CHAPTER 2
BASIC CONSIDERATIONS AFFECTIVE FOUNDATION DESIGN

2-1. Thermal effects.

a. As indicated in chapter 1, ground temperatures and presence or absence of permafrost are the product of many interacting variables. In addition to reflecting the effects of such factors as snow cover, air temperatures, and net radiation flux, the natural ground temperatures of a specific area can be related to the recent history of the terrain. For example, in flood plains of meandering streams permafrost may be absent in areas which were under water in recent decades and present elsewhere. Again, destruction of vegetation by forest fires in past years may have produced special subsurface temperature conditions.

b. The fundamental properties of soil or rock which determine the depths to which freezing and thawing temperatures will penetrate below the ground surface under given temperature differentials over a given time are the thermal conductivity, the volumetric specific heat capacity, and the volumetric latent heat of fusion[30]. These factors, defined in TM 5-852-6/AFM 88-19, Chapter 6 [14], vary in turn with type of material, density, and moisture content. Figure 2-1 shown typical values of thermal conductivity vs dry unit weight and porosity. More detailed plots are presented in TM 5-852-6/AFM 88-19, Chapter 6[14] The specific heat of most dry soils near the freezing point may be assumed to be 0.17 Btu/lb°F. Specific heats of other construction materials are given in TM 5-852-6/AFM 88-19, Chapter 6 [14].

c. Permafrost will thaw when sufficient heat is transferred to it from the underside of a structure. The rate of thaw depends on the temperature of the building, the insulation in the floor, air space or ventilation provisions, layers of special material (such as gravel) on the ground, and the natural characteristics (such as moisture content and temperature) in the annual frost zone and permafrost. Insulation retards and reduces but cannot prevent heat flow from the structure. Thaw will progress in permafrost below a heated structure unless provisions are made for removing the escaping heat such as by an air space beneath the building, natural or forced circulation in ducts, or even artificial refrigeration. During the winter, such a system must provide sufficient cooling and refreezing of any foundation material warmed or thawed during the summer so that progressive thermal changes and degradation will not occur.

d. Ground temperatures influence bearing and adfreeze strengths and creep rates of the permafrost. Normally, in areas of discontinuous permafrost, where the most difficult foundation engineering problems are encountered, the mean annual temperature of the permafrost is not far below freezing. Here tangential adfreeze and other strength values are low, and creep rates are high. In far northern areas of continuous permafrost and lower ground temperatures, strength values and creep factors are more favorable.

e. Ground temperatures and relative thermal stability at sites in permafrost areas are important not only in determining the amount and rate of progression of permafrost degradation which may be initiated by construction, but also the rate of creep closure of underground openings and the extent and dimensions of protective measures such as ventilated foundations or refrigeration, which are required for heat producing facilities. Ground temperatures influence installation procedures and the rate of freezeback of slurried piles. The dynamic response characteristics of foundations are also a function of ground temperatures.

f. Thaw of permafrost from below will result if a long-duration increase occurs in the surface ground temperature. This may be visualized by assuming that the entire temperature gradient curve in figure 1-4 is moved to the right. However, as Terzaghi[197] has shown, the natural heat flow out of the earth can at most produce only quite slow upward thaw of permafrost, that is, somewhat under 2 cm/yr for permafrost containing 30 percent ice by volume. If a normally developed and stable geothermal gradient exists in the permafrost and the foundation is designed to maintain original permafrost temperatures, there will be no thaw from below. If the permafrost is in process of warming from prior colder climatic conditions or is even in an isothermal condition at the thawing temperature, as may sometimes happen, thaw will occur from below at rates up to the maximum possible under the geothermal gradient existing in the sub-permafrost materials. In any case, thaw of permafrost from below is not normally a significant factor in design of foundations for structures, except that a flow of warm sub-permafrost groundwater might rarely present a special situation.

g. Designers must also keep in mind that both manufactured and natural construction materials experience significant linear and volumetric changes with changes in temperature. Linear coefficients of thermal expansion for some common materials are shown in table 2-1. Note particularly that values for asphalt and ice are mush higher than for soil or rock. The higher the percentages of asphalt or ice the greater the degree of

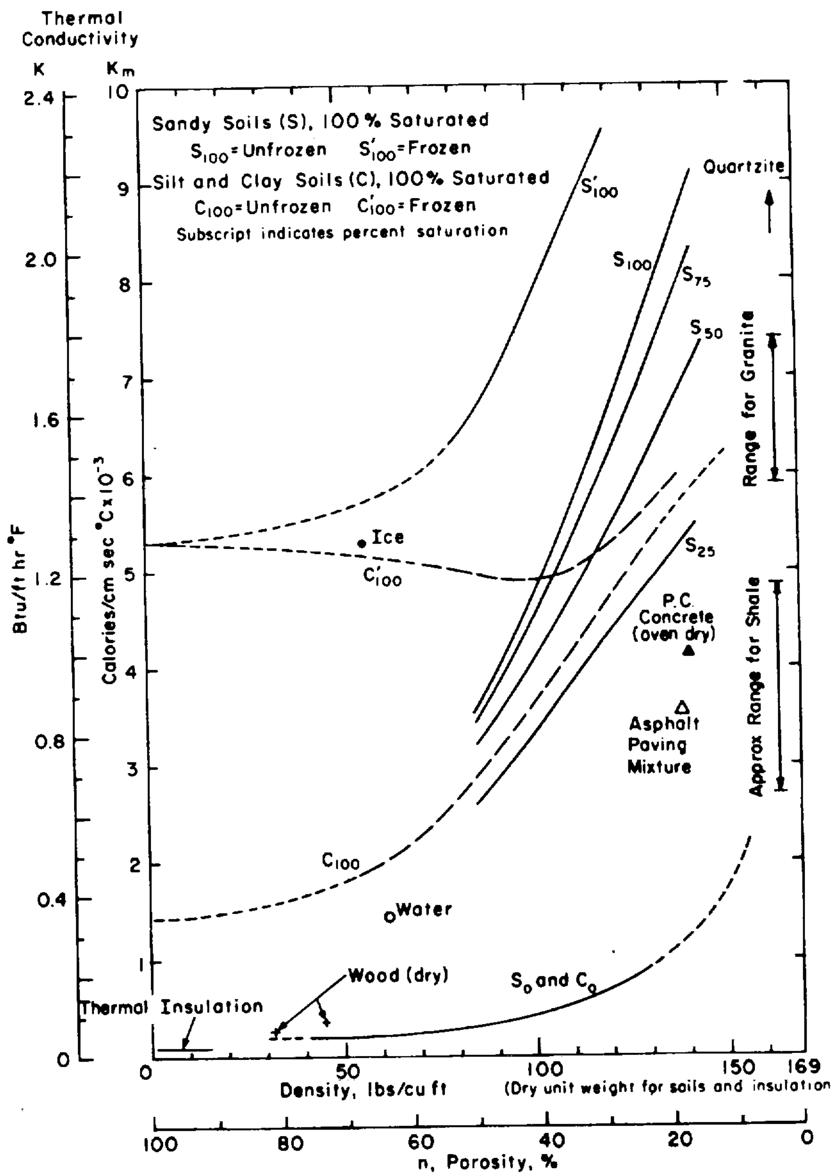

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Figure 2-1. Thermal conductivity vs density and porosity of typical materials[58].

Table 2-1. Approximate Coefficients of Linear Thermal Expansion per°C.

	$\times 10^{-6}$
Granite and slate	8
Portland cement concrete	10
Soil (109 lb/ft^3, 23 % water content, +20 to -160°c)	22
Ice	51
Steel	12
Copper	14-17
Aluminum	18-23
Sulfur	64
Coal tar pitch	160
Asphalt	215
Roofing felt	11-33
Built-up roofing membranes	15-53
Bakelite	22-33
Some other plastics	35-90
Wood (pine), parallel to fiber	5.4
Wood (pine), perpendicular to fiber	34

Note: The coefficient of cubical expansion may be taken as three times the linear coefficient.

shrinkage with lowering temperature.

h. The ground surface experiences substantial contraction as it is cooled in the fall and winter months, resulting in cracking of the surface[207]. In arctic and subarctic areas patterned ground is formed [160], [206], with ice wedges at the boundaries of the resulting polygons, as illustrated in figure 2-2. (For additional information on surficial features such as patterned ground, see TM 5-852-8[17].) In far northern areas the maximum surface cracking effects tend to develop in the spring, even as late as May or June, as the effects of the winter low temperatures reach substantial depths below the surface. Shrinkage cracking of flexible pavements is observed in all cold regions and ground cracking has been observed in seasonal frost areas as well as the permafrost regions[207]. During the summer and fall, expansion of the warming ground may exert substantial horizontal thrust if cracks have become filled with soil or ice. Any construction features embedded in the layers of ground subject to these seasonal thermal contraction or expansion effects, or supported on them, may in consequence have stresses imposed upon them. Where items such as power cables or pipes cross contraction cracks, stresses may be sufficient to rupture or damage these members. Structures supported above the surface may also experience such effects if the strains are differential and if these can be transmitted through the supporting members. Structures of sufficient strength may also serve to alter and control contraction cracking of the ground.

i. Thus, foundation materials and structures in arctic and subarctic regions must be viewed as subject to continual changes in conditions and in their states of stress and strain. It must be the designer's objective to keep such movements and stresses within acceptable limits and without progressive changes detrimental to the facility.

2-2. Seasonal frost heave and settlement.

a. Frost heave may be anticipated whenever freezing temperatures advance into frost-susceptible soil and adequate moisture is available, provided it is not restrained by a countermeasure. Seasonal heave and settlement of frost-susceptible soils occur in both permafrost and seasonal frost regions in the surface strata subject to cyclic freezing and thawing. Heave or settlement may also occur on a nonseasonal basis if progressive freezing or thawing is caused in the foundation.

b. During the freezing process the normal moisture of the soil, and that drawn up from greater depths, is converted into ice as crystals, lenses or other forms. In

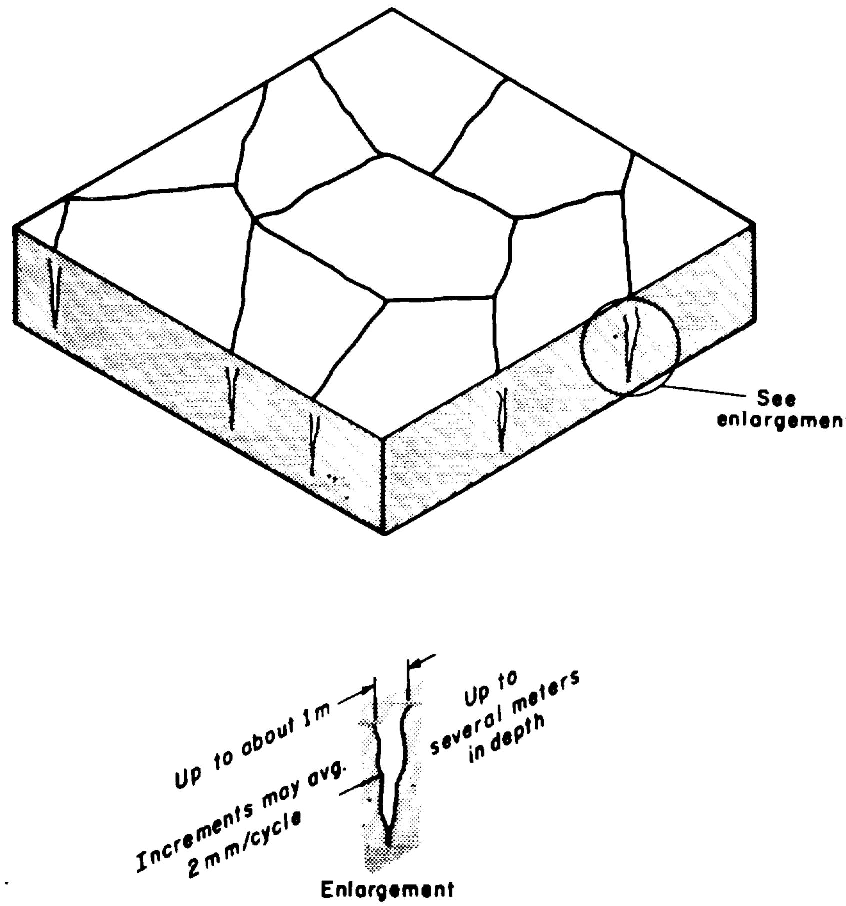

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Figure 2-2. Ice wedges in polygonal ground area.

frost-susceptible soils the formation of ice lenses at (and in the finer-grained soils behind) the freezing plane during the freezing process tends to produce an upward movement or heaving of the soil mass. Ice segregation is most commonly in the form of lenses and layers oriented principally at right angles to the direction of heat flow; the surface heave is approximately equal to the total thickness of the ice layers. The raising or heaving of the ground surface in a freezing season may vary from nothing in confined, well-drained non-frost-susceptible sands and gravels to a foot or more in saturated, frost-susceptible silts and some clays if there is an unlimited supply of moisture. The magnitude of seasonal heaving is dependent upon such factors as rate and duration of frost penetration, soil type and effective pore size, surcharge, and availability of moisture[40,59,60]. Figure 2-3 illustrates, in a laboratory freezing test, the typical gain in moisture content in upper layers of soil, with withdrawal of moisture from lower strata even though water was provided at the base. In frost-susceptible soils, ice segregation can occur by such withdrawal of moisture from lower layers even without an outside source of moisture.

c. Clean GW, GP, SW, and SP gravels and sands with a negligible percentage of material smaller than 0.02 mm are so relatively nonheaving that foundation design on such materials is usually not governed by seasonal frost effects. (Frost susceptibility criteria are discussed in more detail in TM 5-818-26). If saturated, it is possible for such soil to heave a small amount on freezing because of the expansion of water on changing to ice. However, if the expansion of the water which freezes can be balanced by movement of an equal volume of unfrozen water away from the freezing plane, there will be negligible expansion of nominally confined non-frost-susceptible materials. The same expansion relief can be obtained in non-frost-susceptible soils by a condition of partial saturation. Superficial fluffing of the surface of unconfined, relatively clean sands and gravels is frequently observed, but this effect is small to negligible when these materials are confined. In permafrost areas, however, the fact that the foundation materials are non-frost-susceptible does not justify assuming, without investigation, that ground ice masses are not present or that settlement on thawing will not occur.

d. When fine-grained soils thaw, water tends to be released by melting of segregated ice more rapidly than it can be drained away or redistributed in the thawed soil. This results in a very wet, soft condition of the soil, with substantial loss in shear strength. The shear strength of the thawed soil is dependent upon the same factors as would apply under non-frost-related conditions but is very difficult to measure meaningfully by conventional approaches.

2-3. Groundwater.

a. If a freezing soil has no access to free water beyond that contained in the voids of the soil immediately below the plane of freezing, frost hicave will necessarily be limited. However, if free water can be easily drawn to the plane of freezing from an appreciable distance below the plane of freezing or from an underlying aquifer, heave can be large. A water table within 5 feet of the plane of freezing is favorable for significant frost heave[6]. However, lowering of a water table to even great depth cannot be depended upon to eliminate frost heave; the percentage of water that can be drained by gravity from most frost-susceptible soils is limited and may be negligible"'. The remaining water in the voids will still be available to migrate to the plane of freezing. In permafrost areas the supply of water available to feed growing ice lenses tends to be limited because of the presence of the underlying impermeable permafrost layer, usually at relatively shallow depths, and maximum heave may thus be (but is not necessarily) less than under otherwise similar conditions in seasonal frost areas. Uplift forces on structures may nevertheless be more difficult to counteract in these more northerly cold regions because of lower soil temperatures and consequently higher effective tangential adfreeze strength values.

b. As illustrated in figure 2-4, the water table may disappear rapidly in the first part of the freezing period as water is withdrawn from the unfrozen layers of soil to form ice lenses t the plane of freezing. However, even when the free water table disappears a substantial volume of water remains still available, and ice segregation and frost heave may continue for many weeks thereafter, while availability of moisture, surcharge weight at the freezing plane and rate of frost penetration are progressively changing. Full saturation is not necessary for ice segregation in fine-grained soils, though below about 70 percent saturation some soils do not heave significantly. Perched water tables can be as important as base groundwater tables.

c. Susceptibility of the soil to particle break-down under freeze-thaw cycles is a function not only of the durability characteristics of the particles themselves (which can be evaluated by standard laboratory tests), but also of the degree of saturation. For building stone and mineral aggregate it has been found that 87 percent saturation of the material itself is a good maximum if the stone or aggregate (or concrete made therefrom) is to resist freeze-thaw damage (communication from K.B. Woods). Thus, the degree of natural drainage existing in foundation soils during freeze-thaw cycles may be an important design consideration if particle degradation may significantly affect the long range performance of the construction.

