on the TRRL test. The major change is the addition of a self-contained refrigerated unit (Fig. 58). Its main advantage over the coldroom is that it does not require a defrosting cycle and thus gives better temperature control. Jones has also added a Mariotte vessel that automatically maintains a constant water table; in the original test, water had to be added manually every 24 hours. Jones also suggested using a vibratory

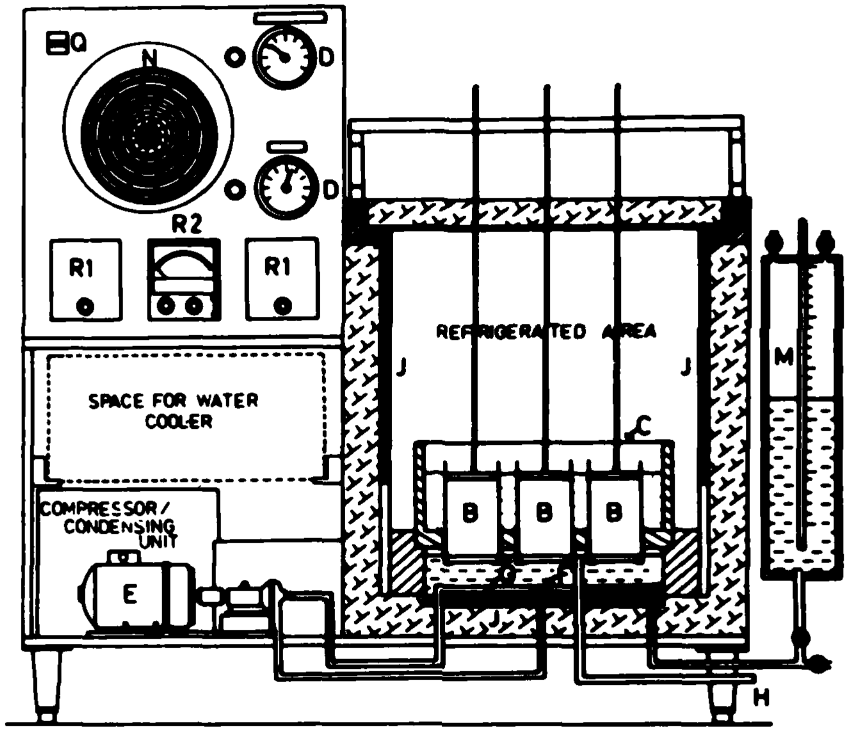


Figure 58. Self-refrigerated unit to improve temperature control in the TRRL frost susceptibility test. (From Jones 1980.)

B	specimen	J	refrigerating coils
C	removable box	M	Mariotte vessel
D	temperature indicator/control	N	chart recorder
E	water pump	Q	hour recorder
F	sparge pipe	R1	thermocouple selector
G	heater	R2	thermocouple readout
H	overflow		

hammer test (BS 1377, test 14) to compact granular materials.

Jones and Dudek (1979) developed a method to improve on the TRRL FS test. Changes have been made to the methods of temperature control and to the sample size. Jones and Dudek referred to their apparatus (Fig. 59) as the precise freezing cell (PFC).

Samples for the PFC are smaller than TRRL samples; the height and diameter are both 10.2 cm. The body of the cell is formed of thin PVC tubes closed at their ends and separated by 50 mm of vermiculite insulation.

The soil specimen, which is wrapped in waxed paper and surrounded by 50 mm of sand for further insulation, sits on a porous disk connected to a constant-head water supply. The base temperature is maintained at +4 ± 0.1°C by circulating water from a constant-temperature bath.

A thermoelectric (Peltier) cooling device is placed on the copper cold plate that rests directly on top of the sample. A thermistor embedded between the copper plate and the upper surface of the sample is coupled to a feedback control unit for the thermoelectric device, which is capable of maintaining a constant cold-side temperature to within ±0.1°C. The thermoelectric device is cooled by tap water running to a drain. The PFC is placed in a refrigerated box maintained at +4°C, and the controls for the thermoelectric cooling device are set at -6 ± 0.1°C.

A unique feature of the PFC is the guard ring that is placed in the annular space adjacent to the copper plate. By circulating an alcohol solution through the guard ring, its temperature can be maintained to within ± 0.5°C. This minimizes radial heat flow and thus allows a better simulation of field conditions.

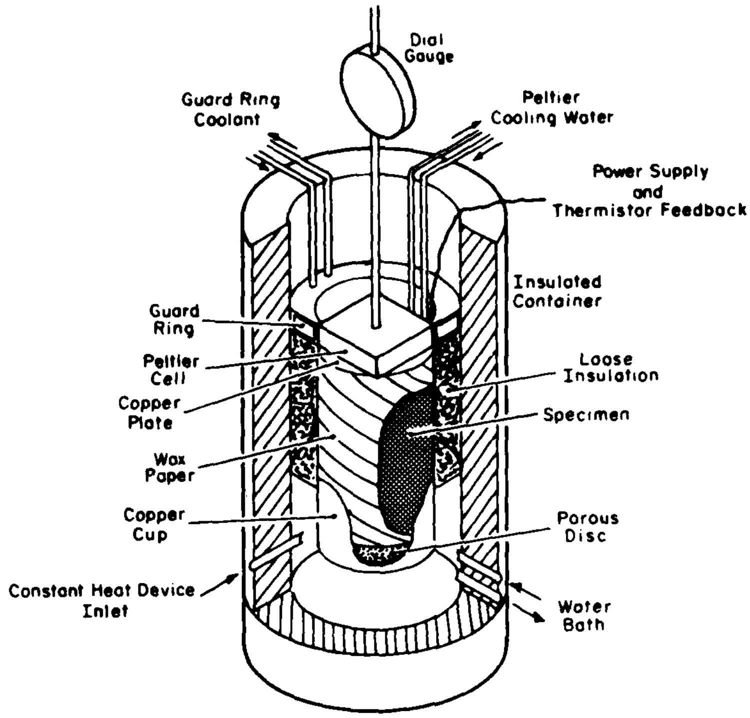


Figure 59. Precise freezing cell according to Jones and Dudek (1979).

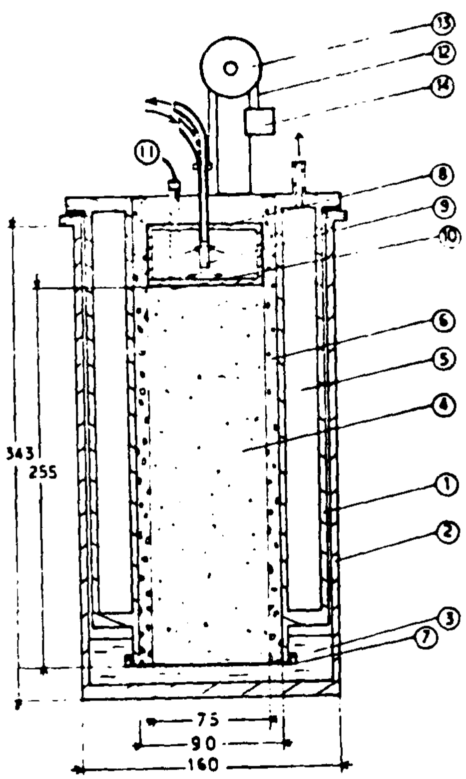


Figure 60. Experimental frost heave apparatus used at the Laboratoires des Ponts et Chaussées. Dimensions are in millimeters. (From Aguirre-Puente et al. 1972.)

1 double-wall cylindrical cell
2 reservoir
3 water supply for specimen
4 soil specimen
5 evacuated space
6 foam rubber tube
7 metal screen
8 cold plate
9 refrigeration line
10 thermocouple for measuring surface temperature
11 potentiometer
12 nylon cord
13 pulley
14 counterweight

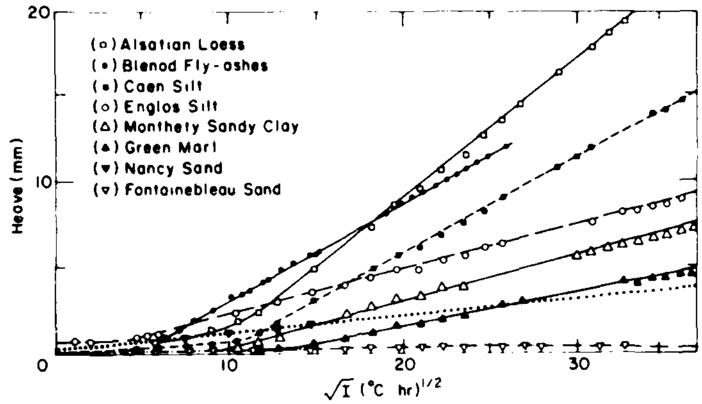


Figure 61. Heaving as a function of the square root of the freezing index I. (After Aguirre-Puente et al. 1974.)

Specimens heave much less in the PFC than in the TRRL units because the temperature at the top of the specimen stays constant as it heaves.

Jones and Dudek did not propose that this test replace the standard TRRL tests, because the cost is much higher (three times that of the standard test). In addition, no FS classification system has been established for the PFC test.

France

J. Aguirre-Puente and his colleagues at the Laboratoires des Ponts et Chaussées have been developing test procedures for determining the FS of soils since the late 1960's (Aguirre-Puente and Dupas 1970, Aguirre-Puente et al. 1972, 1973, 1974).

Their experimental apparatus (Fig. 60) includes a double-walled plexiglass cylindrical cell with the annular space evacuated and maintained near 0°C to minimize radial heat flow. The inside diameter is 7.5 cm and the height is 25 cm. Samples are compacted to a height of 20 cm and soaked for 18 hours in the cell, with water freely available at the base. A temperature of -5.7°C is applied to the upper cold plate by means of a circulating bath. The bottom temperature is maintained at 1°C. The heave is observed for 150-200 hours. When the amount of heave is plotted as a function of the square root of the freezing index (the product of the cold-plate temperature and the lapsed time), the characteristic slope of the resulting straight line is determined (Fig. 61). Caniard (1978) reported that the FS classification in Table 43 has been adopted by the Laboratoires des Ponts et Chaussées after considerable experience.

Table 43. Frost susceptibility according to the Laboratoires des Ponts et Chaussées.

Frost susceptibility	Limiting slope value, p [(mm/°C × h)$^{1/2}$]
None	$p < 0.05$
Low to medium	$0.05 < p < 0.40$
High	$p > 0.40$

Norway

The Norwegian Road Research Laboratory (NRRL) frost heave test has been described by Loch (1979b). The multi-ring apparatus freezes the samples from the top down. Samples 10 cm high and 9.5 cm in diameter can be tested. The cylindrical surface of the sample is coated with rubber, and the sample is placed in the stacked ring holder. The 2-cm-high rings are made of plastic.

Tests are carried out in a controlled-temperature room at an ambient temperature of +0.5°C. The multi-ring mold is surrounded by styrofoam beads to minimize radial heat flow. Temperatures at the top and bottom are controlled by circulating an alcohol-water solution. The base plate is maintained at a fixed temperature slightly higher than 0°C, and the temperature of the top plate is used to control the rate of heat extraction. Early tests were conducted with a fixed top-plate temperature of -17°C, which froze samples to the bottom within two days.

However, since experiments by Loch (1979b) and Horiguchi (1978) indicated that the heave rate depends strongly on the heat extraction rate and that the correlation is not always positive (Fig. 8), Loch concluded, as had Penner (1972), that the rate of heat extraction is the basic variable in the frost heave process.

Loch suggested that a heave test should be carried out at a standard rate of heat extraction. He observed that if the heat removal rate is fixed, then the heave rate will be constant and the test can be conducted in less than 24 hours. Loch found that natural heat extraction rates occurring in southern Norway approximated the optimum values for most of the soils tested in the laboratory. He concluded that a heat extraction rate of 124 W/m^2 should be used in the NRRL test to determine the maximum heave rate for southern Norway. Furthermore, he concluded that this rate of heat removal will cause the frost penetration rate to become small or negligible in later stages of the test, thus simulating the field condition where the frost front is fairly stationary over much of the winter.

Loch also observed that there may be a substantial difference in laboratory frost heave response between undisturbed and disturbed samples of the same soil, and he therefore recommended that the test samples be representative of field conditions.

There has been little experience reported with this test, and it has not been adopted for general use by the Norwegians. Flaate (1980) suggested that they are still considering modifications or other methods for FS compliance testing.

Romania

Vlad (1980) reported on a direct freezing test being developed by the Road Research Station of the Polytechnic Institute of Jassy in Romania. With the exception of the sample size, this test is very similar to the CRREL test.

The samples are 10 cm in diameter and 20 cm high. They are compacted to the optimum density in five layers in a steel mold and are transferred to tapered plexiglass molds for freezing. Any space remaining between the sample and the plexiglass cylinder is filled with paraffin. The samples are saturated under a vacuum.

For freezing, four samples are placed in a freezing cabinet, the bottom of which is open to the +4°C ambient temperature of the laboratory (Fig. 62). The samples are frozen from the top down at an average frost penetration rate of 1 cm/day until 15 cm are frozen; the air temperature in the cabinet is adjusted to as low as -25°C to maintain the constant rate of frost penetration. Samples are frozen with and without a water supply to test the extremes of water availability. Overburden pressures are approximated with lead weights. Heave is measured with dial gauges. Thermocouples are placed at 33-mm intervals within each specimen to obtain temperature profiles.

After freezing, two of the samples are cut open to obtain data on the water content and the shape and size of the ice lenses. The other two samples are thawed in place and subjected to CBR tests. Two other samples, which were maintained in the +4°C laboratory during the freezing test, are also subjected to CBR tests to provide a basis for comparing the thaw CBRs.

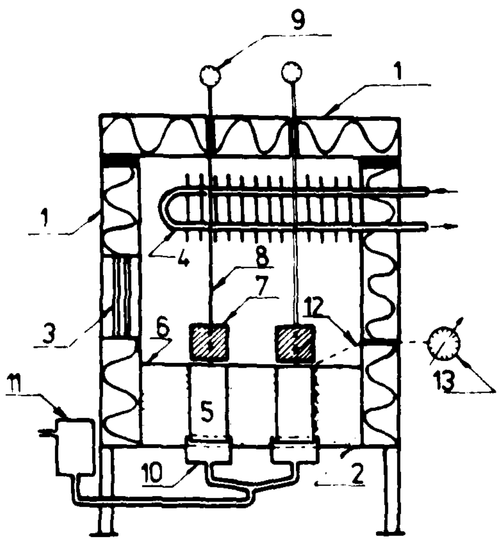


Figure 62. Frost susceptibility apparatus. (From Vlad 1980.)

1. insulated freezing cabinet
2. exposed (uninsulated) bottom of cabinet
3. insulated glass window
4. cooling pipe
5. test specimens
6. foam plastic insulation
7. surcharge
8. heave rods
9. dial gauges
10. water vessels
11. water supply
12. thermocouples
13. multi-channel recorder

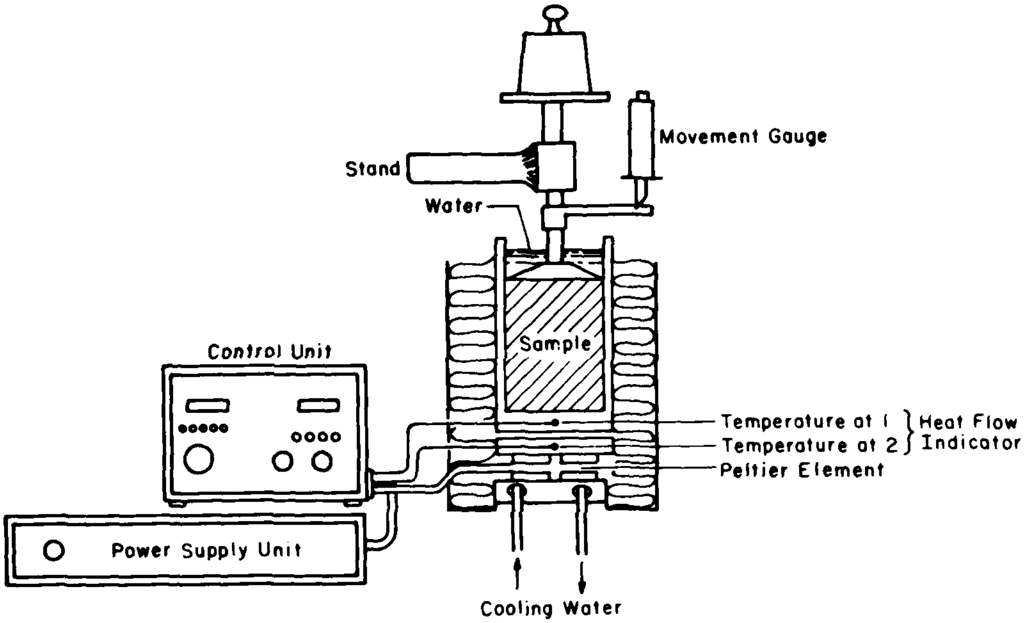


Figure 63. Swedish equipment for measuring the frost heave of soils. (After Fredén and Stenberg 1980.)

The FS is assessed using the following:

1. The maximum heave in 15 days.
2. The average rate of heave.
3. The frost heave ratio.
4. The ratio of the thawed water content to the liquid limit.
5. The consistency index.
6. The reduction in CBR in percent.

No FS criteria were given by Vlad for the Romanian frost heave test. Experience with the test appears to be limited.

Sweden

The Swedish National Road and Traffic Research Institute frost heave test was described by Fredén and Stenberg (1980). Compacted samples are frozen from the bottom up at a constant rate of heat flow, similar to the method suggested by Penner and Ueda (1978). The soil specimen is tamped in an acrylic cell (Fig. 63) 11.3 cm in diameter and 20.0 cm high and saturated by capillary rise for 1–10 days. A Peltier device coupled to a heat-flow sensor is used to keep the heat extraction rate at the base at 490 W/m². Tap water is used to cool the warm side of the Peltier battery. During freezing, water is free to flow into the sample through the top. The surcharge pressure can be varied from 2 to 18 kPa.

