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FROST ACTION IN SOIL FOUNDATIONS AND CONTROL OF SURFACE STRUCTURE HEAVING

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16. ABSTRACT

This is a report on frost action in soil foundations that may influence the performance of irrigation structures. The report provides background information and serves as a general guide for design, construction, and operation and maintenance. It also includes information on the mechanics of frost action, field and laboratory investigations of potential frost problems, case histories of frost damage to hydraulic structures, and measures to control detrimental freezing to avoid damage. The structures mentioned include earth embankment dams with appurtenant structures, canals with linings, and various other concrete canal structures.

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by

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UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. Administration.

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INTRODUCTION

In 1973, a Bureau Ice Management Committee was formed to investigate the problems of ice on the operation and maintenance of hydraulic structures and to propose solutions to these problems. One problem area was the formation of ice lenses in soil foundations, which causes frost heave and damage to overlying structures. The committee decided that this problem was of sufficient importance that a special team should be formed to study the problem. The Frost Action Team was then formed. The principal members of the team, who compiled this report, were Chester W. Jones (Chairman) and David G. Miedema, Geotechnical Branch, Division of Research; and John S. Watkins, Water Conveyance Branch, Division of Design. Other short-term members were Jerry R. Dexter, Robert B. Hayes, and Wolfgang P. Gersch.

The objective of the study team was to compile a report describing: (1) nature and extent of damage to Bureau structures from frost heave, (2) conditions conducive to frost heave with the general principles involved, and (3) procedures for controlling frost heave. In addition to conventional measures, new methods not previously used by the Bureau are included in this report. Although there are various other types of freezing problems on structures such as deterioration of concrete where water infiltrates and freezes, the scope of this report is limited to frost action in soil foundations and structures. This report should serve as a convenient reference to Bureau designers and operations and maintenance personnel.

It is important to distinguish between damage from frost action and damage from other causes. For example, some clays and shales are expansive and, when subject to a change in moisture content, will change in volume and cause damage similar to that from frost action. Also, in thin structures such as concrete canal linings, a back pressure from a ground-water level higher than the water level in the canal can cause cracking and differential movement of portions of the lining which might, from a cursory examination, be attributed to the effects of frost heave.

A review of existing literature shows that most of the previous investigations on frost action have been in connection with pavements on highways and airfields, and with buildings. For these features, various design methods, such as those developed by the Corps of Engineers for airfields, have been devised. There appears to be little published information pertaining directly to hydraulic structures. Although the principles of frost action are universal, the structural and moisture conditions of hydraulic structures are somewhat different than those for paved areas and building foundations.

SUMMARY

A Bureau-wide survey showed that seasonal frost action in soil foundations was a consideration in all regions except the Mid-Pacific, Lower Colorado, and Southwest Regions, and was of most concern in the Upper Missouri Region.

Significant frost heave in soils occurs in cold climates when certain fine-grained soils have access to sources of water and an open-freezing system exists. As frost penetrates the ground during the winter, ice lenses are formed with resulting high pressures that cause overlying structures to heave or walls to tip. Under heterogeneous soil and moisture conditions, damage to structures is caused by differential movement during periods of freezing and thawing. In compacted soils where there is no source of water except for in the soil voids, closed-system freezing occurs, usually without significant heaving.

Investigations for potential frost problems should include: (1) appraisal of the climatic conditions at the proposed structure site based on past weather records, (2) surveys on the performance of existing structures in the area of concern, (3) foundation exploration and soil sampling, (4) laboratory testing on physical properties, (5) laboratory freezing tests on soil specimens where frost action is questionable and the importance of the proposed structure warrants it, and (6) location of ground-water depth, not only at the time of exploration but also after the structure has been placed in operation.

The conventional methods of controlling frost action to prevent damage to structures are: (1) location of the foundation below the frost penetration level, (2) replacement of frost-susceptible soil throughout the expected depth of frost penetration with soil that is not affected by frost, (3) restriction of the supply of water to the frost zone so that ice lenses cannot form (most commonly done by drainage), and (4) the use of

insulation to reduce the depth of frost penetration. The selection of the methods to control frost action depends on previous experience and economics based on the use, value, and expected life of the structure or project.

Bureau hydraulic structures which have suffered the most damage from frost action include: (1) spillways, particularly walls which have become displaced inward; (2) chutes and drop structures which heave, crack, and deteriorate; and (3) checks, pipe turnouts, pier-supported pipe, and canal linings.

Frost action is also a consideration in soil canal linings and in earth embankment dams constructed in cold climates. After a winter construction shutdown period, particular attention should be given to density testing in soil that has been frozen and to recompaction to specified limits where necessary.

A promising new technique to control frost heave is the use of polystyrene insulation, which was originally developed for highway applications. Field experiments with this insulation were conducted on the Riverton Unit, Pick-Sloan Missouri River Basin Program; and has also been used by the Bureau for small installations where special frost protection was required.

REGIONAL SURVEY OF SOIL FROST PROBLEMS

In 1973–74, surveys were made at all the Bureau's seven regions to determine the extent of frost damage to water conveyance, power generation, transmission, and other structures. The following subsections give a summary of the comments from these surveys. Details on specific major damages are presented in appendix A.

Pacific Northwest

Although this region is in a cold weather area, structures are designed accordingly and damage from frost is relatively minor. Freezing and thawing action has occasionally damaged or moved some of the smaller structures; however, this has generally been caused by the development of drainage problems not anticipated at the time of design. Suggestions for controlling frost action are the standard methods for placing footings below the frostline, replacement of frost susceptible foundation materials, and proper surface and subsurface drainage.

During investigations of the Teton Dam failure, which was subsequent to the regional frost survey, questions were raised as to the possibility of freezing action in the embankment contributing to the failure of the dam. These questions have not been completely resolved.

Mid-Pacific

Only minor damage from frost action reported.

Lower Colorado

No damage from frost action reported.

Upper Colorado

Major failures occurred on the spillways of Lemon and Vallecito Dams, and damage to concrete linings occurred on Steinaker Canal. Some heaving of roadways, foundations for mobile homes, and asphalt patios occurred at the Duchesne Government Camp.

Southwest

No significant damage from frost action reported.

Upper Missouri

This region reported the most frost action with their estimate that from 500 to 1000 hydraulic structures had suffered some degree of damage from frost action in a dozen separate areas.

Lower Missouri

Some damage has occurred to concrete canal linings in this region. Some slabs have heaved with the main damage being horizontal cracking along the bottom third of the side slope slabs. Repairs have been made on the Charles Hansen Feeder, Sherman Feeder, and Ainsworth Canals.

MECHANICS OF FROST ACTION IN SOILS

There are two important aspects of frost action in soils: (1) the frost *heave* of certain soils in contact with water and subjected to freezing temperatures, and (2) the *loss of strength* of the soil upon thawing.

Frost heave requires a frost-susceptible soil, subfreezing temperatures, and a supply of water for the growth of ice lenses. Under favorable conditions, a frost heave of 305 mm (12 in) or more can occur in northern parts of the United States, and pressures up to 931 kPa (135 lb/in^2) have been measured in the laboratory. The amount of heave is rarely uniform over a significant area and heaving pressures may damage hydraulic structures, roads, bridges, retaining walls, pipes, buildings, and other structures. Frost action effects are illustrated on figure 1.

In spring, thawing of ice lenses near the ground surface causes the ground to become soft and saturated with a resulting decrease in the strength and rigidity of the soil. This condition will exist until the underlying ground completely thaws and the excess water can drain away. Thawing beneath structural elements can result in structural damage and even failure because the thawing soil may have little ability to support heavy loads, and differential settlement may occur.

Frost Heave

Near the end of the last century, it was thought that frost heave was solely due to the expansion of water on freezing of the soil. However, two things were observed that could not be explained by this theory: (1) the large amount of heave, and (2) the considerable increase in water content of the soil in the frozen zone. It is now known that the freezing of water is accompanied by a volume increase of about 9 percent. Therefore, in a saturated soil that becomes frozen, the void volumes will increase about 9 percent, resulting in an overall increase from 2.5 to 5 percent, depending on the void ratio.

A much greater increase in volume can occur because of the formation of ice lenses, sometimes causing a soil column to grow several times its original height. Ice lens formation is illustrated on figure 2(a), where water rises from the ground-water table toward the freezing front. As ice lenses form, the ground surface heaves. The amount of heave is about equal to the total thickness of the ice lenses. Figure 2(b) shows the water content prior to and after freezing. As one would expect with the formation and growth of ice lenses, the water content increases above the frostline. Perhaps not expected, is the decrease in water content just below the frostline and the consolidation of that part of the soil.

The necessary conditions for frost heave (availability of water, frost-susceptible soil, and freezing temperatures) must all be present for ice lens formation and growth; each condition is interrelated. For example, water movement is influenced by the permeability of the soil and changes in temperature. For the sake of clarity, water, soil, and temperature will be considered separately.

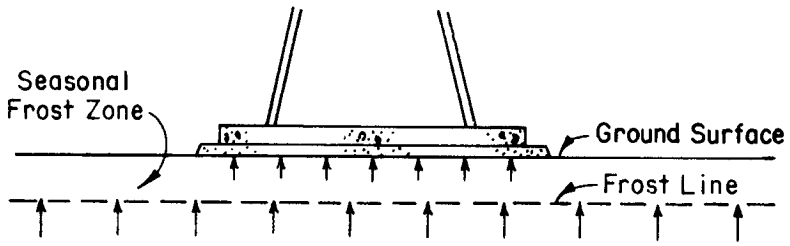
Water

There must be a supply of water for the growth of ice lenses. The ground-water table is generally the source of this water although it may be provided by surface infiltration, seepage from canals, broken waterlines, or other means. The ground-water table needs to be close to the ground surface—within the height of capillary rise, which is usually about 1.5 to 3 m (5 to 10 ft), depending on soil type, gradation, and density. The smaller the sizes of the interconnected pores between the solid particles, the higher the water can rise above the water table. The following tabulation shows the approximate heights of capillary rise for various types of soil [1][1]:

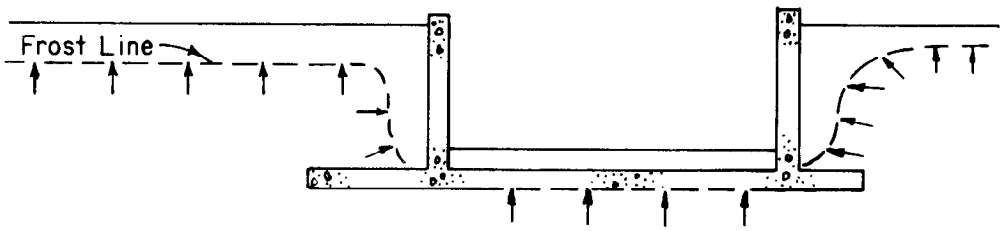
Soil type	Approximate height of capillary rise, m	(ft)
Coarse grained	1.5	(5)
Silt		
Varied silt and clay		
Silty, very fine sand	3	(10)
Saturated, compressible clay	>3	(>10)

Figure 3 illustrates two possible freezing conditions that might be observed in two different soil columns, A and B. Column A is where there is no source of water available during the freezing process beyond that originally in the voids of the soil at and near the zone of freezing. This type of freezing is known as closed-system freezing and may occur where there is a low water table below the height of capillary rise, or where the upward movement of water is obstructed by an impermeable member or by a layer of clean, coarse-grained soil which breaks the capillary rise. This type of freezing occurs in compacted

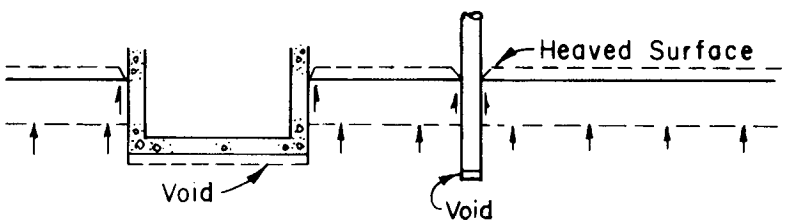
[1]Numbers in brackets refer to entries in the Bibliography.



(a.) Heaving of soil in seasonal frost zone causing direct upward thrust on overlying structural elements.



(b.) Freezing of frost-susceptible soil behind walls causing thrust perpendicular to freezing front.



(c.) Force at base of freezing interface tends to lift entire frozen slab by applying jacking forces to lateral surfaces of embedded structures and creating voids underneath. Structures may not return to original positions on thawing.

Figure 1.—Frost action effects.

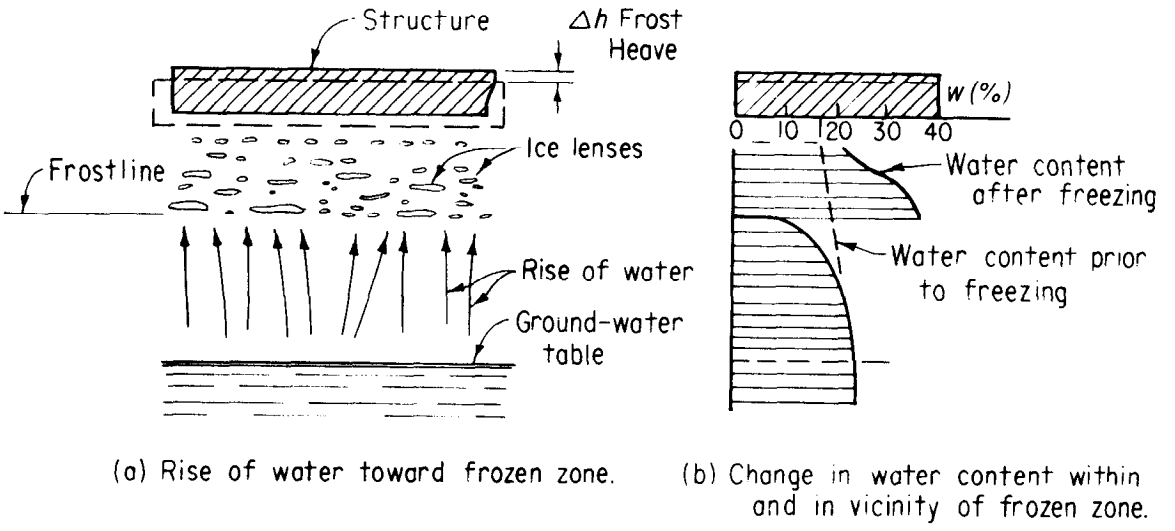


Figure 2.—Formation of frost heave.

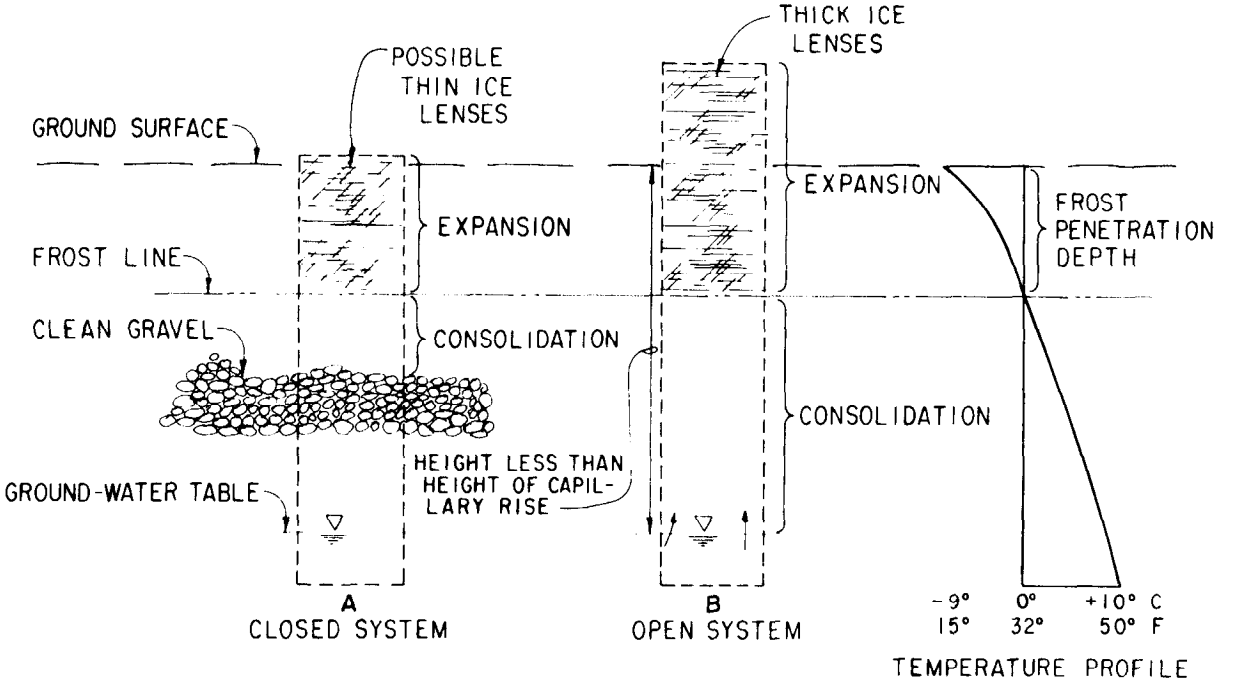


Figure 3.—Open- and closed-system freezing.

soil linings when water is withdrawn from a canal or reservoir and in uncompleted earth dams during winter shutdown periods. A water and density redistribution occurs which may have important effects on these structures. Depending on the soil moisture and density conditions, thin ice lenses may or may not form. During such freezing, the ground surface may heave a small amount (soil volume increase up to 5 percent) or the soil volume decreases causing a small amount of settlement and possible crack formation. The subject of closed-system freezing discussing both the positive and negative effects on soil linings and earth dams is covered in a separate report [2].

In soil column B, water is drawn toward the freezing front with the possible initial growth of ice lenses as in column A, except that additional water is drawn in from the available ground water. Within the freezing zone, ice lenses form more or less perpendicularly to the direction of heat flow. This type of freezing is known as open-system freezing.

What causes water to move into the zone of freezing and form ice lenses during frost heave? In general, movement occurs because equilibrium has been disturbed by a drop in the ground surface temperature and a thermal potential has been set up. Energy is available, due to heat transfer, to draw free water to the ice lenses. This has been explained in terms of capillary flow, suction force, thermodynamics, and vapor transfer of moisture in references [3,4]. Sowers [5] presents the following explanation:

> "Above the frostline, the temperature is below the ordinary freezing point for water. However, in very small openings such as the voids of fine-grained soils, the freezing point may be depressed as low as −5 °C (23 °F). Thus, just above the frostline, water will freeze in the larger voids but remain liquid in the adjacent smaller ones. When water freezes in a larger void, the amount of liquid water at that point is decreased. The moisture deficiency and the lower temperature in the freezing zone increase the capillary tension and induce a flow toward the newly formed ice crystal. The adjacent small voids are still unfrozen and act as conduits to deliver the water to the ice. The ice crystal grows until an ice lens or layer forms. The capillary tension induced by the freezing and the low temperature sucks up water from the water table below or can even dehydrate and shrink adjacent compressible strata such as clays and micaceous silts when the water table is beyond reach. The result is a great increase in the amount of water in the frost zone, and segregation of the water into ice lenses."