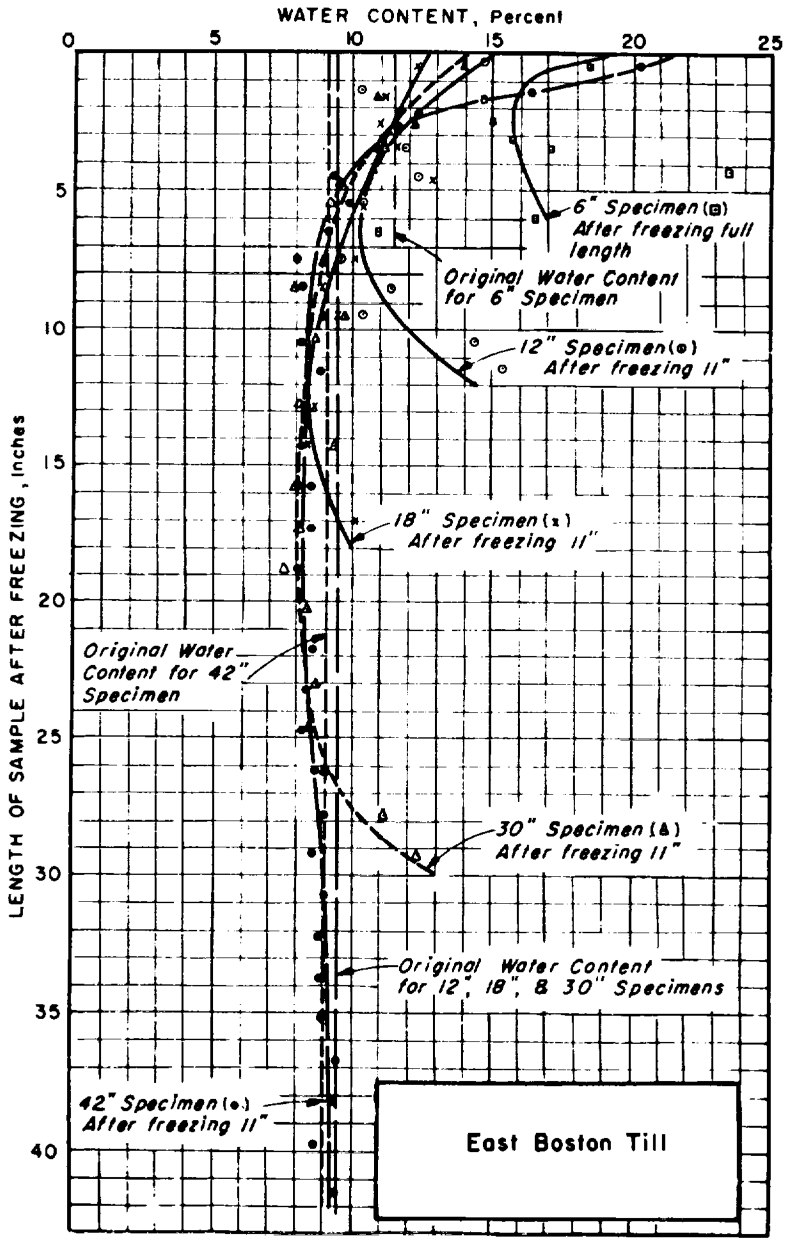

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Figure 2-3. Moisture content changes caused by freezing[40]. Water available at base of specimen during test. See figure 2-11 for soil characteristics.

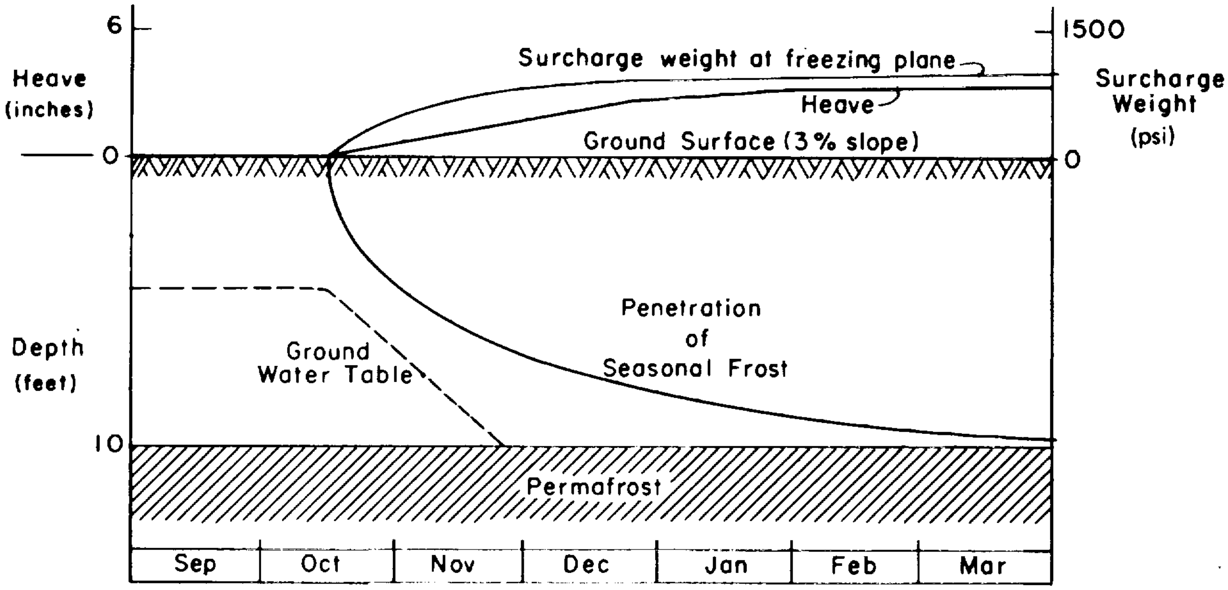

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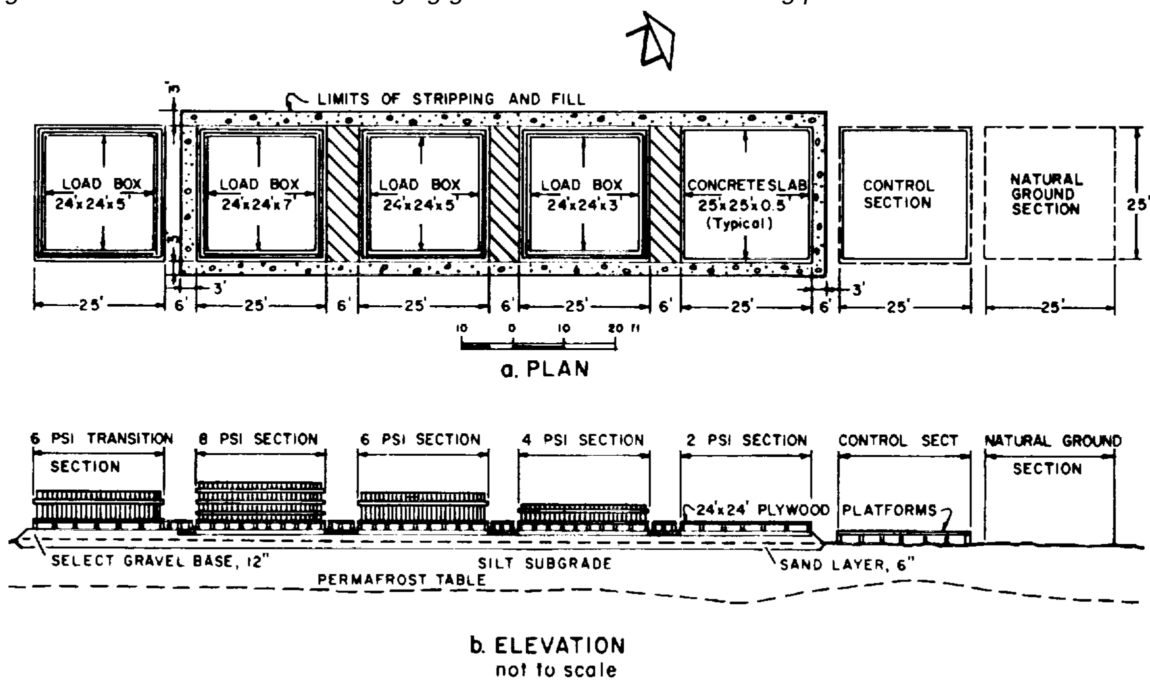
Figure 2-4. Generalization of changing ground conditions as freezing penetrates into the annual frost zone[35].



Notes

Concrete transition sections, shown as crosshatched on plan, were separated from main slabs by one inch expansion joints, and were covered by 6x24-ft plywood platforms.

All slabs were reinforced with 616 welded wire mesh.

Transition section, 6 psi, constructed 2 years after original sections

The groundwater table was at or only slightly below ground surface at the beginning of each freezing season.

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Figure 2-5. Plan and elevation of surcharge field experiment, Fairbanks, Alaska[28].

d. The water content of a soil exerts a substantial effect upon the depth of freeze or thaw penetration which will occur with a given surface freezing or thawing index. An increase in moisture content tends to reduce penetration by increasing the volumetric latent heat of fusion, as well as the volumetric specific heat capacity. While increase in moisture content also increases thermal conductivity, the effect of latent heat of fusion tends to be predominant.

2-4. Effect of surcharge.

a. It has been demonstrated beyond question in both laboratory and field experiments that the rate of frost heaving is decreased by increase of loading on the freezing plane [28,125,166] and that frost heaving can be entirely restrained if sufficient pressure is applied[66]. In foundation design the heave-reducing effect of load may be readily taken advantage of by placing mats of non-frost-susceptible materials on the surface of frost-susceptible soils to reduce the magnitude of seasonal frost heaving. Where the depth of winter freezing is not limited by the presence of underlying permafrost, the heave-reducing effect of such mats is not solely the effect of load; it is also partly a result of the reduction of subgrade frost penetration. The load imposed by the structure and foundation members also contributes to heave reduction.

b. In a field experiment on a silt subgrade near Fairbanks, Alaska, a series of 25-feet-square areas were loaded to values ranging from 0 to 8 psi as shown in figure 2-5 [28]. Ventilation ducts were incorporated in the construction so as to achieve essentially equal depths of subgrade frost penetration in the test sections. As shown in figure 2-6 the seasonal maximum frost heave in this field experiment was reduced from about 0.5 feet to about 0.3 feet with only a 2-psi applied surcharge load and to less than 0. I feet with an 8-psi applied load.

c. Figure 2-7 presents the same data in the form of total stress at the freezing interface (which includes weight of both frozen soil and applied surcharge) versus seasonal heave and frost penetration. This type of presentation is more basic than that in figure 2-6 because it takes into account the total stress against which ice segregation is acting at any point during the freezing. These data indicate that for 5 ft of seasonal subgrade frost penetration an increase in total stress at the freeze/thaw interface from 4 to 10 psi reduced seasonal frost heave from 0.4 to 0.15 feet.

d. On the plot of rate of heave versus applied loading

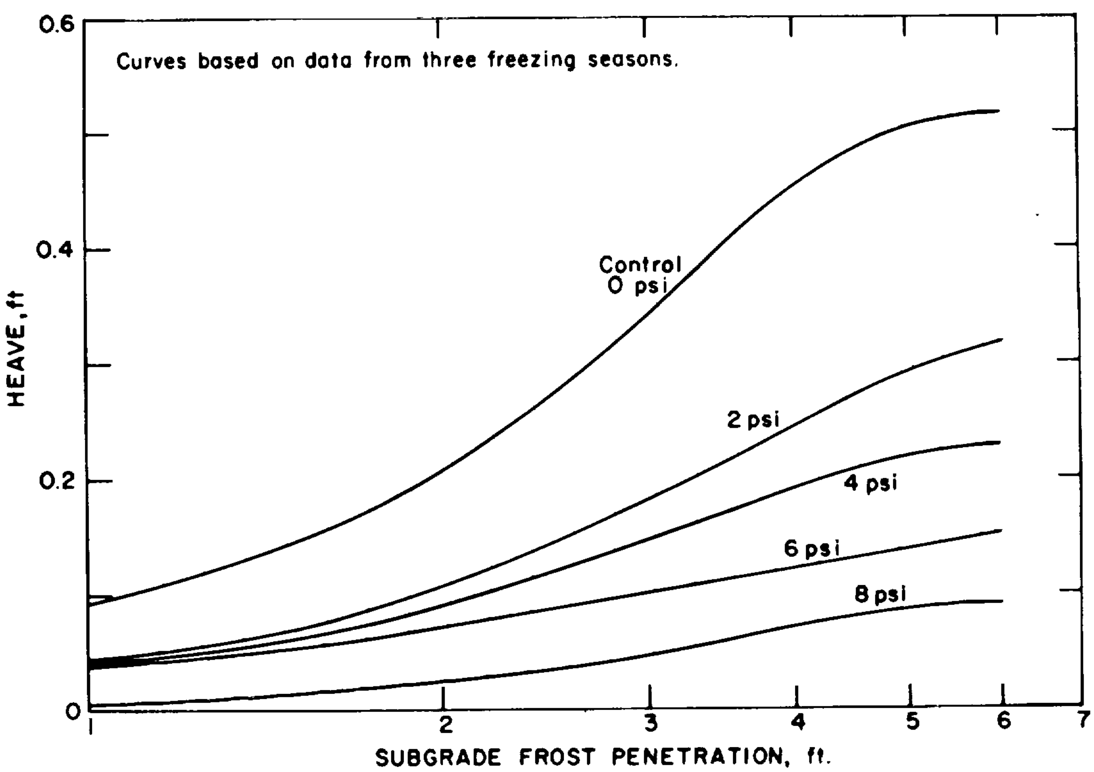

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Figure 2-6. Heave vs frost penetration for various applied loadings, surcharge field experiment". See figure 2-5 for plan and elevation of test installation.

in figure 2-8, comparison is made between laboratory and field test results for silt soils. While the results of the two types of experiment correspond approximately in magnitude, the laboratory tests indicate a more rapid lowering of the rate of heave with increase in surcharge than the field tests. It is believed that this may have been caused by edge effects in the small laboratory specimens; the field values are unquestionably more representative of real construction situations.

e. Also shown in figure 2-8 are laboratory results for WASHO clay (liquid limit = 37.0 percent, plasticity index = 13.0 percent). The flatter curve indicates less rapid reduction of heave with increase in applied load than in the laboratory tests on silts[40]. No quantitative field-scale test on a clay subgrade has been performed and the field quantitative validity of this WASHO clay curve has not been proved; however, there is no question that clays should be less affected by surcharge than silts (see, for example, fig 2-9a). While laboratory data are available on several soils, little reliable field information is available on effect of surcharge for other soils than the Fairbanks silt. Therefore, where advantage is taken of the effect of surcharge and where justified by the scope and details of the construction project, test footings of prototype dimensions using the actual proposed loadings should be constructed in order to obtain data on actual frost heave values which will occur. This is important not only because of the differing behaviors

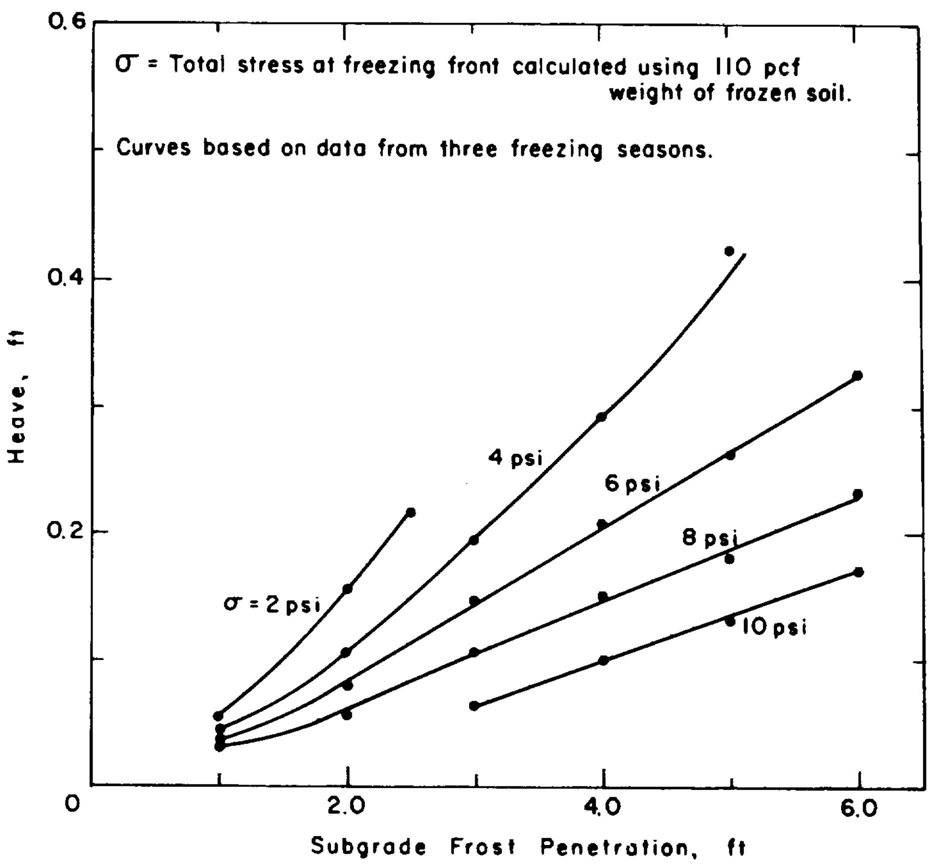

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Figure 2-7. Heave vs frost penetration for various total stresses, surcharge field experiment". See figure 2-5 for plan and elevation of test in stallation.