The heave ratio is used as an index of FS. However, FS criteria have not yet been developed.

Stenberg (1980) reported on larger-scale field tests to validate the Swedish frost heave test. He observed that in test cells 1.5 m in diameter the heave was 20–25% greater than would be predicted from laboratory tests. He attributed the differences to higher porosity in the segregated ice in the field tests. Stenberg also reported difficulty in relating laboratory freezing conditions to field conditions, particularly when using the freezing index as a link between the laboratory and field tests. Problems result because of the effects of radiation and wind velocity on frost heave and because the freezing index has little effect on frost heave in late winter because of the dampening effect of the overlying frozen material.

Switzerland

The Balduzzi and Fetz (1971) frost heave test is similar to the TRRL test. The sample is 5.64 cm in diameter and 10.00 cm long, essentially the same size as the Proctor mold. Samples are compacted in the mold lined with acetylcellulose foil; they are ejected from the mold, placed in holes in insulating blocks, and saturated at room temperature until the samples cease to take up water. When moisture equilibrium is reached, the samples are placed in a freezing cabinet, where -17°C air is circulated over the top and +4°C water is maintained at the base. Heave is

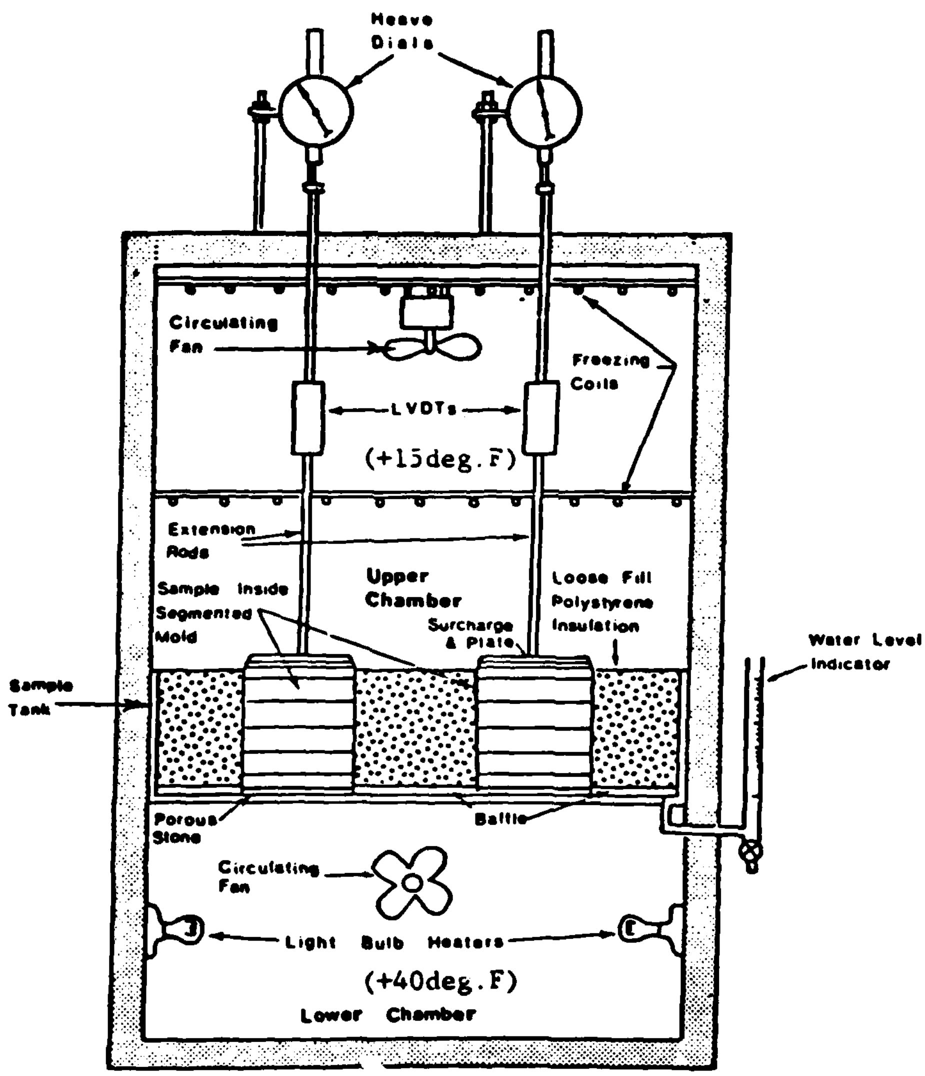


Figure 64. Interior diagram of Alaska Department of Transportation and Public Facilities frost heave test cabinet. (From Esch et al. 1981.)

observed until it stops, generally after 50–70 hours.

No FS criteria are given. However, Recordon and Rechsteiner (1971) reported that the Swiss government has adopted standards that include a CBR-after-thaw test for gravels. The Swiss require that the CBR after one freeze–thaw cycle (or after four days of soaking) be at least 30 for unbroken materials or at least 80 for crushed materials. An additional limitation is that the thaw-CBR value cannot be less than 50% of the normal value.

United States

Alaska Department of Transportation and Public Facilities. Esch et al. (1981) reported some details on the Alaskan direct frost heave test. The multi-ring freezing cell has a 15.2-cm inside diameter and is 14 cm high.

Samples are compacted with a vibratory hammer and saturated by soaking overnight. Only material smaller than 1.91 cm is included. The samples are frozen four to a cabinet (Fig. 64) by maintaining a fixed -9.5°C air temperature above the samples and a +4.5°C temperature

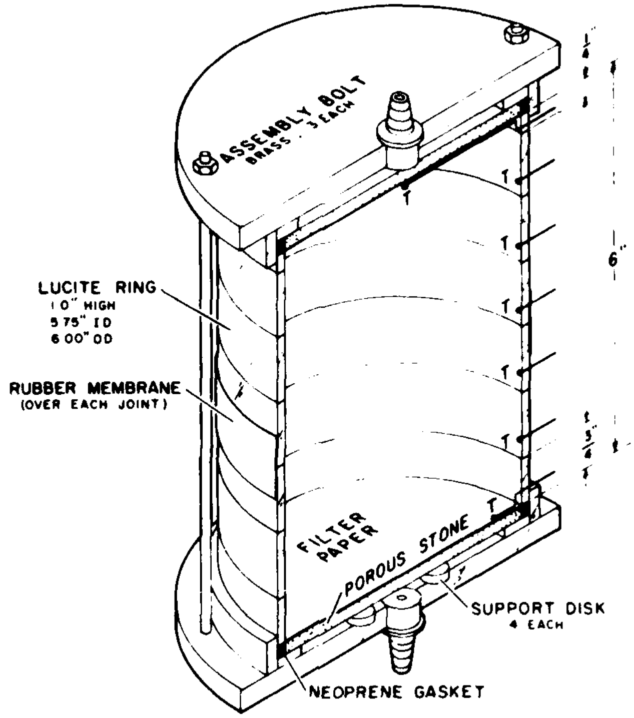


Figure 65. Inside-tapered freezing cell used in CRREL frost heave test. (After Kaplar 1974.)

beneath the samples. Samples are frozen for 72 hours and are classified on the basis of heave occurring between 48 and 78 hours, when the average rate of frost penetration is approximately 1.3 cm/day.

The FS classification system developed for the U.S. Army Corps of Engineers is applied to the test results. This method has not been incorporated into any specifications as yet, but it is presently being field-validated.

U.S. Army Cold Regions Research and Engineering Laboratory. The CRREL frost heave test was originally developed by its parent organization, the U.S. Army Arctic Construction and Frost Effects Laboratory for the U.S. Army Corps of Engineers. Since 1950 the Corps of Engineers has used it (with modifications) as a standard laboratory test procedure for evaluating the FS of soils. According to Linell and Kaplar (1959), this procedure is based on the work of Taber (1929, 1930a,b), Casagrande (1931), Beskow (1935), Winn and Rutledge (1940) and others. Details of the test were first published in 1952 (Haley and Kaplar); more recent details have been published by Kaplar (1974) and Chamberlain and Carbee (1981).

The freezing cell is illustrated in Figure 65. The plexiglass cell is tapered inside from 14.0 cm at the bottom to 14.6 cm at the top to reduce friction during freezing. It is 15.2 cm high. Samples are normally compacted in a steel mold and transferred to the plexiglass cell. They are then degassed and saturated with degassed water. The cells are placed four to a freezing cabinet (Fig. 66) to temper at 3.5°C for 18–24 hours. Degassed water is supplied at the base of each cell; the level is maintained at the top of the cell during tempering and 0.5 cm above the bottom of the sample during freezing. Except for special tests a surcharge of 3.5 kPa is placed on the sample to simulate the minimum field situation of 15 cm of pavement and base. The specimens are frozen from the top down by lowering the air temperature in the cabinet gradually; this maintains a constant rate of penetration of the 0°C isotherm of approximately 0.6–1.3 cm/day.

The FS classification (Table 44), developed by Casagrande, is based on the rate of heave for a constant rate of frost penetration. Figure 29 shows the FS classification as a function of the percentage of particles smaller than 0.02 mm. Table 44 and Figure 29 are the result of several

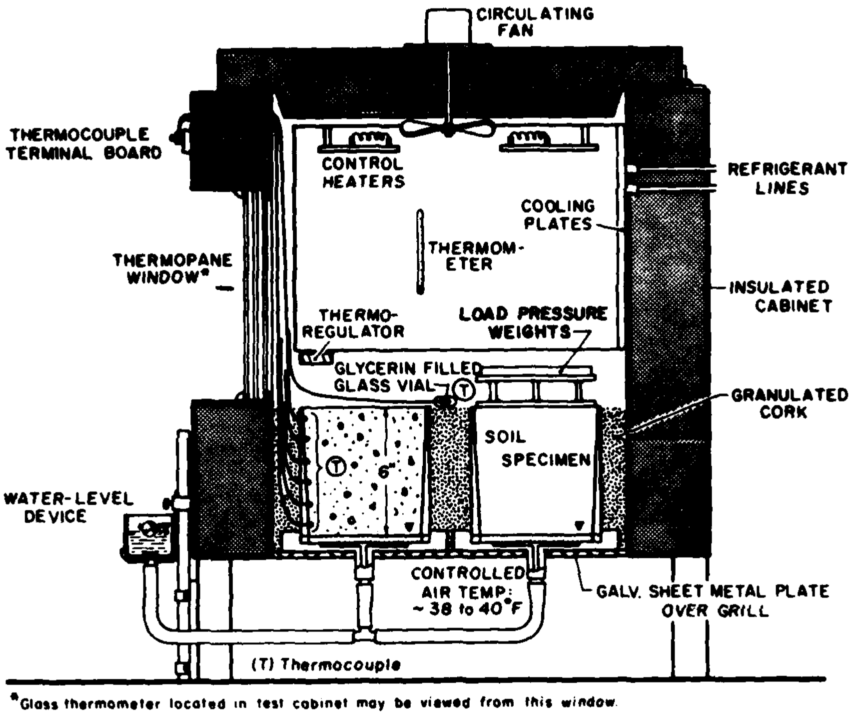


Figure 66. Details of soil freezing cabinet used at CRREL. (After Kaplar 1974.)

Table 44. CRREL frost susceptibility classification.

Frost susceptibility	Average rate of heave (mm/day)
Negligible	0-0.5
Very low	0.5-1.0
Low	1.0-2.0
Medium	2.0-4.0
High	4.0-8.0
Very high	>8.0

hundred laboratory tests; they represent a relative FS classification for the severest conditions of moisture availability and surcharge load.

A tabulation of the results of all the tests performed by CRREL is given in Appendix B. As previously discussed, one can use this table to estimate the FS of a soil if its index properties are known.

There are two major difficulties with the CRREL test: the relatively high variability in heave rate for a given soil and the long period of time (approximately 14 days) required to conduct this test. Studies have been conducted (Kaplar 1971) to identify the causes of the variability in the frost heave rate. Kaplar found that a variable degree of friction may exist between the specimen and its container during frost heave. Freezing tests conducted on soil samples contained in horizontally segmented cells usually showed higher heave rates than did the tests conducted in the tapered, solid-wall cells. Other factors that may cause variability are specimen heterogeneity, variations in the rate of heat extraction, and interruption of the water supply.

Kaplar (1971) also studied methods of decreasing the time required to conduct the tests; he found that useful data could be obtained in freezing times of two days or less by applying a constant subfreezing temperature to samples confined in friction-free containers.

National Crushed Stone Association. Kalcheff and Nichols (1974) combined the CRREL and TRRL methods to develop a method for testing the FS of soil aggregate mixtures. Compacted samples (the dimensions are not specified but the samples appear to be approximately 15 cm in diameter and 20 cm high) are placed 18 to a freezing cabinet and are separated and insulated by loose granular insulation. A surcharge of 1.4 kPa is applied, and the samples are allowed to draw up water by capillary action for two to three days at room temperature. The air temperature above the samples is lowered to -12°C and the heave observed for 200 hours. The heave rate was constant and linearly related to the percentage of fines between 0.075 and 0.020 mm. No classification system was proposed, as field performance of the materials tested had not been adequately quantified.

University of Washington. Sherif et al. (1977) reported on a direct frost heave test being used to study the variables affecting frost heave. This is a constant cold-plate temperature test, with fixed temperatures of -2°, -5° and -10°C employed to determine the frost heave for a range of temperature conditions. The freezing cell (Fig. 67) is an acrylic cylinder 30 cm high and tapered on the inside from 12.62 cm in diameter at the bottom to 11.35 cm in diameter at the top. The specimens are frozen from the top down in a walk-in coldroom, while the base is maintained at +4°C and water is freely available. Thermocouples are used to measure the temperature of the test samples.

Each soil specimen is prepared at optimum water content and tempered in a plastic bag for 24 hours in the +4°C coldroom. The inside of the freezing cell is lubricated with silicone grease, and the samples are molded in four 2-in. layers with a compactive effort equal to that used in the standard Proctor compaction test. The compacted samples are allowed to soak for 24 hours at +4°C with the water level about 1 cm above the bottom of the specimen.

Correlations of frost heave with the amount finer than 0.02 mm, the cold-plate temperature and the length of the freezing period were made for a few soils. However, no attempt was made to relate the results to field observations nor were FS criteria suggested.

University of New Hampshire. Zoller (1972, 1973) and his associates (Biddescombe et al. 1966, Leary 1967, Leary et al. 1967, 1968, Kittridge and Zoller 1969), after several years of development, have developed the University of New Hampshire rapid freeze test. The test equipment is illustrated in Figure 68. Compacted samples are placed in a freezing mold consisting of seven plexiglass rings with inside diameters of 13.7 cm and a total height of 15.2 cm. The multiring mold is then placed in a cylindrical hole cut in the center of a block of rigid foam insulation and lined with waxed cardboard. A constant-head water supply is attached to a porous stone at the bottom of the specimen. A Peltier thermoelectric device is placed in contact with a cold plate at the top of the specimen. The sample is saturated by raising the level of the water table to the top of the specimen for 16 hours, during which the sample is cooled until the temperature at the upper surface is just above freezing. The water level is then lowered to 0.5 in. above the bottom of the specimen. The input current to the battery is increased to begin freezing the specimen and is adjusted so that heat is removed at the constant rate of approximately 675 W/m². At this rate the cold ends of most soil specimens become stabilized at approximately -4°C. Heave is observed for 12 hours and the average heave rate determined. Table 45 compares the resulting frost heave classification with the CRREL system. The heave rates are con-

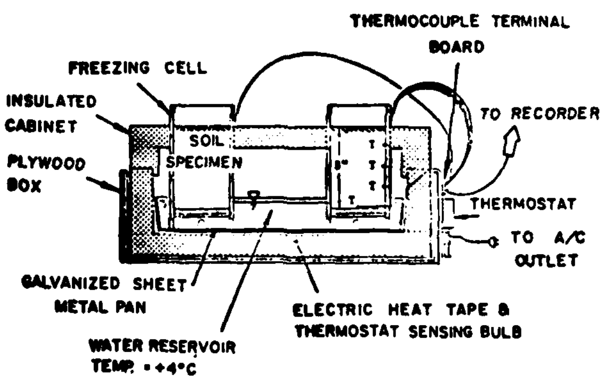


Figure 67. Details of the University of Washington soil freezing cabinet. (From Sherif et al. [1977], courtesy of Cold Regions Engineers Professional Association.)

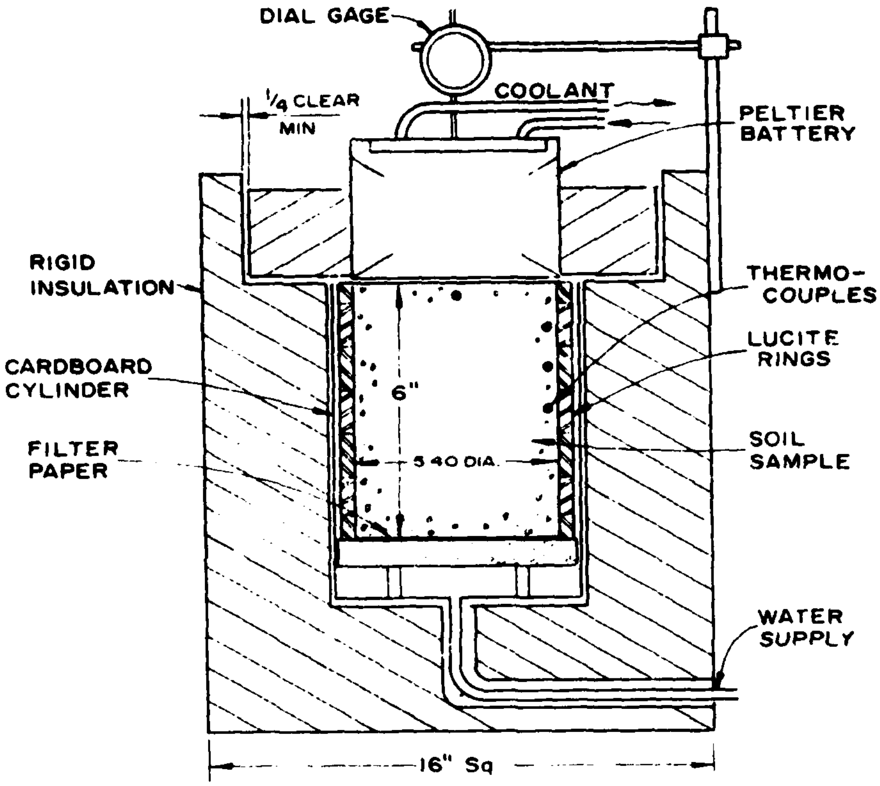


Figure 68. University of New Hampshire rapid freeze test equipment. (From Zoller 1973.)