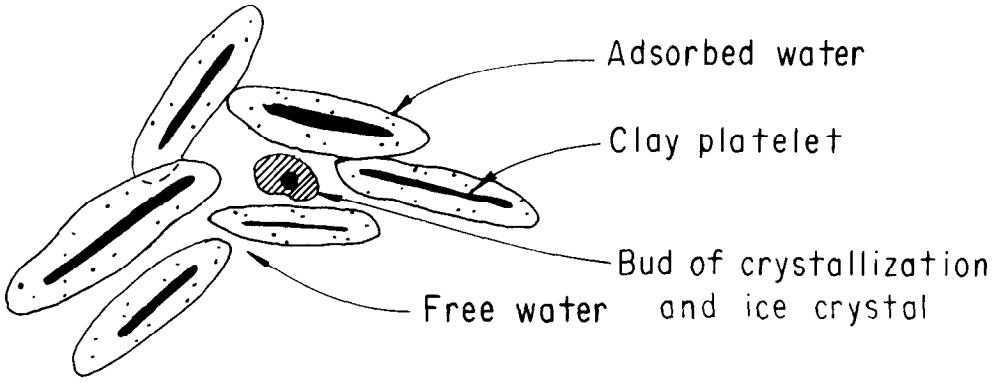
Water in the voids of fine-grained soils exists as free and adsorbed water. Clay consists of mineral particles that are generally plate-shaped and have large surface areas. Surface forces attract and hold a thin layer of adsorbed water to each soil particle. Because of surface forces and dissolved ions, the adsorbed water film remains unfrozen until the temperature drops well below 0 °C (32 °F). For example, clay may have an unfrozen moisture content of 35 percent at 0 °C and 15 percent at −10 °C (14 °F) with the largest proportion of the decrease occurring between 0 °C and −5 °C (23 °F), and no significant unfrozen moisture below −20 °C (−4 °F) [6]. Other soils with less clay would have a smaller unfrozen moisture content at 0 °C and less decrease at lower temperatures. For example, a clayey silt might have an unfrozen moisture content of 12 percent at 0 °C and 7 percent at −5 °C with no significant decrease below the −5 °C temperature. A sand might have a small fraction of 1 percent unfrozen moisture at 0 °C and this would not change significantly as the temperature decreased to −5 °C.

Three stages in the growth of an ice lens in a fine-grained soil are illustrated on figure 4. These stages are:

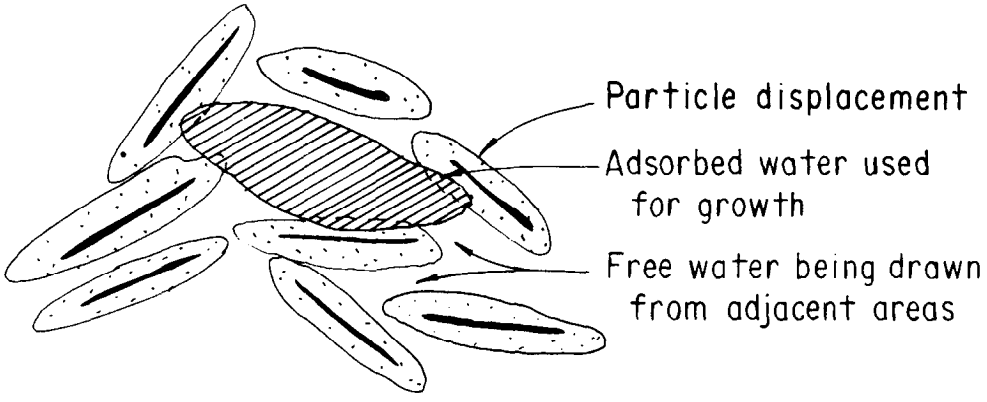
(1) An ice crystal bud is formed and grows, supplied by free water within the immediate pore space.

(2) Free water is drawn from adjacent pores, along with some adsorbed water, as the temperature decreases and the ice crystal continues to grow. Particle displacement occurs as a result of both water loss and crystal growth.

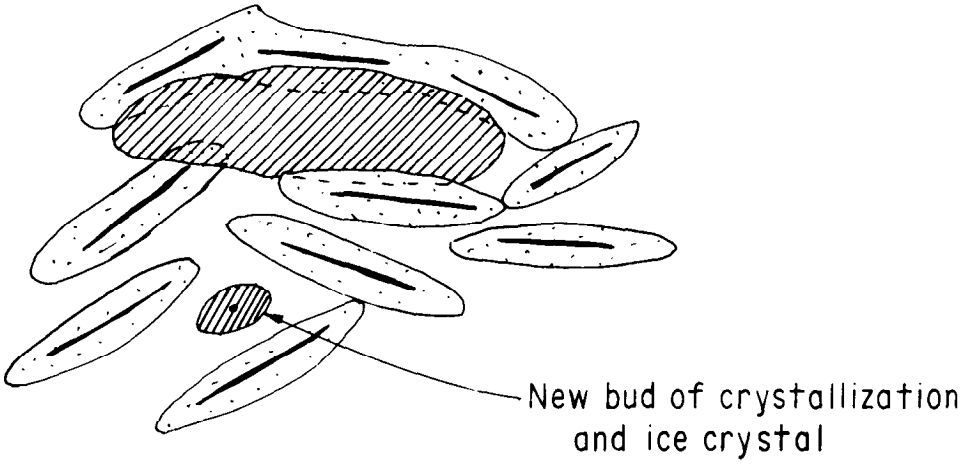
(3) A point is reached when the energy required to bring in water to sustain growth becomes too large and growth stops. When this happens, another bud of crystallization is formed further ahead of the frostline. This process is repeated as long as the frostline moves downward and a supply of water is available.



STAGE 1. Free water within immediate pore supplies water for growth of ice lens.



STAGE 2. Free water from adjacent pores, and adsorbed water, aids growth of ice lens.



STAGE 3. A new ice crystal forms.

Figure 4.—Stages in the growth of ice lenses.

Soil

In addition to the availability of water, the ease that water can move through a soil (permeability) is important. For ice lenses to grow and frost heave to occur, water must move from the ground-water table or other source and pass through the soil pores to reach the freezing front. Permeability of a soil is markedly affected by the amount of fine-grained particles in the silt- and clay-size range. Particles passing the 75-μm (No. 200) sieve are commonly called fines. Fine-grained soils, such as silts and clays, contain more than 50 percent fines by mass. The fines in a soil exert an influence that is out of all proportion to their mass.

In clays without cracks, water migration is restricted and consequently, ice lens development is slow. In a coarse-grained soil with little or no fines, pore spaces are large and permeability is high. Water within the pores is almost all free water, with little adsorbed water, and ice can extend from one void to the next (fig. 5). Consequently, the soil freezes homogeneously with depth. The only volume increase is due to the 9 percent increase in the volume of water upon freezing.

Soils having a high percentage of silt-sized particles are the most frost susceptible. Such soils have a network of small pores that promote migration of water to the freezing front. Silts (ML, MH)[2], silty sands (SM), and clays of low plasticity (CL, CL-ML) fall into this category.

Unfortunately, pore size cannot be measured directly or easily. Pore size is directly related to particle size and to soil density, and to a lesser degree is a function of gradation, grain shape, mineral composition, and plasticity. Thus, grain (particle) size criteria have come into common usage in gaging frost susceptibility.

One of the earliest grain size criterion was proposed in 1932 by A. Casagrande [7]; it is still widely used today. After extensive laboratory and field studies, Casagrande stated that:

> "Under natural freezing conditions and with sufficient water supply, one should expect considerable ice segregation in nonuniform soils containing more than 3 percent of grains smaller than 0.02 mm, and in very uniform soils containing more than 10 percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1 percent of grains smaller than 0.02 mm, even if the ground-water level was as high as the frostline."

A preliminary estimate of a soil's frost susceptibility may be obtained in the field without waiting for laboratory gradation tests. For this estimate, soils are classified into one of three groups based on type of soil and the characteristics shown in the following tabulation from reference [8]:

The Unified Soil Classification System, shown in the Bureau's *Earth Manual* [9] and although not

Group	Frost susceptible	Soil	Field Identification
A	No	Clean gravel and sand	Soil, when dry, does not form into lumps
B	Maybe	Very fine sand, silty sand, and sand containing organic matter	Soil, on drying, forms weakly cemented lumps which can be held between the fingers but break under slight pressure
C	Yes	Cohesive soil and cohesive soil containing organic matter	Soil, when dry, has a slight to very high strength

[2]Soil symbols used in the Unified Soil Classification System [9].

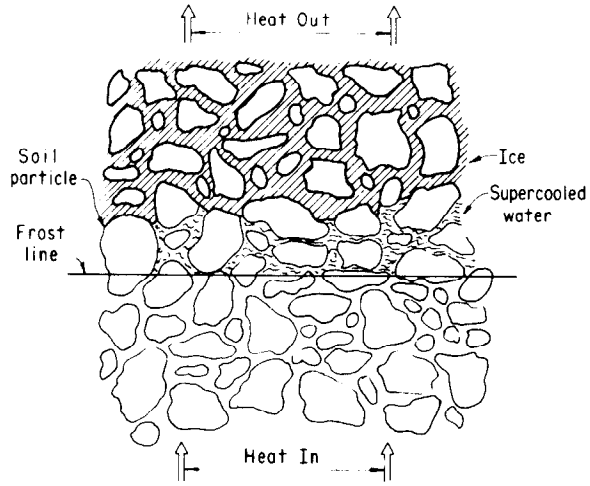


Figure 5.—Schematic view of soil freezing in coarse-grained soil.

expressly a frost-susceptibility classification, has been correlated with laboratory freezing tests by the Corps of Engineers. Tests made from 1950 to 1970 form the basis for figure 6, which relates frost susceptibility in terms of average rate of heave to percent by mass finer than 0.02 mm. This figure shows that most soil types have a wide range of frost susceptibility with no sharp dividing line between frost-susceptible and non-frost-susceptible soils. Nevertheless, silts, clayey silts, and silty sands have the highest potential for frost heave followed by the gravelly and sandy clay, clayey sand, and clayey gravel. Soils with the lowest potential to heave are sandy gravels, clean sands, and silty sands with less than 3 percent grains finer than 0.02 mm.

Temperature

Freezing temperatures must penetrate into a soil for ice lens buildup to occur. The penetration depth depends primarily on air temperature and soil thermal properties and to a lesser degree on ground surface cover, solar radiation, and exposure to wind. Frost penetration depths to 3.6 m (12 ft) are possible in the colder regions of the United States.

For a particular site in question, frost depth can be measured directly or it can be calculated approximately by various formulas. Direct measurements can be made by frost tubes, coring, test pits, and temperature sensors. Calculated depths are based on a knowledge of the air-freezing index, type of surface, and thermal properties of the soil. The air-freezing index (fig. 7), is a measure of the freezing temperature duration over 1 year. Additional information on calculating this index is given later in this report.

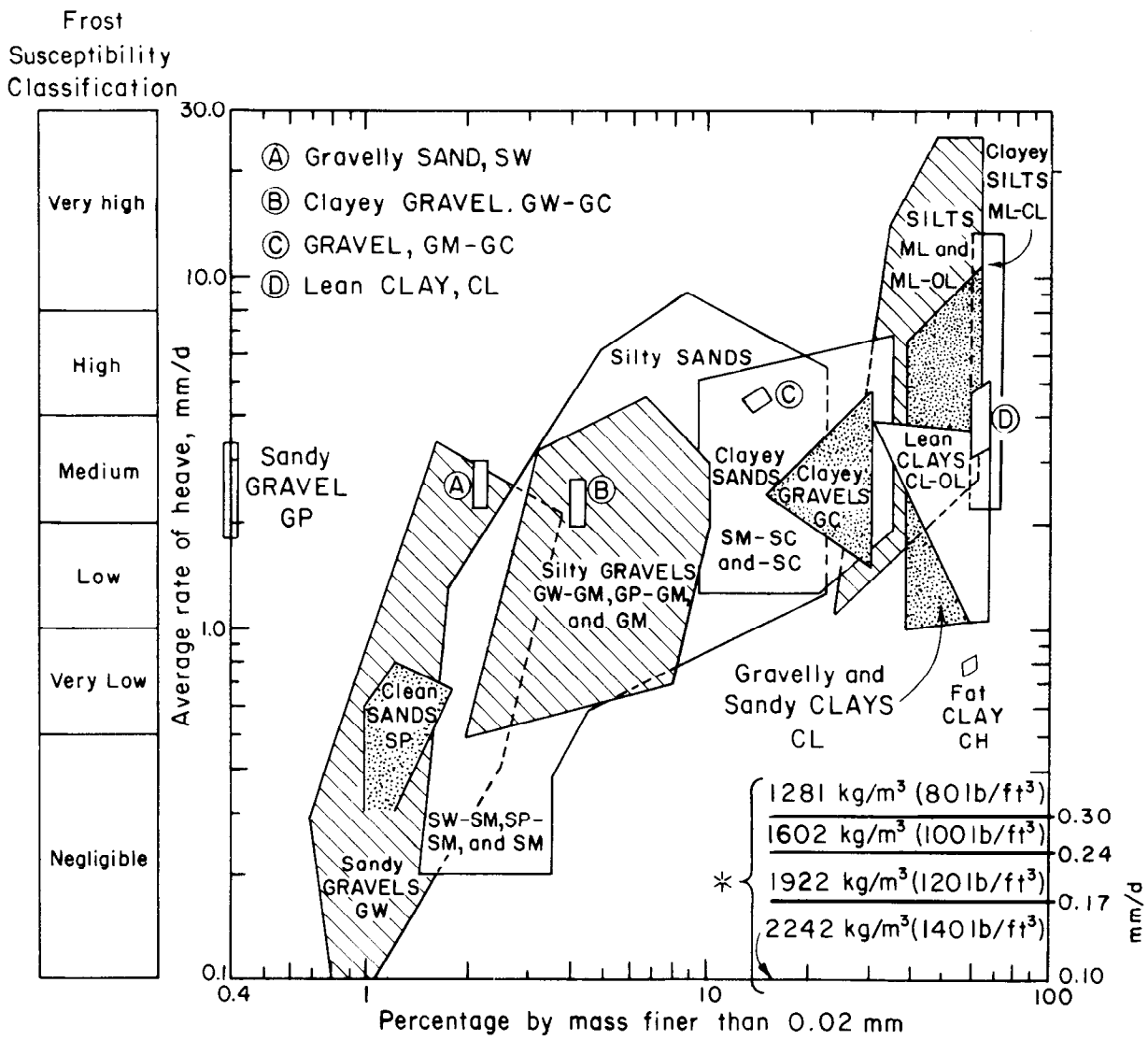
The ground temperature is modified by surface conditions. For example, snow cover may retard or even prevent frost penetration. Vegetation, turf, and organic matter also act as insulation. The effects of wind, sunlight or shade, and surface color are more difficult to evaluate.

Thermal properties of a soil are complex because soil contains mineral particles, air, and free and adsorbed water (which undergoes phase transitions in the temperature range of concern). In general, soil thermal properties are related to moisture content; the state of the soil (frozen or thawed); the size, shape, and nature of the mineral particles; and the temperature. In heat-transfer terminology, the basic thermal properties of soil are thermal conductivity, volumetric heat capacity, and volumetric latent heat of fusion.

Thermal conductivity is a measure of the rate at which heat moves through a medium under a unit thermal gradient. Thermal conductivity increases with soil moisture, with density, and when the soil is frozen. For example, undisturbed dry sand is a much better insulator than moist compacted sand. The volumetric heat capacity is a measure of the heat required to raise the temperature of a unit volume of material by 1°. The volumetric latent heat of fusion is the heat which is liberated on freezing (or is required to thaw) a unit volume of soil. These three properties are basic requirements for calculating the depth of freezing, which is covered in more detail later in this report.

Water content also plays a prominent role in thermal soil considerations. When water changes to ice, the thermal conductivity of the ice increases by a factor of four, the volumetric heat capacity of the water decreases by one-half, and the water releases a latent heat of fusion of 335 kJ/kg (80 cal/g) [11, p. 118].

The rate of cooling will also affect the amount of frost heave, assuming other factors remain constant. When the temperature is gradually lowered, frost penetration is slow and ice lenses



* Indicated heave rate due to expansion in volume if all original water in 100 percent saturated specimen was frozen, with rate of frost penetration 6.35 mm (0.25-in) per day.

Figure 6.—Summary of average rate of heave versus percentage of natural soil finer than 0.02 mm. From [10].

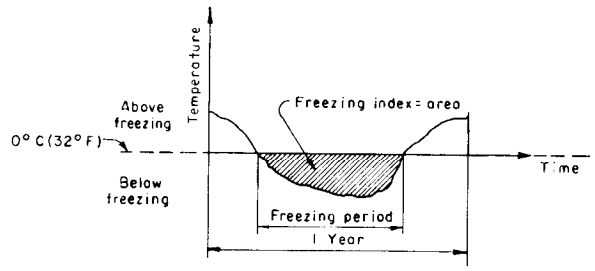


Figure 7.—Freezing index from annual variation in surface temperature.

tend to be thick, 13 mm (0.5 in) or more in thickness is not uncommon, and the amount of heave is large. As the rate of cooling increases, the ice layers become thinner and the amount of heave decreases. A rapid temperature decrease may freeze the soil before the water has time to migrate to form ice lenses; heave in this case is due only to the volume expansion of water.

INSTRUMENTATION FOR MEASURING FROST PENETRATION

Frost penetration in uncompacted soil can usually be determined by penetrating the frozen zone with an auger or other exploration tool and noting the depth where the penetration resistance decreases. Also, a rod with a short 90° pointed projection can be used to scratch the side of an auger hole to determine the frost depth. However, in compacted embankment, the soil may be so dense that it is difficult to determine the frost depth by these methods. For this condition, it is necessary to obtain soil temperatures at short intervals as an exploration hole is advanced. This can be done by taking the temperature of cuttings immediately after their removal from the hole.

A simpler way to follow frost penetration during the winter is to use a shop-made frost penetration tube of the type shown on figure 8, which can be installed before the onset of freezing weather. The dye solution in the inner plastic tube will freeze in the frost zone and become clear ice in contrast to the unfrozen blue solution below. This tube can then be removed from the casing as often as desired and the frost penetration measured. Instructions for the installation of the frost tube and for recording snow depth and climatic data are given in appendix C.