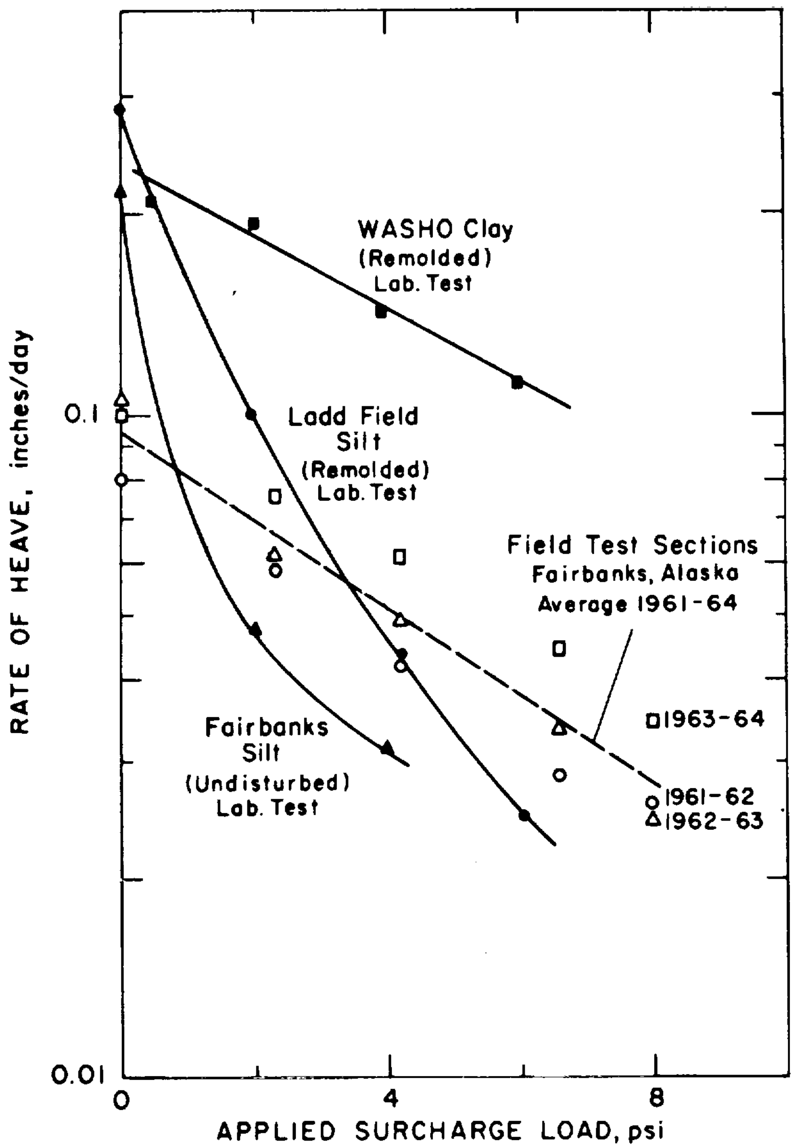

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Figure 2-8. Comparison of laboratory and field measurements of effects of surcharge[28,40].

of different soil types but also because variations in climatic and groundwater conditions may also be expected to affect the field behavior.

f. In laboratory freezing experiments, heaving pressures of the magnitudes shown in figure 2-9 have been measured under conditions of essentially complete restraint. Therefore, if foundation loadings at the freezing plane equal or exceed these pressures, heave will be prevented completely. For many engineering structures such as pavements, such complete prevention is unnecessary and uneconomical. For structures particularly sensitive to movement, however, complete prevention may be essential, and in some cases it may be feasible to achieve this result by providing sufficient foundation loading, allowable foundation bearing values permitting. However, uplift computations cannot be made simply by applying the pressures of figure 2-9 to the areas of direct foundation loading, as frost heave uplift acts on the base of a frozen slab of soil whose effective area may be much greater than the area of the structure foundation, as illustrated in figure 4-42a. The heave-reducing effect of surcharge is presently taken into account in the limited subgrade frost penetration method of pavement design (see TM 5-818-2[6]). This approach to limiting differential movements due to frost heaving is also applicable to unheated warehouses, POL facilities and transmission towers, and may enter into the design of many other types of facilities.

2-5. Foundation materials.

a. Soils.

(1) Permafrost soils cover the entire range from very coarse, bouldery glacial drift through gravels, sands, silts and clays to organic soils. Slightly undersaturated coarse, bouldery frozen soils at low temperature, such as are encountered in northern Greenland, behave in excavation and tunneling as if they are granite[24]. At the other extreme, in fat clays at temperatures not far below the freezing point, only a relatively small percentage of the soil water may actually be frozen, and the behavior of such soil may be only slightly altered by the freezing temperatures. In some

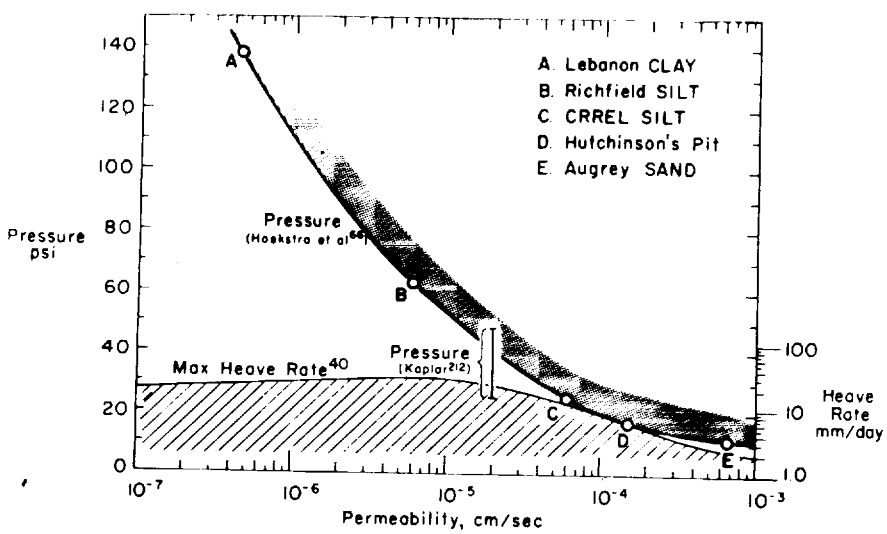

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Figure 2-9a. Maximum Frost Heave Pressures. (Pressure and Heave vs Permeability.)

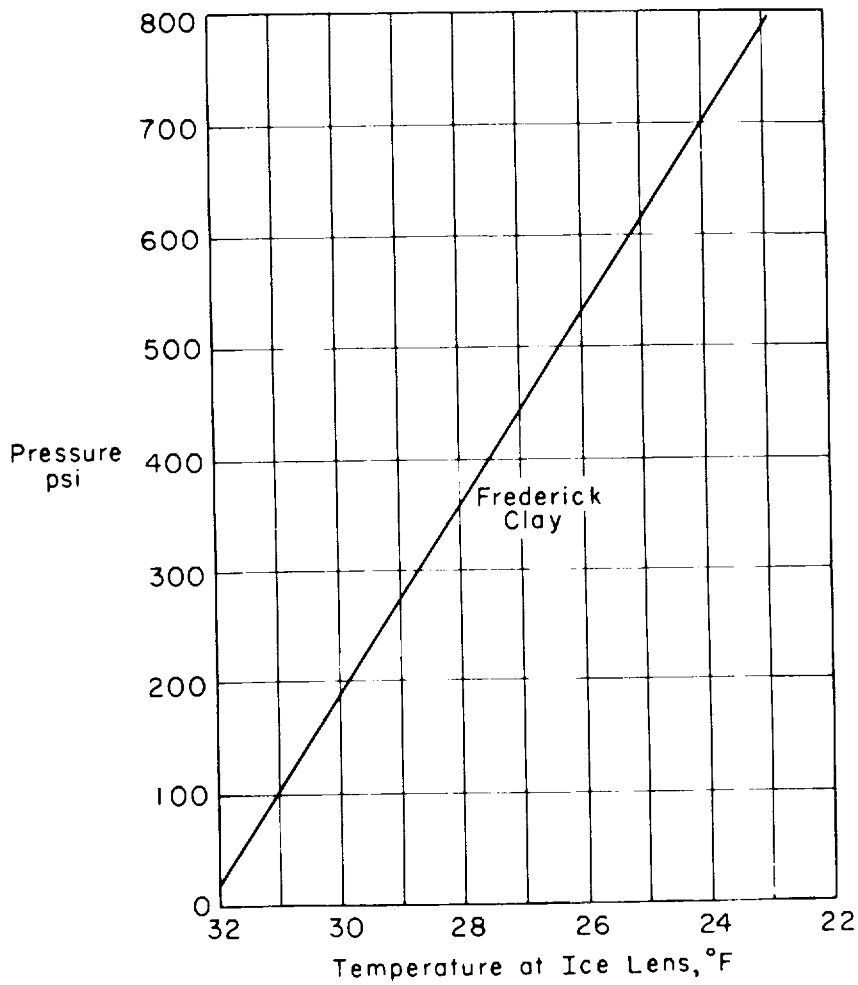

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Figure 2-9b. Maximum Frost Heave Pressures. (Temperature at Growing Ice Lens vs Maximum Pressure Developed at that Depth.[213].)

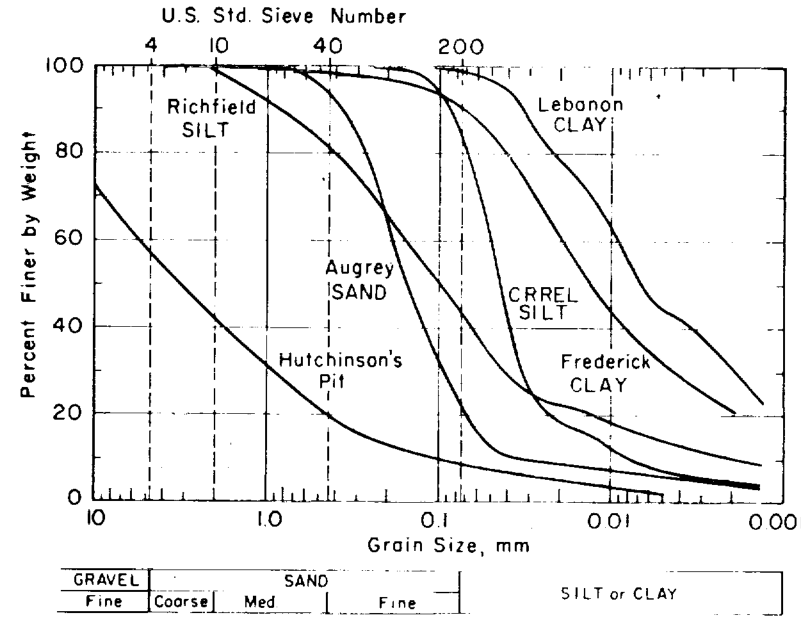

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Figure 2-9c. Maximum Frost Heave Pressures. (Grain, Size Distribution, of Soils.)

areas, layers of salty groundwater and unfrozen strata may be encountered in the soils[208]. Methane pockets are common because of entrapment by the impervious frozen soil, and animal and vegetative remains are often found surprisingly intact, their decomposition rates slowed in the permafrost. Organic soils are common, both in permafrost and seasonal frost zones. Organic soils range from only slightly organic mineral soil to 100 percent organic muskeg or peal. They cover about 10 percent of the land area of Alaska and present both transportation and construction obstacles[189].

(2) Figure 2-10 shows typical compressive strength values for nine types of frozen soil including peat, in laboratory tests performed at 400 psi/min rate of stress increase. Properties of these soils are summarized in figure 2-11. Figure 2-12 presents a similar summary for tension tests performed on these soils at a rate of stress increase of 40 psi/min. Figure 2-13 shows a summary for shear tests performed at a rate of stress increase of 100 psi/min.

(3) Strength properties of frozen ,oils are dependent on such variables as gradation, density, degree of saturation, ice content, temperature, percentage of moisture in specimen frozen, dissolved solids, and rate of loading. Available data on effect of rate of loading on strength of frozen materials are summarized in figure 2-14. Frozen soils characteristically exhibit creep at stresses as low as 5 to 10 percent of the rupture strength in rapid loading[53,187]. Typical creep relationships are shown in figure 2-15.

(4) The effect of ice content on the compressive strength of Manchester fine sand is shown in figure 2-16.

(5) Values of dynamic moduli and Poisson's ratio determined by flexural vibration are summarized in figure 2-17 [68]. See also paragraph 4-6. Values of adfreeze strength for tangential shear on various materials are discussed in paragraph 4-8.

(6) It will be apparent from the information presented above that it is not feasible to give categorical

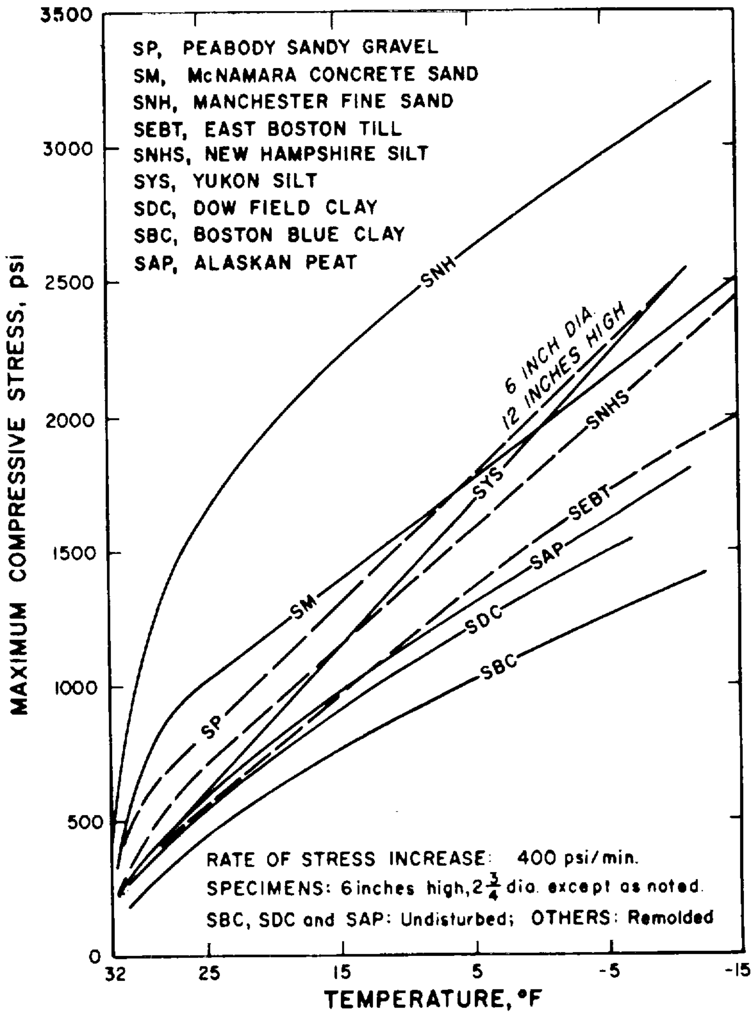

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Figure 2-10. Summary of maximum stress in compression vs temperature. See figure 2-11 for soil characteristics[34].