Table 45. Frost susceptibility classes according to the U.S. Army Corps of Engineers and the University of New Hampshire.

	Avg. rate of heave (mm/day)	
Frost susceptibility	*Corps of Engineers*	*UNH*
Negligible	0-0.5	0-6.5
Very low	0.5-1.0	6.5-8.0
Low	1.0-2.0	8.0-10.3
Medium	2.0-4.0	10.3-13.0
High	4.0-8.0	13.0-15.0
Very high	>8.0	>15.0

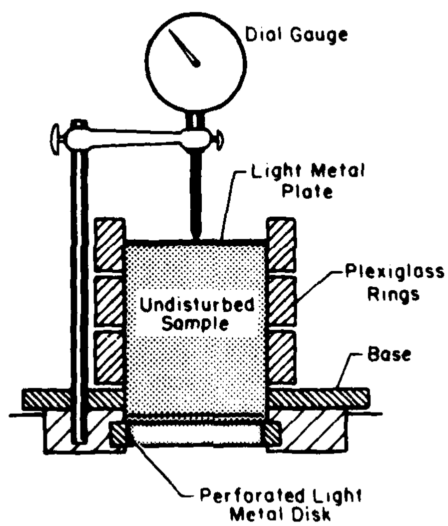


Figure 69. Experimental apparatus of Alekseeva (1957).

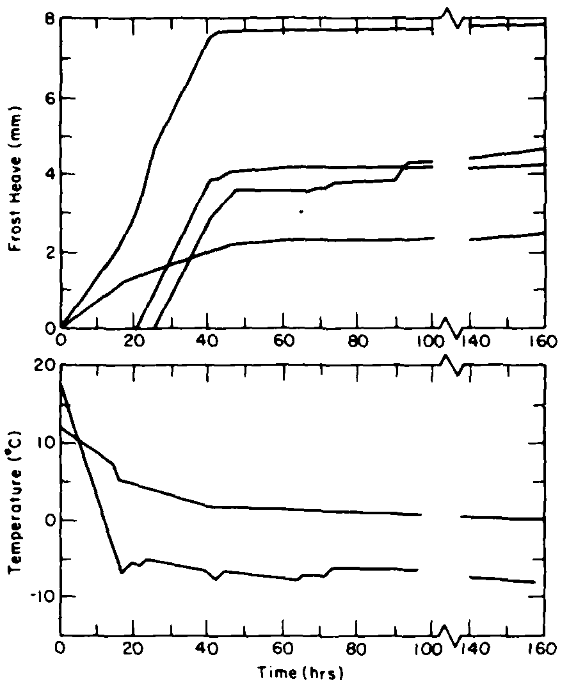


Figure 70. Experimental results of Alekseeva (1957).

siderably greater than in the CRREL tests, probably because of the larger amount of side friction in the CRREL tests.

U.S.S.R.

Alekseeva (1957) proposed a frost heave test using multiple plexiglass rings to contain the test sample (Fig. 69). The sample is frozen by applying a temperature of -2° to -5°C at the upper plate while maintaining a temperature of 0.5-1°C at the base, where water is available. The inside diameter of the multi-ring container is 6 cm and the height is 10 cm. Details of the tests are sketchy; however, it appears that undisturbed specimens are placed in the container for testing. The small diameter probably precludes testing coarse gravels.

Figure 70 illustrates results obtained with this device. Frost penetration appears to be very rapid, with the samples apparently completely frozen within 48 hours. No FS classification was given.

Ganeles and Lapshin (1977) developed another method but gave few details. It is a frost heave test where the frost penetration rate varies from an initial rate of 5-7 cm/day to 0.5 cm/day in the final stages. The test requires one to two weeks of freezing time, and the heave rate is determined as a function of frost penetration rate. No classification system is provided.

The report of Kronik (1973) has not been translated but it appears that a frost heave test is involved. The criteria in Table 46 were established for frost penetration rates of less than 10 cm/day.

Table 46. Frost susceptibility classification according to Kronik (1973).

Frost susceptibility	*Heave ratio (%)*
None	<2
Low	2-5
Medium	5-10
High	>10

Vasilyev (1973) developed a test but provided few details of his laboratory apparatus and procedures. However, it appears that a metal, stacked multi-ring mold is employed; its dimensions are 10 cm i.d. and 8 cm in height (each ring is 1 cm high). The test is conducted to evaluate the heave ratio of subgrade soils. Samples are compacted and allowed to soak at the base. The sample is frozen from the top down by placing the apparatus in a cold box maintained at -4° to -6°C. The average rate of frost penetration is 1.2-1.5 cm/day. Although it is not stated, a test at this rate would take approximately eight or nine days. The frost heave ratio is the critical factor for the FS classification. Subgrade soils with heave ratios exceeding 2-4% are considered to be frost susceptible.

West Germany

Ducker (1939) was probably the first to attempt to determine FS with a laboratory test. Ducker's apparatus is illustrated in Figure 71. The sample mold is made up of four glass rings 1 cm high and 3.85 cm i.d. stacked atop a 7-cm-long glass cylinder. Air-dried soil is placed within the glass rings in contact with coarse sand in the glass cylinder below. The assembly is placed in a pan of water maintained at +4° to +5°C, and the sample is allowed to draw up water from the wetted sand by capillary action for an undisclosed amount of time. The apparatus is then placed in a small double-chambered refrigerator, the lower chamber maintained at a temperature just above 0°C and the upper chamber at -15° or -10°C. Heave is observed for four hours, after which the sample is removed and the depth of frost and the water content in the unfrozen and frozen zones are determined.

Ducker found that for soils with particle diameters ranging from 0.5 to 0.006 mm the amount of frost heave increased dramatically as the particle size decreased. The cold-side temperature also affected the frost heave rate; the largest heave always occurred when the cold-plate temperature was set at -15°C.

Ducker proposed that the ratio of the frost heave to the depth of frost penetration (the heave ratio) be used to express the degree of frost danger *F*. He proposed that the boundary between frost-susceptible and non-frost-susceptible soils be $F = 3\%$. However, since the *F* values differed by 10-20%, depending on the cold-plate temperature, this criterion clearly has some limitations. Ducker appeared to be aware of this problem as well as of the effects of surcharge and moisture availability, and therefore he did not propose that this criterion be the sole factor in determining the FS.

Jessberger and Heitzer (1973) proposed a freezing test where the CBR after seven freeze-thaw cycles is used to determine FS. The samples are frozen in a tapered PVC cylinder (Fig. 72) lined with Teflon foil. The diameter at

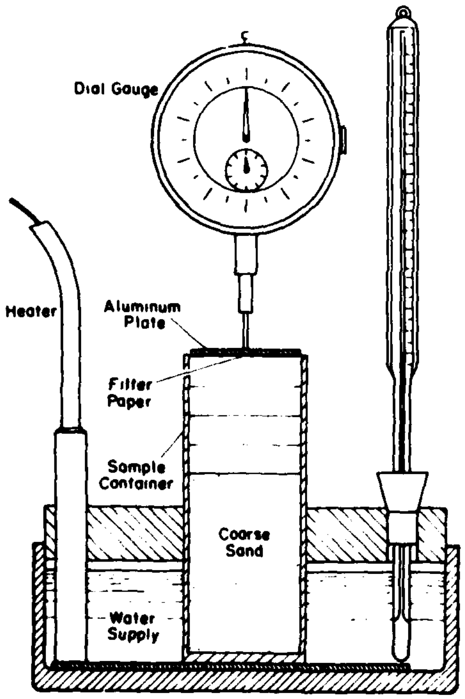


Figure 71. Frost heave apparatus. (After Ducker 1939.)

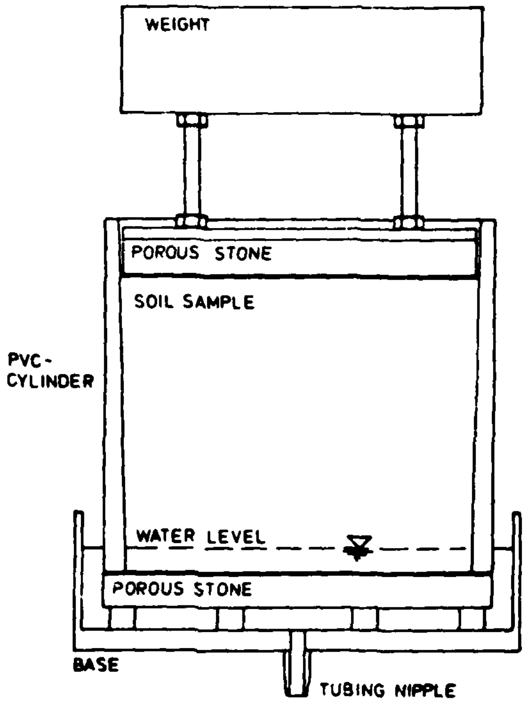


Figure 72. Freezing cylinder used in West Germany. (From Jessberger and Heitzer [1973], courtesy of the Organization for Economic Cooperation and Development.)

the top is 15 cm and at the bottom, 14.5 cm. The sample height is 12.5 cm. The samples are compacted at optimum water content in five layers in a steel container having the same dimensions as the PVC cylinder. The method of compacting is similar to that used in the standard Proctor test.

The freezing cabinet consists of two chambers, with separate cooling systems for controlling the top and bottom temperatures. The air temperature in the upper chamber is maintained at -18°C during freezing and +18°C during thawing. The lower chamber temperature is maintained between +2° and +6°C. Prior to freezing, the samples are placed in a humid room for three days at +20°C and then allowed to soak for 24 hours with the water level 1 cm above the top of the sample. The saturation and the freezing and thawing are conducted with a 5.9-kPa surcharge on the sample. Water is freely available at the base of the sample during freezing.

CBR values for a penetration depth of 2.5 mm are determined 1) after three days in a humid room (CBR_0), 2) after four days of soaking (CBR_u), and 3) after seven freeze-thaw cycles (CBF_F). The CBR_0 and CBR_u values are used to determine the suitability of the material without freezing and thawing.

Jessberger and Heitzer (1973) did not propose FS criteria based on the CBR. However, in a later report (Germany 1979) provided by Jessberger the criteria in Table 47 are suggested. Along with the grain size criteria discussed earlier these standards are being considered for adoption by the West German government. Although there has been little experience with this method, Jessberger has convinced his government that thaw weakening is more important than frost heave in determining the FS of soils and granular materials.

Table 47. West German frost susceptibility criteria (Germany 1979).

Frost susceptibility	CBR_F
None	>20
Low to medium	4-20
High	<4

EVALUATION OF INDEX TESTS

Five fundamentally different approaches to determining the frost susceptibility of soils have been identified. These approaches were based on 1) particle size characteristics, 2) pore size characteristics, 3) soil/water interaction, 4) soil/water/ice interaction, and 5) frost heave. The reliability of any approach is largely dependent on how well it addresses the factors affecting frost heave.

Another important factor in selecting a FS index test is complexity. The simple particle size criteria are the most popular because the tests are faster and because they require less additional testing than is normally required for road construction projects. Time, in fact, may be the deciding factor in selecting the method to be used, as few road builders are willing to wait weeks for test results from more complex methods before deciding about the suitability of materials. Thus, both reliability and complexity must be considered in evaluating FS index test methods.

Tests using particle size characteristics

This group of FS index tests includes those methods where particle size is the principal factor. The simplest of these methods requires only a sieve analysis of the portion larger than 0.074 mm. This type of criteria is popular with government agencies in the United States and Canada. An example is the 10% limitation on the amount of particles finer than 0.074 mm set by Connecticut. The range of allowable percentage of particles passing a given size of sieve is considerable, with only 5% finer than 0.074 mm allowed in Wisconsin and as much as 60% permitted in Manitoba. Of the 97 grain size methods surveyed, 43 require only the sieve analysis, obviously a concession to the simplicity of this test. Ten more require only the addition of the Atterberg limit test.

The remainder of the grain size methods require the determination of the distribution of the particle sizes smaller than 0.074 mm. This requires two tests in addition to the sieve analysis: the hydrometer analysis and the specific gravity test. Eight methods employing only these three tests were reported. Illinois is an example of a state using this type of criteria.

Another 21 criteria require the Atterberg limit test in addition to the grain size distribution

Table 48. Index properties of soils for frost susceptibility performance analysis.

Soil no.	Soil class.*	Percent finer than 4.76 mm	0.42 mm	0.074 mm	0.02 mm	0.01 mm	0.005 mm	$C_u^\dagger$	C_c^{**}	LL	PI	Frost class.††
1	GW	49.0	10.0	3.0	0.8	0.8	0.5	17	1.4	–	–	NFS
2	GW-GM	42.0	14.0	5.3	2.1	1.2	0.7	38	2.2	–	–	NFS
3	GM-GC	54.0	30.0	20.0	15.0	9.0	5.0	485	1.9	–	–	NFS
4	SP	72.0	7.0	3.0	1.3	0.9	0.5	5.3	2.0	–	–	NFS
5	SP-SM	100.0	100.0	6.3	2.6	2.2	1.7	1.9	1.0	–	–	NFS
6	GW	49.0	12.0	4.7	2.4	1.7	0.9	20	1.1	–	–	M-H
7	GW-GM	44.0	18.0	7.0	2.9	2.1	1.5	57	2.0	–	–	L-M
8	GC	48.0	36.0	22.0	17.0	15.0	12.0	4000	1.2	–	–	M-H
9	SW	58.0	15.0	4.9	2.3	1.5	1.1	23	1.3	–	–	M
10	SP-SM	77.0	27.0	7.1	3.3	3.0	2.6	13	0.7	–	–	L-M
11	ML	100.0	100.0	98.0	35.0	18.0	8.0	–	–	29.5	12.7	VH
12	CL	100.0	98.0	91.0	33.0	24.0	19.0	–	–	28.0	12.0	H
13	GP-GM	45.0	25.0	11.0	6.8	6.0	4.0	258	0.7	–	–	L-M
14	GM	55.0	28.0	15.0	6.3	4.4	3.0	193	3.6	–	–	M-H
15	GP-GM	47.0	23.0	9.1	3.2	2.1	1.5	120	0.6	–	–	L-M
16	GC	68.0	52.0	41.0	30.0	25.0	18.0	945	0.1	22.1	7.8	H-VH

* Unified Soil Classification.

† Uniformity coefficient.

** Coefficient of curvature.

†† Frost susceptibility classification according to CRREL (Kaplar 1974).

data. The U.S. Army Corps of Engineers criteria are an example of one of these methods. The remainder of the particle size methods require other tests, such as permeability, capillarity, CBR, and mineral type tests.

The reliability of particle size methods for determining the FS of soils is difficult to assess. The performance of only a few of the methods has been rigorously evaluated. Most may be satisfactory for the conditions in the region where they are used. Manitoba's criteria (60% finer than 0.074 mm) are probably satisfactory for the clay soils of that province but would obviously be inappropriate for the silty soils of Connecticut, where only 5% finer than 0.074 mm is allowable.

Townsend and Csathy (1963b), in a study of the field performance of FS criteria, found that grain size criteria were generally very successful in rejecting frost-susceptible soils, but they also frequently rejected non-frost-susceptible soils. In other words, these are safe but conservative criteria. The most reliable of the nine methods evaluated were the Casagrande (1931), Linnell and Kaplar (1959), and U.S. Army Corps of Engineers (1965) criteria. The latter two methods, which are modifications of the Casagrande criteria, gave practically the same reliability figures.

To determine the reliability of all the grain size methods for determining FS, they were compared with the laboratory frost heave performance of 16 soils from the report of Kaplar (1974). (Comparisons with field observations would have been preferable, but sufficient information was not available for enough sites and material types.) The materials represent a range of soil types; however, the majority of materials were sands and gravels (Table 48).

Table 49 shows the results of this analysis. For each of the criteria the reliability in predicting the performance of non-frost-susceptible and frost-susceptible soils was determined. In addition the overall reliability was determined. Where borderline conditions prevailed, it is noted. For some of the criteria the reliability was not determined because of insufficient information on such properties such as mineralogy, permeability and Atterberg limits.

The most reliable of the criteria based on grain size characteristics were the Swiss (Association of Swiss Road Engineers 1976) and the U.S. Army Corps of Engineers (1965) methods. Few others approached the overall reliability of these two methods (0.91). It is no coincidence that the Swiss and Corps methods agree, as the Swiss criteria are based on the Corps criteria.

Table 49. Performance of grain size frost susceptibility criteria.