If only maximum frost penetration is desired, it is possible to install (before freezing conditions) a

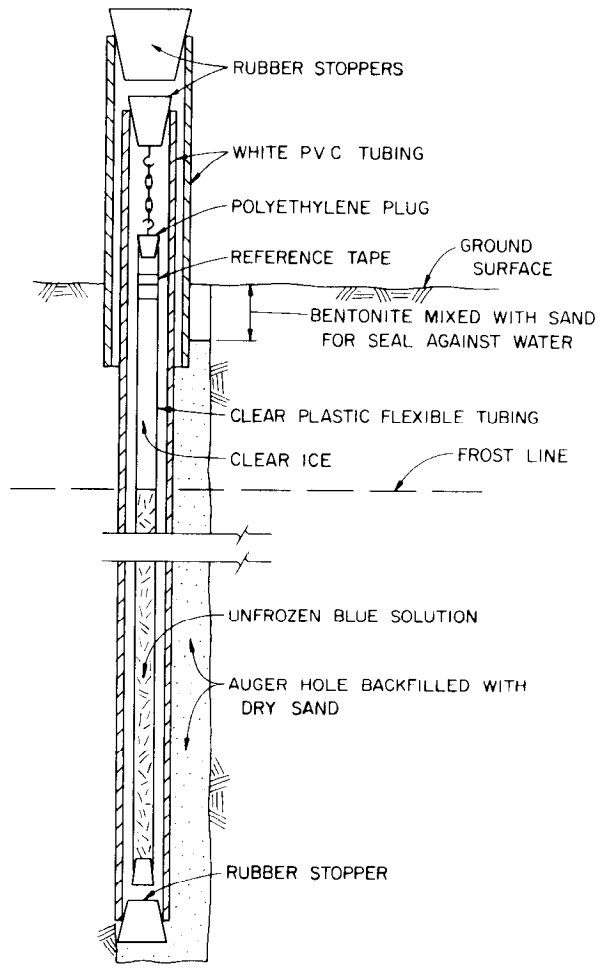


Figure 8.—Assembly of frost tube for measurement of frost penetration in soil.

frost tube which contains a string of water-filled glass capsules. After the ground has thawed, the tube can be removed and the approximate frost depth will be registered by the depth of the lowest capsule broken by freezing.

A more complex system to measure frost penetration is to install a series of thermocouples at short depth intervals in an auger hole backfilled by tamped native soil or a soil-water slurry. The thermocouples can be connected to a multiple point switch, and the temperatures can then be read by a digital thermometer. The frost penetration can be approximated by interpolation between the temperatures at the thermocouple points.

Thermocouple systems for measuring temperature, and dye-filled frost tubes for frost depth

measurements were used during field experiments on the Riverton Unit. These experiments are described in field test reports [12,13,14].

If there is an unusual amount of salt in the soil, the freezing point will be depressed and a temperature-sensing method will not accurately define the freezing level. Also, during the spring thawing period, the soil may remain at 0 °C (32 °F) for a significant subsurface depth interval because of the latent heat of fusion in the soil water. At times, unfrozen soil occurs between two frozen layers.

For an even more accurate method of determining freezing levels, a soil resistivity method has been developed [15]. This method is based on the fact that the electrical resistance of soil moisture rises sharply when water freezes.

INVESTIGATIONS FOR POTENTIAL FROST PROBLEMS

Performance of Existing Structures

In foundation investigations for structures, one of the first considerations is to determine the performance of existing structures in the vicinity with similar foundation and structural characteristics. This is particularly true for frost action investigations. For construction, such as the rehabilitation of irrigation structures damaged by frost action, there is usually at least a partial record of past conditions prior to the damage. Such background information should be carefully studied so appropriate changes in design for the new construction can be made. In areas where such information is lacking, it is advantageous to study the performance of other types of structures, and judgment is required to estimate frost effects upon the particular structure to be built.

Foundation Exploration

Representative geologic logs of subsurface conditions and samples of frost susceptible soils should be obtained. Classification of the soils as the excavation progresses will reveal those soils that should be sampled for laboratory testing. An auger hole is usually sufficient for this purpose. For frost action to develop, a continuous depth of frost susceptible soils within the capillary rise zone above the water table is required. Although the location of the water table at the time of foundation exploration is pertinent, of more importance is its level after the structure is built and put into operation. This expected location needs to be estimated especially if the water table is below the expected frost depth when the exploration is performed. Monitoring the water table during the early project life should provide valuable information. For highway purposes, the maximum capillary potential for frost action to develop is, from a practical standpoint, considered to be about 2 m (6 ft). Although the capillary rise in fine-grained soils can be much greater than this, the flow of water usually would be too small to build up ice lenses of significant thickness within the normal temperature ranges and freezing duration in the United States.

Disturbed soil samples are usually sufficient for laboratory tests to determine frost potential; the size of the samples depends on the tests to be conducted. The minimum size would be about 600 g (1.3 lb) of the minus 4.75-mm (No. 4) material for classification and mechanical analysis. For frost heave and associated physical properties tests, a 41-kg (90-lb) sample would be required.

Laboratory Testing

The gradation test is the most common laboratory test to estimate the potential of a particular soil to frost action. As previously mentioned, the distribution of particle sizes, particularly the amount of fines, is a good indication of soil capillarity and permeability, which are both involved in the formation of ice lenses.

There have been attempts to determine the effects of soil pore conditions by measuring capillarity. Csathy and Townsend [16] determined the capillary rise of water in soil placed in plastic tubes, and computed the effective pore size based on moisture contents obtained at different levels of the capillary soil column. Test results were correlated with pavement performance where the soil samples were obtained. Also, some comparisons with other test methods were made. Such capillary tests have the disadvantage of requiring considerable time to perform, about 1 month.

Laboratory freeze tests on soils have been conducted by a number of organizations. In this type of test, undisturbed or recompacted cylindrical soil specimens are frozen in a special freeze cabinet. The Geotechnical Branch of the

Bureau has such equipment (fig. 9) and has conducted tests on soils to determine frost potential. The specimens are surrounded by an insulation such as ground cork. Early investigators of frost heave, and the Bureau, have used dry sand around specimens, but this is not a good insulating material because the thermal conductivity of the sand is higher than that of moist, fine-grained soils. This would tend to cause some lateral as well as vertical freezing. Freezing should progress from the top downward with temperatures above the specimens controlled; control of temperatures below the specimens would also be desirable. From thermocouples placed at intervals on the specimens or opposite them in the center of the cabinet, a record of the temperatures is obtained as freezing progresses. For an open system representing the presence of a water table, a source of water is provided at the base of each specimen. As the upward transmission of water and the development of ice lenses progress, any heaving of the specimens can be measured by a remote control device at the top of the cabinet and the rate of heaving calculated. For a closed system representing the absence of a water table, no outside source of water is provided. In this case there is a redistribution of water content and density, and the development of ice lenses may or may not occur.

The results of a freeze test on specimens of soil from Whitestone Flats Canal in the State of Washington are shown on figure 10, and the gradation of the soil is shown on figure 11. The soil was classified as a silty clay (CL-ML) with a liquid limit of 23 and a plasticity index of 5. The maximum laboratory Proctor density was 1794 kg/m^3 (112 lb/ft^3) and the optimum moisture was 14 percent.

The Corps of Engineers has performed laboratory freezing tests [10], and has developed frost potential criteria for pavement design.

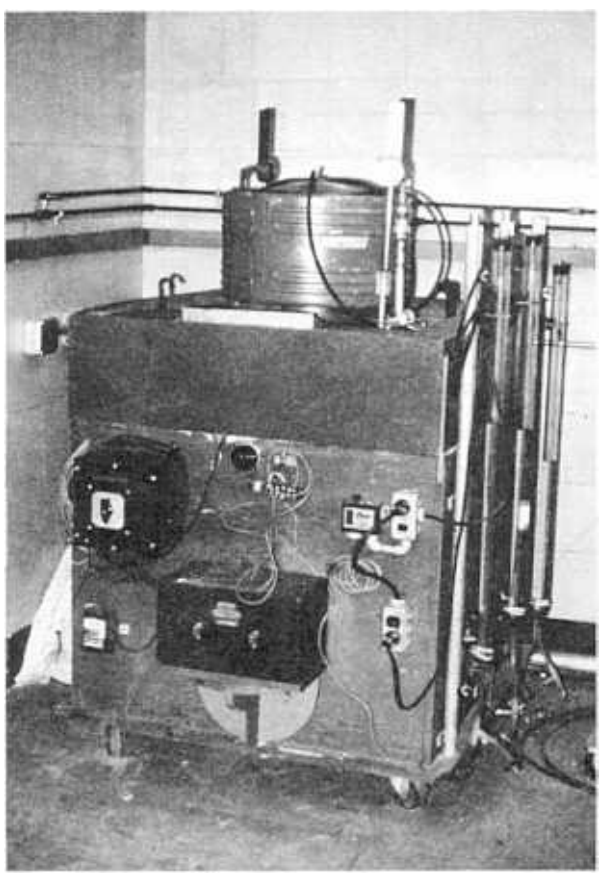


Figure 9.—Frost heave cabinet for conducting freezing tests on soils. P801-D-79856

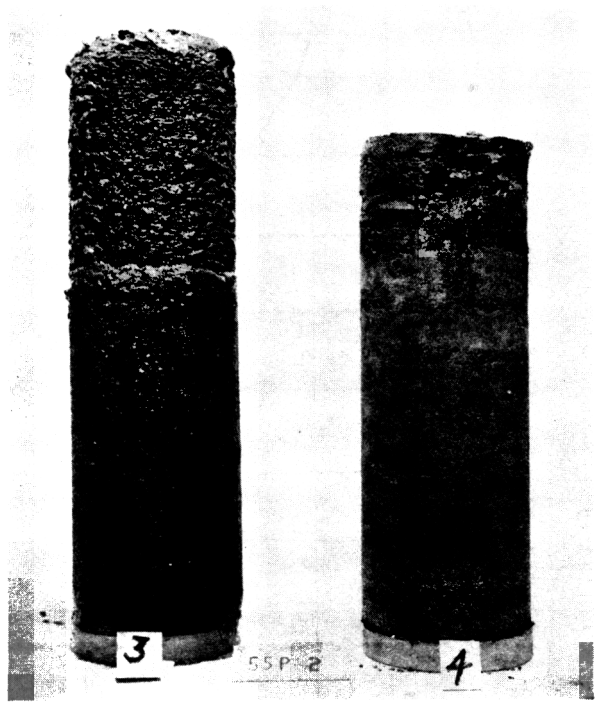


Figure 10.—Soil samples from Whitestone Flats Canal tested in frost heave cabinet. Sample 3, with access to water at base of specimen, heaved about 50 mm (2 in) after 7 days of subfreezing temperatures. Sample 4, with no access to water, heaved about 5 mm (0.2 in). P801-D-79857

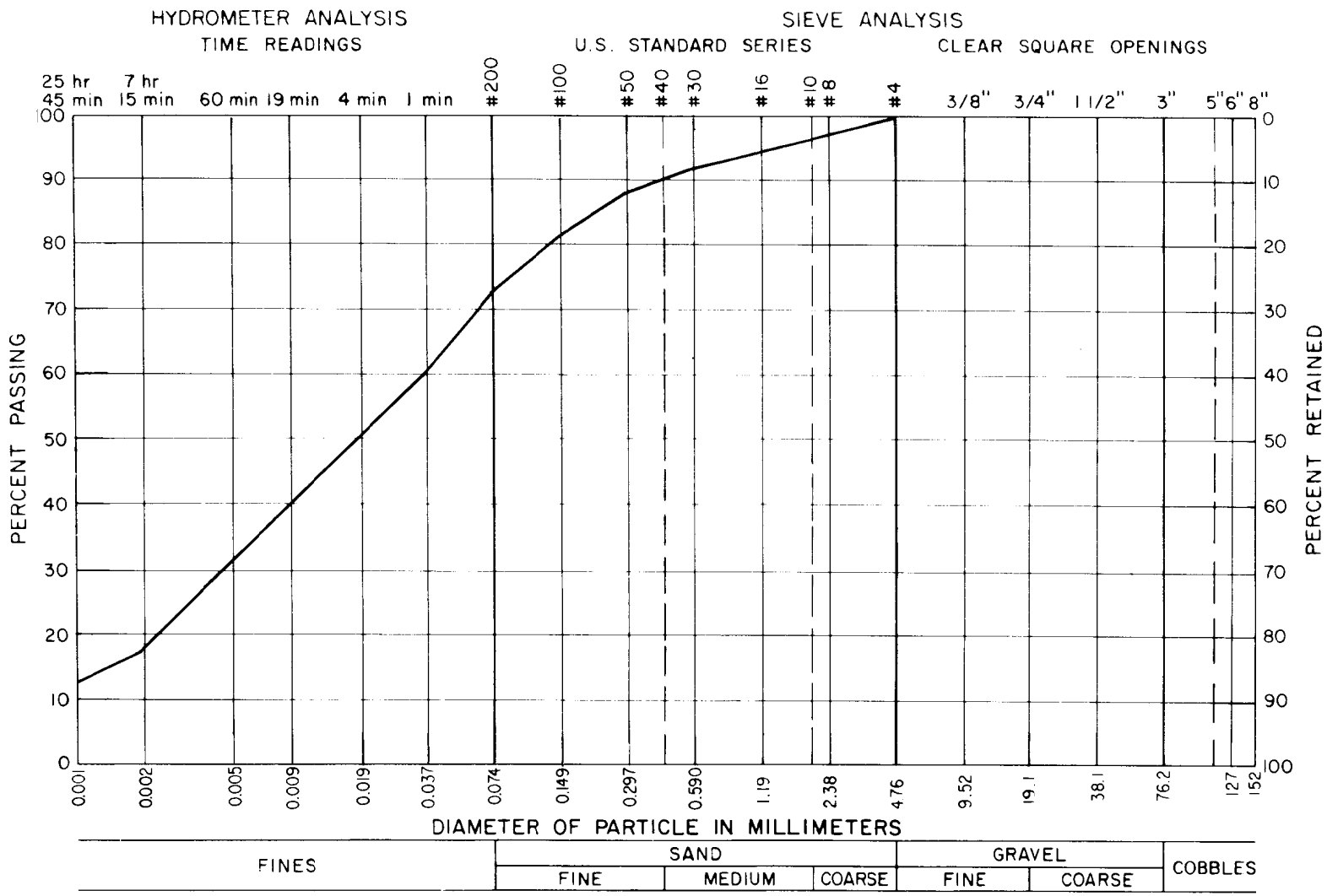


Figure 11.—Gradation of soil from Whitestone Flats Canal tested in frost heave cabinet.

Water Sources

Ground water is the most common source of water contributing to frost action problems; however, the water level may be too low to be drawn up into the freezing zone at the time of the investigation for a structure. After construction, the water table may rise within the capillary zone because of seepage through the structure (as in a canal) or from the percolation of surface water. In some cases, as on the Riverton Unit, adjacent irrigation supplies the water to structure foundations; this causes a high subsurface water condition in the fall which may decrease after the irrigation season. In some valleys, springs supply the water for local ground-water buildup.

If a hydraulic structure is operated during the winter, damage to the structure may be lessened because the flowing water will prevent the foundation from freezing, see subsequent subsection "Pipe Crossing on Piers". However, winter canal operation can supply ground water to cause frost heave to structures adjacent to the canal. Also, frost heave has been known to occur on concrete canal lining above the canal water level, particularly if the canal is operated at a water level below normal.

MEASURES TO REDUCE FROST ACTION

If design for seasonal frost action is neglected, the cost of repair may substantially exceed the cost of adequate initial protection. Frost action can be controlled through control of the soil, the water supply, or the ground temperature. Choice of best design is based on soil properties, source and quantity of water, depth of frost penetration, the structure, and economic considerations. Recommendations and frost design alternatives for foundations and walls of buildings and hydraulic structures are discussed in the following subsections.

Control of Frost Action

Measures to reduce frost action in foundations are more successful than designing a heavier, stronger, overlying structure when frost heave forces are high and unpredictable. Frost action can be controlled by one or more of the following four procedures:

1. *Replace the frost-susceptible soil throughout the depth of frost penetration with a material that is not affected by frost.* If sands and gravels are readily available, soil replacement with these materials may be a solution.

2. *Restrict the supply of water so ice lenses do not grow.* Capillary potential may draw water from the ground water as deep as 3 m (10 ft) in silts, clays, or silty, very fine sands; and from lesser depths in more coarse-grained soils. The capillary rise of water may be controlled by lowering the water table or by obstructing the upward movement of water. Drainage can lower the water table, but this alone may not entirely eliminate frost heave in frost-susceptible soils. The percentage of water that can be drained from most frost-susceptible soils is usually small and the remaining water in the voids will still be available to migrate to the freezing zone, which may still cause a small amount of heaving. In level areas and in areas of excessive rainfall or irrigation, even the use of extensive drains may not result in a sufficient lowering of the water table.

An impervious layer of asphalt, plastic, or clay will prevent the upward movement of water. Also, a blanket of clean, coarse-grained soil is effective if the blanket is thicker than the height of capillary rise. Such blankets must be well-drained and protected by filters where necessary so that finer soils do not fill the voids.

3. *Use thermal insulation to reduce the depth of frost penetration.* A thick layer of well-drained, coarse-grained soil is often used, but it has relatively poor insulating properties. Plastic or glass foam is an alternative which is becoming economically feasible. Design examples of an insulated building foundation are given later in this report.

4. *Use soil additives to stabilize and change the permeability of the soil.* Calcium chloride, sodium chloride, lime, portland cement, chemical dispersing agents, and waterproofing materials have been used as additives in highway construction. However, soluble additives are subject to leaching from water in the soil voids.

Data Required for Design

Design for frost action should be considered in relation to the use, value, and expected life of a project or structure. Alternative solutions should be examined and compared on a cost-value basis. The designer may require the following information:

1. Soil

   a. Properties (classification, grain size, density, water content and, for special cases, thermal conductivity)
   b. Frost-heaving potential
   c. Shear strength on thawing

2. Water

   a. Location of ground-water table (before and after construction)
   b. Source of free water (perched ground water, surface infiltration, seepage from waterways, or leakage from pipes)
   c. Precipitation

3. Depth of frost penetration

   a. Site location
   b. Freezing index
   c. Surface conditions (vegetation, snow cover, wind, shade, heated buildings)

4. Structure

   a. Type
   b. Purpose or use
   c. Heated or not heated
   d. Cost
   e. Expected life (temporary or permanent)

5. Available materials and labor

6. Alternative solutions and costs

Depth of Frost Penetration

Depth of frost penetration depends on temperature, surface cover, and the thermal properties of the soil as previously discussed. In developed areas, building codes and local practice can be used to estimate penetration for average cover and soil conditions. Frost penetration based on the freezing index and type of soil for basic designs is discussed in the procedural steps that follow. For unusual conditions, reference should be made to the U.S. Army and Air Force Manual [17]. For more complex designs and problems, heat flow finite element computer programs can be used [20]. However, many restrictive assumptions must be made in any design, particularly with respect to determining ground surface temperatures.

The following four procedural steps are for estimating the depth of frost penetration:

Step 1.—The *air-freezing index* is used as a measure of the combined magnitude and duration of below-freezing air temperatures. This freezing index is the number of freezing degree-days between the highest and lowest points on a curve of cumulative degree-days versus time for one freezing season. The measure of average or normal freezing conditions is the *mean freezing index,* determined on the basis of mean temperatures averaged over 10 or more years. This index is sufficient for the design of temporary structures. For permanent structures, it is necessary to consider the colder years which will occur during the life of the structure. The measure of these conditions is the *design freezing index*, which is defined as the average of the air-freezing indexes for the three coldest winters in the latest 30 years of record or, if sufficient records are unavailable, the coldest winter in the latest 10-year period. Figure 12 illustrates typical mean and design air-freezing index curves.

Step 2.—Freezing index values should, insofar as possible, be obtained from actual daily maximum and minimum air temperatures from stations nearest the construction site or interpolated between two stations located at two different elevations. Considerable variations in freezing indexes at points short distances apart may be caused by differences in elevations, topographic positions, and distances to cities, bodies of water, or other sources of heat or heat loss. These variations are of greater relative importance in areas with a design freezing index of less than 556 °C·d (1000 °F·d) than they are in colder regions. Figures 13 and 14 show isolines of mean and design air-freezing indexes, respectively, based on National Weather Service data [18].