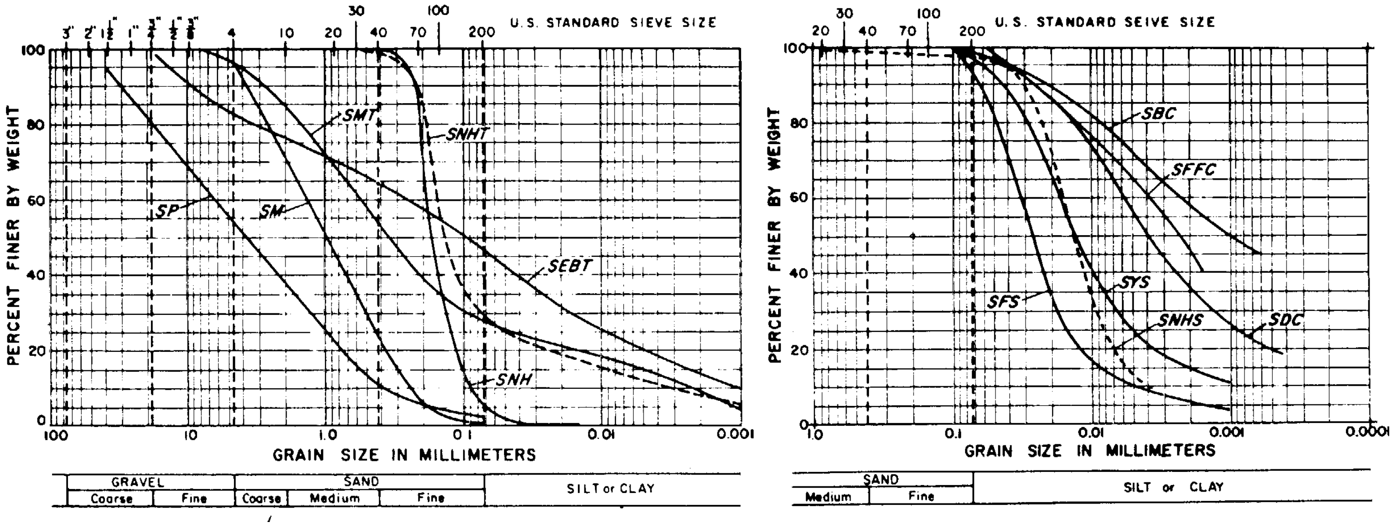

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Figure 2-11a. Summary of Soil Characteristics[34] (Gradations)

SOIL LEGEND	NAME	SOURCE	DEPARTMENT OF THE ARMY UNIFIED SOIL CLASSIFICATION		ATTERBERG LIMITS			SPECIFIC GRAVITY	COMPACTION CHARACTERISTICS		TYPICAL PROPERTIES OF SPECIMENS BEFORE FREEZING (4)					MAX. UNCONFINED COMPRESSIVE STRENGTH	DIRECT SHEAR TEST ANGLE OF INTERNAL FRICTION - ϕ
			DESCRIPTION	GROUP SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	SHRINKAGE LIMIT		MAX. DRY UNIT WT.	OPTIMUM WATER CONTENT	SIZE DIAM.	SIZE HT.	DRY UNIT WT.	VOID RATIO	COEF. OF PERMEABILITY		
									pcf	%	inches	inches	pcf		$cm/sec \times 10^{-4}$	psi	degrees
SP	Peabody Gravelly Sand	Peabody, Mass.	Bank run gravelly SAND	SP	non-plastic		-	2.72	134 (1)	-	6.00	12	175	0.356	120.0	-	-
SM	McNamara Concrete Sand	Needham, Mass.	SAND, brown, angular, processed for concrete.	SP	non-plastic		-	2.72	123 (1)	-	2.75	6	117	0.456	84.0	-	39
SMN	Manchester Fine Sand	Manchester, N. H.	Uniform, fine SAND, light brown, clean.	SP	non-plastic		-	2.68	106 (1)	-	2.75	6	101	0.658	52.0	-	35
SMT	Blend, McNamara Concrete Sand and East Boston Till	SM blended with 35% minus 40 mesh SEBT	Silty SAND	SM	non-plastic		-	2.72	142 (2)	6.8	2.75	6	135	0.262	0.002	-	38
SMNT	Blend, Manchester Fine Sand and East Boston Till	SMN blended with 35% minus 40 mesh SEBT	Silty SAND	SM	non-plastic		-	2.72	128 (2)	9.2	2.75	6	121	0.398	0.106	-	32
SEBT	East Boston Till (-3/4")	East Boston, Mass.	Clayey, gravelly SAND (Glacial Till)	SC	21	7	-	2.76	137 (2)	8.1	2.75	6	130	0.327	0.006	-	28
SNHS	New Hampshire Silt (B)	Manchester, N. H.	Light gray brown, inorganic clayey SILT	CL-ML	26	5	-	2.70	107 (2)	15.6	2.75	6	102	0.653	0.032	-	30
SFS	Fairbanks Silt	Fairbanks, Alaska	Brown and gray SILT, containing traces of mica and some organic matter.	ML-OL	28	4	-	2.68	112 (2)	15.7	2.75	6	105	0.590	0.160	-	32
SYS	Yukon Silt	Whitehorse, Yukon Territory, Canada	Gray, well graded, inorganic clayey SILT	CL-ML	28	9	-	2.73	121 (3)	12.8	2.75	6	115	0.479	0.007	-	38
SDC	Dow Field Clay	Dow Air Force Base Bangor, Me.	Very stiff, lean CLAY, some fractures, brown organic stain and trace of fine roots.	CL	34	17	20	2.79	-	-	2.75	6	100	0.738	0.004	38.1	-
SBC	Boston Blue Clay	No. Cambridge, Mass.	Stiff lean CLAY, relatively homogeneous and free of fractures and varves.	CL	47	27	22	2.81	-	-	2.75	6	85	1.057	0.002	21.9	-
SFFC	Fargo Clay	Fargo, North Dakota	Dark gray, friable, highly plastic homogeneous fat CLAY, with honey-comb structure (organic content 8%)	CH-OH	68	46	15	2.76	-	-	2.75	6	87	0.980	0.035	9.2	-
SAP	Alaskan Peat	Fairbanks, Alaska	Dark brown to black PEAT; fibrous, partially decomposed (organic content 82 percent)	Pt	Tests inapplicable			1.52	-	-	2.75	6	-	-	-	-	-

NOTES: (1) Providence Vibrated Density
(2) Modified AASHO Density
(3) Standard Proctor Density
(4) All specimens de-aired and saturated at 40F prior to freezing. In saturation procedure, vacuum applied at top with water supplied at bottom for minimum of 12 hrs for non-frost-susceptible soils and 24 hrs for frost-susceptible soils.

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Figure 2-11b. Summary of Soil Characteristics (Void Ratio vs Coefficient of Permeability)

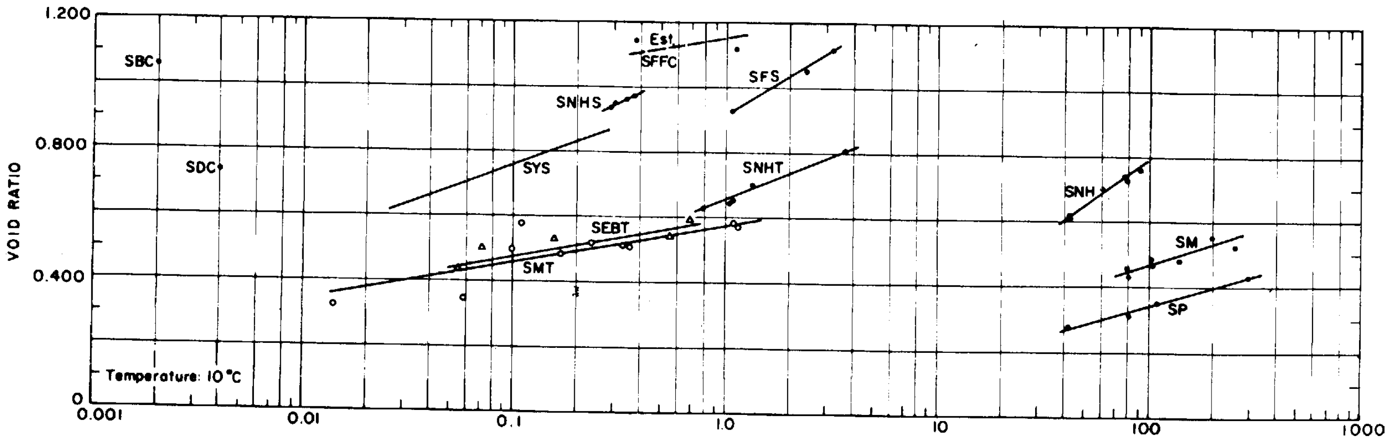

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Figure 2-11c. Summary of Soil Characteristics. (Soils Test Data)

SM..........McNamere Concrete Sd
SNH........Manchester Fine Sand
SMT........Blend, SM and SEBT
SNHT......Blend, SNH and SEBT
SEBT......East Boston Till
SNHS .....New Hampshire Silt
SFS........Fairbanks Silt
SYS........Yukon Silt
SBC........Boston Blue Clay
SFFC......Fargo Cloy
SAP........Alaskan Peat
SI............Artificially-Frozen Ice

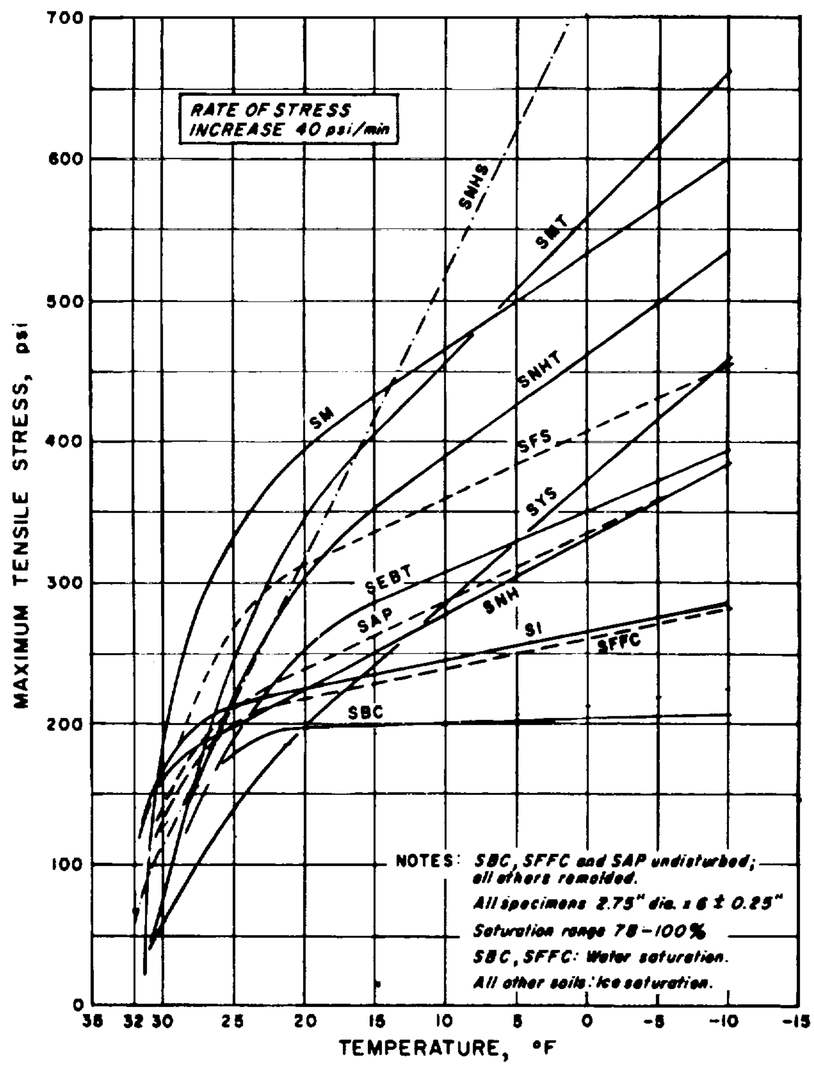

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Figure 2-12. Summary of maximum stress in tension vs temperature. See figure 2-11 for soil characteristics[34].

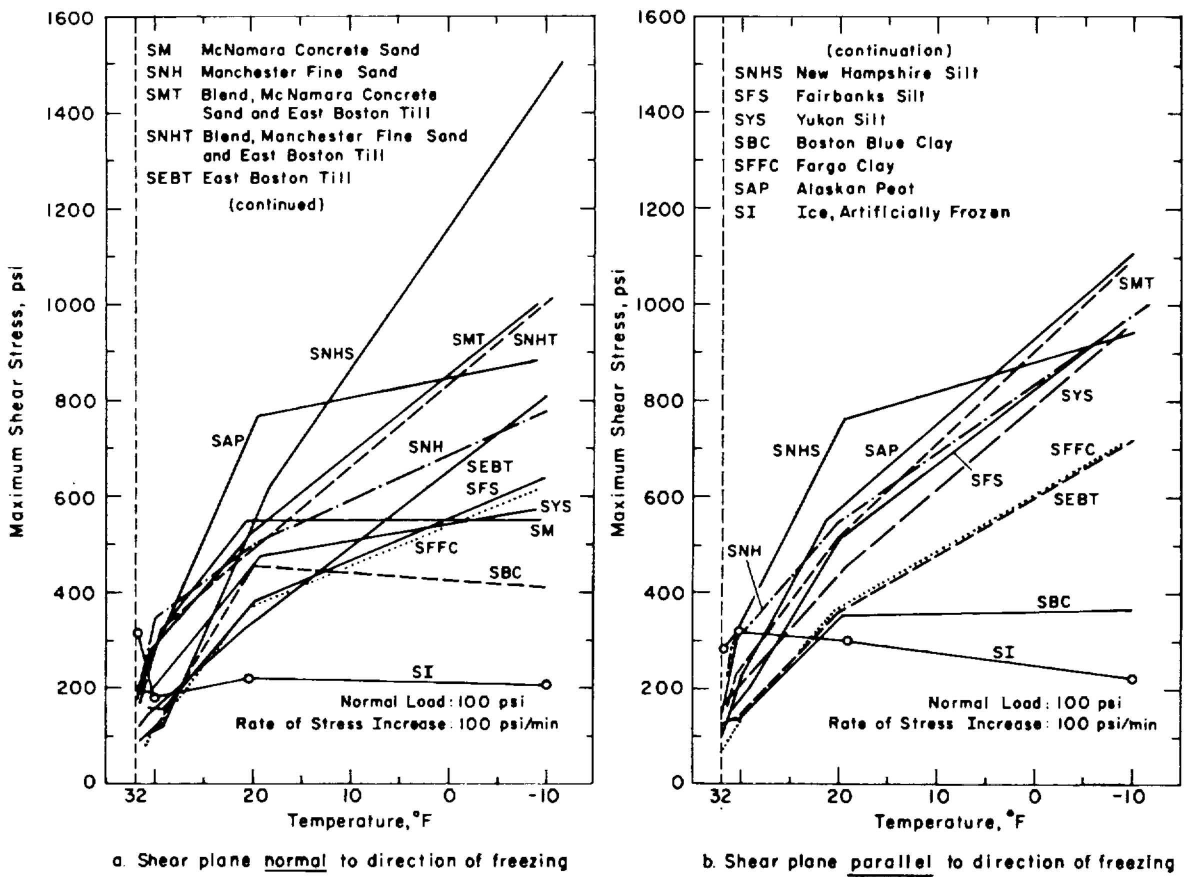

Figure 2-13. Summary of maximum shear stress of frozen soil vs temperature. See figure 2-11 for soil characteristics[34].