User of test*	Reliability in predicting: Non-frost-susceptible soils	Reliability in predicting: Frost-susceptible soils	No of borderline soils	Overall reliability	User of test*	Reliability in predicting: Non-frost-susceptible soils	Reliability in predicting: Frost-susceptible soils	No of borderline soils	Overall reliability
Alberta	0.80	0.36	0		Nebraska	0.80	0.56	2	0.64
Arizona	0.80	0.45	0	0.56	Netherlands	0.80	0.73	0	0.75
Asphalt Institute	0.80	0.82	0	0.81	New Brunswick	0.80	0.64	0	0.69
Beskow	0.80	0.55	0	0.63	Newfoundland	0.60	0.82	0	0.75
Bonnard & Recordon	0.80	0.73	0	0.67	New Hampshire	0.80	0.45	0	0.56
Bonnard & Recordon	0.60	0.73	0	0.69	New York	0.80	0.36	0	0.50
Brudal	0.60	0.45	0	0.50	Nielson & Rauschenberger	1.00	0.36	0	0.56
Can Dept Trans	0.60	0.82	0	0.75	Norway	0.80	0.36	0	0.50
Carothers	0.80	0.55	0	0.63	Nova Scotia	0.80	0.55	0	0.63
Casagrande	0.80	0.73	0	0.75	Ohio	0.80	0.36	0	0.50
Colorado	0.67	0.75	5	0.73	Ontario	1.00	0.27	0	0.50
Connecticut	0.80	0.55	0	0.63	Ontario	1.00	0.64	0	0.75
Croney	1.00	0.36	0	0.55	Orama	0.40	0.82	0	0.69
Delaware	1.00	0.27	0	0.50	Oregon	0.80	0.55	0	0.63
Denmark	0.80	0.55	0	0.63	Oregon	0.80	0.18	0	0.38
Ducker	0.80	0.73	0	0.75	Pietrzyk	0.80	0.40	1	0.53
Floss	0.80	0.55	0	0.63	Quebec	0.60	0.73	0	0.69
Idaho	0.40	0.82	0	0.69	Riis	0.80	0.73	0	0.75
Illinois	1.00	0.18	0	0.44	Ruckli	0.80	0.36	0	0.50
Iowa	0.80	0.45	0	0.56	Saskatchewan	0.80	0.55	0	0.63
Japan	0.60	0.82	0	0.75	Schaible	1.00	0.36	0	0.56
Jessberger & Hartel	0.60	0.64	0	0.63	Switzerland	0.67	1.00	5	0.91
Jessberger	0.80	0.64	0	0.69	Turner & Jumikis	1.00	0.27	0	0.44
Kansas	0.80	0.36	0	0.50	U.S. CAA	0.80	0.55	0	0.56
Linell & Kaplar	0.80	0.82	0	0.81	USAE WES	0.67	0.71	6	0.70
Maag	1.00	0.30	2	0.50	U.S. Army Corps of Engrs	0.67	1.00	5	0.91
Maine	0.40	0.82	0	0.69	Vermont	0.80	0.73	0	0.67
Maryland	0.80	0.45	0	0.56	Vlad	0.80	0.50	0	0.56
Massachusetts	0.80	0.55	0	0.63	Washington	0.80	0.64	0	0.69
Mass. Tnpk. Auth.	0.80	0.55	0	0.63	West Germnay	0.60	0.82	0	0.75
Minnesota	0.80	0.55	0	0.63	Wisconsin	0.40	0.82	0	0.69
Morton	0.80	0.45	0	0.56	Wyoming	1.00	0.36	0	0.73

*A description and reference for these tests are given in the text. Many of the tests reviewed in the text are not included in this analysis because they required more information than was available.

Tests using pore size characteristics

Earlier in this report, details of three FS criteria based on pore size distribution tests were discussed. Csathy and Townsend (1962) and Townsend and Csathy (1963b) used a capillary rise method to determine pore size distribution. They established that when $P_u < 6$ (where $P_u = P_{90}/P_{70}$), the soil was non-frost-susceptible. When they compared the reliability of this method with several grain size criteria, they found that it was significantly more reliable in determining the FS of soils. This criterion was developed and tested only for the climatic and moisture conditions in Ontario, Canada; the authors did not suggest that it could be applied elsewhere without further study.

Csathy and Townsend did not suggest that this method indicates the true pore size distribution, but that it gives a realistic picture of the effective pore conditions as reflected by unsaturated upward moisture flow. They suggested that this method for determining pore size is only approximate because 1) non-uniform void ratio changes occur due to swelling (affecting the degree-of-saturation calculations) and 2) the capillary bundle concept is a drastically simplified model for a pore system.

As previously discussed, Gaskin and Raymond (1973) evaluated the pressure-plate suction tests and the mercury-intrusion test to improve on the time required for determining the pore size distribution (up to 35 days). For the pressure-plate test, equilibrium moisture conditions required two to five days at each pressure differential, or up to one month per test. The mercury-intrusion test was much faster, requiring only 30 minutes to complete.

Gaskin and Raymond observed significant differences between the pore size distribution curves obtained from the three tests (Fig. 37). The relatively small differences between the capillary rise and the pressure-plate wetting test results were attributed to the differences in void ratio. The hysteresis between the pressure-plate wetting and drying test results is a typical phenomenon in suction tests and is believed to be caused by small pores restricting the drainage from large pores. The greatest differences were observed between the mercury-intrusion and capillary rise results. Gaskin and Raymond attributed these differences to the small pores restricting the movement of mercury into larger pores.

They found that the capillary rise method produced results that had the best correlations with field observations. (None of the pore size distribution methods were more reliable in determining the FS of soils than criteria based on particle size.) Nonetheless, Gaskin and Raymond concluded that because of the short time of testing required, any further correlation of pore size distribution with FS should use the mercury-intrusion test, even though it appears to be the least accurate of the three methods evaluated. The more recent study by Reed et al. (1979) using the mercury-intrusion method is an attempt to follow up on this conclusion.

Reed et al. (1979) pointed out that the advantage of using the pore size distribution criteria is that they consider the compaction variables of moisture and density. The correlation that they found between the predicted and observed values was good ($r = 0.91$). However, the scatter in predicted values is considerable (Fig. 38); the calculated values differed from the actual heave by 50% and more. In addition to the uncertain reliability of the mercury-intrusion method, there is another important disadvantage: the mercury-intrusion method has not yet been used successfully on sands and gravels. This is a serious limitation, as it is these materials that are most often in question.

Soil/water interaction tests

This group of tests for determining FS includes 1) moisture-tension tests, 2) capillary rise tests, 3) saturated hydraulic conductivity tests, 4) unsaturated hydraulic conductivity tests, and 5) centrifuge moisture content tests.

Williams (1966) concluded that the air intrusion value determined from the moisture-tension test can be used as a guide for determining the susceptibility of soils to frost heave. Williams's approach may be valid for materials with single-size pores, but for soils with pores of many sizes, the air intrusion value may not be well defined. Moreover, Chamberlain (1980) recently observed that considerable moisture movement and frost heave can occur at tensions well above the air intrusion value.

Furthermore, Jones and Hurt (1978) reported that there is no well-defined air entry value for aggregates. Ingersoll (1981) also observed this. Jones and Hurt (1978) suggested that FS can be determined according to tension values at 70% saturation if there is no well-defined air intrusion value.

Wissa et al. (1972) proposed that the product of the air entry value and the corresponding unsaturated hydraulic conductivity be used to determine the FS of soils. Obermeier (1973) criticized this interpretation because it assumes that the air entry value is unique and that the hydraulic conductivity and air intrusion values are of equal importance. He suggested that FS criteria be based on the shape of the unsaturated hydraulic conductivity tension curve to account for the movement of water over a wide range of suction regimes. He stated, after corresponding with Wissa and Martin, that bands or regions could be established graphically to distinguish frost-susceptible and non-frost-susceptible soils.

In addition to the lack of well-developed criteria based on moisture-tension and unsaturated hydraulic conductivity tests there are other problems with this approach. Cumberledge and Hoffman (1976) had considerable difficulty in obtaining reproducible results with a production unit provided by Wissa. Problems occurred principally from clogging of the piezometer tips, assembly of the apparatus, and accurate determinations of the head loss during the permeability tests. Recommendations were made for increasing the size of the piezometer tips and the thickness of the sample, but no plans were made to continue testing with the device.

The FS classification method based on saturated hydraulic conductivity suggested by Onalp (1970) is rather simple. It assumes that frost heave is uniquely related to the saturated hydraulic conductivity of soils. This may be a good assumption when the water table is high. However, most frost heave problems occur in partially saturated soils where the hydraulic conductivity depends on other factors, such as the level of moisture tension and the pore size distribution. Because of this and because little detail is known of this method, it will not be considered further, nor will there be any further discussion of the centrifuge moisture content method of Willis (1930), as little is known of the method and its application.

Soil/water/ice interaction tests

The two tests in this category, the frost heave stress test and the pore-water suction test, involve freezing soils but not measuring frost heave.

Rice (1978) evaluated the frost heave stress test prepared by Wissa and Martin (1968) and concluded that the use of the slope R of the logarithm of the heave stress versus time curve in predicting the relative FS of a soil appears unreliable because the R value is very sensitive to fluctuations in test conditions and because it depends on the judgement of the investigator. Furthermore, the correlation of R values with Casagrande's and the Corps of Engineers' FS classification systems indicated that this parameter is not a sensitive indicator of relative FS.

The test of pore-water suction during freezing proposed by Wissa and Martin (1968) uses equipment similar to that used in their heave stress test, but it is much more complicated. They concluded that the pore-water suction test provides the same information as the heave stress test and that the heave stress test is preferable because it is much simpler to conduct.

Riddle's (1973) pore-water suction test has the additional limitation that it applies only to soils with suctions no greater than 1 atm, which probably eliminates all clayey soils.

Frost heave tests

The literature review of frost heave tests revealed a wide variety of methods for determining the FS of soils with a direct frost heave test. It is clear from this review that no one test is the most desirable.

Some of these methods can be immediately excluded, as they cannot accommodate coarse-grained base materials because of the small diameters of their freezing cylinders. These include the Ducker (1939), Alekseeva (1957), Aguirre-Puente and Dupas (1970) and Balduzzi and Fetz (1971) methods. Others, including the Vasilyev (1973), Penner and Ueda (1977), Jones and Dudek (1979), Loch (1979a) and Vlad (1980) methods, are marginally acceptable as they can be used with coarse-grained materials only by removing the larger particles. The earlier method of Brandl (1970) is also probably unacceptable as a universal technique for all material types, as it requires a very large sample (30 cm in diameter and 50 cm long). To avoid removing all but the coarsest gravel particles (approximately 25 mm in diameter), there appears to be a concensus among the various researchers that the sample diameter should be between 12 and 15 cm. A sample height in the same range also seems desirable.

Of those remaining, only the CRREL (Kaplar 1974) and TRRL (Croney and Jacobs 1967) methods have established FS criteria and have been compared with field performance. These tests, however, require 10 and 12 days, respectively, to conduct and there appears to be a problem with side friction in the CRREL test, particularly with coarser-grained materials (Kaplar 1968).

Side friction, in fact, appears to be one of the major problems in direct frost heave testing. The multi-ring freezing cell (MRFC) is by far the most popular method of minimizing the side friction during frost heave. Nine of the twenty direct frost heave tests surveyed used this method (for example, Brandl [1970, 1980], Loch [1979a] and Gorlé [1980]). The next most popular method of minimizing side friction is the tapered-cylinder freezing cell (TCFC), which is used by Jessberger and Heitzer (1973), CRREL (Kaplar 1974), Sherif et al. (1977) and Vlad (1980). Other methods to minimize side friction include bottom-up freezing (Penner and Ueda 1977, Loch 1979a), and the use of cellulose foil, waxed paper, polyethylene film (Croney and Jacobs 1967, Balduzzi and Fetz 1971, Kalcheff and Nichols 1974, Jones and Dudek 1979), or lubricated rubber tubes (Aguirre-Puente and Dupas 1970).

Both Zoller (1973) and Kaplar (1974) observed that side friction during freezing was considerably less with a MRFC than with a TCFC, particularly with coarse-grained materials. Even when the TCFC is lined with Teflon, soil friction is a problem because particles gouge the cylinder wall (Carbee, pers. comm.).

No rigorous comparisons of the other alternatives have been published. However, Kaplar (1974) reported that waxed cardboard cylinders were abandoned in favor of the TCFC to reduce side friction, and Zoller (1973) noted that when the tape used to hold the MRFC together during compaction was inadvertently left in place during freezing, frost heave was considerably suppressed. It appears, then, that the MRFC offers the least resistance to frost heave and that the amount of the resistance depends on the friction characteristics between the soil and the side-wall material, the stiffness and strength of the side-wall material, and the amount of frost heave.

The bottom-up freezing cell (BUFC) appears to be equal to or better than the MRFC in minimizing side restraint for fine-grained soils. However, for coarse-grained soils friction, problems similar to or worse than those of the TCFC would be expected.

These arguments regarding side restraint are subjective at best, as few comparative studies have been reported. The uncertainty can only be resolved by rigorous testing.

Of all the tests only the CRREL (Kaplar 1974) and Vlad (1980) tests employ constant frost penetration rates. Keeping the ratio constant is clearly a liability, as the temperature must be adjusted frequently and the freezing conditions may not necessarily be the most severe nor similar to those in the field. The literature review showed that the rate of heat removal and the temperature gradient, not the rate of frost penetration, are the critical thermodynamic factors. The cold- and warm-plate temperatures can be adjusted to simulate field conditions in all the tests except the Zoller (1973) and Fredén and Stenberg (1980) methods, which do not have temperature controls for the warm plate. However, these tests could be easily modified.

Most of the direct frost heave tests use fixed boundary temperatures during freezing and thus impose a variable rate of frost penetration. The freezing temperatures ranged from −25°C for the Vlad (1980) test to −4°C for the Zoller test. The warm-side temperatures generally ranged from near 0° to +4°C. The Fredén and Stenberg and Zoller tests are conducted at normal room temperatures, with the warm end insulated from the ambient temperature.

The only test to employ a constant rate of heat removal is the Fredén and Stenberg method. Both Penner and Ueda (1977) and Loch (1979b) suggested that a constant rate of heat removal be used, but they have not incorporated the method into their tests because of the complicated control system required. The Fredén and Stenberg test uses a closed-loop control system, where a heat flow indicator on the underside of the cooling plate senses the rate of heat removal and feeds a signal to an electronic control system, which automatically adjusts the current source for the Peltier cooling device. This procedure is probably the best method for simulating field conditions. However, because of the complicated equipment required, it is not suitable for routine FS testing in highway laboratories.

The method of temperature control is also important in selecting a FS test. Peltier thermoelectric cooling devices (used in the Zoller, Fredén and Stenberg, and Jones and Dudek [1979] methods) have potentially the best temperature control, while circulating non-freezing liquids (used in the Loch [1979a], Penner and Ueda [1977], and Aguirre-Puente and Dupas [1970] tests) are probably nearly as good. Circulating air is less desirable because of the larger temperature variations inherent to air cooling systems. From another point of view, however, the methods employing circulating air are more desirable as they allow multiple samples to be tested (nine in the TRRL [Croney and Jacobs 1967] and Kalchef and Nichols [1974] tests, seven in the Balduzzi and Fetz [1971] test, and four in the Jessberger and Heitzer [1973], CRREL [Kaplar 1974], Sherif et al. [1977], Vlad [1980], and Esch et al. [1981] tests). The Zoller and Fredén and Stenberg tests are unique in that they provide the best temperature control at the cold plate while providing none at all at the warm plate.

In most of the tests, radial heat flow is minimized using foam insulation. The TRRL test uses dry sand on the theory that the sand and the samples will have nearly the same thermal conductivities. When the upper surface of the samples and the surrounding sand is exposed to freezing air temperatures, the heat flow is unidirectional upward toward the cold air. However, when cooling plates are used, this method is less desirable because of temperature discontinuities at the upper surface. Jones and Dudek overcame this problem by adding foam insulation and a temperature-controlled guard ring around the cooling plate. Aguirre-Puente and Dupas (1970) surrounded the test vessel with a vacuum maintained at just above freezing. The last two methods require very complicated test equipment that is valuable for conducting research under precise conditions but is much too complex and expensive for routine testing. The best and simplest solution for controlling radial heat flow is to use foam insulation backed by an ambient temperature of 0°C, as in the TRRL test.

The surcharges used in the direct frost heave tests reviewed ranged from 0 to as much as 18 kPa; most used little or no surcharge (the TRRL, Aguirre-Puente and Dupas [1970] and Loch [1979a] tests, for example). The next most frequently used surcharge was one designed to simulate the load due to pavement. These surcharges ranged from 2.2 to 5.9 kPa; the 3.6-kPa surcharge of the Zoller, CRREL, Gorle (1980) and Esch et al. (1981) tests is typical. The surcharge could be varied in the Vasilyev (1973), Penner and Ueda (1977) and Fredén and Stenberg (1980) tests. Other tests could probably be modified readily to make the surcharge variable, as the CRREL test has been on several occasions (Carbee, pers. comm.).

All of the tests surveyed provided free access to water at the base of the sample during freezing. One test (Vlad 1980) specified that the water availability should approximate the in situ conditions. If specific site conditions are not known, free access to water is probably the preferable method because it simulates the worst moisture conditions.

Because of the heterogeneity of soils, it is important that a FS test integrate the heave response over a sufficient sample length to account for material variations. The TRRL and CRREL tests are the only tests that subject the entire length of the specimen to freezing. In the other tests the freezing zone does not reach the warm end. In most of these, however, the critical FS factor (the magnitude or rate of frost heave) develops over several centimeters of frost penetration and thus probably satisfies this requirement.

None of the FS tests provides for varying the moisture tension, as all are directed toward the most severe condition of saturation. In all but the Penner and Ueda test and the Fredén and Stenberg test, the moisture-tension profile could be readily adjusted to simulate the depth to the water table if modifications were made to allow the water table to be lowered or a vacuum to be applied. These methods are, however, limited to moisture tensions of 1 atm at the sample base. The problem with freezing samples upward from the bottom is that the effects of lowering the water table cannot be simulated precisely because the tension developed in the freezing zone is reinforced by the tension developed due to gravity. In top-to-bottom freezing, this tension is opposed by gravity.

This may or may not be a problem, depending on the relative levels of the moisture tensions due to freezing and the tensions in situ. If the tensions developed during freezing are very large compared to the in situ values, as they would be in fine-grained materials, then there may be little effect on the test. However, if the freezing tensions are only slightly greater than the in situ tensions, then the direction of freezing may have some effect on the observed frost heave. Whether or not this is a problem is uncertain and can only be resolved with controlled tests.