Step 3.—Empirical relationships between the air-freezing index and frost penetration are

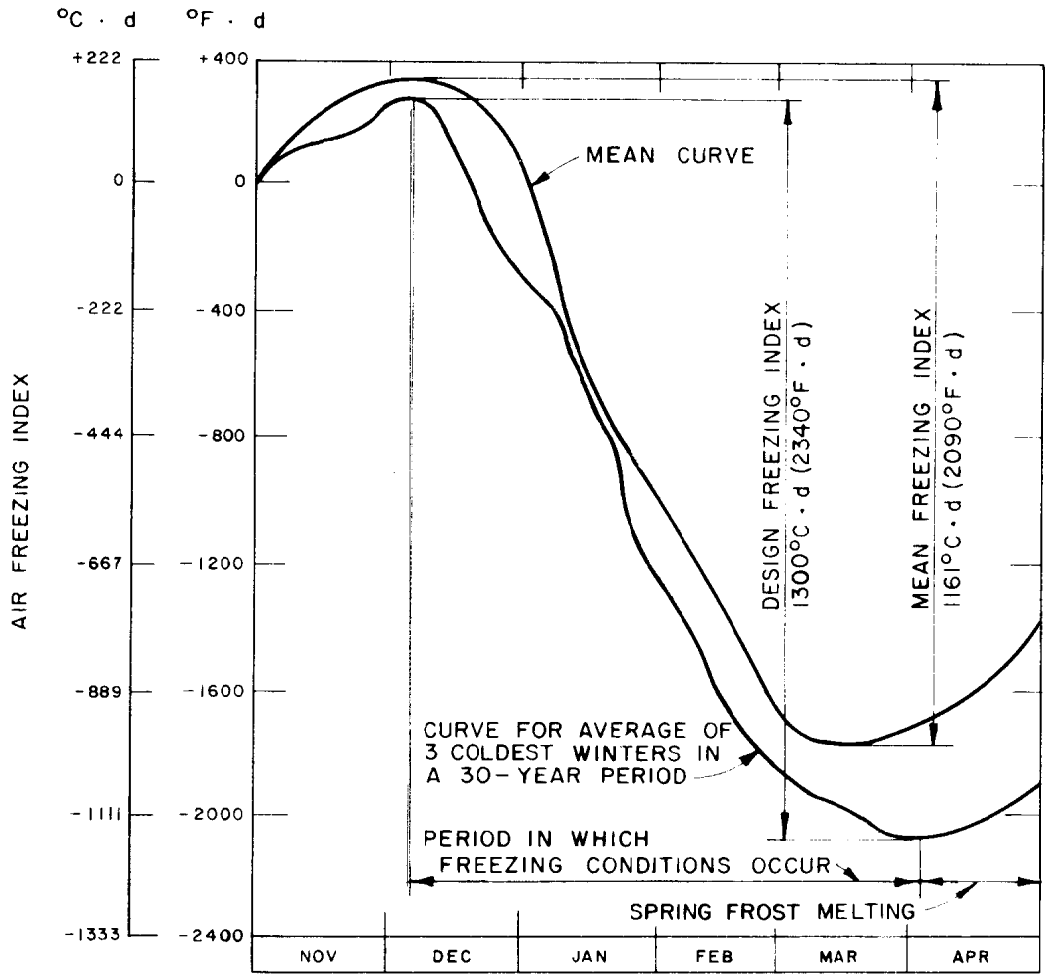


Figure 12.—Determination of the design and mean air-freezing indexes. From [20].

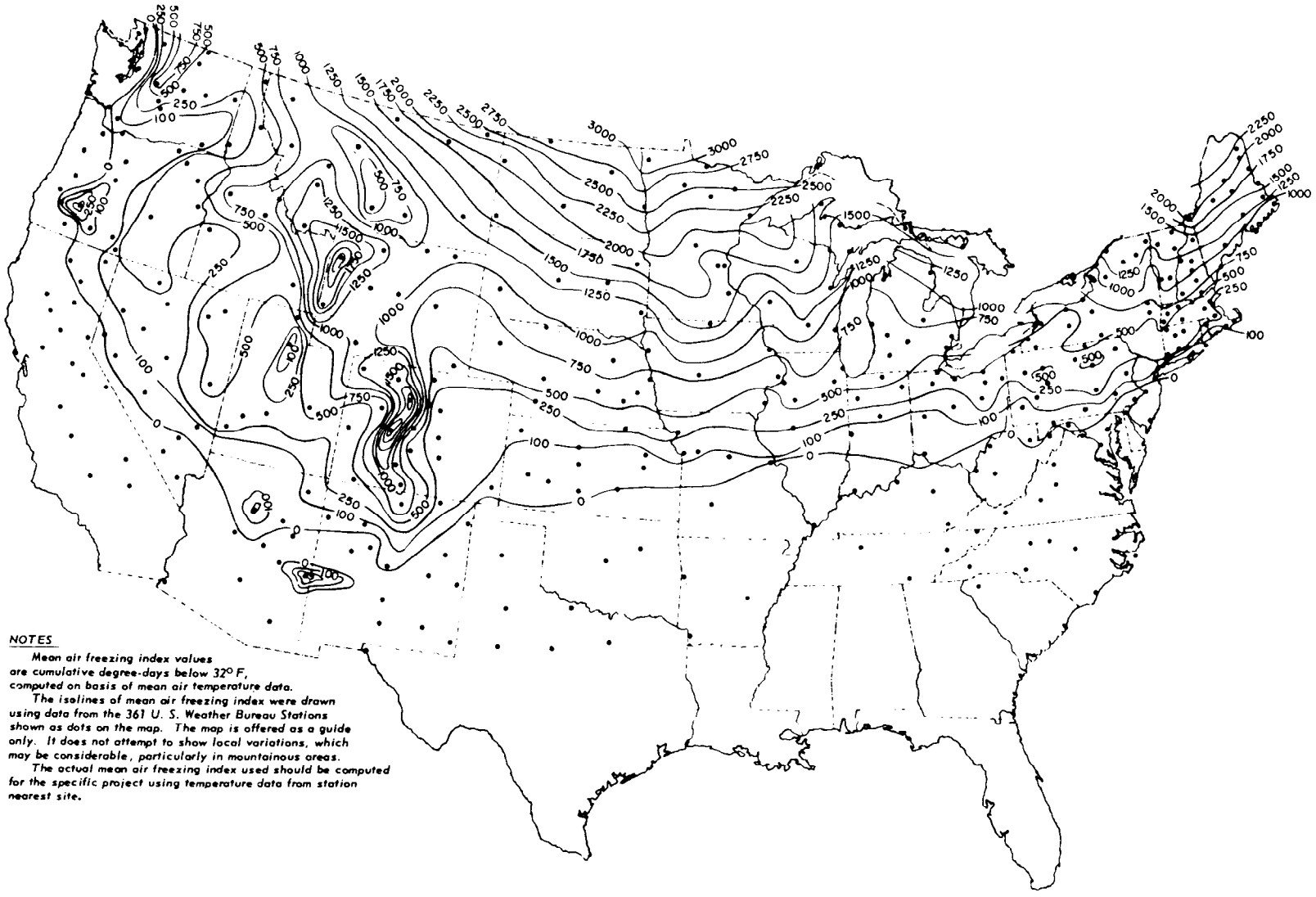


Figure 13.—Distribution of mean air-freezing index values in continental United States. From [18].

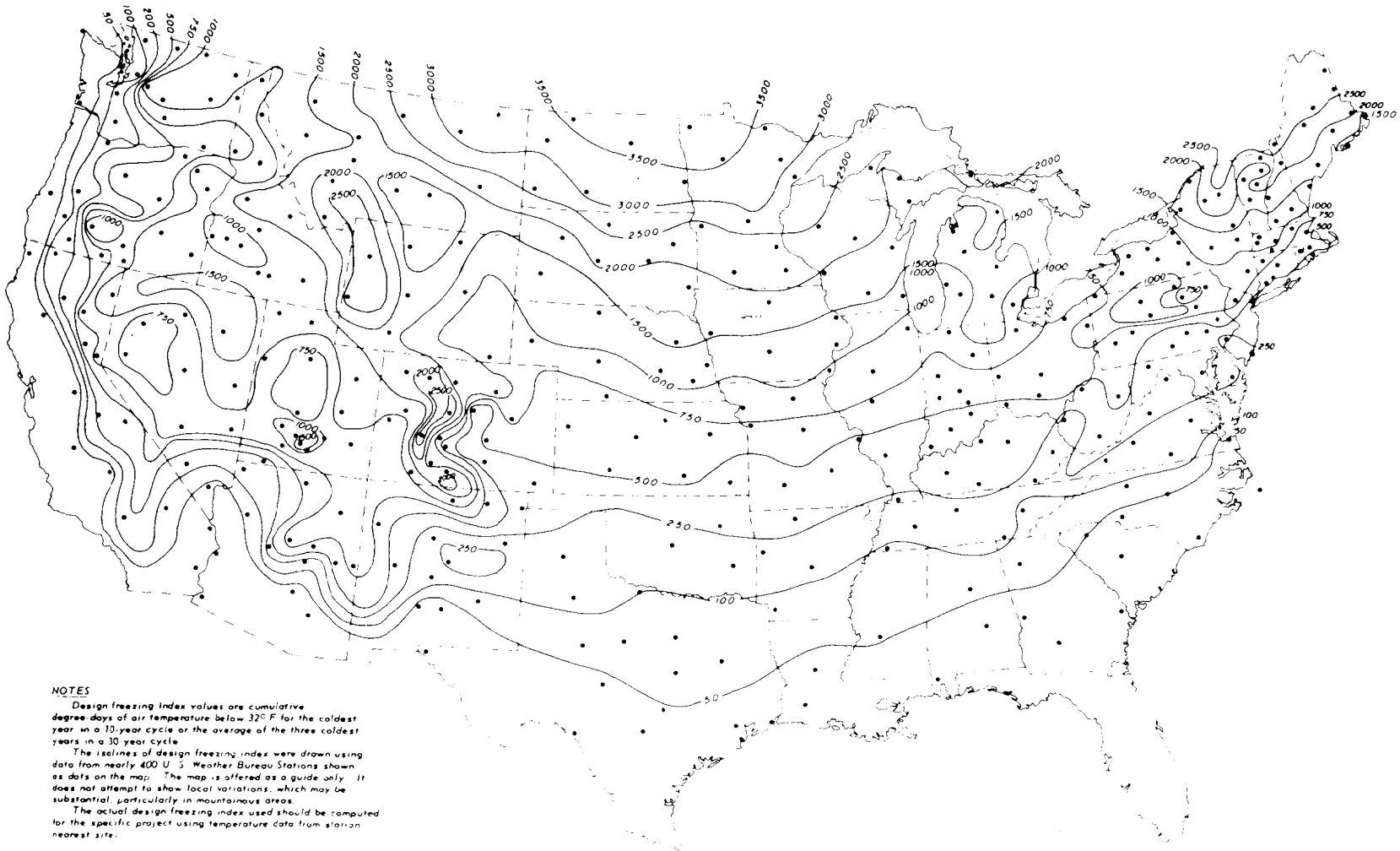


Figure 14.—Distribution of design air-freezing index values in continental United States. From [18].

shown on figure 15 for frost penetration into a free-draining granular soil beneath a paved area kept free of snow and ice. Frost penetration for unheated structures surrounded by cleared, paved areas is similar. The curves on figure 15 were computed for an assumed 305-mm (12-in) portland cement concrete pavement using the modified Berggren formula, and the correction factors were derived by comparison of theoretical results with field measurements under different conditions. The "n" factor shown on figure 15 is the ratio between the surface index and air index, and varies with the type of surface [19]:

Type of surface	"n" Factor for freezing conditions
Snow surface	1.0
Shaded surface	0.9
Portland cement concrete	0.75
Bituminous pavement	0.7
Bare soil	0.7
Turf	0.5

Step 4.—Figure 16 relates the frost penetration to the air-freezing index for various types of soil and surface cover. For example, given

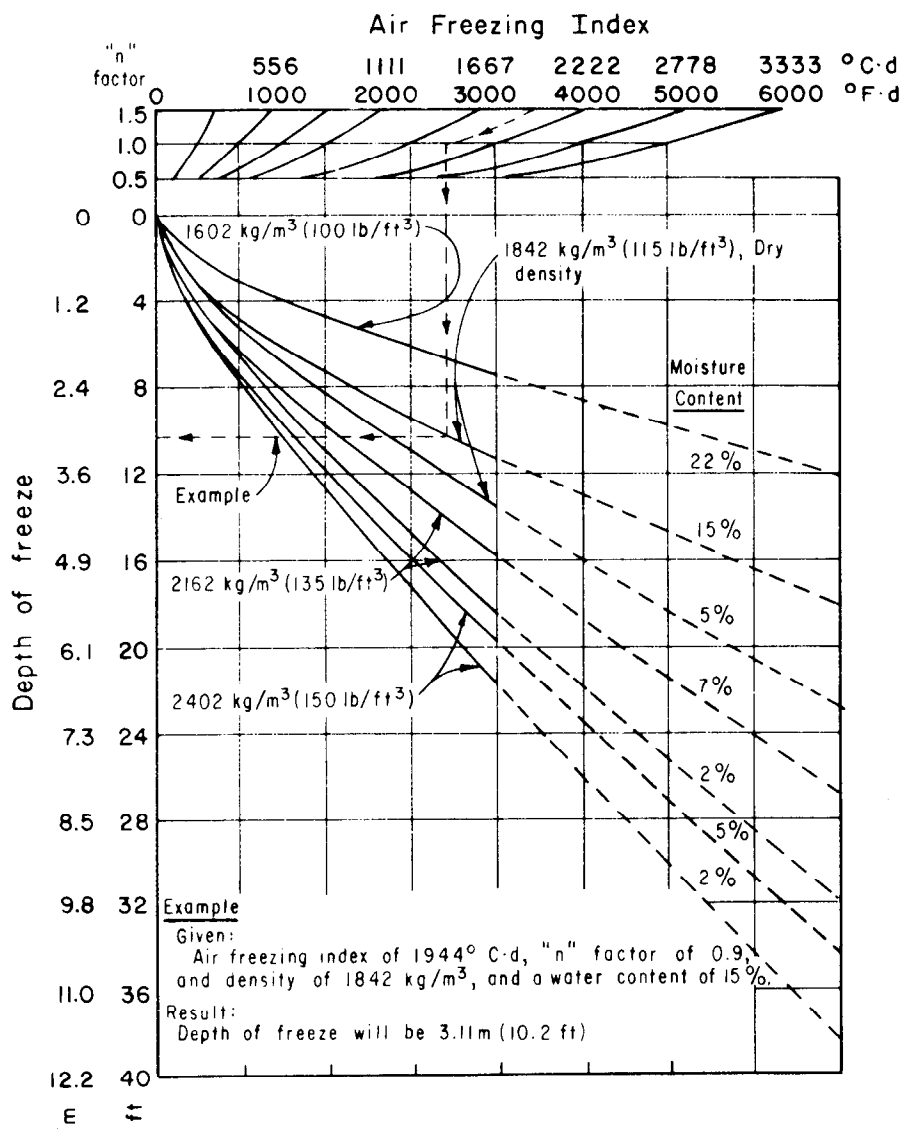


Figure 15.—Frost penetration in granular soil beneath a 305-mm (12-in) concrete paved area kept free of snow and ice. From [19].

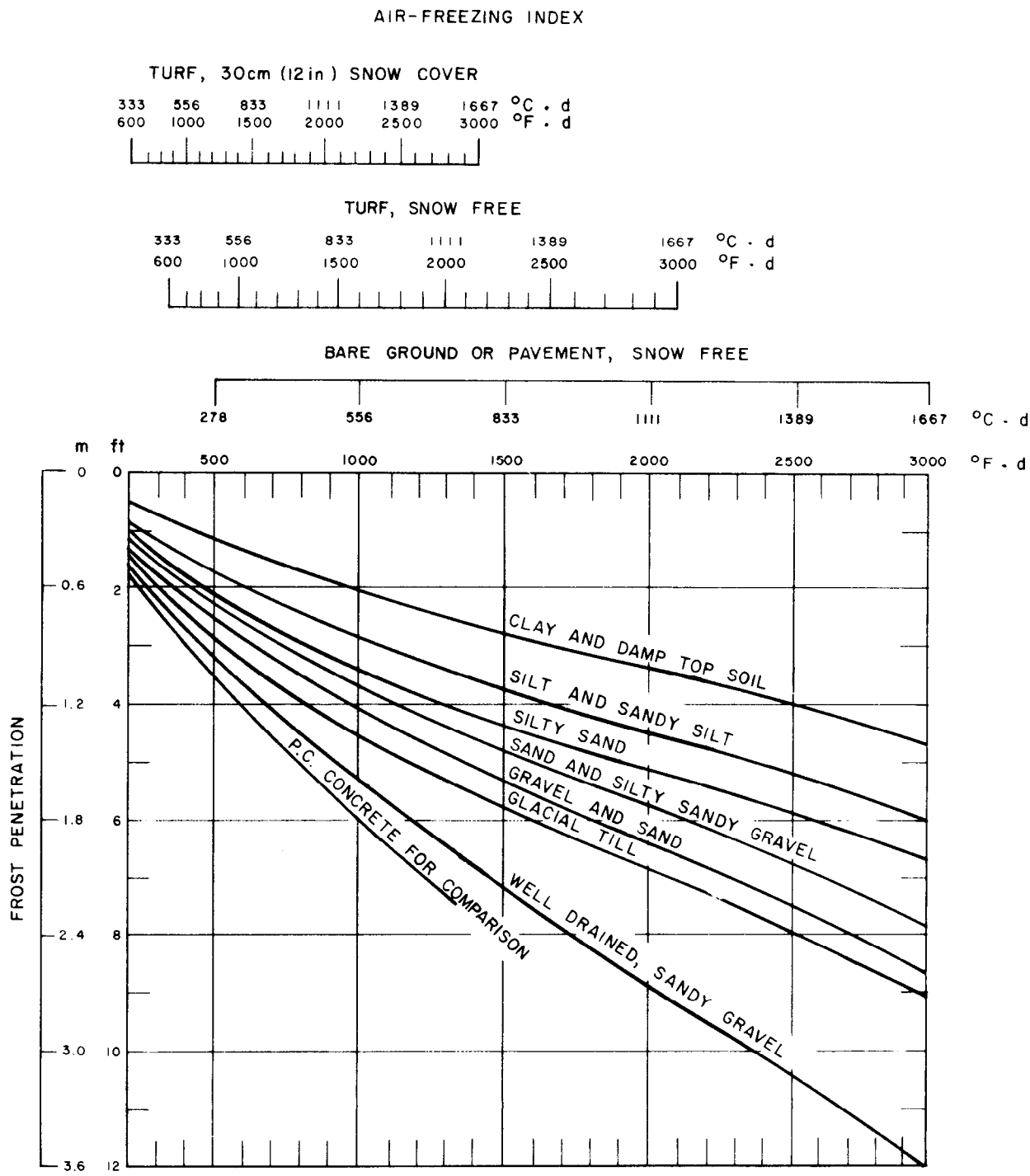


Figure 16.—Relationship between air-freezing index, surface cover, and frost penetration for homogeneous soils. From [20].

a silty sand subjected to an air-freezing index of 556 °C·d (1000 °F·d):

If the soil is covered with turf and 30 cm (12 in) of snow, frost penetration is 0.52 m (1.7 ft); if the ground is bare, the depth is 1.04 m (3.4 ft). The turf-snow cover thus reduces frost penetration by one-half.