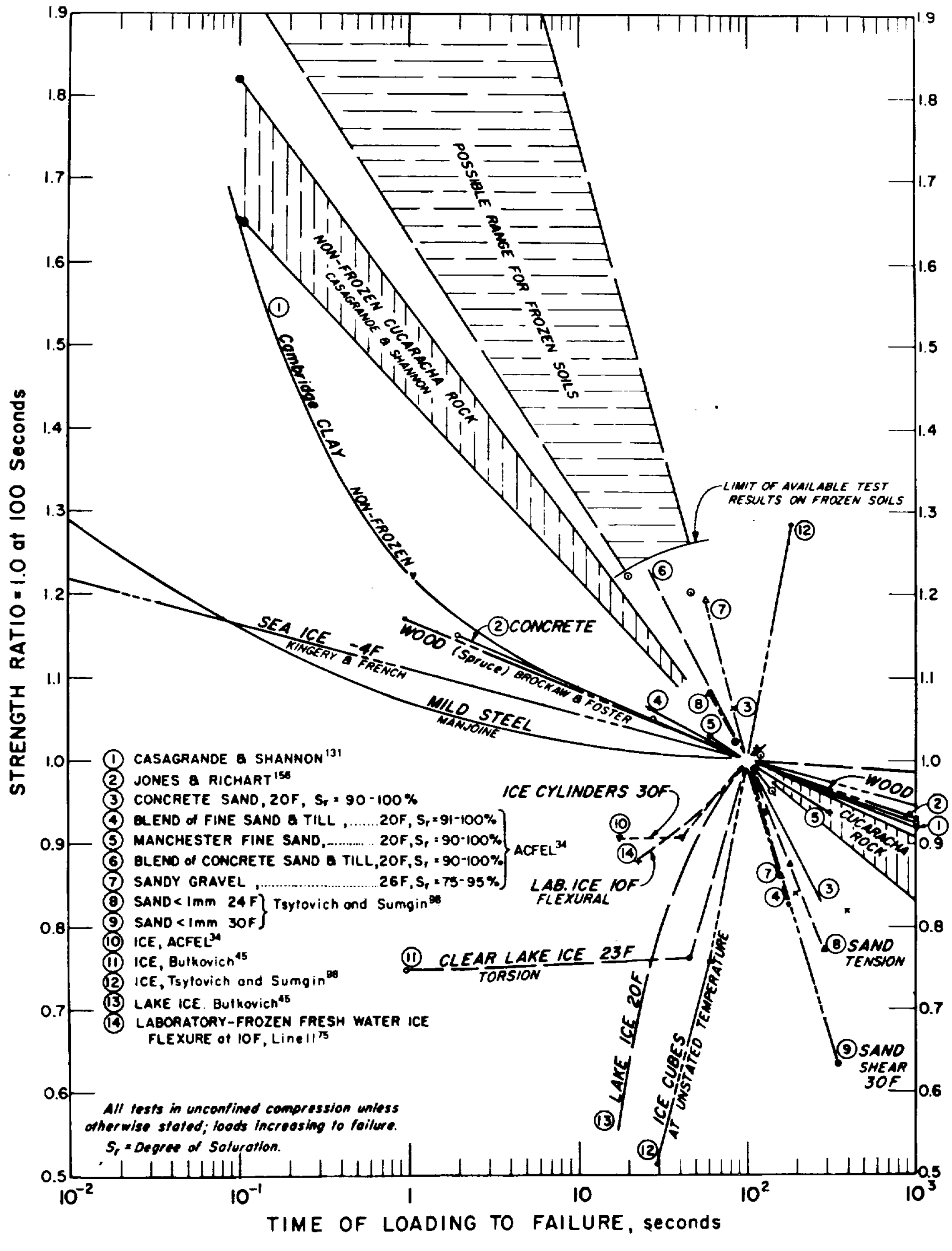

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Figure 2-14. Effect of rate of stress application on failure strength[69].

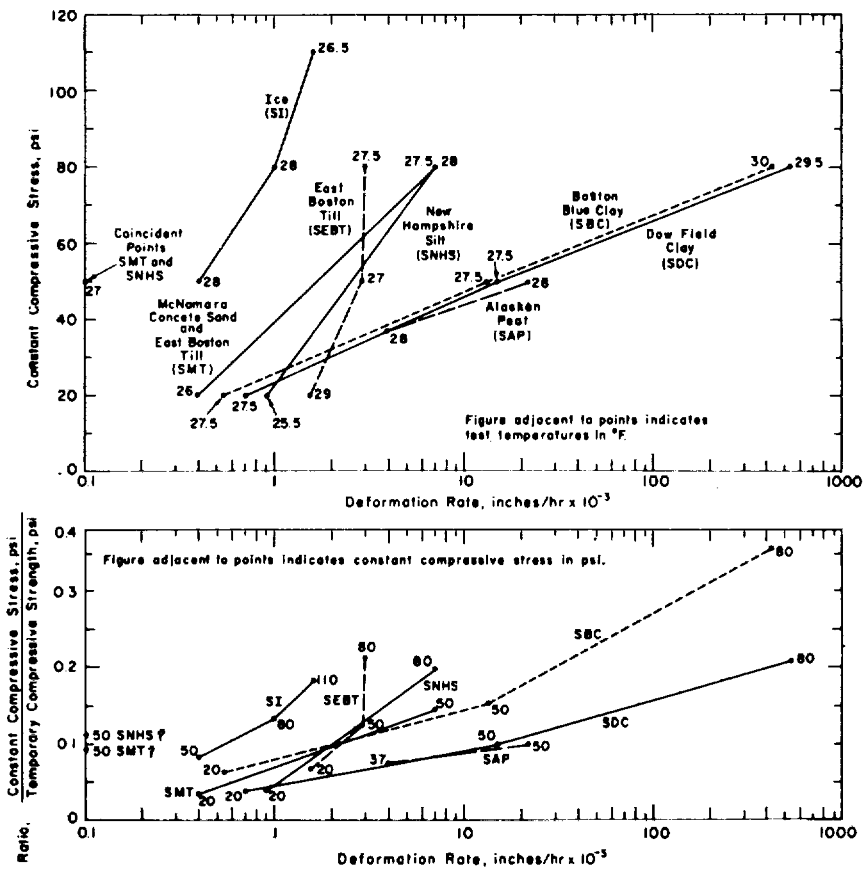

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Figure 2-15. Plastic deformation of frozen soils under constant compressive stress. See figure 2-11 for soil characteristics[33].

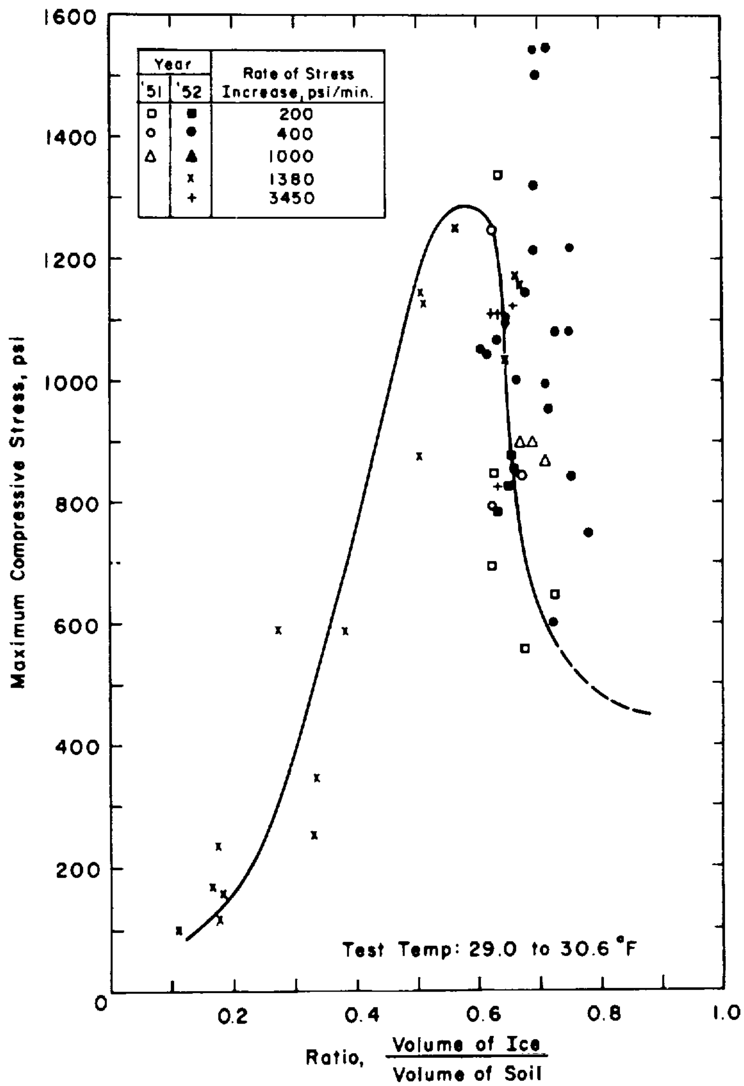

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Figure 2-16. Compressive strength vs ice content, Manchester fine sand[34].

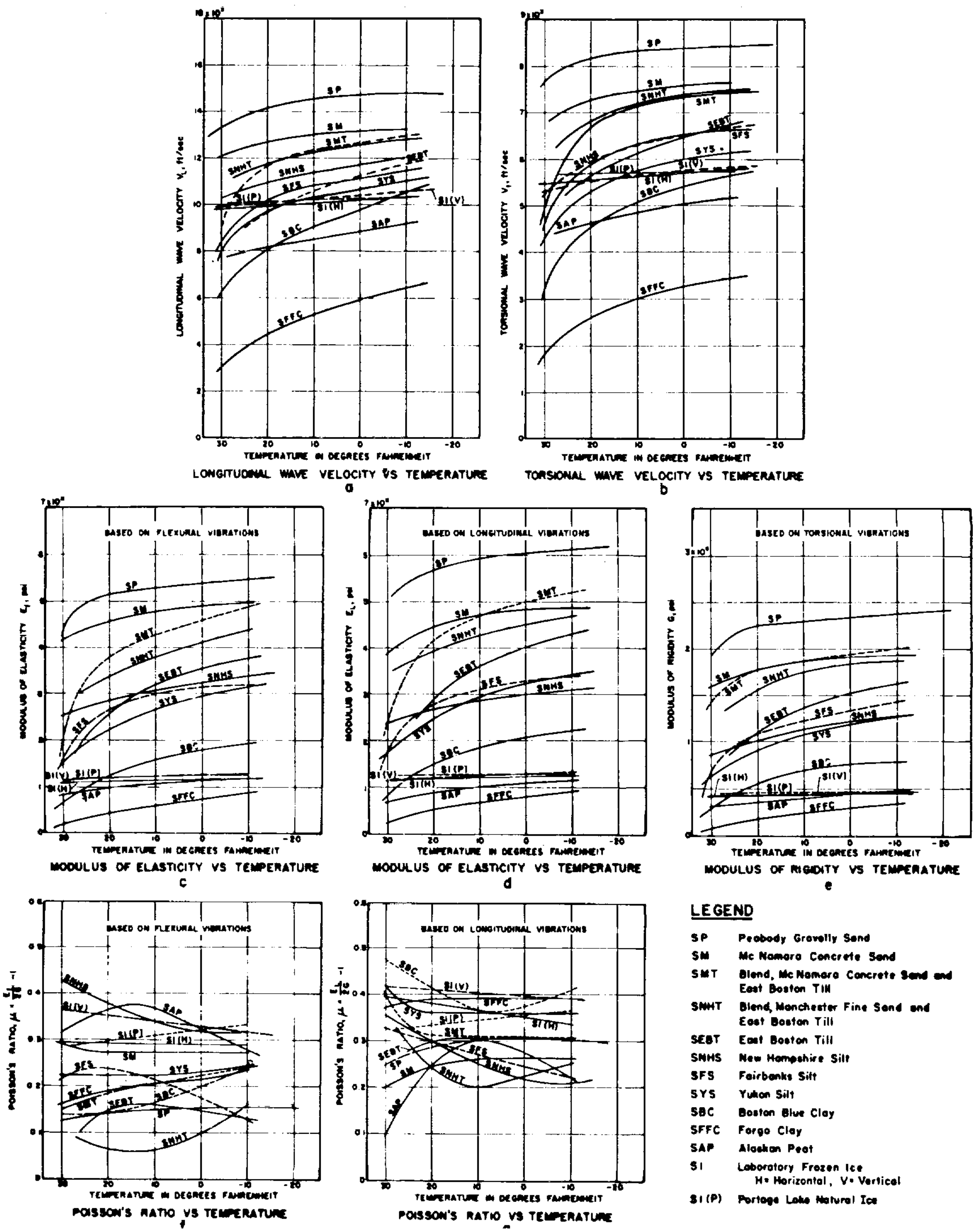

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Figure 2-17. Longitudinal and torsional wave velocity, dynamic moduli of elasticity and rigidity, Poisson's ratio vs temperature. Each curve represents test results of two to six specimens. See figure 2-11 for soil characteristics[68].

property values which are typical for frozen soils under all conditions.

b. Ground ice. When ice is found in permafrost in sufficient amounts that significant settlement would occur upon thawing, it may have originated by the conventional ice segregation process with ice lensing primarily parallel to the ground surface as the permafrost was gradually developed in the geologic past; it may have been formed as vertical ice wedges in the horizontal contraction-expansion process which results in the typical patterned ground features so evident in far northern areas; it may be "fossil ice" buried by landslides or other events and preserved as permafrost; or all three of these types may occur together. Often several soil formations of different ages may be superimposed one on another, each containing ground ice. Figure 2-18 shows a typical cross section in silt near Fairbanks, Alaska; note that ice concentrations do not necessarily decrease with depth, contrary to a frequent assumption.

(1) In the annual frost zone, ice is of the common ice segregation type; small amounts of ice may also be found in shrinkage cracks. Ice formations in this zone disappear every summer and are formed anew in the winter. Substantial ice concentrations are frequently found on permafrost at the bottom of the annual thaw zone which thaw only in occasional very warm summers.

(2) Occasionally bodies of permafrost may be encountered (such as near the top of a high, well-drained bluff) which are less than 100 percent saturated; such permafrost may lack some of the detrimental characteristics associated with ground ice. However, if ground ice exists in strata at lower levels in the foundation and thaw may reach the ice during the life of the structure, this must be taken into account in the design.

(3) Ice masses in clean, granular deposits are not uncommon in the Arctic because of the severe environment, but their occurrence is less common in the Subarctic. U.S. Army Engineer District, Alaska, personnel have reported the occurrence of major ice inclusions in gravels at Cape Lisburne, Alaska, and at Gambel on St. Lawrence Island off the coast of Alaska, and others have reported ice wedges and masses in gravels at Barrow and Umiat, Alaska, and Inuvik, N.W.T., Canada. Church, Pewe and Andresen[46], in their study of patterned ground in the Donnelly Dome area near Ft. Greely, Alaska, found evidence that ice wedges had formed in the outwash gravels of the area during a period colder than at present and subsequently thawed during a period warmer than now exists. Similar evidence has been observed at Clear, Alaska. Thus, the possibility should be considered that, even in the Subarctic, ice wedges formed in clean granular materials in a previous colder climatic period may be found preserved by overlying accumulations of soil which have protected the ice from thawing, or by other especially favorable local conditions. The possibility of such ice will be greater, the colder the climate of the location.

(4) Open-work gravel containing clear ice tends to give a visual impression that substantial settlement of this material may occur on thaw, but close examinations of random samples of such material near Fairbanks, Alaska, have shown particle-to-particle contacts, and no actual thaw-settlement difficulties with such soils have thus far been reported. On the other hand, personnel of the Alaska District, Corps of Engineers, report that many ice-saturated gravel deposits do not shown particle-to-particle contact, and that tests of sands have shown consolidation on thaw. It may be visualized that if permafrost is formed in clean, granular, non-frostsusceptible material by continuous downward advance of the freezing front under conditions permitting escape of any excess water then there may be no separation of the soil particles which would lead to consolidation on thaw. However, if opportunity for escape of excess water is lacking, either in the original freezing or in refreezing from a partially degraded condition, as from a climatic warming interval, then a bulked condition will exist in the zone of water entrapment; this zone may be horizontally discontinuous. Finally, even essentially clean deposits may develop some ice segregation during downward freezing if even thin inclusions of fine material are present, or possibly if the freezing front advances very slowly or is stationary. Thus, no simple generalizations are safe.

(5) In borderline permafrost areas remnants of permafrost containing ground ice may sometimes be found at depths such as 25 to 40 ft, with non-frozen soils both above and below. Such was the experience in deep granular deposits at Clear, Alaska. Extensive exploration is required to locate these remnants when they occur as small, scattered bodies.