Most of the frost heave tests use compacted soils that are prepared to replicate field conditions or are compacted to some adopted standard, such as the Proctor test. Only Loch (1979b) suggests that undistubed samples be used. (Undisturbed samples are sometimes used in the CRREL test, but the normal procedure is to use remolded samples.) Undisturbed samples are preferable if they can be obtained. For coarse-grained materials this is usually impossible. As an alternative, compacted samples can be submitted to several freeze-thaw cycles, the theory being that the freeze-thaw cycling will condition the soil as it would under natural conditions. Balduzzi and Fetz (1971), Jessberger and Heitzer (1973), Vasilyev (1973), Brandl (1980) and Fredén and Stenberg (1980) all suggest that two or more freeze-thaw cycles be used. As previously discussed, the considerable experience at CRREL in frost heaving testing has revealed that the rate and amount of heave are considerably higher after two or more freeze-thaw cycles.

In most of the methods surveyed, the samples are saturated by capillary action, i.e. by raising the water table to the base of the sample and maintaining that level for one or more days. Jessberger and Heitzer (1973) follow a 24-hour capillary saturation by 72 hours of total submersion. Zoller (1973) submerges his samples for 16 hours. Others, such as Kaplar (1974), Penner and Ueda (1977) and Gorlé (1980), use a vacuum saturation method. The last method, while more complicated than the others, is preferable because the sample can be more completely saturated and the moisture conditions in duplicate samples can be reproduced more accurately.

Most of the tests use the amount of frost heave at a given time or the rate of frost heave as the critical factor in determining the FS of soils. The TRRL, CRREL, Penner and Ueda, Loch, and Fredén and Stenberg tests are examples. Others, such as Ducker (1939), Balduzzi and Fetz (1971) and Vasilyev (1973), use the heave ratio. The Aguirre-Puente et al. test is unique in that it employs the ratio of frost heave to the square root of the freezing index. Still others (Brandl [1970, 1980], Balduzzi and Fetz, Jessberger and Heitzer, and Vlad [1980]) use the CBR after freeze-thaw cycling as the indicator of FS. These last four methods are the only frost heave tests that employ a direct measure of thaw weakening in their FS criteria.

Testing time is perhaps the most significant factor affecting the choice of test for some laboratories. The amount of time required to conduct the freezing and/or thawing portions of the tests ranged from four hours (Ducker 1939) to 28 days (Brandl 1970). Eleven of the tests could be completed in one week or less. The TRRL and CRREL tests, which are among the more widely used tests, require 10 and 12 days, respectively.

SELECTION OF FROST SUSCEPTIBILITY TESTS FOR FURTHER ANALYSIS

The analysis of the literature related to frost heave, thaw weakening and frost susceptibility testing has made it clear that there is much yet to learn. The mechanism of frost heave has not been clearly identified nor have all the factors affecting frost heave been resolved. Most important for this study, no FS index test has emerged as the ultimate solution for selecting non-frost-susceptible materials or for determining frost heave or thaw weakening under field conditions.

Since we need reliable FS criteria, however, it is essential that we analyze further some of the more promising tests. The choices should include tests of several levels of complexity and sensitivity. If an array of tests was available, engineers could select a test with the appropriate degree of reliability and complexity. The prospective FS tests are therefore chosen from four levels in the hierarchy of FS testing. The first and most basic test is based on *grain size characteristics*. The second test is related to the more fundamental *moisture-tension hydraulic-conductivity* aspects of frost heave. The third is an actual *frost heave test*. And the final method is the *thaw-CBR test*.

Grain size distribution test

Three classification systems based on grain size emerge as candidates for further consideration. They are the U.S. Army Corps of Engineers (1965), the Swiss (Association of Swiss Road Engineers 1976) and the West German (Germany 1979) FS classification systems. These have been selected from the list of nearly 100 classification systems (Appendix A) because they appear to be the most rigorously developed. The others have been excluded from further consideration because their data bases are limited, such as for most of the states or provinces where only regional conditions are considered, or because they have evolved into more recent FS criteria, such as those of Casagrande (1931), Beskow (1935, 1938), Ducker (1939) and Schaible (1950, 1953, 1957).

The West German classification system has evolved from the work of Schaible, under the influence of Hans Jessberger of Ruhr University at Bochum. In several reports (Jessberger and Hartel 1967, Jessberger 1969, 1973, 1976), Schaible has evaluated the problem of determining the FS of soils. As a result of these reviews and his own studies (Jessberger and Carbee 1970, Jessberger and Heitzer 1973, and Jessberger 1976) Jessberger concluded that the reduced bearing capacity after thaw is the most important factor in any FS classification system. The standard (Table 38) now under consideration for adoption in West Germany (Germany 1979) relates the FS of soil to the soil type on the basis of thaw-CBR values. Their classification system is similar to that proposed by Jessberger (1976) but has combined the low and medium FS categories into a single class. The standard thus includes three classes of FS, rather than four as originally suggested by Jessberger. The German system also provides a procedure for conducting CBR tests after freeze–thaw cycling when materials of questionable classification are encountered.

The Swiss FS standards were originally developed from Casagrande's (1931) grain size criteria and the Corps of Engineers criteria based on the Unified Soil Classification System (Bonnard and Recordon 1958). Recently Bonnard and Recordon (1969) proposed that the CBR after thaw be included. Recordon and Rechsteiner (1971) introduced further changes for granular materials to incorporate the coefficient of curvature, the optimum water content during compaction, and the CBR after freezing and thawing or soaking.

The current Swiss (Association of Swiss Road Engineers 1976) FS classification system includes three levels of screening (Tables 20–22). The first level is a grain-size criterion based on Casagrande's criterion. This level separates non-frost-susceptible soils from those of unknown FS. Questionable soils are subjected to a second level of screening that is based on soil type. As with the first level of screening, the second level does not distinguish between frost heaving and thaw-weakening potential.

A third level of screening is for sand and gravel subbase and base course materials of still questionable FS. At this level the Swiss separate coarse material into two categories: Gravel I and Gravel II. Gravel I is the material that passes the first two levels of screening and needs no further testing. Gravel II materials must pass additional classification tests and must be submitted to a CBR test after soaking or one freeze–thaw cycle (the criteria for selecting one of these two options are unknown).

The U.S. Army Corps of Engineers FS classification system (Table 32) also has evolved from Casagrande's original work (1931). In the 1930's, Casagrande (1934, 1938) clarified his grain-size criteria; he later (Casagrande 1947) proposed a FS classification system based on the Unified

Soil Classification. Numerous studies by the U.S. Army Corps of Engineers Arctic Construction and Frost Effects Laboratory in the laboratory and in the field led to the development of a FS classification system (Linell and Kaplar 1959) based on three levels of screening: 1) the percentage smaller than 0.020 mm, 2) the soil classification, and 3) a frost heave test. The first two levels of screening are the same as the basis of the Swiss criteria. The third level of screening differs from that of the Swiss in that it calls for a frost heave test rather than a CBR test after freezing and thawing. The standards presently in use are provided in a U.S. Army technical manual (U.S. Army Corps of Engineers 1965).

The soil classification test that emerges as the candidate for further consideration is the U.S. Army Corps of Engineers (1965) Frost Design Soil Classification System. Together with the Unified Soil Classification equivalent groupings and the CRREL standard frost heave data, this method has probably the largest data base of any grain size or soil classification method. The great advantage of this method is that it does not require a higher level test (CBR or frost heave) for soils of questionable FS. The amount of frost heave and thus the FS classification can be estimated from the large tabulation (Appendix B) of previous frost heave test results. Another advantage to this study is that CRREL personnel have ready access to the data, the soils, the CRREL frost heave test equipment, and the field sites on which the method was established.

The disadvantage of the Corps method is that it is based on frost heave, although according to Linell and Kaplar (1959), thaw-weakening characteristics determined from field plate-bearing tests have also been taken into account. This is different from the West German method, which directly incorporates the reduced bearing capacity after thaw.

Whether or not the methods of the Swiss or Germans are better than the Corps method is uncertain, particularly in view of the fact that there has been little field experience reported with the European methods. The only certain way of determining the relative merits of the three criteria is to subject them to a rigorous laboratory and field evaluation. Such an evaluation is beyond the scope of this study.

Moisture-tension hydraulic-conductivity tests

Moisture-tension hydraulic-conductivity tests address the fundamental causes of frost heave more closely than grain size tests do. With test results characterizing the flow of water in soils, one can discuss all but the thermal dynamics of frost heave. Of the index tests reviewed only the critical permeability-suction test of Wissa et al. (1972) allows the determination of the moisture-tension and hydraulic-conductivity characteristics. It is recommended, however, that neither their equipment nor their method of analysis be employed. Their equipment has proven to be unreliable. Their method of analysis assumes that single points on the continuous moisture-tension and hydraulic-conductivity curves are uniquely related to the frost heave mechanism. This is probably not justified, as moisture flow occurs over a range of suction values and hydraulic conductivities. Alternative methods of analysis must be developed that use more data from these curves. The moisture-tension curve also has a side benefit in that an effective pore size distribution curve can be developed from it.

The pressure cell permeameter being used at CRREL (Ingersoll 1981) to determine moisture-tension and hydraulic-conductivity curves has advantages over the apparatus of Wissa et al. The sample is placed in a cell with an inside diameter of 7.5 cm and a height of 10.0 cm (Fig. 73). The porous piezometer cups for measuring the head loss during the hydraulic conductivity part of the test are 6.4 mm in diameter. (They are considerably larger, expose more surface area, and thus respond faster than those used by Wissa et al. [1972].) The distance between the pressure sensors (6 cm) is also a large improvement over the Wissa et al. apparatus because it allows the pressure gradient to be determined more accurately. Positive air pressure is applied through a porous screen on the side of the cell to establish a pressure differential between the water in the soil voids and the atmospheric pressure. Water is expelled until the soil-water tension is in equilibrium with the pressure differential.

After equilibrium is achieved, a falling-head permeability test is conducted while maintaining the pressure differential. The inflow and outflow of water are monitored until they are equal. The process is repeated at increasing increments of pressure. The present moisture-tension limit with the apparatus is 0.85 atm. However, it is now being modified to operate at up to 3 atm of tension.

The hydraulic conductivity is calculated using the following form of Darcy's equation:

$k = QL/hAt$

where k = hydraulic conductivity
Q = quantity of flow in time t
A = area of sample
L = distance between tensiometers
h = head loss between tensiometers.

Figure 74 shows a hydraulic conductivity versus moisture tension plot for silt, sand and till soils. Figure 75 compares the results with moisture-tension values obtained using a standard pressure-plate extractor. This apparatus has proven to be reliable for a range of soil types, including coarse-grained sands and gravels.

This device has several advantages over that of Wissa et al. (1972). First, it appears to be more reliable. The test is being routinely conducted at CRREL in support of a number of research programs. Second, a data base of moisture-tension hydraulic-conductivity curves is being established for a large number of soils. Finally, many of the results are being used as input into the mathematical model now being developed at CRREL. The same hydraulic-conductivity moisture-tension data can thus be useful in two approaches to the FS problem, one complementing the other. Experience with the model may help in selecting the appropriate hydraulic-conductivity moisture-tension characteristics to use as

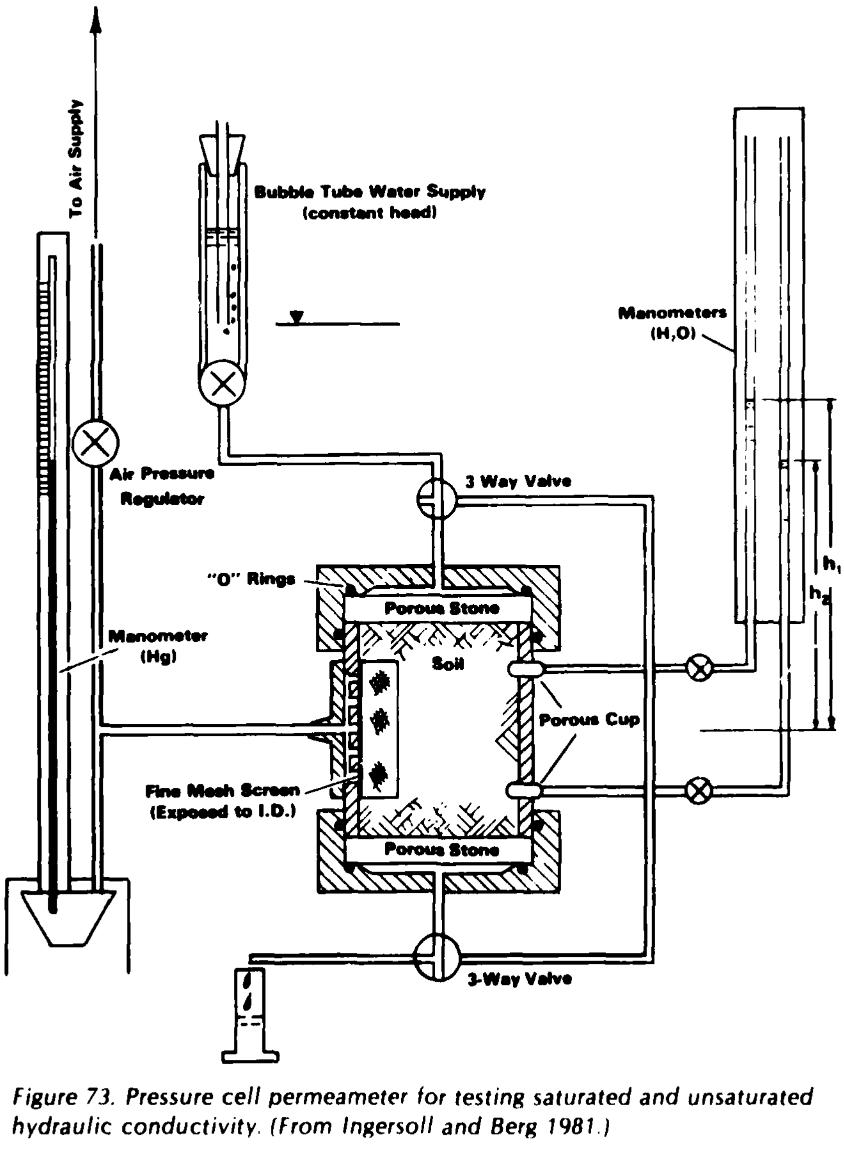


Figure 73. Pressure cell permeameter for testing saturated and unsaturated hydraulic conductivity. (From Ingersoll and Berg 1981.)

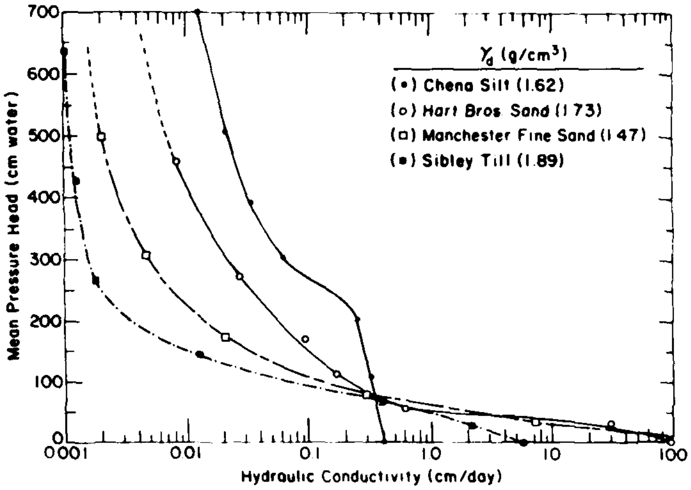


Figure 74. Typical hydraulic conductivity vs pressure head (tension) curves. (From Ingersoll and Berg 1981.)

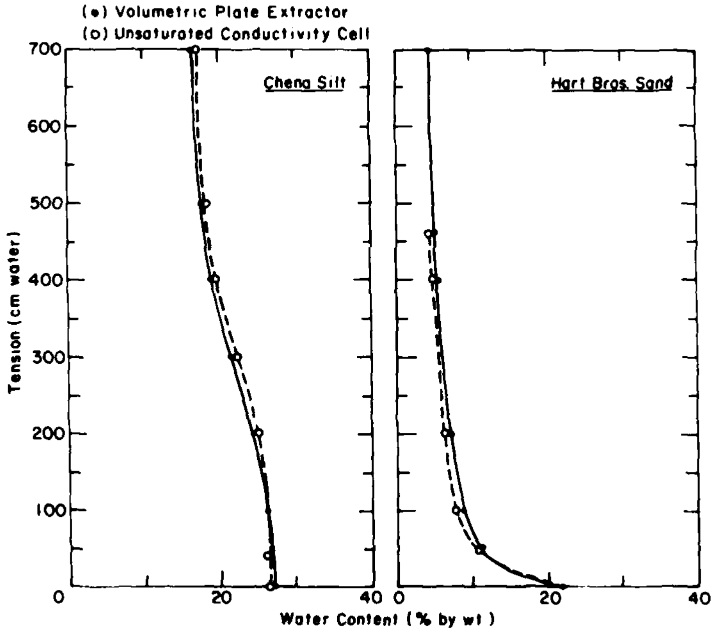


Figure 75. Moisture tension test results using a volumetric plate extractor and an unsaturated conductivity cell. (From Ingersoll and Berg 1981.)

an index of FS, while conducting more moisture-tension hydraulic-conductivity tests may help in developing the frost heave model.

Frost heave test

Before selecting a frost heave test, we must establish some guidelines for making the choice.

1. A good test should be as simple as possible, so that highway and geotechnical laboratories can conduct tests readily and obtain reliable reproducible results.

2. The equipment must be reliable.

3. The test must relate to frost heave in the field.

4. It must be of short duration.

5. It must accommodate the complete range

Table 50. Characteristics of existing frost heave index tests.