An indication of the variability in soil thermal properties can also be seen on figure 16. Assuming the same index, frost would penetrate 0.6 m (2 ft) in a clay soil and over 1.2 m (4 ft) in a gravel and sand soil. It is interesting to note that the frost penetration in portland cement concrete (fig. 16), which is included for purposes of comparison, is greater than for the different soil types.

Structural Foundations

The amount of differential heave or settlement that can be tolerated by a structure varies considerably depending on its design and construction. For example, greater precaution must be exercised in the foundation design of a rigid-frame concrete bulding than for a wood-frame building. General procedures and recommendations for frost design of footings, slabs, posts, walls, and retaining structures are discussed in this section.

The most common approach for designing footings is to place them below the depth of frost penetration. Two exceptions are: (1) footings placed on soil not subject to frost heave, and (2) interior footings of heated buildings, assuming the footings are protected during construction and the building remains heated; the footing depths would then conform to the same criteria as for nonfrost areas. A minimum footing depth of 0.6 m (2 ft) is recommended for areas of very shallow frost penetration, 152 mm (6 in) or less. Footings on the exterior of heated buildings for support of porches, roof extensions, etc. are subject to full frost heave, a condition frequently overlooked.

For unheated slabs surrounded by paved areas kept free of snow, the frost penetration may be obtained from figure 16. Three design approaches are:

1. Replace frost-susceptible material to the depth of frost penetration with compacted, clean, coarse-grained fill. Drainage should be provided. If the fill becomes saturated, some uplift can occur due to freezing of water in the voids.

2. If architectural design permits, build a raised pad of well-drained, non-frost-susceptible material above natural ground.

3. Use insulation to reduce the thickness of select material. Figures 17 and 18 can be used to determine the required thickness of insulation and fill beneath unheated and heated structures, respectively. Frost penetration is reduced substantially beneath heated buildings. Penetration depends primarily on indoor temperature, insulation thickness, and the freezing index.

A design example using figure 17 would be:

Design conditions.—Design the necessary insulation for an interior drop-slab foundation of an unheated structure in Canada that has a mean freezing index of 1111 °C·d (2000 °F·d). The soil consists of a sand with trace of clay.

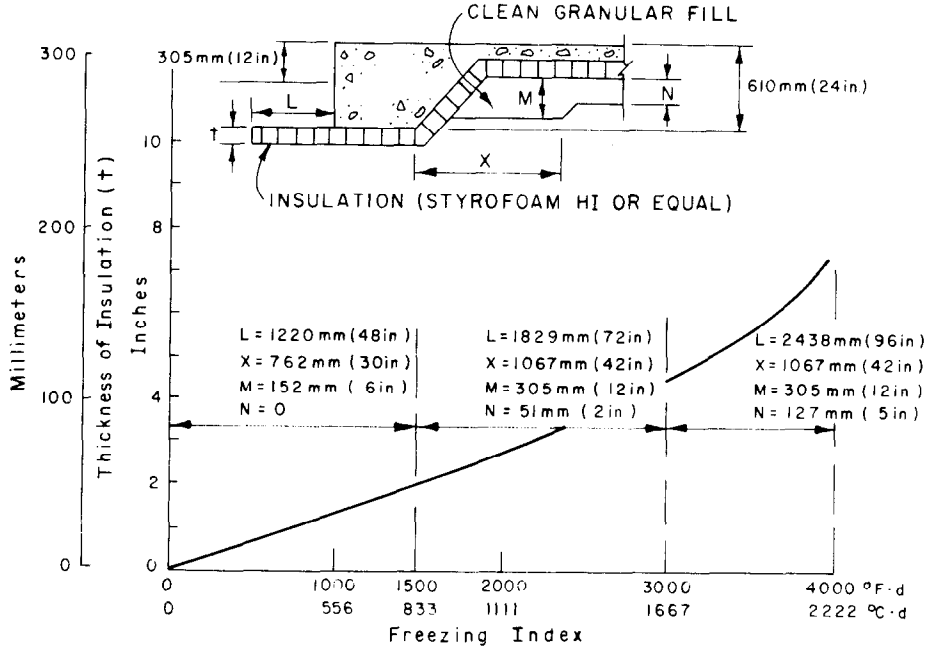
Solution.—From figure 17(b): (1) the insulation thickness is 76-mm (3 in) beneath entire mat and extends 2438 mm (96 in) beyond the limits of the building, and (2) a 305-mm (12-in) gravel thickness, beneath the insulation, extends over a width of 1067 mm (42 in) adjacent to the inside edge of the perimeter strip footing. Similar gravel protection is required around each interior drop-slab footing. The remaining floor slab is to be underlain by 102-mm (4-in) gravel.

A design example using figure 18 would be:

Design conditions.—At the same site, a building is to be erected for which the indoor temperature will never be allowed to fall below 7 °C (45 °F). Determine insulation thickness assuming a silty soil.

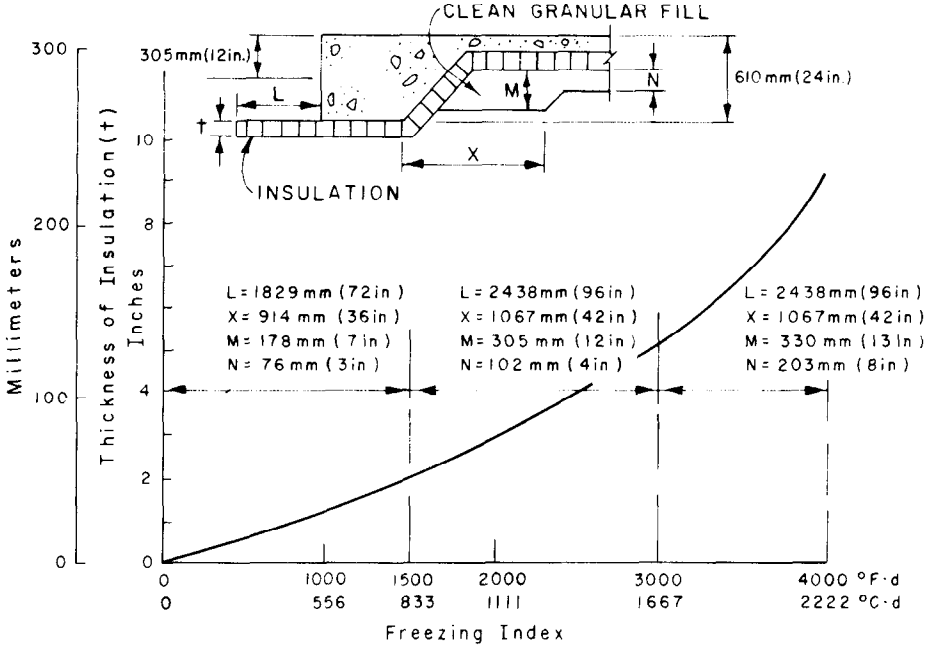
Solution.—From figure 18(a), the required insulation thickness would be about 51 mm (2 in) for an L of 1220 mm (48 in) or a thickness of 38 mm (1.5 in) for an L of 1829 mm (72 in).

In areas of deep frost penetration, the structure may be supported sufficiently above the ground surface to allow for surface heave. Possible ways to support the structure are:



Soil conditions: Dry density = 1362 kg/m^3 (85 lb/ft^3), 30% water content, all soil-water freezes at 0°C (32°F).

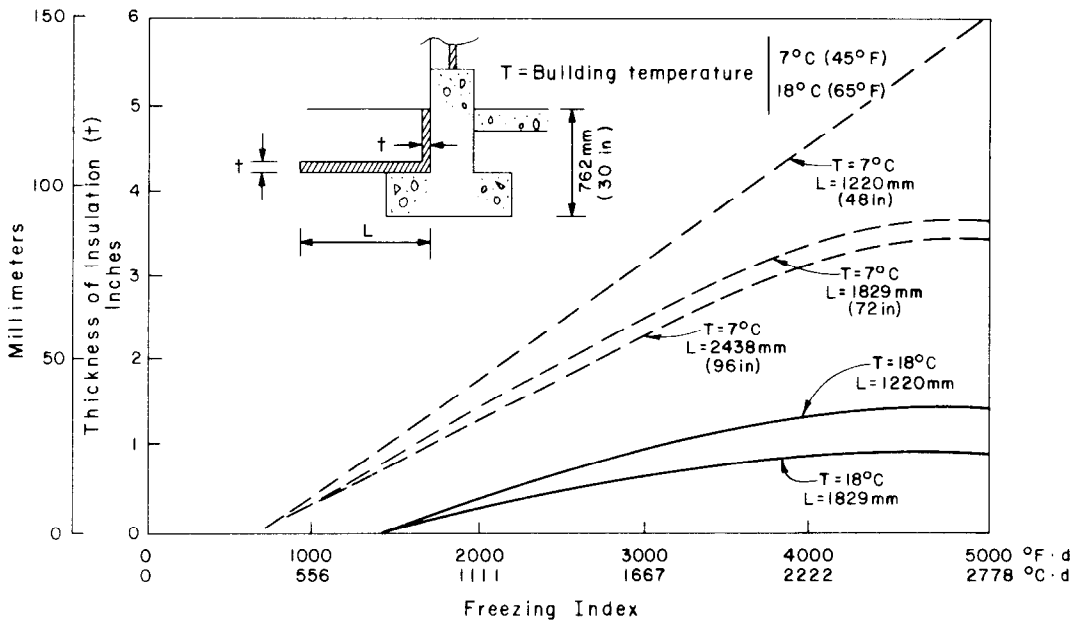
(a) On clayey or silty soil



Soil conditions: Dry density = 1682 kg/m^3 (105 lb/ft^3), 10% water content, all soil-water freezes at 0°C (32°F).

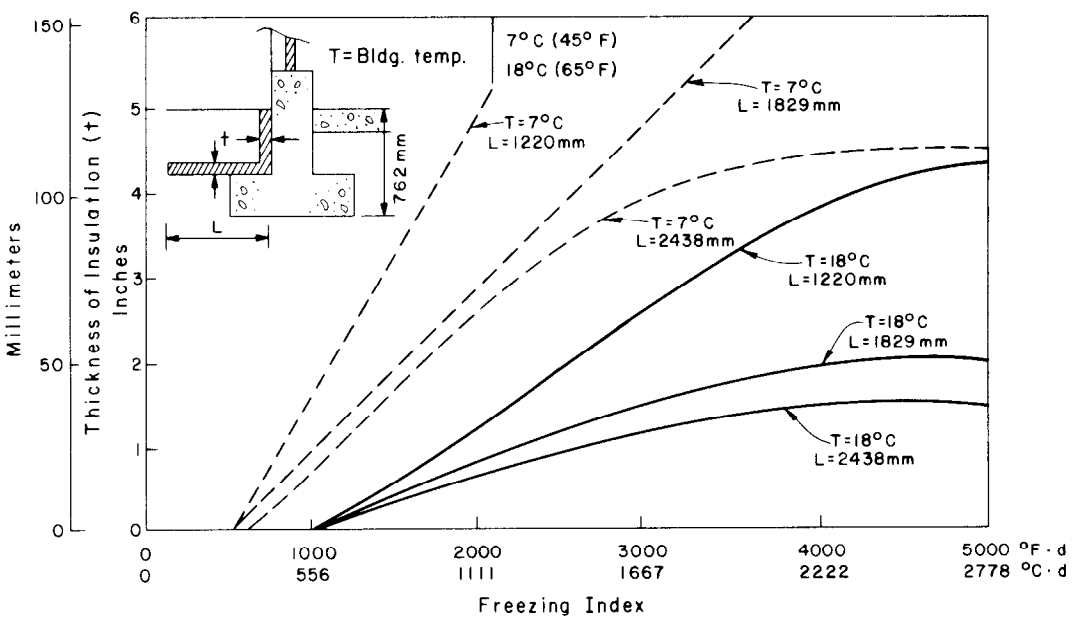
(b) On sandy soil

Figure 17.—Design curves for foundation insulation of unheated structures. From [21].



Soil conditions: Dry density = 1362 kg/m^3 (85 lb/ft^3), 30% water content, all soil-water freezes at 0° C (32° F).

(a) On clayey or silty soil



Soil conditions: Dry density = 1682 kg/m^3 (105 lb/ft^3), 10% water content, all soil - water freezes at 0° C (32° F).

(b) On sandy soil

Figure 18.—Generalized design curves for minimum insulation requirements for heated structures without insulation above grade. From [21].

- Belled-auger footings with the bell placed below the frost penetration

- Piles driven to sufficient depth to resist frost heave (The depth of the piles may be reduced by casings which eliminate adfreeze bond on piles).

- Steel or concrete columns tied to footings below frost penetration

- Conventional continuous foundation wall with footing below frost penetration and designed to resist adfreeze bond

The Bureau has also successfully used "void forms" (cardboard forms providing numerous voids with sufficient strength to support concrete placement) to provide clearance between the slab and ground.

Even if the bottom of a foundation is below the frostline, the overlying soil can freeze to the foundation and transmit heaving forces to it. This is called "adfreezing" or "frost grip." Penner and Gold [22] measured such forces on columns of steel, concrete, and wood; and on a concrete block foundation wall, all in frost-susceptible Leda Clay in Ottawa, Canada. At the field test site, the freezing index was 1133 °C·d (2039 °F·d) and the maximum frost depth during the test was 1.07 m (3.5 ft). For the tests on columns, the adfreezing strength ranged up to about 138 kPa (20 lb/in^2), with the strength for the steel and concrete being somewhat higher than for the wood. For the wall, the maximum adfreezing strength was about 62 kPa (9 lb/in^2).

Possible ways to provide protection against adfreezing are:

- Anchor footings by spreading the base below frost penetration and include suitable reinforcement to resist the tensile forces in the concrete.

- Isolate the structure from the heaving soil with a layer of loose, coarse backfill. A pile or a pipe column may be protected by a larger sized casing which permits free movement of the casing. Oil-wax mixtures and other materials have been used as fill between the column and casing, or between the column and soil if no casing is used. A casing should have an external flange at its bottom end.

- Batter the face of a wall inward so that heaving soil will tend to break contact with the wall.

Unheated walls and retaining structures in contact with frost-susceptible backfill and with access to a source of water, are subject to horizontal frost penetration, ice lensing, and lateral thrust because ice lenses tend to form perpendicularly to the direction of heat loss. Placement of a layer of free-draining granular fill adjacent to a wall is a satisfactory solution. Figure 19 gives the thickness of non-frost-susceptible backfill required for a given freezing index.

Some miscellaneous considerations are:

- The ground surface should be sloped away from structures, and roof drainage should be taken 3 m (10 ft) away from a building.

- Water and sewer lines should be placed below the depth of frost penetration. Ensure that utility lines will not be broken by heave or settlement of foundation walls.

- Concrete exposed to frost action should contain sound aggregates and entrained air, and be provided with good drainage.

Hydraulic Structures

As previously discussed, three basic conditions must exist for frost heaving to develop:

1. Freezing temperatures must exist in the soil.

2. Ground-water table must be sufficiently close to the frostline to provide water for growing ice lenses.

3. Soil must allow capillary movement of water from the water table.

Canals built in cold climates where frost penetrates deeply into the ground are particularly susceptible to frost heave. Seepage from a canal may be sufficient to cause a high ground-water table near the canal prism. Irrigation of land adjacent to the canal may contribute to a rise in the water table. Depending upon the foundation drainage conditions, the water table may or may not be lowered significantly when the canal is

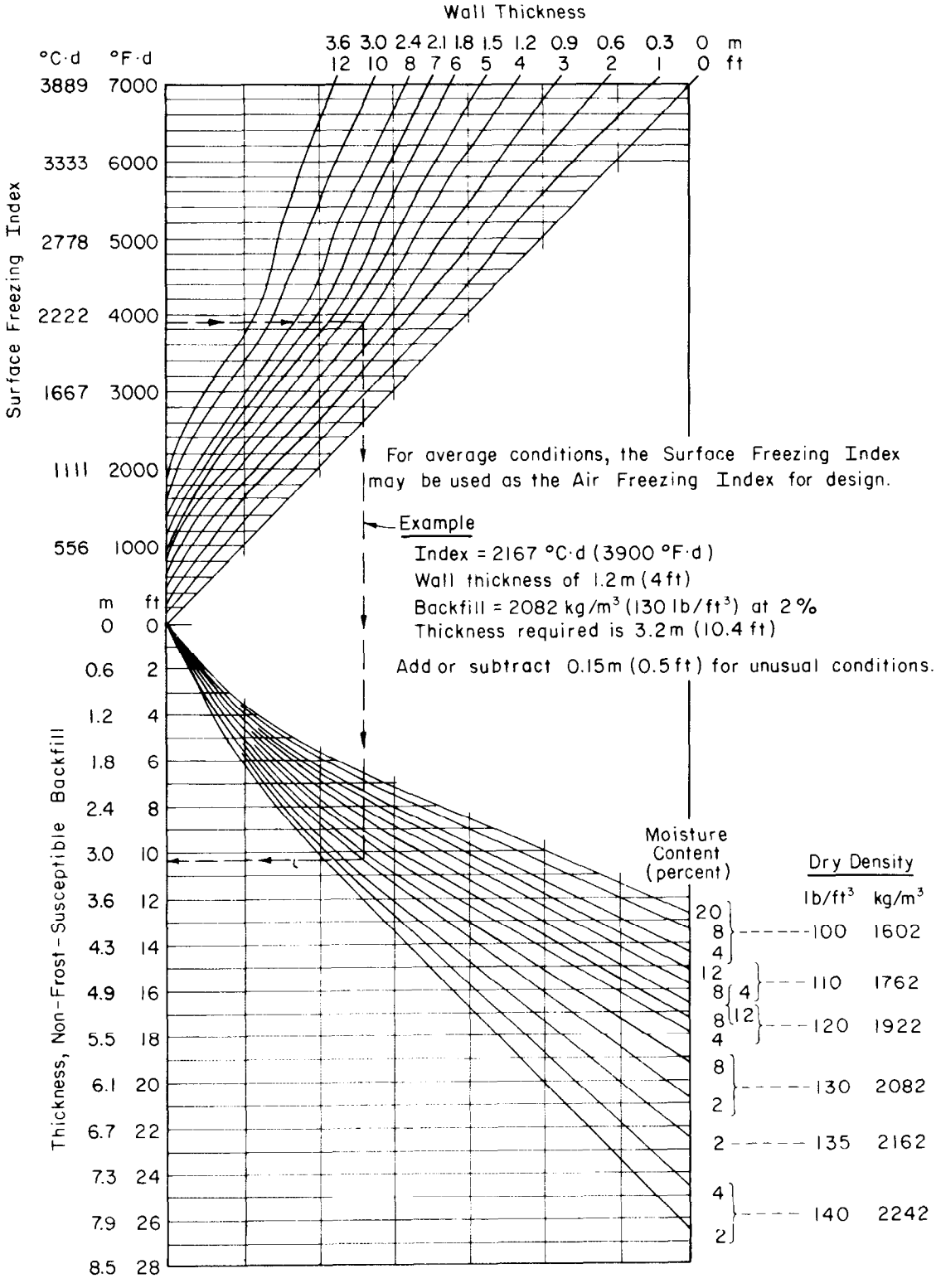


Figure 19.—Thickness of non-frost-susceptible backfill behind concrete walls. From [19].

dewatered between irrigation seasons. Continuous operation of the canal should keep the foundation soil from freezing during the winter and prevent frost action on canal structures. However, there may still be some frost action outside the structure above the water level in the canal, particularly if the canal is operated with a water level below normal.