(6) If fine-grained soils containing ground ice are excavated, very wet, liquid-soft material unusable for earth fill and incapable of supporting equipment may result on thaw. From coarse, free-draining granular soils and broken rock, however, thaw-water may escape almost as rapidly as thaw occurs. The settlement of foundations from degradation of permafrost and the development of frost heave in the annual frost zone both typically occur differentially between points across the foundation. The thawing of polygon ice wedges such as shown by figures 2-2 and 2-18 may tend to develop cavern-like voids in the foundation which may lead not only to sudden and dangerous collapses but also to development of underground drainageways which may serve to extend thaw. Loess-type frozen soils may be particularly susceptible to subsidence and erosion on thaw. Where foundations include anchorages in permafrost, thaw may lead to anchorage failures.

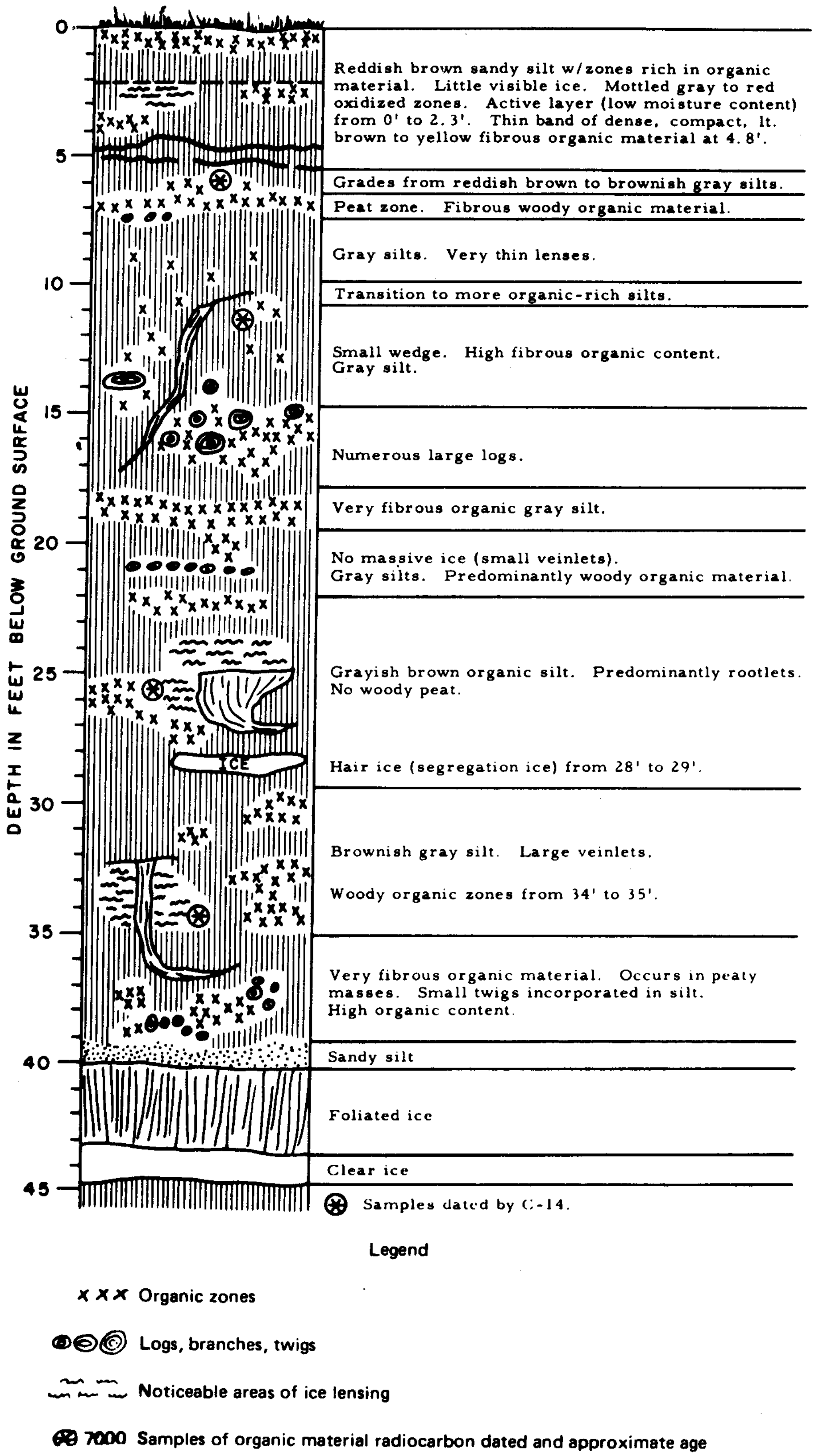

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Figure 2-18. Typical record of ice in permafrost[89].

(7) For engineering purposes it is very important to know whether significant settlement will take place upon thawing of the frozen soil. If the ice present will produce more water upon melting than can be retained within the voids of the soil after thaw, then the material is thaw-unstable to a degree that is dependent upon the amount of excess ice and the soil density. If all the melt water can be absorbed by the soil voids without significant settlement, then the soil can be considered thaw stable. Thaw-consolidation computations are outlined in chapter 3.

c. Rock. Bedrock should never be assumed free of ice if the melting of such ice is possible and the consequences significant. Numerous cases are on record where this assumption was made and substantial volumes of ice were later found to be present in the bedrock. At Thule, Greenland, this situation resulted in substantial settlement of a building; required repair measures included installation of artificial refrigeration in the foundation on a permanent basis. At the same location the disappearance of water from a drainage ditch cut in shale resulted in discovery of ice layers up to 20 inches thick in the rock which were being melted by the drainage waters[76]. The only safe approach is to carry subsurface explorations into bedrock, obtaining undisturbed frozen cores which will reveal the exact thicknesses of any ice strata present.

(1) Structurally, the frozen water in bedrock fissures adds substantially to the competence of the rock, so long as the ice remains frozen. At Yellowknife, NWT, Canada, it has been stated that in mining of ore by the stopping process, excavations can safely be made in 300 foot lifts whereas the standard height in unfrozen rock in Canada is 150 feet. Blasting specialists responsible for mining of iron ore at Knob Lake, Quebec, Canada, have stated that three times as much powder is required for blasting of ore rock containing 10 percent ice as for blasting of unfrozen deposits"'. On the other hand, ore containing this much moisture is very wet when thawed and very substantial problems have been encountered in handling, transporting and using such material.

(2) Bedrock is frequently a source of severe frost heave because of mud seams in the rock or concentrations of fines at or near the rock surface in combination with the ability of fissures in the rock to supply large quantities of water for ice segregation.

2-6. Structural materials.

Foundations of structures in areas of deep seasonal frost and permafrost tend to experience larger and less predictable tensile, compressive, and shear stresses than in more temperate regions as a result of thermal expansion and contraction effects, unstable conditions during and after the construction period, and seasonal frost action. The properties and behavior of structural materials used in the foundations are also affected by the low temperature conditions. Materials susceptible to brittle fracture at low temperatures should not be specified for conditions where they may fail. Consideration must be given to logistics, supply, costs and problems of field fabrication and erection. Shop fabrication may be found more economical than field fabrication in spite of greater logistics problems. Availability of local materials must also be considered. Native materials are often scarce and not very suitable and construction practices may differ because of transportation problems, equipment and labor available and severe weather. The relative cost and availability of all construction materials must be established early in the design; this will influence the selection of the foundation type to be used.

a. Wood. Wood is a satisfactory and widely used construction material because of availability, low thermal conductivity, adequate durability, when selectively used and treated if required, flexibility, unimpaired strength at low temperature, low weight and availability. However, in many arctic and subarctic areas it may be as unavailable as any other construction material. Even when timber is obtainable in the Subarctic, available sizes may be limited and quality low.

(1) A principal use of timber in foundations is in piling. Spread footings have been frequently made of wood. Heavy timbers are widely used for distributing structure loads onto piles or other principal supporting members. Timber is also useful as a semi-insulating pad under a concrete footing. Rock fill wood cribs are useful for many purposes. Although timber piles have lower thermal conductivity than concrete or steel piles, this is usually a negligible advantage for the thermal. stability of foundations. It may be assumed that no decay will occur in portions of piles embedded in permafrost. Within the annual frost zone, temperatures are low throughout the year and below freezing temperatures prevail for many months; groundwater is often near the surface throughout the year. These factors slow decay below ground, though not guaranteeing against it. At the ground line, however, experience shows that, except in the high Arctic, timber members may be destroyed in only a few years even where the mean annual precipitation is low. While wood piles therefore need preservative treatment above the permafrost table, a coating of creosote on the portion of a pile embedded in permafrost reduces the tangential shear stress which can be developed below that which would apply for bare wood. In theory then, only the upper parts of wood piles should be so treated. However, no method exists for pressure creosoting only part of a pile. Therefore when wood piles require pressure treatment, the entire length must be treated and tangential adfreeze working stresses

must be established at sufficiently low values to reflect the presence of the surface coating of treatment material. Wood piles which are embedded entirely in permafrost (capped at the permafrost table) need not be treated.

(2) One of the most important properties influencing strength of wood, especially at low temperatures, is the moisture content. Table 2-2 presents the relationships of specific gravity and various strength properties of wood at room temperature, based on average results of tests on a large number of species. As illustrated in figure 2-19 compressive strength tests at 44°F show higher values at all moisture contents than tests at 68 °F with a maximum at approximately 82 percent. Tests conducted by Kollman[159] as shown in figures 2-20 and 2-21 indicate nearly straight line increases of compressive strength and toughness ratio with decreasing temperatures over a wide range. These effects are proportional to density. Experiments at the Forest Products Laboratory with Sitka Spruce and Douglas Fir showed an increase in bending strength of about 1½ to 2½ in the cold to room-temperature strength ratio. Figure 2-22a shows the effect of temperature on the modulus of rupture and modulus of elasticity of three species of wood tested in flexure and figure 2-22b shows the effect of temperature and moisture content on the modulus of elasticity of plywood. One of the special characteristics of wood in the frozen state is that failure is often sudden, without the usual audible advance warning of overloading. Such failure occurs violently with a loud report and separation of the parts is complete.

(3) Wood piles cannot be driven into permafrost by hammer; this may dictate the use of steel instead of wood piles. Wood and steel must both be protected from the possibility of fire damage which is a problem of major concern in remote cold areas. (See TM 5-852-9/AFM 88-19, Chap. 9 [16].)

b. Metals. The predominant problem encountered with use of metal in cold regions is cold embrittlement fracture. Generally speaking, as temperature is lowered the hardness, yield strength, modulus of elasticity, and endurance limit of most metals and alloys increase. However, many of these same metals become embrittled at reduced temperatures and will shatter or fracture when subjected to stresses (especially due to impact) that would be allowable at normal temperatures. Characteristic of this type of failure is lack of deformation or prior indication of failure.

(1) A relatively small group of pure metals remains ductile at low temperature, including nickel, copper, aluminum, lead and silver. Several other metals such as magnesium, zinc and beryllium are brittle, with little ductility even at room or slightly higher temperatures. The behavior of ferrous metals covers a very broad range of possibilities. The brittleness of carbon steels increases with the carbon content (up to 0.25 percent), and higher carbon steels may be expected to be brittle. Although special alloy steel is not likely to find application in foundation work, it may be noted that addition of nickel in the amount of 2-¼/4 percent will ensure satisfactory properties of steel to temperatures of -50°F and additional amounts up to 9 percent will increase the range to -320°F. Almost all aluminum and titanium alloys can be used at low temperatures (except high strength aluminum in which stress concentrations are likely to occur) and essentially all nickel and copper base alloys. However, because the properties of zinc and its alloys are adversely affected by low temperature, brass, a zinc-copper alloy, is unsuitable for low temperature use. Parts made from brass have been known to shatter under normal handling techniques. Galvanizing has shown cracking and spalling when subjected to cold in arctic areas if the coating was too thick.

(2) Cold embrittlement susceptibility of the steels used should be known, and the possible effects of the temperature extremes and types of loads which will be experienced during both construction and subsequent operation should be examined. As evidenced by years of experience with mild steels in structures throughout Alaska, it should not be necessary to use special alloys in structures and foundations when the number of fatigue cycles or dynamic stress level is low. However, the possibility of brittle fracture in either structural components or construction equipment under other conditions including the construction phase must always be kept in mind. For example, components of excavating equipment often fail when operated at extremely low temperatures unless they are kept heated or are made of special, higher cost steels.

(3) Steel piles can be driven successfully into permafrost composed of fine-grained soil (sand, silt or clay) at soil temperatures down to about 25 F. Steel pipe piles, 6 inches in diameter (schedule 80), were driven with a diesel pile hammer to a depth of 21 feet into the silt-ice soil at the Kotzebue Air Force Station in the late winter with soil temperatures approximately as indicated in figure 1-3 for 11 February 1960. Compressed air, diesel and vibratory hammers have been used successfully, [47,48,49,134]. Examination of steel pipe and H-piles installed for periods of 8-11 years at the U.S. Army CRREL field station at Fairbanks, Alaska, showed that the length of pile embedded in permafrost was unaffected by corrosion and only insignificant effects were observed in the annual thaw zone[183,184]. Protective coating of steel in the annual frost zone, therefore, is optional; it may be needed only under special local conditions. Paint or similar protective coatings reduce the potential tangential shear which can be developed between frozen soil and pile surfaces and care must be taken that such materials are not applied below the level

Table 2-2. Specific Gravity and Strength of Wood[43]

Specific gravity of typical species at 12% moisture content are:

Ponderosa pine	0.40	Western hemlock	0.48
Loblolly pine	0.51	Eastern hemlock	0.41
Sitka spruce	0.40	Lodgepole pine	0.41
Douglas-fir, coast type	0.48	Englelmann spruce	0.34
White fir	0.37	Alaska cedar	0.44

(Forest Products Laboratory,
U.S. Department of Agriculture)
Specific Gravity-Strength Relation

	Green wood	Air-dry wood (12% moisture content)
Static bending:		
Fiber stress at proportional limit, psi	$10{,}200G^{1.25}$	$16{,}700G^{1.25}$
Modulus of rupture, psi	$17{,}600G^{1.25}$	$25{,}700G^{1.25}$
Work to maximum load, in. lb/in.^{3}	$35.6G^{1.75}$	$32.4G^{1.75}$
Total work, in. lb/in.^{3}	$103G^2$	$72.7G^2$
Modulus of elasticity, 1000 psi	2,360G	2,800G
Impact bending, height of drop causing complete failure, in.	$114G^{1.75}$	$94.6G^{1.75}$
Compression parallel to grain:		
Fiber stress at proportional limit, psi	5,250G	8,750G
Maximum crushing strength, psi	6,730G	12,200G
Modulus of elasticity, 1000 psi	2,910G	3,380G
Compression perpendicular to grain, fiber stress at proportional limit, psi	$3{,}000G^{2.25}$	$4{,}630G^{2.25}$
Hardness:		
End, lb	$3{,}740G^{2.25}$	$4{,}800G^{2.25}$
Side, lb	$3{,}420G^{2.25}$	$3{,}770G^{2.25}$

*The properties and values should be read as equations: for example, modulus of rupture for green wood = $17{,}600G^{1.25}$ where G represents the specific gravity of oven-dry wood, based on the volume at the moisture condition indicated.