Country	Source	Temperature control: Cold end	Temperature control: Warm end	Side friction control	Radial heat flow control	Variable surcharge	Variable water table	Undisturbed samples	Duration of test	Simplicity	Complete range of materials	Correlated with field observations	Cost per test	Frost susceptibility classification	Thaw weakening: Field	Thaw weakening: Lab	Freeze-thaw cycling
Austria	Brandl (1970)	fair	good	very good	good	possible	no	possible	poor	simple	no	no	low	no	no	yes	yes
Austria	Brandl (1980)	fair	good	very good	good	possible	no	possible	poor	simple	yes	yes	low	yes	yes?	yes	yes
Belgium	Gorlé (1980)	fair	good	very good	good	possible	no	possible	excellent	simple	yes	no	medium	no	no	no	no
Canada	Penner & Ueda (1977)	good	fair	very good	very good	yes	possible?	possible	good	complex	marginal	no	high	no	no	no	no
England	Croney & Jacobs (1967)	fair	good	good	good/fair	possible	no	possible	poor	simple	yes	yes	very low	yes	yes?	no	no
England	Jones & Dudek (1979)	very good	very good	good	good/fair	possible	possible	possible	good	complex	yes	no	high	no	no	no	no
France	Aguirre-Puente et al. (1975)	good	good	good	very good	possible	possible	possible	fair	complex	no	?	high	yes	?	no	no
Norway	Loch (1979a)	good	good	good	good	possible	possible	yes	very good	complex	marginal	no	medium	no	no	no	no
Romania	Vlad (1980)	fair	fair	fair/good	good	yes	yes?	possible	poor	complex	marginal	no?	low	yes	yes?	yes	yes
Sweden	Fredén & Stenberg (1980)	very good	poor	very good	good	yes	possible	possible	?	complex	marginal	no	high	no	no	no	yes
Switzerland	Balduzzi & Fetz (1971)	fair	good	good	good	possible	no	possible	very good	simple	no	no	low	no	no	yes	no
U.S.A.	Esch et al. (1981)	fair	good	very good	?	possible	no	possible	poor	simple	yes	no?	low	yes	yes?	no	no
U.S.A.	Kaplar (1974)	fair	fair	fair/good	good	possible	possible	possible	poor	complex	yes	yes	low	yes	yes?	no	no
U.S.A.	Kalcheff & Nichols (1974)	fair	fair	good	good	possible	no	possible	fair	simple	yes	no	very low	no	no	no	no
U.S.A.	Sherif et al. (1977)	fair	good	fair/good	poor	possible	no	possible	poor	simple	yes	no	low	no	no	no	yes
U.S.A.	Zoller (1973)	very good	poor	very good	good	possible	possible	possible	excellent	complex	yes	incomplete	high	yes	no?	no	no
U.S.S.R.	Alekseeva (1957)	fair	good	very good	poor	possible	no	possible	excellent	simple	no	no	low	no	no	no	no
U.S.S.R.	Vasilyev (1973)	fair	good	very good	fair	yes	no	possible	?	simple	marginal	yes?	low	yes	no	no	?
W. Germany	Ducker (1939)	fair	good	very good	poor	possible	no	possible	excellent	simple	no	no?	low	yes	no	no	no
W. Germany	Jessberger & Heitzer (1973)	fair	fair	fair/good	good	possible	possible	possible	good	simple	yes	incomplete	low/med.	yes	yes?	yes	no

of material types; in particular it must accommodate granular base and subbase materials and fine-grained subgrade materials.

6. The apparatus should be inexpensive to construct and operate.

One objective is not clear: Should the test replicate field conditions so that actual frost heave can be predicted, or should the test be only an index test that imposes the most severe conditions? Perhaps the best answer would be to develop a frost heave index test for the more severe conditions and to correlate that index test with field observations. From the review it is apparent that the most severe conditions for frost heave include 1) saturation prior to freezing, 2) freely available water, 3) no surcharge, 4) a critical rate of heat removal, and 5) a critical temperature gradient. When there is a sufficient body of knowledge so that the FS criteria developed for these conditions are reliable, the test procedures can then be modified to simulate actual field conditions.

The test should also accommodate both remolded and undisturbed samples and should be readily adaptable to simulate other than the most severe conditions in the field. The surcharge should be adjustable and side friction must be kept to a minimum. To simulate field temperatures, precise temperature control must be available at the top and bottom of the sample, and lateral heat flow must be kept to a minimum. It should also be possible to vary the depth to the water table. Table 50 includes a checklist of these desirable characteristics.

None of the methods surveyed fulfills all these requirements. Thus, one has the choice of accepting an imperfect test or introducing desirable modifications. The question, then, is whether the large data bank on frost heave obtained using the CRREL test should be abandoned in favor of a test that is much faster and has fewer problems with side friction and freezing method.

If a frost heave test is to be successful, it must exclude the known imperfections and resolve the difficulties that limit its reliability. A better frost heave test should be established and a new body of experience developed to support it. Perhaps some correlation with the CRREL frost heave test results can be made. The new test should include 1) a multi-ring freezing cell (MRFC), 2) circulating-liquid-cooled cold and warm plates, 3) an air-cooled room or cabinet for multiple samples, 4) variable surcharge, and 5) adjustable moisture tension.

The MRFC appears to be the best method for minimizing side friction while accommodating the other important factors. Bottom-up freezing is probably better (except for coarse materials), but it is difficult to use this technique, simulate field moisture-tension profiles, and overcome the compaction problems at the same time. The MRFC is not a new development in frost heave testing, as it was employed long ago by Taber (1929) and Ruckli (1950). The considerable experience with this method has revealed certain drawbacks. For instance, it is difficult to completely saturate a specimen in a vacuum (Kaplar 1971), and when non-cohesive sandy soils are being tested, grains tend to fall through the joints between the ring segments beneath the freezing zone (Carbee, pers. comm.).

The MRFC, however, is preferable to the methods employing waxed paper, cellulose foil, polyethylene film, or foam rubber tubing, because it appears to offer less heave resistance. This choice has been made with some uncertainty, and alternatives should also be explored. These include, but are not restricted to, 1) lining the MRFC with a rubber membrane and 2) using polyethylene film or a rubber membrane alone to contain the sample.

The sample should be large enough to accommodate coarse-grained gravels but not so large as to require large amounts of material. A sample with both a diameter and height of about 15 cm would be appropriate.

The sample should be placed in the MRFC in an undisturbed condition when possible. If undisturbed samples cannot be obtained the test specimen should be compacted to approximate the in situ density.

Prepared test samples should be saturated by soaking or by adding degassed water under a vacuum. The latter method produces more repeatable results, but soaking is more practical for most laboratories.

Although the moisture tension should be adjustable, the pore water tension in the standard test should be near zero to simulate a high water table. A constant-head Mariotte table water supply device should be used to maintain the water table near the zone of freezing.

The surcharge should be variable to simulate field conditions. Perhaps the 3.6-kPa value used by CRREL should be used as a standard. Air loading devices have been used at CRREL (Carbee, pers. comm.) and are very simple and reliable.

Temperature control is probably best accomplished by circulating a non-freezing liquid from controlled temperature baths through plates *placed in good thermal contact with the upper* and lower surfaces of the test sample. Thermal contact between the cold plate and the soil specimen must be maintained to prevent needle ice from forming (Carbee, pers. comm.).

The heat extraction rate imposed in the test should represent a severe condition or simulate the actual field conditions. The user of the test should have the option to impose either rate.

According to Horiguchi (1978) and Loch (1979a), the optimum heat extraction rate for silts and clays is near 150 W/m² (no data are available for sands and gravels). Loch (1979b) determined that the heat extraction rate ranges between 20 and 120 W/m² immediately beneath asphalt concrete pavements in southern Norway; he therefore chose a heat extraction rate of 120 W/m² for the Norwegian direct frost heave test. Fréden and Stenberg (1980), however, suggested that 490 W/m² be used in the Swedish test. According to Loch (1979a), a heat extraction rate that high would preclude frost heave in undisturbed silts and clays. Therefore, a standardized heat extraction rate more nearly like that suggested by Loch should be used in the test being proposed.

The temperature gradient should also either represent a severe condition or simulate an actual field condition. Gorlé (1980) showed clearly that this is important, especially for coarse materials. He observed that the rate of heave increased significantly as the temperature gradient increased from 0.1 to 2.5°C/cm. Temperature gradients near 0.05°C/cm in the region immediately beneath the freezing front occur naturally during much of the freezing season in the region near CRREL.

The limits of the apparatus must also be considered. If a 0.05°C/cm gradient is used, the temperature difference over the length of the sample (assuming a 15-cm-long sample) would be 0.75°C. This would be difficult to sustain in such a relatively short column. A temperature gradient of 0.25°C/cm beneath the freezing front would be an appropriate compromise. The temperature difference over the length of the sample would then be 3.75°C.

At least two freeze-thaw cycles should be employed to account for the changes that occur under natural freezing conditions. This is important in determining FS because repeated freeze-thaw cycling is always a factor in freezing soils, whether the cycles are generated during a single season or during several successive seasons.

To limit the test to one week and still complete at least two freeze-thaw cycles will require careful design of the freezing conditions. The first freeze can be accomplished at a relatively high rate of heat removal, so that the full length of the sample freezes and thaws within two days. The second cycle should be designed so that only the upper 5-7 cm of the sample are frozen (at a rate of heat removal of approximately 100 W/m²). The second freezing would occur on the third day and thawing on the fourth day. Additional freeze-thaw cycles will have to be performed to validate this procedure, and the test should be modified if necessary.

The MRFC should be insulated radially with foam insulation extending sufficiently above the cooling plate to ensure that no ring is exposed to the ambient temperature as heaving occurs. The insulated MRFCs should be placed in a cold box or coldroom where the ambient temperature is near freezing. Alternatives are to surround the entire MRFC with a guard ring to maintain the side temperature near the desired ambient temperature or to provide sufficient insulation so that radial heat flow is not a problem. Obviously, if the cold box or room can be eliminated, the test would be simpler and much less expensive.

Moisture tension is probably best varied by adjusting the height of the water reservoir or by applying a vacuum to the reservoir. With these methods the tension is limited to 1 atm by the cavitation pressure of water. This is probably sufficient for most tests, particularly for the dirty gravels which are of much concern.

Thaw-CBR test

The literature revealed few index tests for thaw weakening. The most frequently discussed method is the thaw-CBR test (e.g. Jessberger and Carbee [1970] and Jessberger [1975]). The CBR-after-thawing test procedures included in the methods of Austria, Germany, Romania and Switzerland are the only index test procedures specifically developed for determining thaw-weakening susceptibility. Others, such as the U.S. Army Corps of Engineers soil classification system, consider thaw weakening only indirectly.

The repeated-load triaxial test now being conducted at CRREL (Chamberlain et al. 1979) may also be considered to be a thaw-weakening test. Its use, however, requires a commitment to an elastic layer analysis for pavement design. It is not an index test, as it provides specific values for the resilient modulus and resilient Poisson's ratio for the entire freeze, thaw and recovery periods.

Other methods, such as the unconfined compression test (Dempsey and Thompson 1973), the triaxial compression test (Broms and Yao 1964), and the direct shear test (Thomson and Lobacz 1973), have been used but are too complicated or are incompatible with a frost heave test. Still other methods, such as the shear vane and cone penetrometer, cannot be used with coarse-grained materials.

Using the CBR test for thaw weakening is a rational approach, particularly where the CBR test is used in designing pavement systems. It is a standard test conducted by many transportation departments and geotechnical laboratories; it is much simpler to conduct than the repeated load triaxial test. It is also readily adapted to a frost heave test.

The CBR test after freezing and thawing should be considered as one additional procedure in the frost heave test for use with specific soil types, particularly sands and gravels. Procedures should be developed to include both frost heave and thaw weakening in the FS criteria.

CONCLUSIONS

This review identified over 100 methods for determining the frost susceptibility of soil. Of these, most are based on particle size characteristics, four on pore size characteristics, five on the interaction of soil and water, three on the interaction of ice, soil and water, and twenty on frost heave.

The criteria based on particle size characteristics are the most popular, probably because they require little or no more testing than is normally done for roads and other heavy construction projects. However, particle size methods are often unsuccessful because they address only part of the problem of frost susceptibility. Few address the effects of mineralogy, moisture, density, structure, freezing conditions, and surcharge.

The pore size, soil/water interaction, and ice/soil/water interaction tests are all closer to addressing the causes of frost heave, but none have proven to be the universal solution to determining susceptibility. Even frost heave tests, which may appear to be the ultimate solution, have not proven to be so.

Because we need reliable frost susceptibility tests, however, it is essential that the more promising tests be analyzed further. The tests should be of several levels of complexity and sensitivity to allow project or design engineers to select a method with the appropriate degree of reliability and complexity.

The simplest test should be based on grain-size characteristics. The frost susceptibility classification system developed by the U.S. Army Corps of Engineers appears to be the best of this type. The second test recommended for further evaluation is the more fundamental moisture-tension hydraulic-conductivity test presently being used at CRREL. The third is a frost heave test that will allow the frost heave susceptibility to be determined for both severe conditions of heat flow and moisture availability and actual field conditions. Because of limitations in all available frost heave tests, a new frost heave test should be developed incorporating the best features of the present tests. Finally, a CBR test after thawing should be evaluated as an index for thaw weakening susceptibility, and procedures should be developed to include both frost heave and thaw weakening in a new frost susceptibility classification method.

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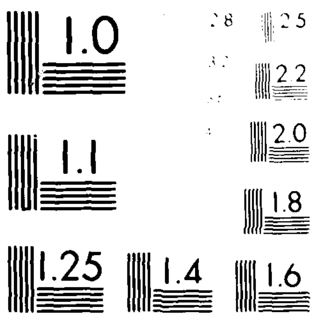

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APPENDIX A. FROST SUSCEPTIBILITY CLASSIFICATION METHODS BASED ON GRAIN SIZE CHARACTERISTICS

APPENDIX A. FROST SUSCEPTIBILITY CLASSIFICATION METHODS BASED ON GRAIN SIZE CHARACTERISTICS.

Organization	*Source*	*Year*	Allowable percentage passing *0.074 mm*	*0.020 mm*	*Other size*	*Uniformity*
Austria						
	Brandl	1976	–	3	–	–
	Brandl	1979	–	3	–	–
Canada						
Alberta	Johnson et al	1975	–	3	–	yes
				10	–	yes
Canadian D.O.T.	Armstrong & Csathy	1963	15	–	–	–
Canadian Natl. Parks	Armstrong & Csathy	1963	36	–	–	–
Manitoba	Armstrong & Csathy	1963	60	–	·30%	–
New Brunswick	Armstrong & Csathy	1963	50	–	–	
			6-8	–	–	–
Newfoundland	Armstrong & Csathy	1963	6	–	–	–
Nova Scotia	Armstrong & Csathy	1963	10	–	–	–
Ontario	Townsend & Csathy	1963a	40	–	45% fine sand and silt	–
	Johnson et al	1975	8	–	–	–
Quebec	Armstrong & Csathy	1963	10	–	· 20% fine sand and silt	–
	Johnson et al	1975	10	–	· 3% · 0.053 mm	–
Saskatchewan	Johnson et al	1975	7-10	–	–	–
			20	–	–	–
Denmark						
State Road Lab	Riis	1948	–	10	· 50% · 0.125 & · 35% · 0.062 mm	–
			–	3	–	–
	Christensen & Palmquist	1976	10	–	–	–
East Germany						
	Klengel	1970	–	–	· 10% · *0.10 mm*	–
England						
Road Res. Lab	Croney	1949	–	·70 or · 20	–	–
Finland						
	Orama	1970		3-10	–	yes
Greenland						
Greenland Tech. Org	Nielsen & Rauschenberger	1957	5	–	–	–
			35	–	· 50% · 0.125 mm	yes
			–	–	–	yes
Japan						
	Jessberger	1969	6	–	–	–
Netherlands						
	von Moos	1956	–	–	· 5% · 0.05 mm	–
Norway						
	von Moos	1956	–	–	· 25% · 0.25 & · 20% · 1.0 mm	–
	Christensen & Palmquist	1976	–	–	· 20% · 0.125 mm	–
Poland						
Cracow Tech. Univ	Pietrzyk	1980	–	–	grain size curves	–
Romania						
Polytech. Inst. Iassy	Vlad	1980	–	10	· 1% · 0.002 & · 20% · 0.1 mm	–
Sweden						
	Beskow	1935	–	–	· 30% · 0.062 & · 55% · 0.125 mm	yes
			–	–	15% · 0.062 & · 22% · 0.125 mm	yes
	Beskow	1938	40	–	–	–
			19	–	–	–
Natl. Road Res. Inst	Rengmark	1963	–	–	–	–
	Freden & Stenberg	1980	16	–	–	–
Switzerland						
Swiss Fed. Gov't	Ruckli	1950	17	–	or 22% · 0.125 mm	–
	Bonnard & Recordon	1958	–	3	–	yes
	Bonnard & Recordon	1969	–	3	–	yes
			–	3	–	yes
			–	3-10	–	
	Recordon & Rechsteiner	1971	–	3	–	yes
			–	3		yes
				10		*no*
	Jessberger	1976		1.5		yes
				10		yes
				3		yes
				3	–	yes
				3-10	–	no
United States						
Alaska	Johnson et al	1975		3	–	yes
	Esch et al	1981	6			
Arizona	Erickson	1963	8-12			