PROBLEMS AND CORRECTIVE MEASURES IN HYDRAULIC STRUCTURES

The Bureau of Reclamation is responsible for many types of water conveyance structures including spillways, stilling basins, chutes and drops, checks, wasteways, pipe turnouts, pipe crossing on piers, canal linings, and earth embankment dams. This section contains information on problems resulting from frost action, and corrective measures and comments collected from years of operating experience. More detailed information on some of the field problems can be found in appendix A.

Vertical walls in retaining structures, turnout headwalls, and stilling basins may require special backfill. Backfill usually becomes partially saturated due to normal seepage at joints and cracks during the irrigation season. Consequently, to avoid frost action, drainage of the backfill must occur after the end of the irrigation season and before the first deep-ground freezing. To improve drainage, a filter material may need to be placed adjacent to the wall (fig. 20). Grading of the filter material should be in accordance with established criteria such as shown in the Bureau's *Earth Manual* [9]. Where counterforts are used on a wall, a collector drain pipe may be placed through the counterfort to join each bay. To increase the rate of seepage water collection at the drain pipe, a 305-mm (12-in) filter layer is a definite improvement where there are counterforts.

The chart shown on figure 19 may be used to estimate the thickness of non-frost-susceptible backfill required behind concrete walls. For average wall conditions, assuming essentially vertical wall faces and with shade part of the time, a surface freezing index equal to 0.9 of the air-freezing index should be used. If the wall receives no sunshine during the freezing period, is exposed to substantial wind, and remains free of snow or ice, an n-factor of 1.0 should be used. For walls with southerly exposure and sheltered from the prevailing wind, the n-factor may be as low as 0.5 to 0.7. There is less difference in the n-factor values at higher latitudes where the net radiational heat input may be much smaller and the n-factor may be from 0.7 to 0.9 [19].

Figure 19 may also be used for estimating the depth of frost penetration vertically into granular soil below a snow-free horizontal ground surface. For example, the surface freezing index for bare ground would be taken as 0.7 of the air-freezing index and the chart would be entered at zero wall thickness. Adfreeze contact between the backfill and vertical walls heave the walls upward if they do not have footings. When built with footings, the walls may encounter excessive tensile loads due to the upward thrust of the soil. This problem can be reduced by sloping the side of the wall in contact with the soil. This slope (batter) allows the heaving soil to break contact with the wall as the soil heaves upward. It should not be assumed that this will completely eliminate uplift forces. Anchorage against the uplift forces should be provided by extending the batter well down below the zone of frost penetration and/or by using an adequately widespread base. In any case, sufficient reinforcing steel must be incorporated in the

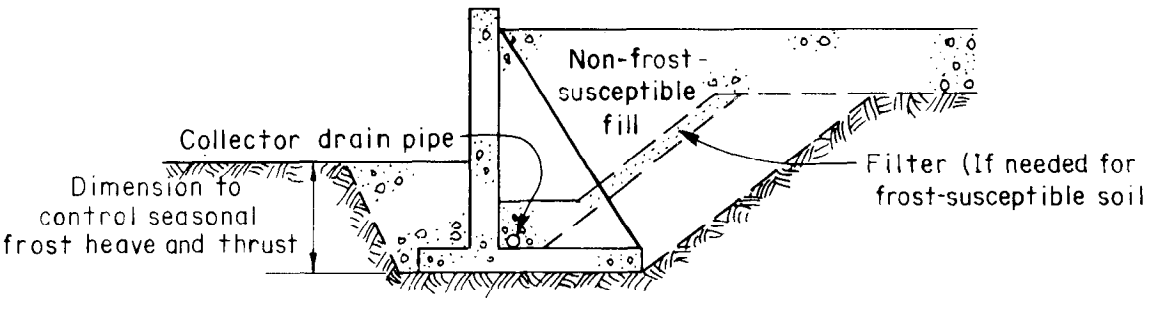


Figure 20.—Design of wall to prevent frost damage.

concrete to sustain tensile forces developed therein without cracking the concrete.

Adding insulation at critical points, such as at turnout headwalls and stilling basin walls, can minimize the problem of drain outlet freezing. The insulation is placed between the concrete wall and the backfill to reduce heat loss through the concrete from the soil (fig. 21). This problem is particularly critical at turnouts where the soil freezes and the headwall and/or pipe heave causes a break between the two. It is often difficult to find the break until the water is turned into the pipe and an earth cavity is eroded by the seepage flow around the pipe. To prevent this, 50 mm (2 in) of polystyrene insulation was found to be sufficient on the Riverton Unit [12, 13].

Operation and maintenance (O&M) personnel have developed some very practical solutions to frost heave problems. The Jamestown Dam stilling basin wall rehabilitation is a good example of using a combination of underdrains and non-frost-susceptible soil for wall backfill. The original indication of a problem was horizontal cracking in the stilling basin wall (fig. 22). Figure 23 shows how backfill was removed to the bottom of the wall. The collector drain pipe was then placed at the bottom of the excavation (fig. 24). The excavation was backfilled with pit-run gravel up to the original ground elevation (fig. 25), and riprap was replaced upon completion of the work (fig. 26).

Other practical suggestions by O&M personnel for reducing frost damage in the design of new structures and the rehabilitation of existing ones are:

1. Prevent frost heave damage behind canal walls by using free-draining, non-frost-susceptible backfill with pipe drainage.

2. Provide drainage behind walls by:

   a. Longitudinal drains when possible
   b. Weep holes (The disadvantage of weep holes is that they allow canal water to saturate the backfill. Flap valves may be used but they may deteriorate and malfunction with time).

Spillways and Outlet Works

Two cases of failure due to frost heave involved cantilevered walls on spillway stilling basins; this type of wall is found in many hydraulic structures. Vallecito Dam (completed in 1941) and Lemon Dam (completed in 1964), both located in southwestern Colorado, were subjected to high rainfall in December 1970 which thoroughly soaked the backfill material behind the spillway walls. In January 1971, following the wet fall season, unusually low temperatures were recorded including several days when the temperature did not rise above −6 °C (21 °F) and the minimum ranged between −30 and −51 °C (−22 and −59 °F). These extreme

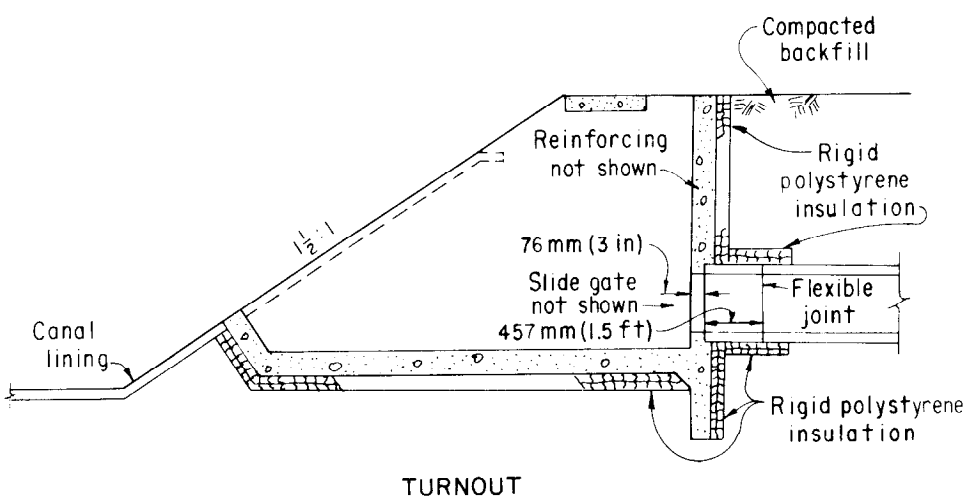


Figure 21.—Design of insulated turnout.

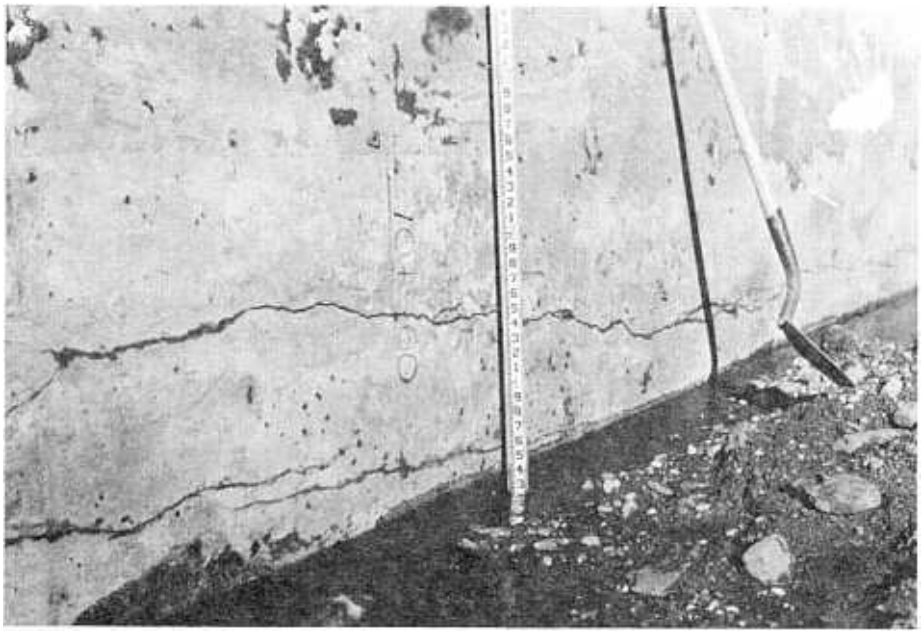


Figure 22.—Cracks caused by frost action in stilling basin wall. Jamestown Dam, 1961 P604-603-128

Figure 23.—Excavation behind stilling basin wall. Soil against wall at lower right is gravel. Jamestown Dam, 1961. P604-603-126

Figure 24.—Collector drain pipe placed at bottom of excavation behind stilling basin wall. Jamestown Dam, 1961. P604-603-127

Figure 25.—Gravel backfill behind stilling basin wall. Jamestown Dam, 1961. P604-603-132

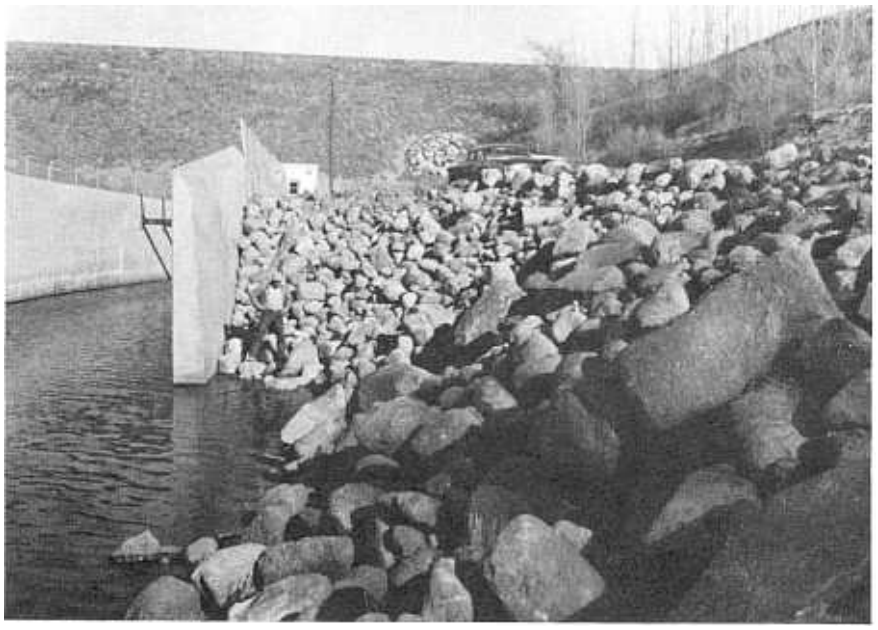


Figure 26.—Replacement of riprap behind stilling basin wall to complete repair. Jamestown Dam, 1961. P604-603-131

weather conditions were believed to have significantly added to a progressive failure of the spillway walls.

Vallecito Dam.—Sometime between April 1 and 9, 1973, two panels of the left spillway wall in the basin area and the upper portions of six counterforts were jacked inward by expansive freezing action, dropped into the basin, and rested in a nearly horizontal position on top of the basin floor baffles and sills (fig. 27). The right wall of the spillway approach channel deflected enough to damage the fence above. After reconstruction of the concrete wall, a pervious backfill with an underdrain system was installed behind the wall. The underdrains had outlets through the wall below the waterline in the basin to prevent water freezing at the drain outlets. An impervious soil blanket was placed on top of the backfill; however, shrinkage occurred in the blanket which allowed surface water to penetrate into the backfill. If such a blanket is to be used, it should be carefully designed as an impervious barrier. The spillway damage at Vallecito Dam is discussed in more detail in appendix A.

Lemon Dam.—In 1966, minor deflections at the spillway wall joints were first noted and some of the impervious backfill behind the walls was replaced with pervious material. In May 1973, the second wall panel on the left spillway entrance collapsed into the spillway (fig. 28). The panel on the opposite side was deflected inward about 305 mm (12 in) and was on the verge of collapsing. The primary cause for this damage was frost action behind the wall. During the repair operations, 76-mm (3-in) thick rigid polystyrene insulation was attached to the outside of the wall (fig. 29). The new drains, which emptied into the spillway as before, were equipped with electric heaters to prevent freezing and water stoppage at the drain outlets. The damage at Lemon Dam is discussed in more detail in appendix A.

Jamestown Dam.—Jamestown Dam is located on the James River near Jamestown, N. Dak. Immediately after construction was completed in 1953, a portion of the left wall of the outlet works stilling basin had been displaced inward. The 1961 O&M inspection report stated that a 12.2-m (40-ft) long section of the wall had tilted inward 102 mm (4 in) at the top. At the time, there was considerable seepage from the bank to the left side of the outlet works. The movement of the wall was attributed to frost action in the backfill. This was particularly true in the left

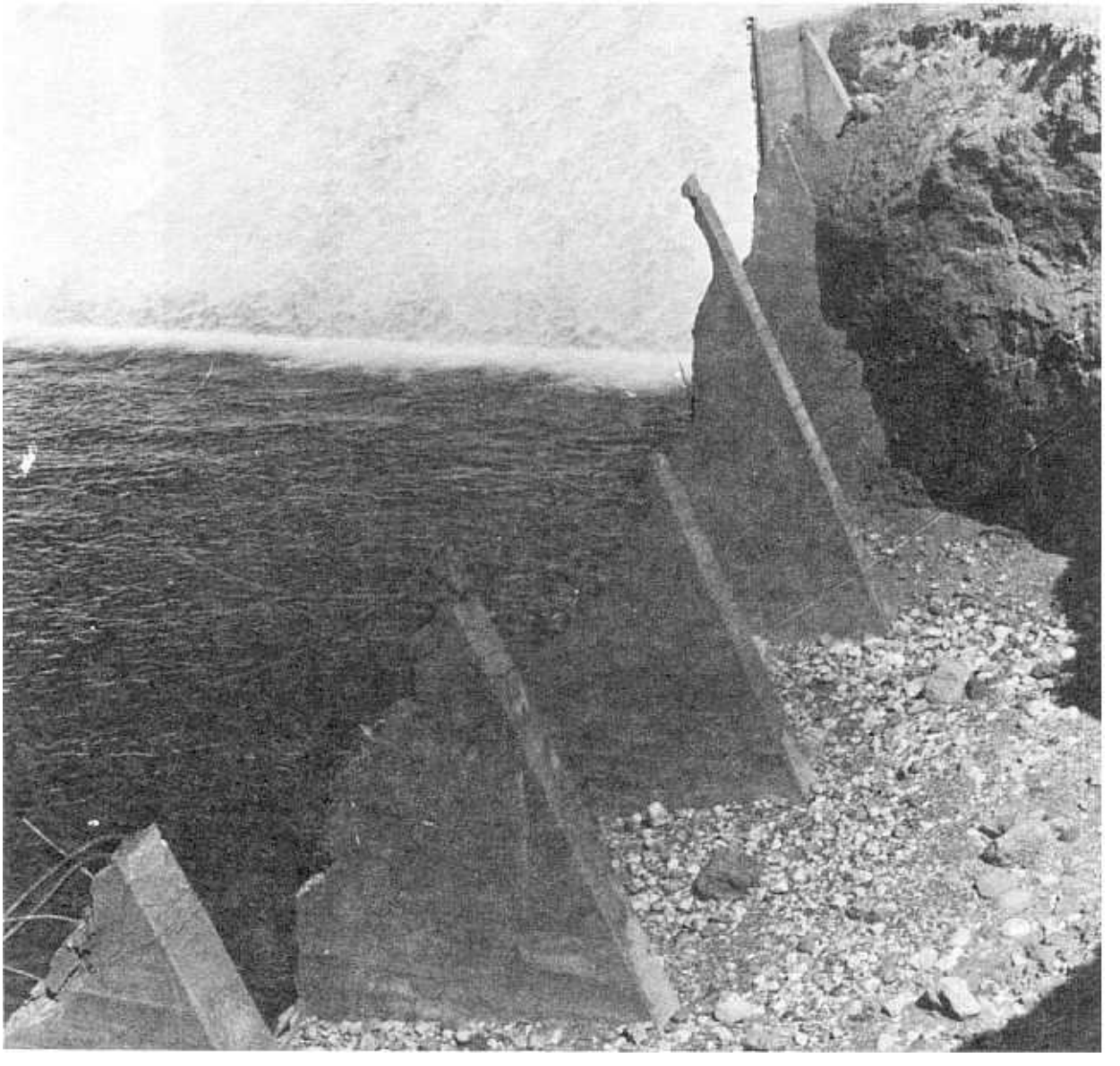


Figure 27.—Failure of left stilling basin wall on Vallecito Dam from frost action in backfill. September 1973. CN191-D-73902