(Courtesy of Forest Products Laboratory USDA)

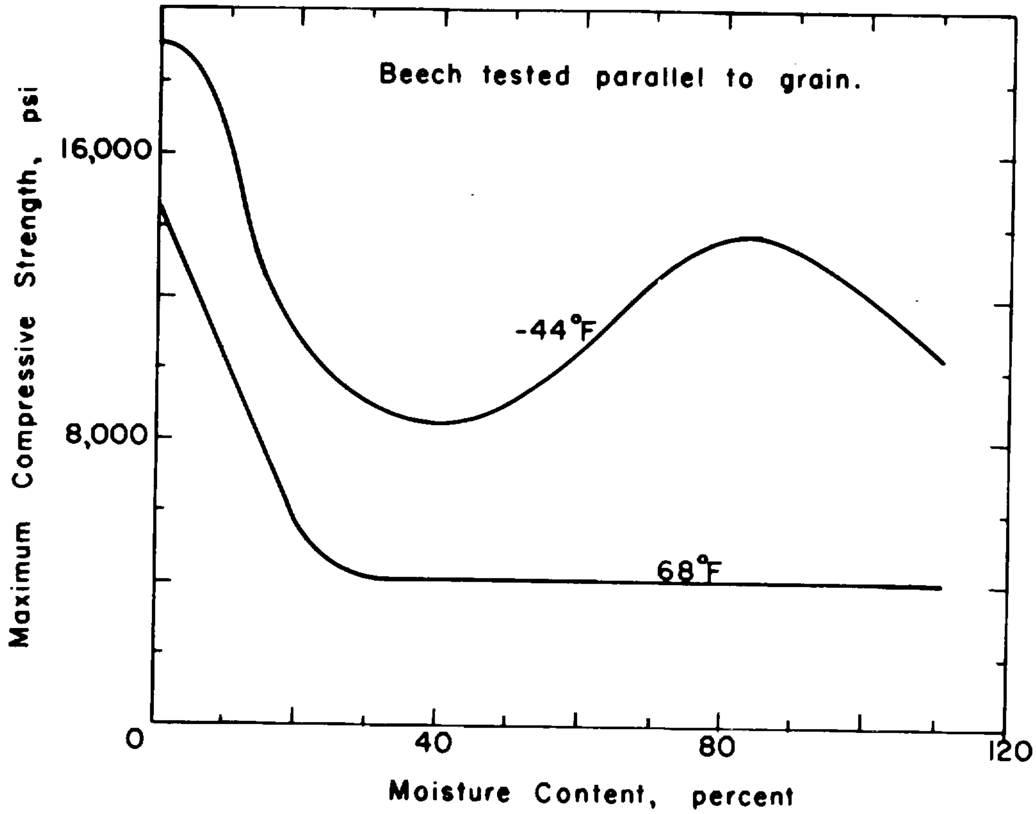


Figure 2-19. Influence of moisture and temperature on strength of wood. (Adapted from Kollman[159] by Boiler[127].)

(Courtesy of Advisory Board on Quartermaster Research and Development, NAS-NRC)

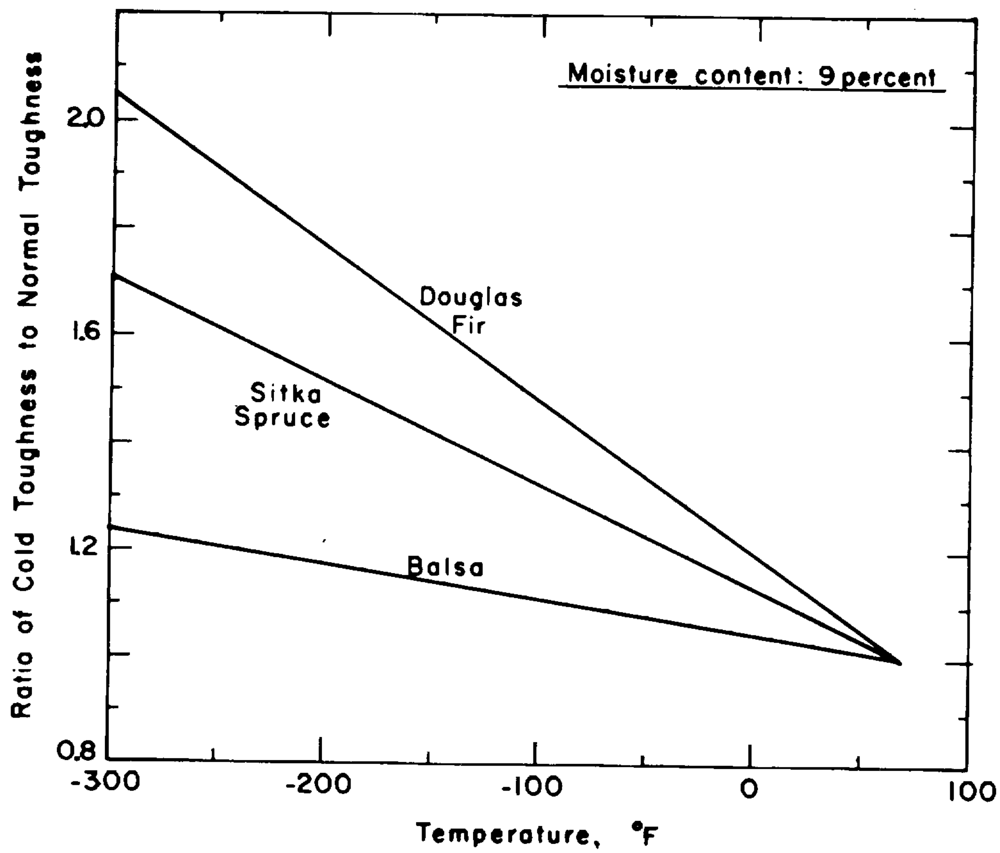


Figure 2-20. Effect of temperature on strength of wood. (Adapted from Kollman[159] by Boiler[127].)

(Courtesy of Advisory Board on Quartermaster Research and Development, NAS -NRC)

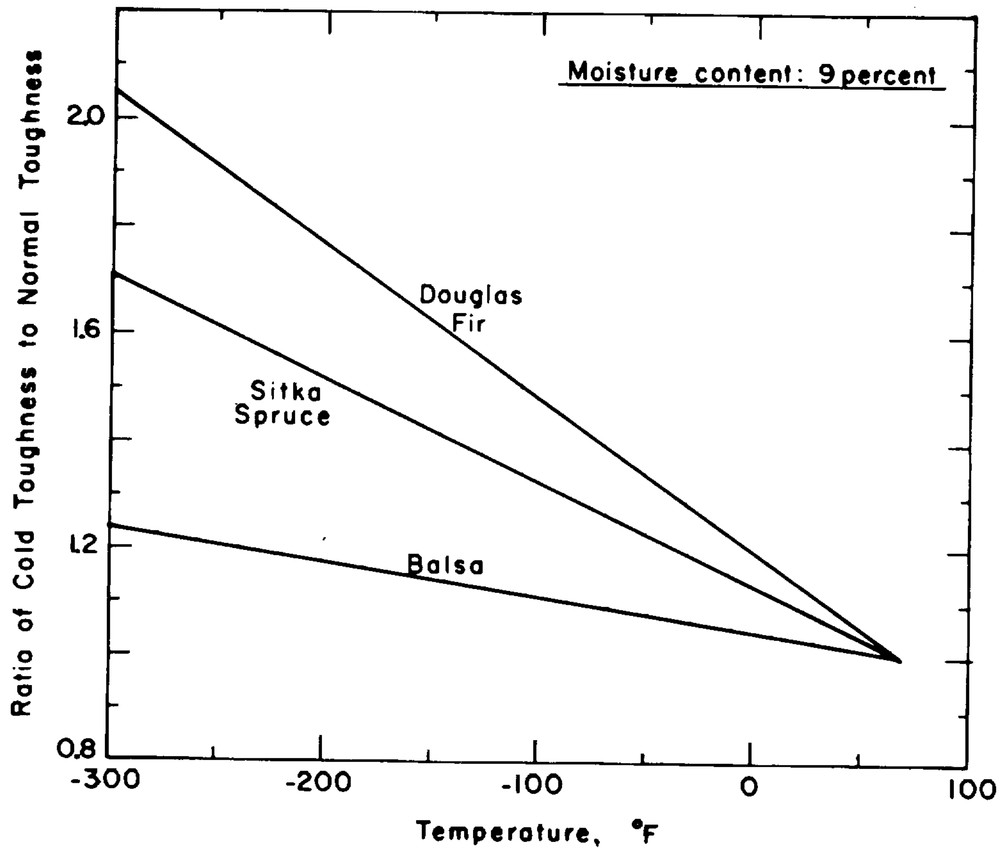


Figure 2-21. Toughness shown in flexure test[127]. 'est

(Courtesy of Advisory Board on Quartermaster Research and Development, NAS-NRC)

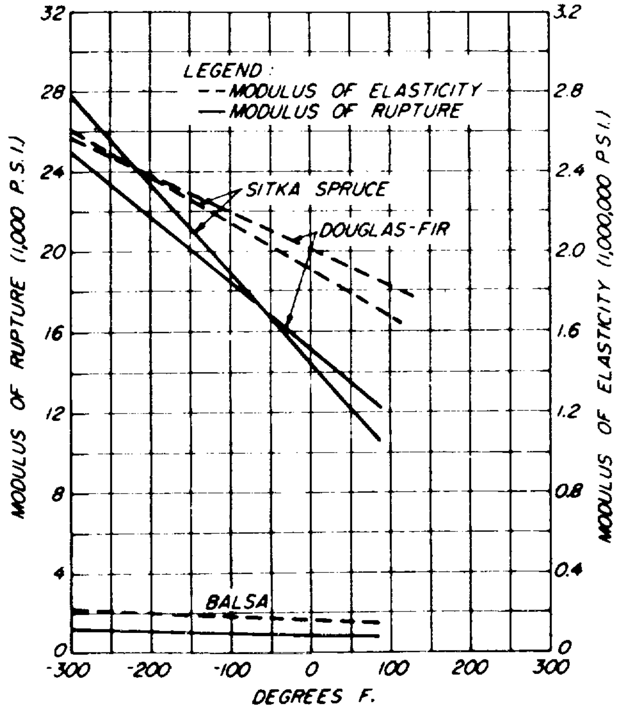


Figure 2-22a. Effect on Flexure Properties and of Temperature and Moisture Content on Modulus of Elasticity of Wood (Effect of Temperature on Flexure Properties, Moisture Content = 9 percent for Three Species of Wood)

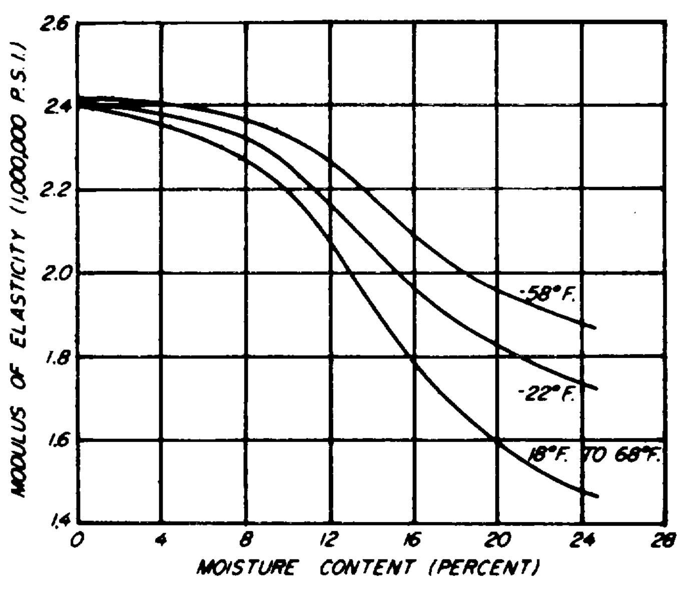


Figure 2-22b. Effect of Temperature on Flexure Properties and of Temperature and Moisture Content on Modulus of Elasticity of Wood (Effect of Temperature and Moisture Content on Modulus of Elasticity of Plywood Specimens - from Bending Tests. (Adapted from Kollman[159] by Boller[127].)

(Courtesy of Advisory Board on Quartermaster Research and Development, NAS-NRC)

of the permafrost table which will exist after construction[23]. Protective coatings which are brittle or which have linear shrinkage coefficients widely different from steel should be avoided.

c. Concrete and masonry. Portland cement concrete is an extremely useful material in the cold regions. Gravel and sand deposits and/or rock exposures are usually readily available (though this is not necessarily so). With proper processing of these deposits and with suitable water available, only the cement and air entraining component has to be imported. Good concrete is durable:' and presents no fire hazard. Because concrete needs to be warm during mixing, placement and curing and because it generates heat internally for a considerable time, difficulties arise when concrete is to be placed on or near Permafrost (See TM 5-852-6[14] for computations on heat of hydration and its effects.)

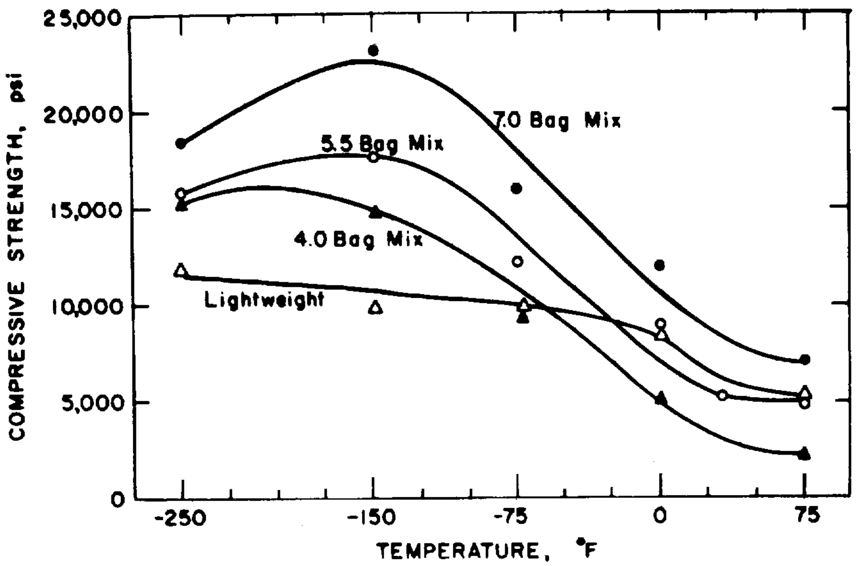
(1) Concrete which is exposed to freezing at very early periods may be damaged sufficiently to seriously lower the strength and durability which otherwise will be attainable. It is necessary, therefore, in all cases to carefully protect concrete against freezing for the first 48 hours and for such additional time as may be needed to meet minimum strength and curing criteria (para 6-4). Beyond this critical point, concrete hardens very slowly at low temperatures, and below freezing there is almost no increase in strength or hardness. Concrete which has been kept at a low temperature for a period may resume hydration and strength gain at an increased rate when favorable conditions are provided. If subjected to large drops in temperature at early periods cracking may occur, particularly if some degree of restraint exists against shrinkage. However, concrete also shrinks and cracks if cured quickly by too much heat. Some research has been done to develop concretes which will set and gain strength at below freezing temperatures". However, these depend on use of salt admixtures. They have not yet been tried in actual construction and the possible effects of the salts on long range strength and durability are unknown. They should be used with caution in reinforced or pre-stressed concrete because of corrosion hazard and never in the presence of zinc or aluminum.

(2) Freezing and thawing cycles in completely cured good quality air-entrained concrete are not generally harmful but if the concrete is below standard or if especially adverse factors exist at the time of the freezing thaw cycles the effects may be serious. Concrete which will be exposed to frost action should have durable aggregates, 4 to 7 percent entrained air depending on aggregate gradation, proper consistency for good placement without segregation, adequate curing, and best possible drainage afterwards. Concrete which is saturated prior to freezing tends to be more susceptible to freeze-thaw damage. For large or critical exposed structures, investigation and testing of available aggregates including freeze-thaw testing, petrographic analysis, and detailed mix-design studies are justified. Frost action is less detrimental in areas which are so cold that materials remain frozen throughout the winter than in warmer areas where frequent freeze-thaw cycles occur during the winter months.

(3) Tests performed by Monfore and Lentz[173] to determine the suitability of concrete for use in underground storage structures for liquefied natural gas ascertained the characteristics of concrete in very cold temperatures down to -250 °F (-157 °C). Sand and gravel mixes with three different cement contents and one using expanded shale for aggregate were used. All samples employed air entrainment in amounts varying from 5.3 to 7.8 percent. Test temperatures ranged from + 75 to -250 °F (+ 24 to -157 °C). Proceeding down through the temperature range, Young's modulus increased approximately 50 percent for the saturated samples, increased only 8 percent for those stored at 50 percent relative humidity, and remained the same for the oven-dried. Poisson's ration remained essentially unchanged throughout the temperature range at approximately 0.22. Over the 325 °F (181 °C) range, the contraction of the samples was in the neighborhood of 1.4×10^{-3} in./in. Compression tests showed an increase in strength with an increase in the cement factor. Strength curves for various cement contents exhibited similar trends with temperature (fig. 2-23a). The compressive strength of the samples with 5.5 bags/yd' cement content showed little change from + 75 to + 40 °F (+24 to + 4 °C) at about 5000 psi. At +40°F (+4°C) an upward trend started, reaching a maximum of approximately 18,000 psi at -150°F (-101 °C). This overall increase in strength is due to moisture within the samples (fig. 2-23b). Tests on oven-dry samples showed an increase in strength of 20 percent from +75 to -150°F (-101 °C), 50 percent moist samples gained only slightly more, while 100 percent moist samples increased by 240 percent.