Atterberg limits	Other factors	Type of classification	Material type	Comments
—	mineralogy	pass/fail	base	
—	mineralogy	pass/fail	base	
yes	soil classification	degree	subgrade	after U S Army Corps of Engrs (1965)
yes	soil classification	degree	base/subbase	
—	grain size curves	degree	all	
yes	—	pass/fail	silts, clays	after Beskow (1935) and Casagrande (1931)
yes	—	pass/fail	all?	
—	mineralogy	pass/fail	silts	
—	mineralogy	pass/fail	gravels	
—	—	degree	base	
—	—	degree	all	
—	—	degree	all	
yes	—	pass/fail	all	
—	—	degree	all	
—	—	pass/fail	subgrade	
—	—	pass/fail	base	
—	—	pass/fail	subbase	
—	—	pass/fail	homogeneous moraines	
—	grain size curves	pass/fail	heterogeneous sediments	
—	—	pass/fail	base/subbase	
—	—	pass/fail	gravels, crushed stone	
—	grain size curves	degree	all	
—	capillarity/grain size curves	degree	all	after Casagrande (1931)
—	—	pass/fail	all	
—	grain size curves	pass/fail	homogeneous soils	in % of fraction · 2 mm, based on frost heave
—	grain size curves	pass/fail	heterogeneous soils	
—	—	pass/fail	sands and gravels	also crushed rock
—	organic content	pass/fail	all	
—	—	pass/fail	all	
—	—	pass/fail	all	
—	surcharge	pass/fail	all	based on lab frost heave tests
yes	—	—	all	after Schaible (1957), Romanian std
—	capillarity, hygroscopocity	degree	homogeneous moraines	
—	capillarity, hygroscopocity	degree	heterogeneous sediments	in % of fraction · 2 mm
—	capillarity	pass/fail	homogeneous moraines	
—	capillarity	pass/fail	heterogeneous sediments	
—	soil type	degree	all	based on frost heave and thaw weakening
—	capillarity	degree	all	after Beskow (1935)
—	soil type	degree	all	also water table and permeability
yes	soil classification	degree	all	after Casagrande (1931), Swiss std
—	coefficient of curvature	pass/fail	all	after Casagrande (1931), Swiss std, in % of fraction · 100 mm
—	coefficient of curvature	pass/fail	sand	after Casagrande (1931), proposed new std
yes	frost heave/thaw CBR	pass/fail	gravel, crushed stone	in % of fraction · 100 mm
—	coefficient of curvature	pass/fail	sand, gravel, crushed stone, undisturbed	after Casagrande (1931), Swiss std, in % of fraction < 200 mm
—	coefficient of curvature, w_{op}	pass/fail	sand, gravel, crushed stone compacted	after Casagrande (1931), Swiss std, in % fraction < 100 m, · 2% increase in % · 0.02 mm after compaction
yes	thaw or soaked CBR, w_{op}	pass/fail	sand, gravel, crushed stone	after Casagrande (1931), Swiss std
—	—	degree	most soils	of % of fraction · 60 mm
—	—	pass/fail	homogeneous sands C_u · 5	
yes	soil classification	degree	all	based on U S Army Corps of Engrs (1965)
—	coefficient of curvature, w_{op}	pass/fail	sand, gravel, crushed stone	in % of fraction · 100 mm
yes	(thaw CBR, w_{op}	pass/fail	sand, gravel, crushed stone	
yes	soil classification	degree	all	after U S Army Corps of Engrs (1965)
—	—	pass/fail	base and subbase	based on lab & field frost heave observations
—	elevation	pass/fail	all	

Appendix A (cont'd). Frost susceptibility classification methods based on grain size characteristics.

Organization	Source	Year	Allowable percentage passing 0.074 mm	0.020 mm	Other size	Uniformity
United States (cont'd)						
Asphalt Institute	Johnson et al	1975	7	—	—	—
Bureau of Public Roads	Morton	1936	—	—	—	yes
California	Johnson et al	1975	5			
Colorado	Johnson et al	1975	5-10	—	—	—
Connecticut	Johnson et al	1975	10	3	—	yes
			10	10	—	yes
Delaware	Haley	1963	35	—	—	—
Idaho	Erickson	1963	36	—	—	—
	Johnson et al	1975	5	—	—	—
Illinois	Johnson et al	1975	36	—	—	—
			70	—	—	—
Iowa	Johnson et al	1975	15	—	—	—
Kansas	Johnson et al	1975	15	—	—	—
Maine	Johnson et al	1975	5	—	—	—
			7	—	—	—
Maryland	Johnson et al	1975	12	—	—	—
Massachusetts	Haley	1963	15	—	—	—
	Johnson et al	1975	12	—	—	—
			10	—	—	—
Mass. Inst. Tech.	Casagrande	1931	—	3	—	yes
			—	10	—	yes
	Casagrande	1947	—	—	—	yes
Mass. Turnpike Auth.	Johnson et al	1975	10	—	—	—
Michigan	Johnson et al	1975	—	—	< 7% fines lost by washing	—
Minnesota	Johnson et al	1975	10	—	—	—
Montana	Erickson	1963	12-35	—	0.42 mm & 2.0 mm	—
Nebraska	Johnson et al	1975	8-12	—	—	—
			5-13	—	—	—
New Hampshire	Haley	1963	10	—	—	—
	Johnson et al	1975	3	—	—	—
			8	—	—	—
			12	—	—	—
New Jersey	Turner & Jumikis	1956	25	—	—	—
New York	Haley	1963	—	3	—	yes
			—	10	—	yes
Ohio	Johnson et al	1975	15	—	—	—
Oregon	Erickson	1963	10	—	—	—
	Johnson et al	1975	8	—	—	—
Texas	Carothers	1948	16	8	—	—
U.S. Civil Aero. Admin.	Townsend & Csathy	1963a	15-25	—	—	
U.S. Army Corps of Engrs	Linell & Kaplar	1959	—	3	—	—
	U.S. Army Corps of Engrs	1965	—	1.5	—	yes
			—	3	—	yes
				10		yes
U.S. Army Engr. WES	USAE WES	1957	—	—	—	—
Utah	Erickson	1963	25	—	—	—
Vermont	Haley	1963	10	or 3	—	—
	Johnson et al	1975	36	—	—	—
Washington	Johnson et al	1975	10	—	—	—
Wisconsin	Johnson et al	1975	5	—	—	—
Wyoming	Erickson	1963	20	—	—	—
West Germany						
	Ducker	1939	—	3	see text	—
	Floss	1973	—	—	—	
Fed. Trans. Ministry	Jessberger	1969	—	—	· 10% · 0.1 mm	—
			—	—	grain size curves	—
	Germany	1979	—	—	· 5% · 0.063 mm	yes
Tech. Univ., Munich	Jessberger & Hartel	1967	—	—	grain size curves	yes
Ruhr-Univ., Bochum	Jessberger	1976	—	—	· 8% · 0.06 mm	—
	Koegler et al	1936	—	3	—	—
			—	10	—	—
	Maag	1966	—	—	· 15% · 0.063 mm	—
	Schaible	1950	—	20	—	—
	Schaible	1954	—	10	· 20% · 0.10 mm	—
	Schaible	1957	—	10	· 20% · 0.10 & · 1% · 0.002 mm	—

Atterberg limits	Other factors	Type of classification	Material type	Comments
–	–	pass/fail	all	
yes	soil classification	degree	all	
–	–	pass/fail	subgrade soils	
–	–	pass/fail	all	
–	–	pass/fail	heterogeneous soils	based on Casagrande (1931)
–	–	pass/fail	homogeneous soils	
–	–	pass/fail	all	
yes	–	pass/fail	silty and organic clayey soils	
–	sand equivalent	pass/fail	base and subbase	
yes		pass/fail	silty soils	
yes		pass/fail	all when *PI* [illegible] 10% & *LL* [illegible] 40%	
–	–	pass/fail	all	
–	–	pass/fail	base and subbase	
–	–	pass/fail	base	
–	–	pass/fail	subbase	
–	–	pass/fail	base and subbase	
–	–	pass/fail	all	
–	–	pass/fail	subgrade soils	
–	–	pass/fail	base and subbase	
–	–	pass/fail	heterogeneous soils	
–	–	pass/fail	homogeneous soils	
yes	soil type	degree	all	
–	–	pass/fail	base and subbase	
–	–	pass/fail	base and subbase	
–	–	pass/fail	all	
yes	–	pass/fail	granular	after U.S. Army Corps of Engrs (1965)
yes	–	pass/fail	base	
yes	–	pass/fail	subbase	
–	–	pass/fail	silty soils	
–	–	pass/fail	subgrade	
–	–	pass/fail	crushed stone	
–	–	pass/fail	sand and gravel	
yes	–	degree	all	
yes	–	pass/fail	heterogeneous soils	after Casagrande (1931)
yes	–	pass/fail	homogeneous soils	
–	–	pass/fail	base and subbase	
–	–	pass/fail	all	
yes	sand equivalent	pass/fail	all	
–	grain size curves	pass/fail	base and subbase	
yes	–	pass/fail	subbase	
yes	soil type	degree	all	based on lab frost heave & field thaw weakening
yes	soil classification	degree	gravels, heterogeneous soils	
yes	soil classification	degree	most inorg. materials	
yes	soil classification	degree	homogeneous sands	
–	soil classification	degree	all	based on frost heave and thaw weakening
–	–	pass/fail	fine sands and silts	
–	–	pass/fail	all	
–	–	pass/fail	silt and clays	
–	–	pass/fail	all	
–	–	pass/fail	base and subbase	
yes	–	pass/fail	base and subbase	
–	–	pass/fail	cohesionless soils	after Casagrande (1931)
–	soil classification	degree	all	bearing capacity after thaw considered
–	grain size curves	degree	gravels	after Schaible (1957)
–	organic content	degree	sands	
	thaw CBR	degree	all	thaw CBR considered, based on Jessberger (1976)
–	–	degree	all	based on frost heave
yes	–	degree	all	thaw CBR considered
–	permeability	degree	heterogeneous soils	after Casagrande (1931)
–	permeability	degree	homogeneous soils	
–	water table/capillary rise	pass/fail	all	
–	permeability	pass/fail	all	
–	permeability	degree	all	
–	–	degree	all	based on frost heave and thaw weakening

APPENDIX B. SUMMARY OF FROST SUSCEPTIBILITY TESTS ON NATURAL SOIL.

Specimen Number	Material Source	Unified Soil Classification Symbol (2)	Maximum Size in.	Soil Gradation Data (As Frozen): Percent finer, mm 4.76	0.42	0.074	0.02	0.01	0.005	Coefficients (3) C_u	C_c	Physical Properties of Basic Soil: Atterberg Limits (4) LL	PI	Specific Gravity	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 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	N	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	N	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	24.8	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	13.6	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	99	-	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	100	-	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	M-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	1.84	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	N	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	N	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	M	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	38.1	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	18.0	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	Martle 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	Martle Point		2	45	25	11	6.8	6.0	4.0	256	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	19.1	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
CBG-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	3.0	4.8	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	2.8	4.5	1.60	M-H	SC

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

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Table B1 (cont'd). Summary of frost susceptibility tests on natural soils' — open system nominal load 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 Vol. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
														CLAYEY SANDY GRAVELS															
MDD-1	Washington, D.C.	GM-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
MDD-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	[illegible]	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	12	8.7	6.9	-	315	32	22.3	8.1	2.75	-	-	131	96	0.250	98	-	8.8	16.2	19.9	1.5	2.0	1.33	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	M	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	3.3	2.0			3.20	139.1(b)	-	139	100	0.440	86	-	11.7	11.7	7.5	0.3	0.4	1.33	N	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	-
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 B1 (cont'd).

Specimen Number	Material Source	Unified Soil Classification Symbol (2)	Maximum Size in.	Soil Gradation Data (As Frozen): Percent finer, mm 4.76	0.42	0.074	0.02	0.01	0.005	Coefficients (3) C_u	C_c	Physical Properties of Basic Soil: Atterberg Limits (4) LL	PI	Specific Gravity -	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 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 Vol. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
												SILTY GRAVELLY SANDS																	
KA-5	Keflavik	SW-SM	3/4	57	16	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	M	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
MIR-1	Minnesota		2	69	20	9.6	3.8	-	-	28	1.8			2.73	135.6(d)	6.5	135	106	0.268	97	-	9.4	22.0	37.0	2.8	4.3	1.54	M-H	-
SA-2	Stewart		2	58	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	29	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	21.3	1.2	2.0	1.66	L	-
NDG-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	7.7	1.26	H	-
NDG-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	-
NDG-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			2.73	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	21	5.3	1.9	1.7	-	4.0	1.6			2.75	123.5(b)	-	121	98	0.421	100	-	15.3	17.3	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	M	-
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	M	-
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	M	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	5.00	M	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	M	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	5.9	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-2	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	-
NDG-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	56	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	6.5	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.[illegible]	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	5.5	4.5	-	81	0.3			2.70	129.1 (b)	-	125	97	0.343	87	-	11.3	23.4	27.6	1.8	3.3	1.83	L-M	-
VF-7	Volk Field		2	94	83	10	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	1.39	M	-
LSG-15	Loring		½	65	14	8.[illegible]	7.1	5.2	4.6	260	34	24	5	2.71	139.1 (b)	-	135	97	0.254	93	2.3	8.6	14.6	17.4	2.0	3.8	1.70	M	SC
LSG-39	Loring		½	88	17	10	8.2	7.2	[illegible]	[illegible]	[illegible]			2.72	139.1 (b)	-	135	97	0.259	93	-	8.5	31.6	59.8	2.0	4.2	2.10	MH	EC

Table B1 (cont'd). Summary of frost susceptibility tests on natural soils[1]—open system nominal load 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 (%)	G, at Start of Test (6) (%)	Permeability k (7) cm/sec x10^{-4}	FREEZING TEST DATA — Avg. Water Content Before Test (%)	Avg. Water Content After Test (%)	Total Heave (8) (%)	Rate of Heave mm/day (9) — Avg	Max	Heave Rate Ver. 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
W -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	M-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	45	0.6			2.71	136.0(c)	7.0	129	95	0.312	88	-	10.1	28.5	36.3	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	M	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	-	13.2	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	1.96	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		½	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	[illegible]
TD-9	Truax		3/4	92	73	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	2.8	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	M	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		¼	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	[illegible]	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	68.5	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	130	99	0.378	100	0.66	14.2	19.6	17.1	1.5	2.7	1.80	L-M	SC
PT-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 B1 (cont'd).

Specimen Number	Material Source	Unified Soil Classification Symbol* (2)	Maximum Size (in.)	SOIL GRADATION DATA (As Frozen): Percent finer, mm: 4.76	0.42	0.074	0.02	0.01	0.005	Coefficients (3): C_u	C_c	PHYSICAL PROPERTIES OF BASIC SOIL: Atterberg Limits (4): LL	PI	Specific Gravity	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 $\times 10^{-4}$)	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)
														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	2[illegible].0	11.0	2.75	138.7(c)	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(c)	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(c)	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															
CB-3	Goose Bay	ML	-	100	99	54	6.0	-	-	-	-			2.74	102.0(c)	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	-	-	-	26.0	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	11.3	1.33	VH	SC
HAN-1	Hanover		-	100	100	98	35	18	8	-	-	29.5	12.7	2.75	[illegible]	16.9	101	96	0.695	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(c)	-	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(c)	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(c)	16.5	104	98	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(c)	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(c)	16.5	105	98	0.611	100	0.15	22.6	116.7	239.2	12.8	13.7	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(c)	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(c)	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(c)	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(c)	14.7	107	97	0.577	99	-	21.2	42.4	50.8	3.7	4.5	1.22	M-H	SC
NH-49A	New Hampshire		-	100	100	99	73	37	13	-	-	23.7	6.0	2.70	110.1(c)	14.7	106	96	0.596	100	-	22.0	36.4	29.8	4.0	5.3	1.32	H	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	1.29	M	SC
LFT-13	Fairbanks		-	100	100	97	42	22	12	-	-	32.6	6.2	2.67	107.4(c)	17.1	101	95	0.646	100	0.20	24.2	112.6	223.6	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(c)	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															
DFT-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(a)	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	1.52	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
FB-1A	Fort Belvoir		1/4	98	90	61	49	41	34	-	-	43.8	20.3	2.73	114.9(a)	15.6	110	96	0.536	100	0.148	19.8	81.5	188.4	7.7	12.0	1.56	VH	SC
FB-7A	Fort Belvoir		1/4	98	90	61	49	41	34	-	-	43.8	20.3	2.73	114.9(a)	15.5	117	102	0.456	90	0.098	15.0	22.3	18.2	1.5	3.7	2.45	L-M	SC
FB-9A	Fort Belvoir		1/4	98	90	61	49	41	34	-	-	43.8	20.3	2.73	114.9(a)	15.6	113	98	0.504	100	0.130	18.5	27.4	22.1	1.3	3.0	2.30	L-M	SC
FB-14A	Fort Belvoir		1/4	98	98	61	49	41	34	-	-	43.8	20.3	2.73	114.9(a)	15.5	118	103	0.441	100	0.089	16.2	27.0	27.6	2.2	3.2	1.45	M	SC
P*FB-5	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
EBT-28	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	37.4	72.5	7.8	12.7	1.62	H-VH	SC
EBT-29	East Boston		3/4	86	73	57	49	42	30	-	-	21.0	7.0	2.76	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	H	SC
DFC-12	Dow Field		3/4	96	93	86	51	38	27	-	-	26.4	8.4	2.69	112.9 (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	89	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.[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	26.4	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	2.09	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	-
BOBG-4	Bong		3/4	97	90	80	60	48	36	-	-	28.6	12.6	2.80	128.8 (c)		125	98	0.375	96	0.00002	13.5	16.1	16.1	1.5	1.7	1.13	L	
BOBG-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	
BOBG-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		1¼	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		1¼	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	77.7	3.3	3.8	1.15	M	SC

* 4-inch diameter

Table B2. Summary of frost susceptibility tests on natural soils[1] — open system nominal load pressure 0.5 psi.