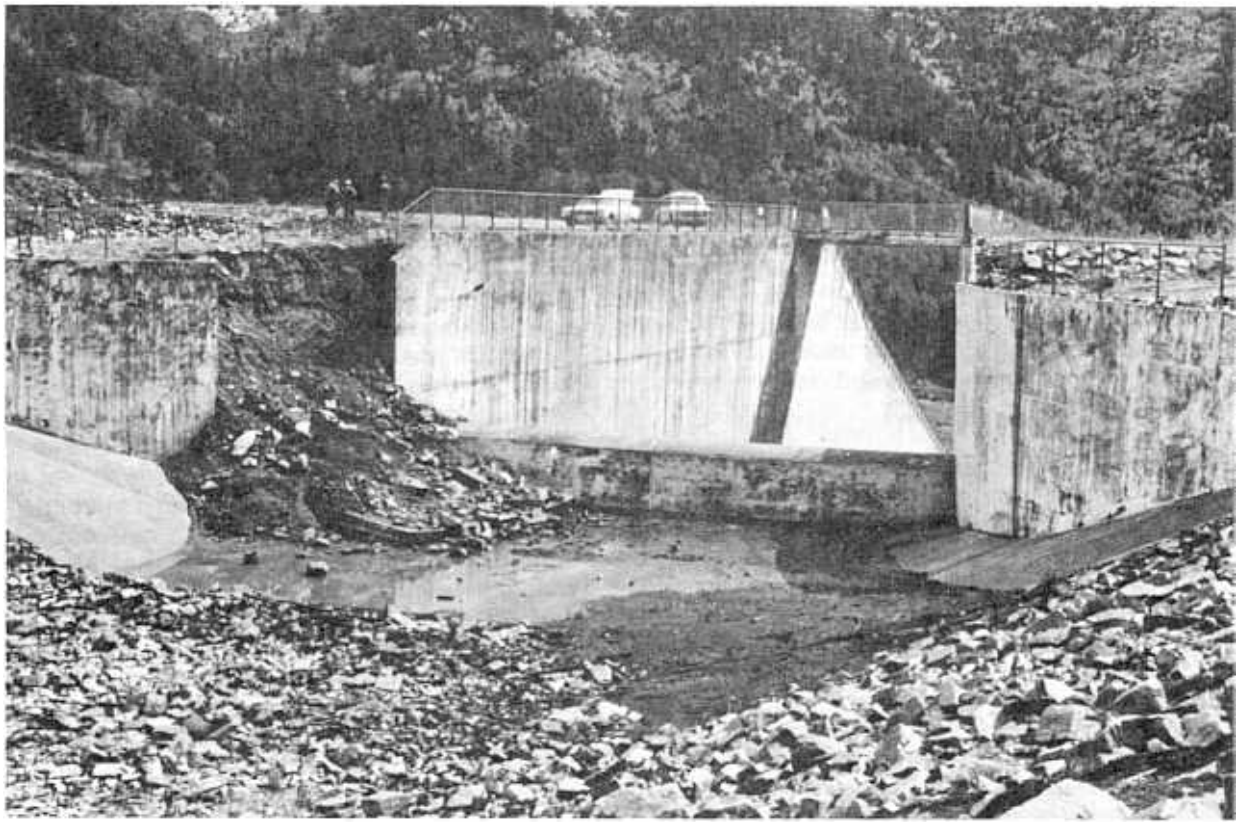


Figure 28.—Failure of spillway wall panel on Lemon Dam from frost action in backfill. September 1973. P519-429-3NA

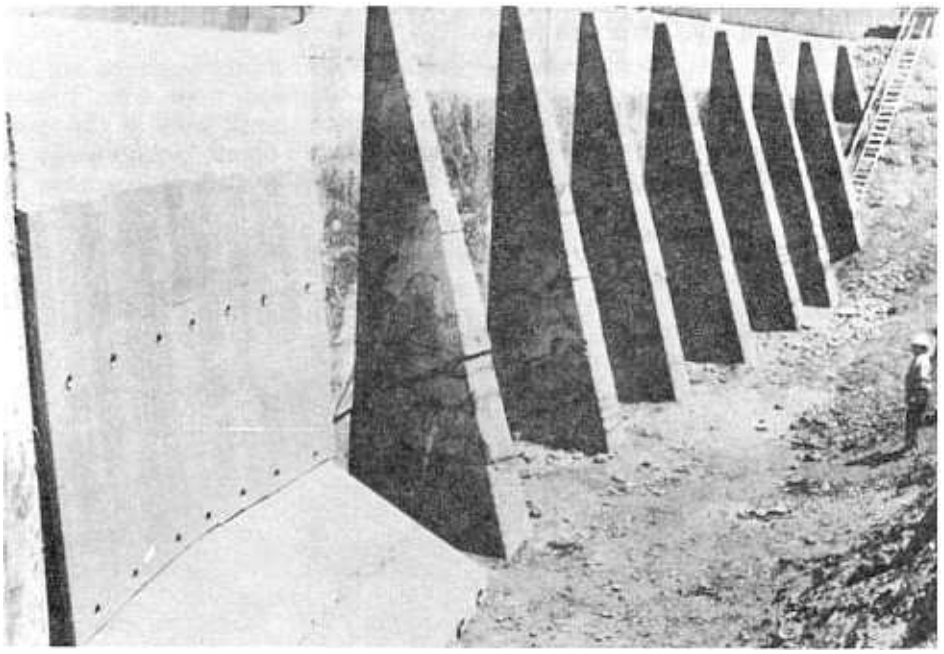


Figure 29.—Rear side of right spillway wall with repaired counterforts and polystyrene insulation. Lemon Dam, May 1974. P519-429-28A

wall where the backfill became saturated by ground water seeping along the surface of the bentonite layers which were part of the bedrock series. These layers ranged from 50 to 152 mm (2 to 6 in) in thickness and occurred in the valley wall at intervals varying from 3 to 4.6 m (10 to 15 ft). Apparently, adequate drainage had not been provided in the original construction to remove seepage water. A large mass of ice had built up during cold weather on the valley wall above the outlet works stilling basin as a result of seepage from springs at outcrops of bentonite. A drain was installed to remove the seepage water.

Dickinson Dam.—Dickinson Dam, completed in 1949, is located on the Heart River near Dickinson, N. Dak. In April 1954, the spillway failed during the spring runoff season when several concrete floor slabs were washed out (fig. 30). The failure was attributed to improper functioning of the underlying filter blanket. The probable cause was freezing of the filter. Construction photographs showed the filter being compacted by immersion-type concrete vibrators during water application from a hose without a sprinkler head. Probably, the filter was not as free-draining as planned because of segregation of particles by this method of placement. The normal construction procedure should have included a method of uniform water application and compaction of layers by a surface type vibrator. Repair measures for the spillway included a zoned filter blanket with a drainage system under the concrete spillway floor, an anchor system on the concrete panels, cutoff walls under the spilway apron to increase the seepage path, and metal water seals in the longitudinal joints and in some of the transverse joints. The damage at Dickinson Dam is discussed in more detail in appendix A.

McClusky Canal Structure.—The concrete radial gate structure and right wingwalls on reach 3A of the McClusky Canal in North Dakota were built during the 1971 construction season. No backfill was placed and water in the bottom of the canal was diked off for the 1971–72 winter season. In March 1972, movement of the wingwalls at the construction joints resulted from frost action in the wall foundation. Measurements showed a maximum misalinement of 61 mm (2.4 in) at the top of the wall. As the ground thawed in the spring, the structure returned to its normal position and adjacent construction resumed on schedule. The remaining concrete work and backfilling were completed during the 1972 construction season. The dikes were removed from the canal and water was allowed to flood the structure to a depth of 1.5 m (5 ft) above bottom grade. No further movement of the structure has been observed. A possible way to prevent a recurrence of this type of movement, if construction is shut down for the winter, would be to backfill around the structure before work is stopped.

Chutes and Drop Structures

Chutes and drop structures are both subject to severe damage from frost heave. Differential movement, particularly at the lower end of the downstream apron, often causes cracking in the basin, leaving slabs susceptible to erosion. In

Figure 30.—Failure of Dickinson Dam spillway in April 1954. P203-603-347NA

laterals and canals not lined with concrete, small concrete structures have a tendency to heave higher than the adjacent invert, also causing problems of erosion. Vertical side walls tip inward and cause concrete cracking adjacent to the bottom joints. Sometimes, broken pieces of concrete from cracked areas are pushed along the floor by turbulence generated by the obstruction to flow, and thus cause abrasion on the existing concrete. A drop structure having a trapezoidal cross section is even more subject to damage because forces from frost action act on the side slope lining causing vertical and horizontal displacement.

At the Riverton Unit in Wyoming, a series of small trapezoidal concrete chutes and drop structures were often placed on unlined or earth-lined canals on relatively steep slopes. One solution for the damage to these structures from frost heave was the construction of one long chute with an energy dissipator at the bottom. On Pilot Canal, a chute about 1950 m (6400 ft) long replaced a number of deteriorated drop structures (fig. 31). The chute was built as close to the original ground line as practicable, and shallow subsurface drains were placed on both sides. For a long chute, it is necessary that the interior of the chute have concrete with a very smooth surface to reduce resistance to water flow. Also, there should be a minimum of lateral joints, even small displacements in the flow prism may be sufficient to initiate a hydraulic jump which could cause water to spill over the walls of the chute. The work on the Riverton Unit is discussed in more detail in appendix A.

In the Upper Missouri Region, rectangular, inclined, drop structures have probably received the most damage from frost heave. Cantilevered stilling basin walls have a tendency to tilt inward. When movement is detected on structures that are not too large, the walls are sometimes braced apart laterally by timbers, steel channels, or precast concrete slabs. In areas such as the Riverton Unit, where tumbleweeds are very prevalent, this method is not too successful since the lateral bracing may act as a trash catcher which in turn may plug the structure.

Figures 32 and 33 show damage to canal structures on the Greenfields Irrigation District on the Sun River Project in Montana.

Checks and Wasteways

Check structures may be of three general types: (1) flow-through trapezoidal cross section with side bay weirs, (2) rectangular check with a transition from trapezoidal to rectangular and back to trapezoidal, and (3) trapezoidal section to rectangular check to tunnel or pipe entrance. Some wasteways have a radial gate for control. All of these gate structures have common problems; they must remain rigid enough to keep the gates from binding, and they all must maintain the same relative position with the canal flow prism. Frost heave would prove detrimental to the gates if the walls were displaced. Displacement of the invert at the upstream or downstream end would lead to certain failure if spring thaw water eroded under an exposed lining edge while still hidden under snow and ice still remaining in the channel.

Pipe Turnouts

Frost heave not only results in concrete walls moving, accompanied by misalinement of gates, but also differential movement may break off a turnout pipe from its headwall. Pipe turnouts often include a vertical wall on which the turnout slide gate is fastened. Design practices call for a flexible pipe joint at one-half the pipe diameter or a minimum of 457 mm (18 in) from the outside face of the wall to accommodate differential settlement. For severe frost heave, this joint may not be sufficient to prevent breakage of the pipe at the outside face of the wall. A break may cause an erosion cavity (fig. 34), and remedial action may be required to prevent frost heave. Insulation behind the headwall prevents heat loss through the concrete and prevents the formation of ice lenses (fig. 21).

Pipe Crossing on Piers

In April 1974, a 600-mm (24-in) steel pipe, near the Thomas Point Pumping Plant crossing the main canal of the Lower Yellowstone Project in Montana, was observed to have been damaged by frost heave. The pipe crosses the canal on two piers located in the canal section. Because of differential heave between the piers, the pipe was dislocated about 76 mm (3 in) at a joint (fig. 35). The maximum dislocation had been about 152 mm (6 in) during the preceding winter. During the preceding 1973–74 winter,

(a) Original canal with frost damaged chute and drop structures on right. 1236-617-1984

(b) New chute. 1236-617-1983

Figure 31.—Pilot Canal on Riverton Unit in Wyoming. October 1975.

Figure 32.—Drop structure with inadequate weep holes and impervious backfill that resulted in frost damage. Lower Turnbull Drop, Sun River Project. P801-D-79858

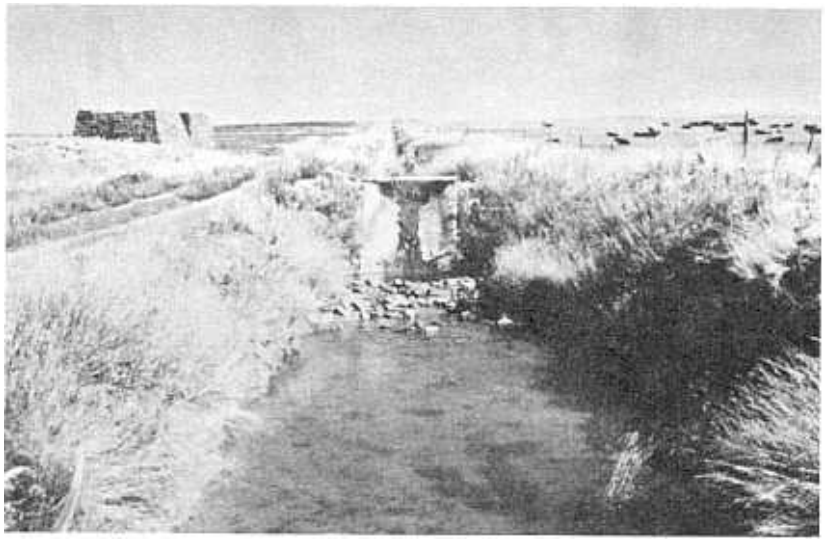


Figure 33.—Drop structure susceptible to frost action to be replaced by pipe. Lateral G.M. 58, Sun River Project. P801-D-79859

Figure 34.—Break between outlet structure and pipe resulted in a cavity forming behind headwall at pipe turnout. Lateral 23.2, Riverton Unit. P801-D-79860

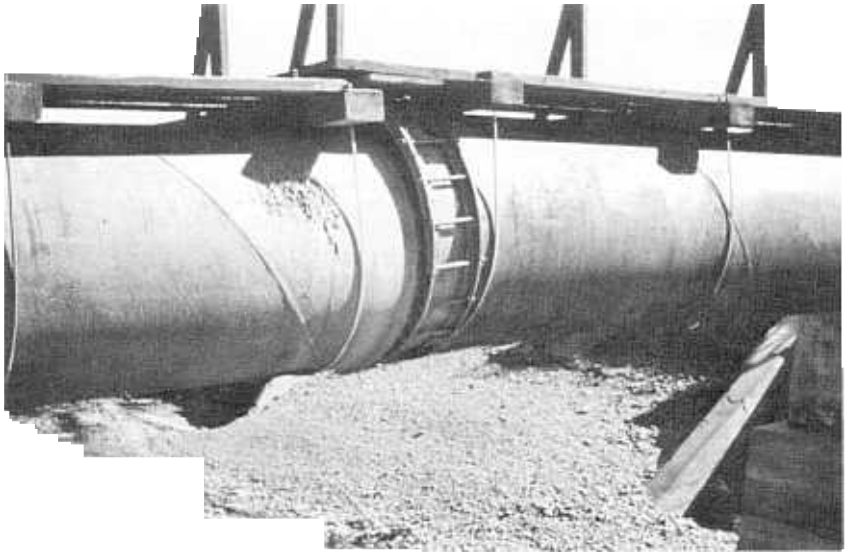


Figure 35.—Displacement at pipe joint due to differential heave between supporting piers. Lower Yellowstone Project in Montana. P801-D-79861

there was no water in the canal when the serious frost heave damage occurred. In previous years, the canal had been operated at partial capacity during the winter and frost heave was not a problem.

Canal Lining

Concrete.—A problem of uplift on concrete canal lining was found on the Steinaker Feeder Canal in the Central Utah Project. The cause was attributed to either frost action or hydrostatic uplift, or a combination of both. The installation of a longitudinal drain was recommended as a satisfactory corrective measure. This canal damage is discussed in more detail in appendix A.

Problems with frost heave on concrete canal linings have been experienced on the Riverton Unit in Wyoming since their construction. The problems were aggravated because the quality of the concrete and construction methods would not meet today's established high standards. These problems, along with damage to other canal structures, are discussed in appendix A.

Damage to concrete canal linings has also been a continuing problem in Montana where frost is often present at depths of 1.5 to 1.8 m (5 to 6 ft). Frost heave and the resulting settlement after the spring thaw are commonplace, not only because winter temperatures may plummet to −40 °C (−40 °F), but also by the fact that in many areas of Montana, varying temperatures cause many freezing and thawing cycles each year. Since a majority of Bureau structures are located on irrigation projects where the groundwater table remains at a level close to the ground surface, the structural foundation soils have a tendency to remain wet. For larger canals where concrete lining is a necessity, better drainage techniques are being used to lower the water table away from the structures and dry out the backfill.

The use of sands and pea gravels behind the lining has a decreased capillary effect on the ground water, thus reducing ice lens formation and consequently frost heave damage. It has also been pointed out by field personnel that although the first cost of overexcavating for lining and placing select permeable backfill behind the concrete lining is expensive, the alternative of having to replace the lining after the canal is in service is more expensive. The canal can only be dewatered in the off-growing season (winter months), which is the worst time to place concrete. Poor concrete results in more failures and very soon the initial savings are depleted and the price of labor and materials has gone up. Therefore, in cold climates, the initial cost of select material for foundation and backfill is a very good investment.

Canals oriented in an east-west direction, where the south side is shaded from the sun, have more damage on the south side. This damage is due to the fact that the shade-side lining remains frozen to the soil and is not free to move as the rest of the soil foundation thaws. This is illustrated by the broken lining on Fivemile Lateral of the Riverton Unit (fig. 36), placed in the fall of 1978. In this case, the entire lined section heaved uniformly during the unusually cold 1978–79 winter and the fracturing occurred during a 2-week period in March 1979. The ice lenses in the soil under the north (sun-exposed) side thawed faster than those on the south side and the differential settlement caused the lining break. The maximum offset at the break was about 64 mm (2.5 in), which equalled the lining thickness. After the frost was out of the ground, the lining settled and the offset was reduced to about 25 mm (1 in). The lining was a total loss and had to be replaced.