(4) Precast and prestressed reinforced concrete may be used for foundations as well as structures in arctic regions but special care must be taken where frost heave, settlement or freeze-thaw under moist conditions may occur. Heave may, for example, produce tensile stresses and cracking in reinforced concrete bearing piles. While precast reinforced concrete sectional buildings have been used successfully',', exposed joints in such construction may be very carefully sealed and drainage must be provided where joints can be damaged by freezing of accumulated water within the joints.

(5) Unless favorable foundation conditions exist or can be provided, concrete block or brick masonry construction should be avoided because of its poor ability to tolerate differential movements. When masonry is used for interior work, environmental protection must be



(Courtesy of Portland Cement Association)

Figure 2-23a. Effect of Concrete Mix and Moisture Conditions on Strength of Concrete at Low Temperatures[173]. (Effect of Concrete Mix (Moist Concrete).

(Courtesy of Portland Cement Association)

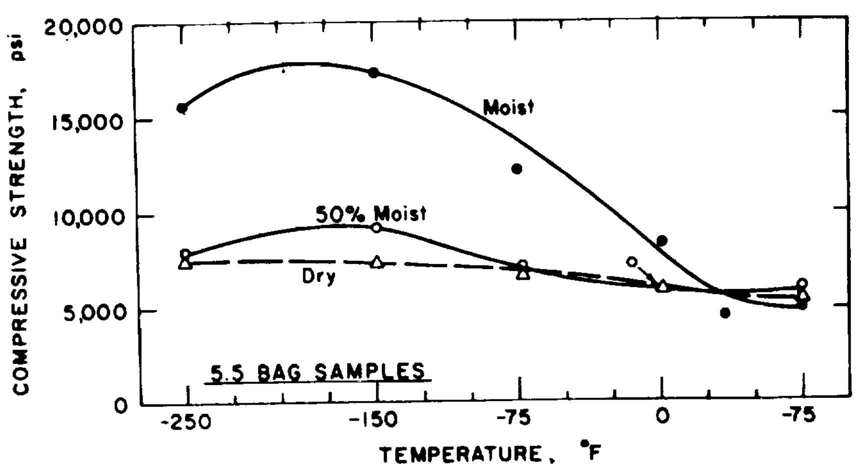


Figure 2-23b. Effect of Concrete Mix and Moisture Conditions on Strength of Concrete at Low Temperatures[173]. (Effect of Moisture Conditions.)

provided during the construction period. For exterior applications, the additional possibility of freeze-thaw damage during the life of the structure must be anticipated. Bricks which will be in contact with frozen soil should be of SW (ASTM) grade or equivalent. The mortar must be durable and moisture resistant.

d. Thermal insulating materials. Thermal conductivity values of construction materials are usually given for the dry condition. However, many of these materials lose much of their insulation value if they become wet. Many insulating materials will absorb significant amounts of water (fig. 2-24 and 2-25)[70,71,139] and care should be exercised to select insulation material for underground use which will absorb minimum moisture and to allow in the design for the degree of insulation impairment from moisture which is expected over the life of the facility. Cellular insulations exhibit extremely varied performance and must be examined closely before acceptance for specific installation usages; details of manufacture may significantly affect moisture absorption rates. Cellular glass has performed fairly well, though not perfectly, in maintaining its low conductivity in wet ground[42,80]. Both laboratory and field experiments show that cellular glass experiences slow but progressive deterioration under wet freeze-thaw conditions. Table 2-3 shows field test data. Figure 2-25d illustrates the effect of freeze-thaw on moisture absorption by various board-type insulations, but since exposure to freeze-thaw in presence of water causes some cell damage in cellular insulations, thermal conductivity tests, after freeze-thaw cycles, are considered a better determination of performance than total amount of moisture absorption after freeze-thaw, especially in those which absorb a great deal of moisture. Vermiculite concrete or other forms of lightweight concrete which will gradually absorb substantial amounts of moisture when placed underground are unsuitable as insulating material in below-surface construction. To prolong effectiveness of insulation underground, it should be placed in positions offering minimum exposure to moisture and to moist freeze-thaw conditions. Thus, from the point of view of maintaining insulation effectiveness, incorporating insulation within a concrete foundation slab is preferable to placing the insulation in the ground below the slab. In usages where complete moisture protection can be readily provided, such as under the floor of a structure, any commercial insulation or insulating composition suitable for general building usage may be employed provided it meets other applicable requirements such as bearing capacity or compression criteria. Thermal properties of a number of materials are given in TM 5-852-6/AFM 88-19, Chapter 6[14]. Insulation only slows down rate of heat conduction; it cannot prevent heat flow. When thick gravel pads contain some moisture, they provide nonfrost-susceptible thermal buffers or heat sinks in which freezing and thawing are absorbed with minimum detrimental effects; if freeze and thaw should penetrate somewhat into the soil below such a gravel pad, the gravel layer helps to minimize frost heave and to smooth out any differential heave or settlement which may occur.

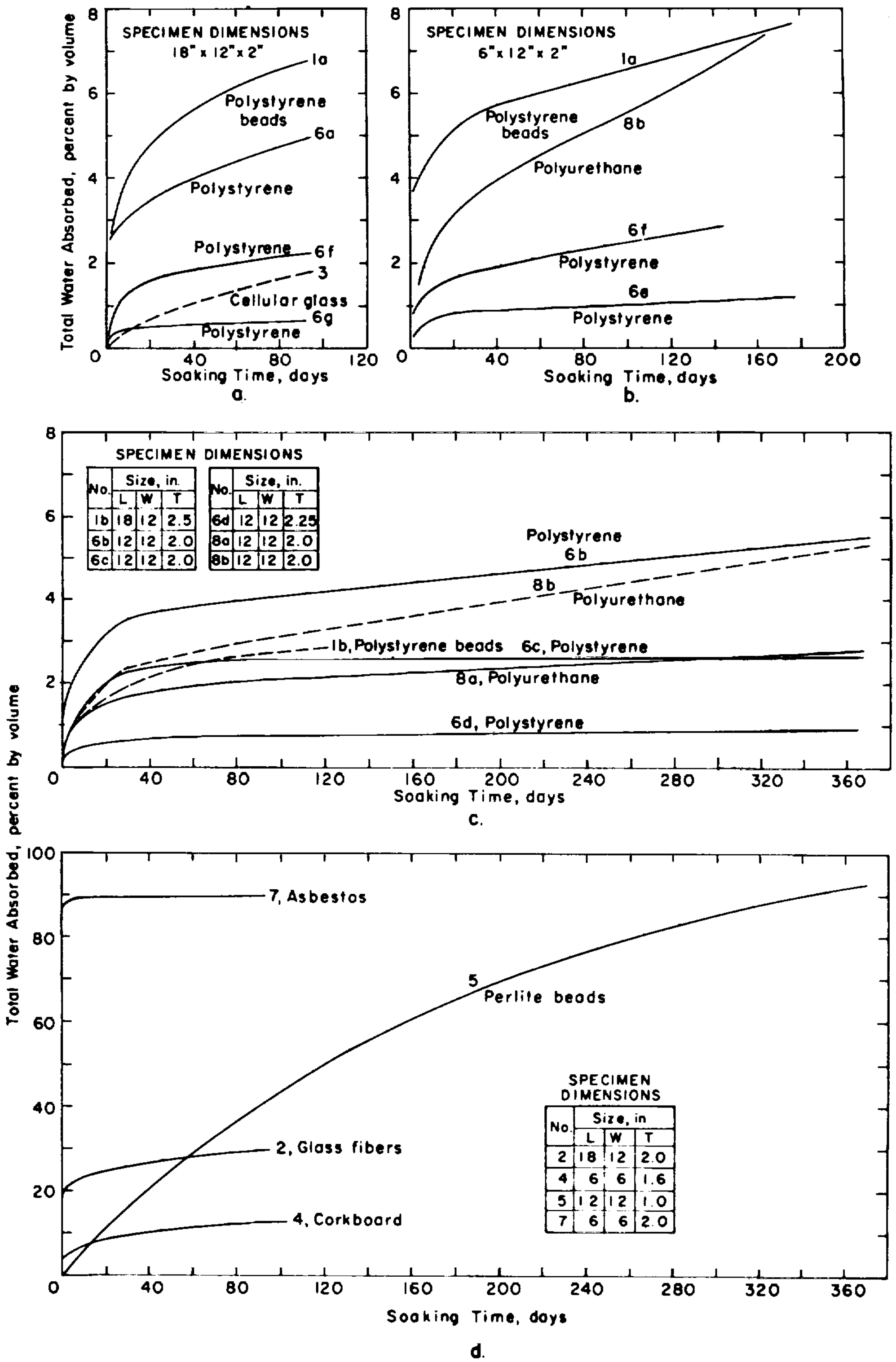


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Figure 2-24. Moisture absorption of insulation board by soaking in water[93]. See figure 2-25 for index of insulating materials.

No.	Board Composition	Approx. Density ρ pcf
1a	Fused expanded polystyrene beads	0.9
1b	" " " "	1.6
2	Glass fibers, sandwich	9.5
3	Cellular glass	9.2
4	Corkboard	15.0
5	Perlite beads with organic fiber	10.7
6a	Polystyrene, extruded	1.7
6b	" "	1.9
6c	" "	2.0
6d	" "	2.5
6e	" "	2.5
6f	" "	3.1
6g	" "	3.6
7	Asbestos with binder	14.8
8a	Polyurethane	1.9
8b	"	2.2

All insulations were plant manufactured

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Figure 2-25a. Internal moisture distribution in insulation board under different rest conditions. (Index for insulating materials)

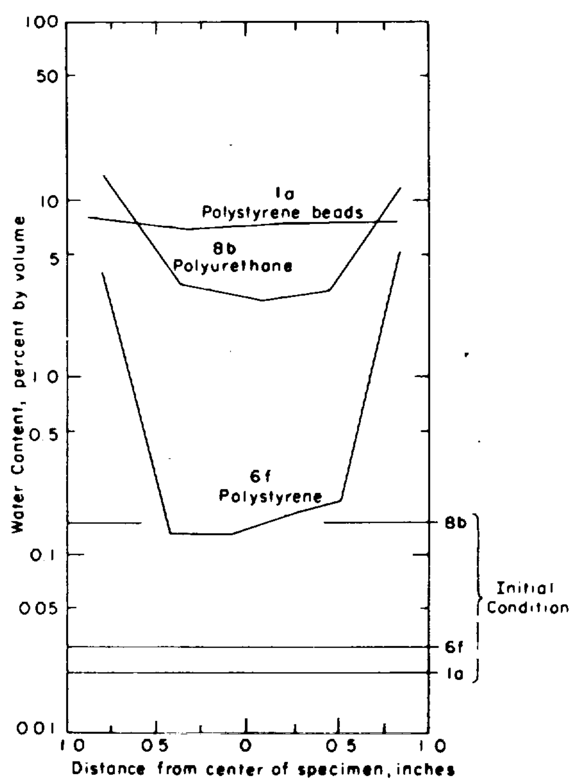

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Figure 2-25b. Internal moisture distribution in insulation board under different test conditions. (After 18 months in water)

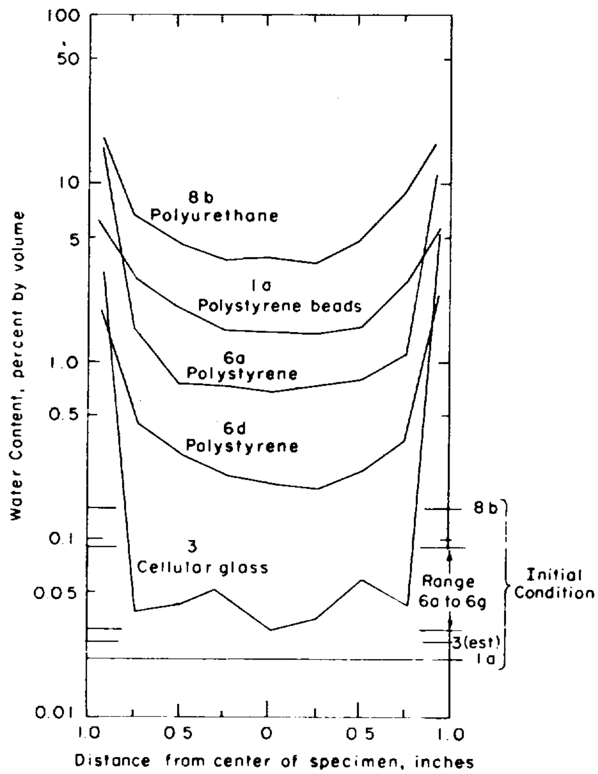

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Figure 2-25c. Internal moisture distribution in insulation board under different test conditions. (After 34 months embedment in moist silt)

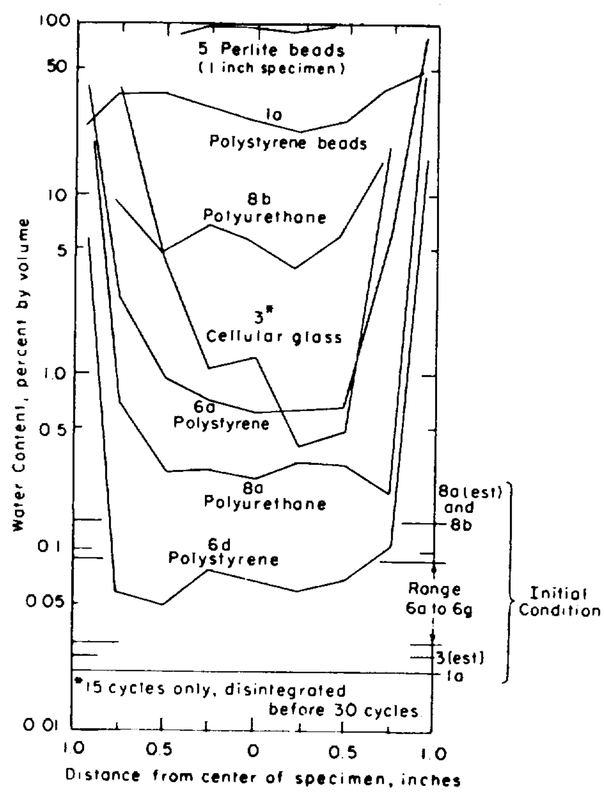

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Figure 2-25d. Internal moisture distribution in insulation board under different test conditions. (After 30 freeze-thaw cycles (over approximately a 30-day period) with speciment immersed in water)

Table 2-3. Moisture Distribution in Cellular Glass after 20 Years Burial in the Annual Frost Zone, Fairbanks, Alaska"

Test Area	0 - 1" %	1 - 2" %	2 - 3" %	3 - 4" %	4 - 5" %	5 - 6" %
RN-5	117.0	0.7	1.4			
	85.0	0.1	5.8			
RN-6	115.5	2.1	2.1	103.2	104.3	1.8
	76.5	4.4	5.3	26.0	0.3	0.6
RN-11	55.5	1.4	12.4	35.5	1.8	1.3
	64.7	2.2	5.3	5.0	1.5	4.4

Notes on Tests Performed at 20 Years;

1. Each test section was 50 ft square, asphalt-surfaced with the insulating layer in the middle of a 4 to 4.5-ft-thick gravel layer on a silt subgrade. The insulation in RN-5 consisted of one 3-in. layer of 12-in. by 15-in. blocks, coated with tar top and bottom, and with joints sealed by tar. The insulation in RN-6 and RN-11 was made up of two 3-in. layers. of 12-in. by 15-in. blocks, coated with tar between layers, and with Joints staggered and sealed by tar.

2. Two samples of insulation from each test section were tested for moisture content as percent of dry weight. Data are shown for each of the two samples.

3. Before moisture measurement, sand and tar were trimmed from the samples. Samples were sliced into approximately 1-in.-thick layers. 0-1 in. is the first 1-in. horizontal layer of cellular glass from the top; 1 to 2 in. the second and so on.

4. Large discrepancies are noted between values for the top inch of the bottom (3 - 4 in.) layer of cellular glass in RN-6 and RN-11 and for the 4 to 5 in. layer in RN-6. This was possibly caused by water passing through the joints between blocks and lying between layers where the tar bond is broken.

5. It was noted that cell walls had deteriorated in the upper 1/4 in. to 1/2 in. of the blocks to the extent that the cell walls could be easily broken by finger pressure.