Specimen Number	Material Source	Unified Soil Classification Symbol (2)	Maximum Size (in.)	Soil Gradation Data (As Frozen): Percent finer, mm: 4.76	0.42	0.074	0.02	0.01	0.005	Coefficients (3): C_u	C_c	Physical Properties of Basic Soil: Atterberg Limits (4): LL	PI	Specific Gravity	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 Vs. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
												GRAVELS AND SANDY GRAVELS																	
BFR-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
BFR-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	5.6	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	M-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
PLJ-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																	
MDO-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
MDO-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	24.8	1.4	1.8	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	M	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	38.7	2.6	5.5	2.12	M-H	SC
JFBB-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
WAC-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	56.4	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	H	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
PLJ-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	15.7	1.6	2.7	1.68	L-M	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	0.325	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	16.5	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	90	0.096	11.7	17.0	13.0	2.0	3.0	2.50	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	12.5	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	2.6	15.9	24.0	20.2	1.4	2.0	1.42	L	SC

Table B2 (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 $\times 10^{-4}$	FREEZING TEST DATA: Avg. Water Content: Before Test %	After Test %	Total Heave (8) %	Rate of Heave mm/day (9): Avg	Max	Heave Ratio Vsr. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
														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	209.1	7.7	10.7	1.38	H-VH	SC
EBR-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	245.0	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
EBR-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	201.8	8.2	12.2	1.48	VH	SC
EBR-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	95.3	4.8	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	214.9	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	91	0.553	100	0.0215	20.3	90.2	156.8	7.2	11.3	1.56	H-VH	T
														LEAN CLAYS															
FBW-2	Greenland	CL	-	100	100	97	60	43	34	-	-	36.5	16.8	2.78	119.4(e)	15.0	98	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	-	-	102	-	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	132.3	4.7	6.7	1.43	H	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	H	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	H	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	241.8	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	-	-	33.8	16.4	2.79	117.0(d)	-	105	90	0.660	92	-	21.8	94.3	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
LC-11*	Boston Blue Clay		-	100	100	99	90	81	72	-	-	47.3	27.4	2.72	106.2(e)	20.2	80	-	1.156	98	-	41.2	124.2	130.8	9.5	15.7	1.65	VH	SC
EC-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	H	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 B2 (cont'd). Summary of frost susceptibility tests on natural soils' — open system nominal load 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 (%)	G, 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	Heave Rate Vs. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
												CLAYEY SILTY SANDS																	
WAB-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																	
FA-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	0.34	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	6.2	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	VH	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	2.02	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	0.899	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(c)	18.1	94	93	0.811	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	M	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 B3. Summary of frost susceptibility tests on natural soils'' — open system nominal load pressure 0.073 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 (%)	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 Rate Var. Index (10)	Frost Susc. Class (11)	Type of Cyc (12)
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	19.3	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
ACP-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	Valparaiso, Indiana Silt		-	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	66.1	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		1½	95	7	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		1½	95	87	74	58	48	38	-	-	27.3	11.9	2.74	121.0 (a)	13.5	114	94	0.457	100	.0084	[illegible]	29.0	31.4	3.8	4.3	1.03	M-H	T
ART-19	AASHO Road Test		1½	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	43.8	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	2.8	1.75	L-M	SL
YS-10	Yukon Silt		-	100	100	100	67	37	29	-	-	[illegible]	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	H	SL

Table B1 (cont'd). Summary of frost susceptibility tests on natural soils' — open system nominal load 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.	Rate Var. Index (10)	Frost Susc. Class (11)	Type of Cyl. (12)
														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	RL
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	1.33	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	0.426	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

APPENDIX B. NOTES FOR TABLES B1, B2 AND B3

1. The data reported in this Appendix 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%.
   b. Molded dry unit weight equal to or greater than 95% of the applicable maximum standard.
   c. Rate of penetration of the 32°F isotherm approximately ¼ to ½ in./day.
   d. Load pressure:
      Table B1–0.5 psi
      Table B2–0.5 psi
      Table B3–0.073 psi (¼-in. steel plate only)
   e. Height of molded specimen approximately 6 in.
   f. Free water supply at base of specimen (water maintained at approximately 38°F).

   The specimens are listed in order of increasing percentage finer than 0.02-mm grain size within each soil classification group.

2. U.S. Army Engineer Waterways Experiment Station, *The Unified Soil Classification System.* Technical Memorandum No. 3-357, vol. 1, Vicksburg, Mississippi, revised 1960.

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

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

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

4. Atterberg limits tests performed on material passing the U.S. Standard no. 40 sieve. If no limits are shown, material is nonplastic. 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. AASHO T99-57[1] Method A.
   b. Providence Vibrated Density Test.
   c. AASHO T180-57 Method D.
   d. AASHO T180-57 Method A.
   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 ¼-in. and ½-in./day. Rate of heave is averaged over as much of the heave versus the 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 ¼-in. and ½-in./day:

Rate of heave mm/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, actylic

13. The specimens listed in supplementary Table B2 do not fulfill requirements given under Note 1a and 1b above; otherwise all other notes apply.

14. The specimens listed in Table B3 have been tested under a load pressure of 0.073 psi, and may or may not fulfill 1a and 1b; otherwise all other notes apply.

APPENDIX C. SUMMARY OF DIRECT FROST HEAVE TESTS

Principal use of test	*Austria* *Brandl (1970)* *Technical University,* *Graz* Special test	*Austria* *Brandl (1980)* *Technical University,* *Graz* Special test	*Belgium* *Gorlé (1980)* *Belgian Road* *Res. Ctr.* Research	*Canada* *Penner & Ueda (1977)* *Natl. Res. Council* *(NRC)* Research	*England* *Croney & Jacobs (1967)* *Trans. & Road* *Res. Lab.* Classification
Description of apparatus					
Side friction control	10-cm, multi-ring, plexiglass	multi-ring?	0.5-cm multi-ring	bottom-up freez., Teflon	waxed paper
Surcharge (kPa)	~ 2.2	5	3.4	variable	very small
Open or closed system	open	open	open	open	open
Sample diameter (cm)	30	12.5	15.24	10.2	10.2 (or 15.2)
Sample height (cm)	50	15	12.7	10.2	15.2
Radial heat flow control	15 cm foam insulation	foam insulation?	insulation	polyureth., 0°C amb.	dry sand
Samples per test	?	multiple, quant. unkn.	multiple, quant. unkn.	1	9
Observations					
Temperature	no	yes	thermocouples	thermocouples	thermocouples?
Frost heave	yes	yes	displace. transducer	displace. transducer	rule
Water flow	no	no	graduated cylinder	automated?	no
Heat flow	no	no	no	no	no
Sample preparation method					
Material types	gravels	all	all	fine-grained	all <5mm (or 37.5 mm)
Compaction method	Proctor, opt. water cont.	modified Proctor	unknown	consolidated slurries	vibr. hammer
Undisturbed samples	no	no	no	no	no
Saturation method	soaking or perc.	soaking	vacuum, soaked 48 hr	vacuum saturation	soaked 24 hr at base
Freezing conditions					
Freezing mode	T_c = constant	T_c = constant	T_c = constant	T_c = constant	T_c = constant
Direction of freezing	top down	top down	top down	bottom up	top down
Cooling method (top)	circulating air	circulating air	circulating air	circulating air	circulating air
Cooling method (bottom)	heated water	heated water	heated water	circ. methanol-water	heated water
Cold side temperature (°C)	-24 freeze, +20 thaw	-15 (+20)	variable	variable	-17
Warm side temperature (°C)	+4	+4	variable	variable	+4
Rate of frost penetration (cm/day)	variable	variable	variable	variable	variable
Number of freeze-thaw cycles	11	2-4	1	1	1
Duration of freezing	10 1-day, 1 7-day	0-2 1-day, 2 7-day	1 day	3-4 days	250 hr
Duration of thaw	10 1-day, 1 1-day	2 1-day	none	none	none
Total freeze-thaw duration	28 days	16-21 days	1 day	3-4 days	250 hr
Comments			temp. grad. variable	suggests using Q = const.	
Method of analysis					
Critical frost suscept. factor	heave & CBR loss	heave & CBR loss	h, I_r, & $\overline{V}$	heave rate	heave
Frost susceptibility criteria[2]	none	*Allowable frost heave* 1-2 cm (main highways) 2.5 cm (sec. roads) *Min. thaw CBR* 20% (main highways) 25% (sec. roads)	none	none	Heave at 250 hr (cm) / Class <1.3 NFS 1.3-1.8 BFS >1.8 VFS
Field validation	yes?	some?	no	no	field experience

Principal use of test	England Jones & Dudek (1979) University of Nottingham Research	France Aguirre-Puente et al. (1970) Laboratoires des Ponts et Chaussees (LPC) Classification	Norway Loch (1979a, b) Norwegian Road Res. Lab. Classification	Romania Vlad (1980) Polytechnic Inst. of Jassy Classification	Sweden Fredén & Stenberg (1980) National Road and Traffic Research Institute Classification
Description of apparatus					
Side friction control	waxed paper	lubricated rubber tube	2-cm multi-rings, plexigl.	tapered plexiglass	bottom-up freez., plexiglass
Surcharge (kPa)	very small	very small	very small	approximate *in situ*	variable, 2-18
Open or closed system	open	open	open	approximate *in situ*	open
Sample diameter (cm)	10.2	7.5	9.5	10	11.3
Sample height (cm)	10.2	25	10	20	20
Radial heat flow control	dry sand, foam, & guard ring	vacuum, 0°C ambient	styrofoam beads, 0.5°C	foam insulation	foam insulation at +20°C
Samples per test	1	1	?	4 frozen, 2 soaked	1
Observations					
Temperature	thermocouples	thermocouples	thermocouples	thermocouples	thermocouples
Frost heave	dial gauge	potentiometer	dial gauge	dial gauges	transducer
Water flow	graduated cylinder	no	graduated cylinder	no	no
Heat flow	calculated	no	heat flow meter?	no	heat flow meter
Sample preparation method					
Material types	all	fine gravels, finer soils	fine gravels, finer soils	sands, finer soils	fine gravels, finer soils
Compaction method	vibr. hammer, static load	standard Proctor	variable or undisturbed	Proctor?, opt. water cont.	tamped dry while soaking
Undisturbed samples	no	no	yes	no	no
Saturation method	soaked 24 hr at base	soaked 18 hr	soaking at base	vacuum	soaked 1-10 days at base
Freezing conditions					
Freezing mode	T_c = constant	T_c = constant	T_c = constant	dz/dt = constant	Q_c = 490 W/m^2
Direction of freezing	top down	top down	top down	top down	bottom up
Cooling method (top)	water-cooled Peltier	circulating glycol-water	circulating alcohol-water	circulating air	no control, top insulated
Cooling method (bottom)	circulating water	heated water	circulating alcohol-water	circulating air	*water-cooled Peltier*
Cold side temperature (°C)	-6.0	-5.7	variable, -7	variable to -25	variable
Warm side temperature (°C)	+1.0	+1	[illegible]	+4	variable
Rate of frost penetration (cm/day)	variable	variable	variable, avg = 6.4	1	variable
Number of freeze-thaw cycles	1	1	[illegible]	1	several
Duration of freezing	4-10 days	150-200 hours	[illegible]	15 days	?
Duration of thaw	none	none	[illegible]	?	?
Total freeze-thaw duration	4-10 days	150-200 hours	< 4[illegible]	> 15 days	?
Comments			suggests using Q = const.		
Method of analysis					
Critical frost suscept. factor	heave at 4 or 10 days	ratio of heave to sq root of freezing index (p)	heave rate	avg. heave rate, CBR change	rate of heave
Frost susceptibility criteria[2]	none	p [mm/(°C·h)$^{1/2}$] — Class[3] <0.05 — NFS 0.05-0.40 — FS >0.40 — VFS	none	CRREL criteria for avg rate of heave, criteria for CBR change not given	none reported
Field validation	no	uncertain	no	yes?	no?

Principal use of test	*Switzerland* *Balduzzi & Fetz (1971)* *Swiss Federal Inst. of Technology* Classification	*U.S.A.* *Esch et al. (1981)* *Alaska Dept. of Trans. & Public Facilities* Special test	*U.S.A.* *Kaplar (1974)* *U.S. Army Cold Regions Res. & Eng. Lab. (CRREL)* Classification	*U.S.A.* *Kalcheff & Nichols (1974)* *Natl. Crushed Stone Assoc.* Classification	*U.S.A.* *Sherif et al. (1977)* *Univ. of Washington* Research
Description of apparatus					
Side friction control	cellulose foil	multi-ring	tapered plexiglass	polyethylene film	tapered plexiglass
Surcharge (kPa)	not specified	3.5	3.5	1.4	none
Open or closed system	open	open	open	open	open
Sample diameter (cm)	5.6	15.2	14.3 (avg)	15.2	12
Sample height (cm)	10	14.0	15.2	20.0	30
Radial heat flow control	foam insulation	?	granular cork	granular cork	none
Samples per test	7	4	4	18	4
Observations					
Temperature	thermocouples	no	thermocouples	thermocouples	thermocouples
Frost heave	metal rules	dial gauge	dial gauge or potentiom.	rods	ruler
Water flow	no	graduated cylinder	graduated cylinder	no	no
Heat flow	no	no	no	no	no
Sample preparation method					
Material types	10 mm max. particle size	<1.91-cm material	<5-cm material	granular base materials	all
Compaction method	AASHO Std.	vibratory hammer	AASHO T180-57 or vibr.	compaction, vibration	std. Proct, ASTM D698-70
Undisturbed samples	no	no	yes	no	no
Saturation method	soaking at base	overnight soaking	vacuum	soaked at base 2-3 days	soaked 24 hr at +4°C
Freezing conditions					
Freezing mode	T_c = constant	T_c = constant	dz/dt = constant	T_c = constant	T_c = constant
Direction of freezing	top down	top down	top down	top down	top down
Cooling method (top)	circulating air	circulating air	circulating air	circulating air	circulating air
Cooling method (bottom)	heated water	heated water	air-cooled water	heated water	heated water
Cold side temperature (°C)	-17	-9.5	variable	-12	-2, -5 and -10
Warm side temperature (°C)	+4	+4.5	+3.5	?	+4
Rate of frost penetration (cm/day)	variable	1.3 bet. 48 & 72 hr	0.6-1.3	variable	variable
Number of freeze-thaw cycles	1	1	1	1	3
Duration of freezing	50-70 hr	3 days	12-24 days	200 hr	3 days
Duration of thaw	none	none	none	none	3 days
Total freeze-thaw duration	50-70 hr	3 days	12-24 days	200 hr	18 days
Comments					
Method of analysis					
Critical frost suscept. factor	heave ratio	avg. heave rate (h_r)	avg. heave rate (h_r)	avg. heave rate	heave at 3 days
Frost susceptibility criteria[2]	none published	same as CRREL	h_r (mm/day) — Class 0-0.5 — NFS 0.5-1 — VLFS 1-2 — LFS 2-4 — MFS 4-8 — HFS >8 — VHFS	none	none
Field validation	no	no?	yes?	no	no

Principal use of test	U.S.A. *Zoller (1973)* *Univ. of New Hampshire* Classification	U.S.S.R. *Alekseeva (1957)* —	U.S.S.R. *Vasilyev (1973)* *Leningrad Branch of All-Union Highway Research Institute* Classification	West Germany *Ducker (1939)* Classification	West Germany *Jessberger & Heitzer (1973)* *Ruhr Univ., Bochum* Classification
Description of apparatus					
Side friction control	2.5 & 1.3 cm multi-ring, Lucite	2.5-cm multi-ring	1-cm multi-ring, metal	1-cm multi-rings, glass	tapered PVC, Teflon foil
Surcharge (kPa)	3.5	very small	variable; 0, 6, and 9	very small	5.9 kPa
Open or closed system	open	open	open	open	open
Sample diameter (cm)	13.7	6	10	3.85	14.75 (avg)
Sample height (cm)	15.2	10	8 and 10	4.0	12.5
Radial heat flow control	rigid foam	none	sawdust	none	insulation?
Samples per test	1	1	?	1	4
Observations					
Temperature	thermocouples	none	yes?	thermometers	thermocouples
Frost heave	dial gauge or transducer	dial gauge	yes?	dial gauge	dial gauges
Water flow	no	no	no	no	graduated cylinder
Heat flow	no	no	no	no	no
Sample preparation method					
Material types	all	fine-grained	sands, silts, clays	sands, silts, clays	all
Compaction method	variable compaction	?	hammer	static load, air dry	Proctor, w_{op}
Undisturbed samples	no	?	no	none	no
Saturation method	submerged 16 hr	soaked at base	?	capillarity	soaked 24 hr at base, submerged 72 hr
Freezing conditions					
Freezing mode	T_c = constant	T_c = constant	T_c = constant	T_c = constant	T_c = constant
Direction of freezing	top down	top down	top down	top down	top down
Cooling method (top)	water-cooled Peltier	circulating air	circulating air	circulating air	circulating air
Cooling method (bottom)	no control, only insulation	heated water	heated water	heated water	circulating air
Cold side temperature (°C)	-4°C	-5 to -7	-5	-15 or -10	-18 (+18)
Warm side temperature (°C)	+25 initially	+0.1 to +1	?	0.0	+2 to +6
Rate of frost penetration (cm/day)	variable, 8-18 cm/day	variable	1.5 to 2.5	variable	variable
Number of freeze-thaw cycles	1	1	3	1	7
Duration of freezing	12 hr	50 hr	?	4 hours	24 hours
Duration of thaw	none	none	?	none	
Total freeze-thaw duration	*12 hr*	*50 hr*	?	4 hours	7 days
Comments					CBR after thaw test
Method of analysis					
Critical frost suscept. factor	avg rate of heave (h_r)	heave	heave ratio (F)	heave ratio (F)	thaw CBR (CBR_F)
Frost susceptibility criteria[2]	h_r (mm/day) Class 0-6.5 NFS 6.5-8.0 VLFS 8.0-10.3 LFS 10.3-13.0 MFS 13.0-15.0 HFS > 15.0 VHFS	none given	F (%) Class <2 NFS 2-3 BFS >3 FS	F (%) Class <3 NFS ≥3 FS	CBR_F (%) Class >20 NFS 4-20 L to MFS <4 VFS
Field validation	no	no	yes?	no	some?

Notes:

1. h = heave rate, I_r = ice segregation ratio, $\bar{V}$ = rate of water in flow.
2. NFS = non-frost-susceptible, BFS = borderline frost susceptibility, VLFS = very low frost susceptibility, LFS = low frost susceptibility, HFS = high frost susceptibility, VHFS = very high frost susceptibility.
3. According to Carriard (1978).

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