Buried Plastic Membrane.—On February 15, 1977, a slide occurred on the PVC- (polyvinyl chloride) lined right bank of Pilot Canal of the Riverton Unit between stations 18+593 and 18+654 m (610+00 and 612+00 ft) (fig. 37). This section of canal has a water depth of 3.0 m (10 ft), bottom width of 9.1 m (30 ft), and 2 to 1 side slopes. The 254-μm (10-mil) thick PVC lining was placed in November 1976 to replace a deteriorated asphalt membrane lining. The cover over the membrane consisted of 230 mm (9 in) of soil from canal excavation and was superimposed with 230 mm of pit-run gravel. The lining was torn below the anchored top portion and slippage occurred between the membrane and the soil underneath. Observations of an adjacent section revealed ice under the lining (fig. 38). Also, for about a 50-mm (2-in) depth beneath the lining, there was a series of ice lenses from about 1 to 3 mm (1/16- to 1/8-in) thick, alternating with layers of soil, which is typical of frost action in soil. The soil beneath the membrane was a sandy clay with a liquid

Figure 36.—Breakage of concrete canal lining during spring thawing period. Fivemile Lateral, Riverton Unit. P801-D-79862

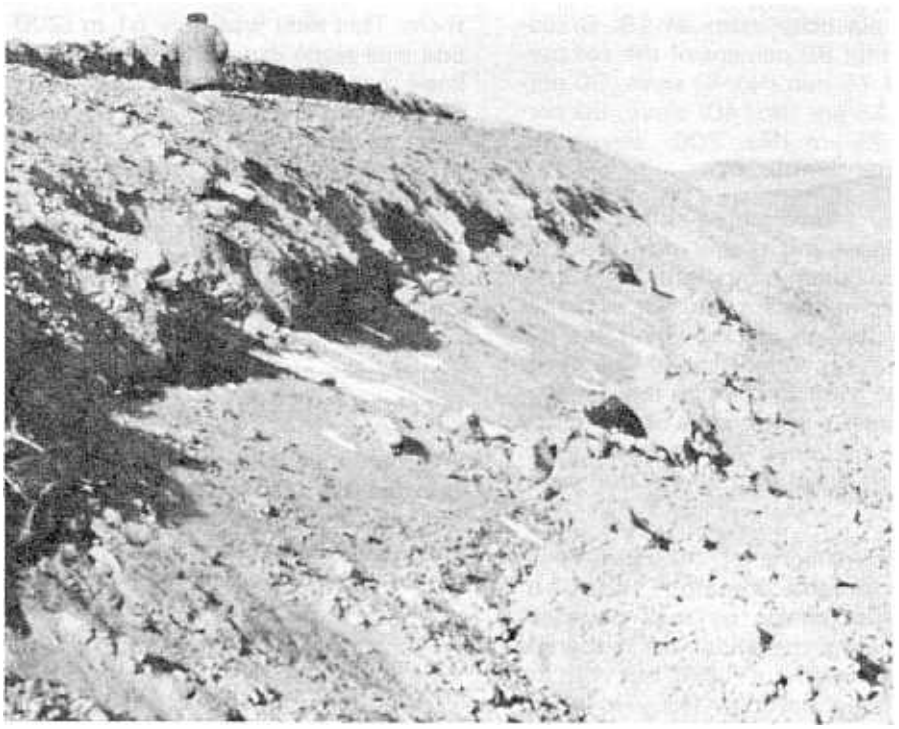


Figure 37.—Slippage of PVC membrane lining on Pilot Canal, Riverton Unit. Beyond the torn lining at extreme left, patches of ice can be seen. P801-D-79863

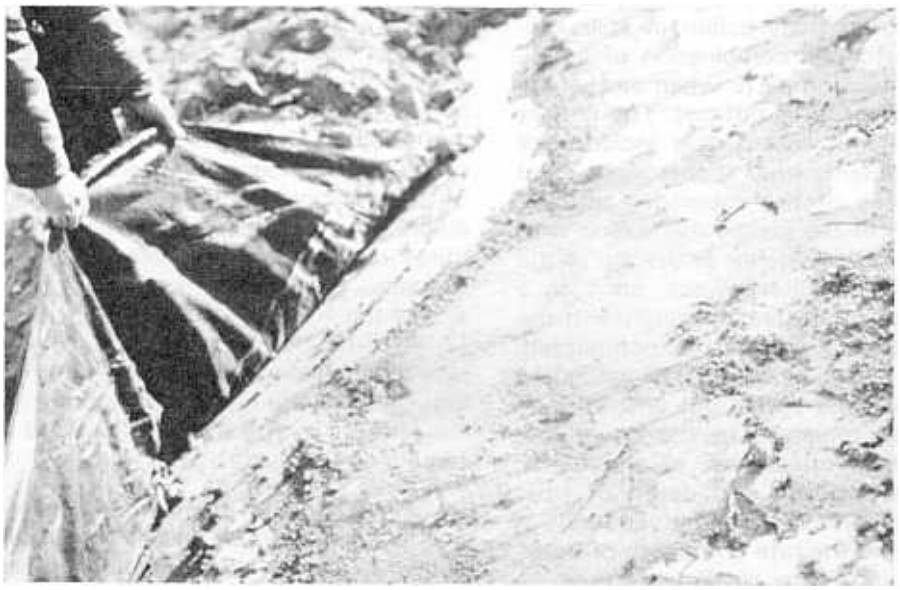


Figure 38.—Ice accumulation beneath PVC membrane lining near slippage area. P801-D-79864

limit of 37 and a plasticity index of 19. Gradation data showed that 96 percent of the soil particles passed the 4.75-mm (No. 4) sieve, 90 percent passed the 425-μm (No. 40) sieve, 60 percent passed the 75-μm (No. 200) sieve, and 25 percent was finer than 5 μm.

The record of daily temperatures shows an unseasonable warming trend a few days prior to the slide, with a maximum temperature of 13 °C (55 °F). A plausible reason for the slide was that thawing occurred through the soil cover and the membrane. The melting water beneath the membrane was not able to drain through the underlying ice, but did form a water-lubricated surface with very little frictional resistance, which caused the slide to occur just under the membrane.

Prior to the slide, the subgrade for the PVC lining was lightly rolled to form a smooth surface to avoid puncturing the plastic by rock particles. For construction after the slide, the subgrade surface was dragged without rolling but with the removal of projecting particles likely to cause puncturing. Also, at the option of the contracting officer, sand was required to be placed below the lining to increase frictional resistance. For large canals subject to similar conditions, a 508-μm (20-mil) thick lining is now recommended, and slopes flatter than 2 to 1 are considered.

The occurrence of a slide of this nature, as with many problems from frost action in soils, depends upon a fortuitous combination of conditions, and any prediction as to when and where such a slide would occur is difficult. The factors influencing stability against sliding include soil type, moisture content, frost susceptibility, and surface smoothness of the subgrade; smoothness and strength of the plastic membrane with the effectiveness of the anchor at the top of the slope; presence of a water source, such as a water table; mass and shearing resistance of the cover soils, which are usually not compacted; amount of snow cover which affords insulation tending to reduce frost action but which, upon melting, may add moisture to the cover and reduce its stability; orientation of the canal, which influences the rate and depth of frost penetration and thawing caused by variations in solar insolation; and the rate and depth of freezing and thawing caused by air temperatures.

If such slides continue as small isolated incidents, it may not be economical to take extensive measures along an entire canal to prevent them. This slide was only 61 m (200 ft) long on one side slope out of 1680 m (5512 ft) of fully lined canal in that particular lining contract. Small slides can readily be repaired in the spring prior to diverting water into a canal, however, for more critical areas where repair delays would be costly, special measures during construction may be justified. For example, for a membrane-lined reservoir in a very cold area where the water level fluctuates widely, there may be recurrent opportunities for the right combinations of conditions for a slide to occur. In this example, there is the added effect of reservoir water temperature to influence thawing.

For preventive measures, in addition to a layer of selected sand under the membrane to increase frictional resistance and break the capillarity to prevent formation of an ice layer under the membrane, it would be well to prevent such a buildup of ice in a smooth interface between the subgrade and sand layer. This could be done by causing an uneven subgrade surface by compacted indentations in the subgrade surface formed by appropriate equipment before placement of sand.

Compacted Soil.—From 1953 to 1963, under the Lower Cost Canal Lining Program, density tests were made periodically in about 25 compacted earth canal linings. Since a major concern was the effect of climatic conditions, particularly freezing and thawing, causing a decrease in density of the linings with a resulting increase in seepage, many of these linings were selected in areas sufficiently cold for frost action to occur. In addition to density tests, field permeability and seepage tests and laboratory permeability tests were conducted in connection with some of these linings. During 1976 through 1978, additional density tests and observations were made on eight of the selected linings in cold climates. These tests have shown that the density of the linings, which are from 0.6- to 0.9-m (2- to 3-ft) thick, changes from year to year, sometimes increasing and sometimes decreasing. Most of the change is in the top portion of the linings, with the density toward the bottom remaining relatively stable.

In a few instances, the overall density of a lining had increased slightly over a period of years. As shown by the results of laboratory freezing tests on specimens of canal lining soils (fig. 39), freezing at the top exposed end of the specimen causes a migration of moisture and decrease in density toward the top of the specimen. When

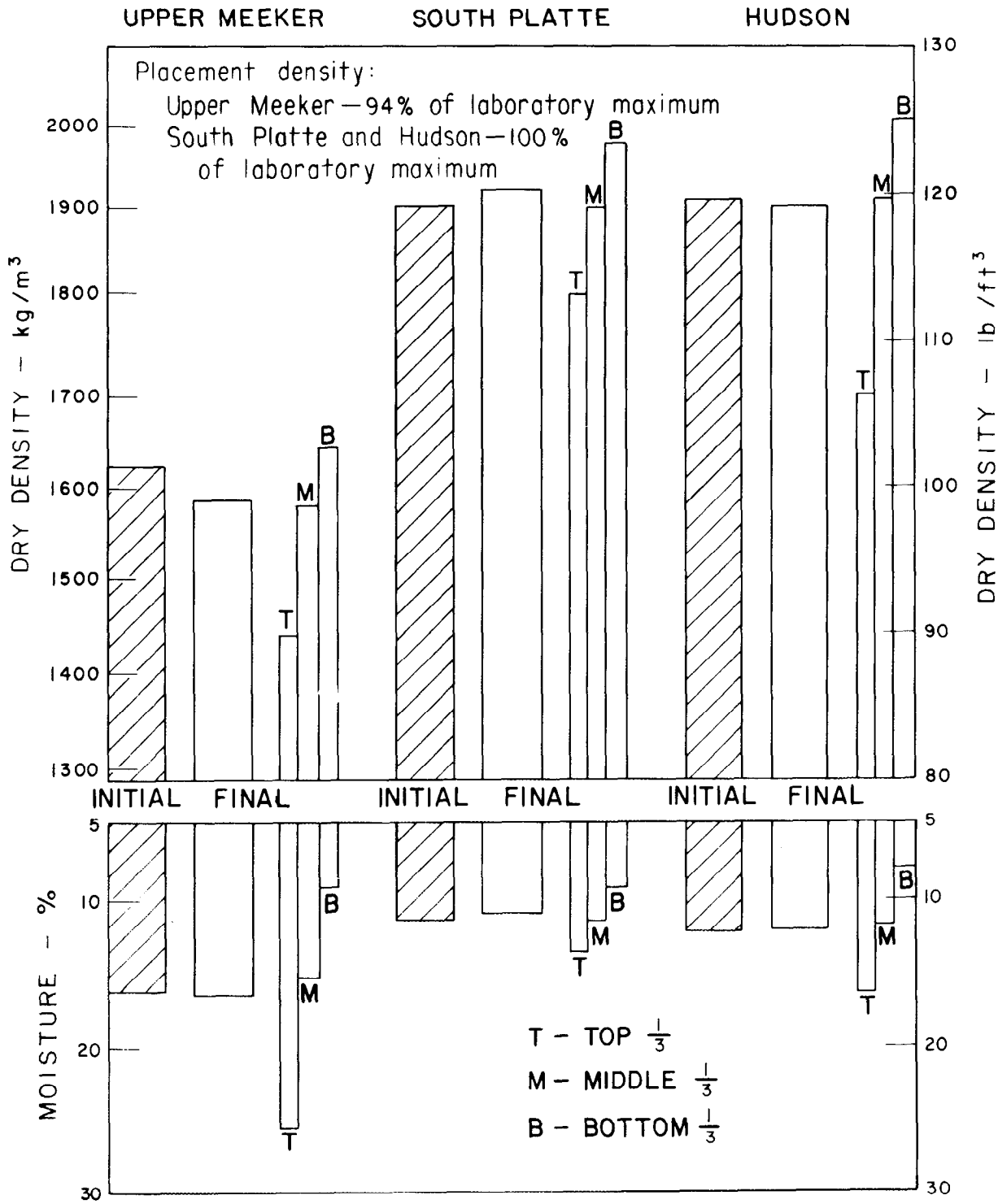


Figure 39.—Density and moisture contents of canal lining soils before and after closed-system freezing tests.

no outside source of water was made available, the moisture content decreased and the density increased toward the bottom of the specimen. Freezing must have this same effect on compacted earth lining during the winter when there is not a source of water below the lining. Apparently, after thawing of the soil in the spring, the moisture and density may not redistribute to prefreezing conditions. This is possibly due to evaporation of moisture from the soil surface. It seems likely that this natural process may, at least in some cases, actually help maintain density toward the bottom of the lining when there is good drainage to a sufficient depth below the lining. For more information on this subject, see reference [2].

There have been conditions of high ground water, high capillarity in the soil, and prolonged cold weather without appreciable snow cover where ice lenses would form in the compacted soil lining and cause it to deteriorate. The only noted instance of this problem on the selected test reaches was on a short reach of the Boulder Creek Supply Canal in Colorado where the canal was located on a hillside and emerging ground water on the hillside was evident. There was a high loss of soil density and some sloughing of the lining.

Earth Embankment Dams

In Bureau specifications for earth dam construction, the standard reference prohibiting frozen soils in embankments is often a statement similar to the following: "No embankment material shall be placed in the embankment when either the material or the foundation or embankment on which it would be placed is frozen." Design and construction personnel should be aware of the effects of freezing on soil properties and of corrective actions necessary to ensure construction of a safe dam.

For at least 1 month before and after a winter shutdown, the collection of weather information and observations of embankment conditions, including contractor operations affecting soil temperatures, are important. A daily record of weather conditions at the elevation of concern on the dam should be maintained. This record should include maximum and minimum air temperatures and precipitation. Also, representative snow depths should be obtained at intervals to show significant changes and cumulative depths. In addition, observations should be recorded of areas where snow is completely or partially removed by drifting or by contractor operations. Since snow is an insulator, it can greatly affect frost penetration. In extremely cold areas, artificial snow has been blown on earth embankments to reduce frost penetration and to speed thawing in the spring by virtue of the warmer soil below the frost line.

In areas where soil is susceptible to frost action and deep frost penetration is expected, it is advisable to monitor frost penetration in selected locations critical to the safety of the dam. This can be done with the frost tubes previously described (fig. 8). These tubes should be installed in the fall, and frost penetration can be recorded at intervals during the winter. See appendix C for frost penetration measurements and collection of related data.

Before resumption of embankment placement in the spring, a thorough exploration for frozen soil in the embankment should be made and this information should be incorporated in the construction record. This can be done by exploring in a definite pattern with an auger or other exploration tool. However, in compacted embankment the unfrozen soil may be so dense that the dividing line between frozen and unfrozen soil is not distinguishable. It would then be necessary to obtain soil temperatures at short depth intervals either by measurements in the exploration hole or on cuttings immediately upon removal from the hole. Areas that are completely or partially shaded during winter, where snow depths are less than 30 cm (1 ft), and in critical areas such as those adjacent to abutments, should receive special attention. For example, a roadway crossing the dam and maintained free of snow would result in a deeper frost penetration than surrounding snow-covered areas.

Since freezing of soil results in a lower density in the vicinity of the ground surface, density tests should be conducted in embankment placed during the preceding fall and the soil recompacted as necessary to meet minimum specifications limits. This would eliminate a low density area that might result in a high permeability and thus produce conditions favorable to piping and possible loss of shear strength which would affect stability. Density test sites should be carefully selected to be representative of the embankment and their locations should be influenced by the exploration for frost.

Since frost penetration can be up to two or more times as deep in rock as in soil, special attention

should be given to the density of frost-susceptible soil at depths adjacent to rock abutments; this would be an area particularly critical to the safety of a dam because of soil erosion, hydraulic separation, or hydraulic fracturing. Particular attention should be given to note any sources of water such as springs, overflow from grouting operations, or melting snow which would cause water to seep into the embankment where it would freeze. Also, adequate density of soil across a key trench and in bonding to rock is very important. Any results of density tests made in embankment placed during the preceding fall should be so indicated in the monthly construction control report.

The contractor should be made aware that density tests in embankment placed in the fall will be conducted in the spring prior to the resumption of embankment placement because the contractor is responsible for recompaction of any soil not meeting specifications. The contractor can then arrange construction operations most efficiently. For example, the contractor could avoid removing the insulating snow cover during cold weather, or place a layer of loose soil over the surface of the dam for insulation and remove it in the spring.

When soil is placed in cold weather with the soil and air temperatures near freezing, soil temperatures should be recorded and a daily examination made to ensure the soil does not become frozen after placement. A contractor once placed backfill when the soil temperatures were only a few degrees above freezing, but the following spring, when earthwork operations resumed, the backfill was frozen about 5.8 m (19 ft) deep. The material had been placed at approximately 1.5 percent dry of optimum moisture, and a loose lift of material had been placed over the compacted backfill each night. During placing operations, air temperatures were barely above freezing for 4 to 6 hours during the day and fell well below freezing at night. For some reason, such as unobserved ice crystals in the soil which joined upon compaction, supercooling effects, or subfreezing night temperatures, the soil became completely frozen. Heat loss in soil and the resulting freezing action increases significantly as the degree of soil compaction increases; unfrozen soil in a borrow area may freeze when placed in a compacted fill.

Personnel in charge of earthwork construction in cold climates should receive training, where necessary, to understand the principles of frost action in soils and should make a concentrated effort to avoid or correct detrimental effects. The consequences of overlooking these effects in critical locations in a dam could result in its failure. For more information on this subject, see reference [2].

REVIEW OF NEW TECHNIQUES TO CONTROL FROST ACTION

Polystyrene Insulation

Under Pavements.—During the past 15 years, various synthetic insulation materials have been developed. Probably the most successful insulation for control of frost penetration under highways and airfields has been a closed-cell polystyrene manufactured by Dow Chemical Co. under the brand name of Styrofoam HI or Styrofoam SM. This is a rigid-type insulation manufactured in sheets that are either 0.6 or 1.2 m wide by 2.4 m long (2 or 4 ft by 8 ft), and in thicknesses of 20, 25, 38, 50, and 76 mm (¾, 1, 1½, 2, and 3 in). This insulation has a water absorption of less than 0.5 percent by volume when immersed in water, and a maximum thermal conductivity of 0.03 W/(m·K), or 0.2 Btu·in/(h·ft^2·°F).

For use as highway insulation, the polystyrene is usually placed on subgrade and covered with gravel subgrade and then pavement. The thickness is based on the design freezing index. About 25 mm (1 in) of Styrofoam is required for each value of 556 °C·d (1000 °F·d) for the freezing index. In general, the polystyrene insulation has been effective in controlling frost penetration and resulting frost heave of pavements where adequate thicknesses for the climate of the area have been used. However, under certain weather conditions, the insulation tends to cause icing on the pavement surface because the insulation prevents ground heat from melting the ice. This problem has limited its use as a pavement insulation. A summary of the experience with polystyrene insulation for airfield and highway pavements, and results of field tests are given in appendix B.

On Bureau Projects.—Polystyrene insulation has been occasionally used in Bureau construction. In March 1977, 50-mm (2-in) thick Styrofoam was placed around a CHO type 4 pipe chute turnout at station 20+49 m (67+22 ft) on Sand Butte B Lateral of the Riverton Unit (fig. 40). The pipe in the turnout had a diameter of 600 mm (24 in), and the base of the concrete inlet structure was about 1.2 by 2.4 m (4 by 8 ft). Observations on this turnout since installation have shown no differential movement between the pipe and the concrete inlet. A suggested design for the insulation of a pipe turnout is shown on figure 41.

As a part of the repair of the spillway on Lemon Dam (app. A), a 76-mm (3-in) thickness of Styrofoam was attached to the outside of the new wall (fig. 29) to prevent frost penetration into the soil backfill. For more on this repair, see Bureau specifications No. DC-7018.

Another installation of Styrofoam insulation was on reach 4B of the McClusky Canal. This work was on the 128-m (420-ft) long fish and trash structure (fig. 42) at station 1057+82 m (3470+53 ft). The structure contains a very fine mesh fishscreen which was thought to be in danger of freezing during the winter. The structure has insulation on the top and sides to a 2.1-m (7-ft) depth below the ground line, and a coarse gravel backfill with a drain outlet at the lower end of the structure.

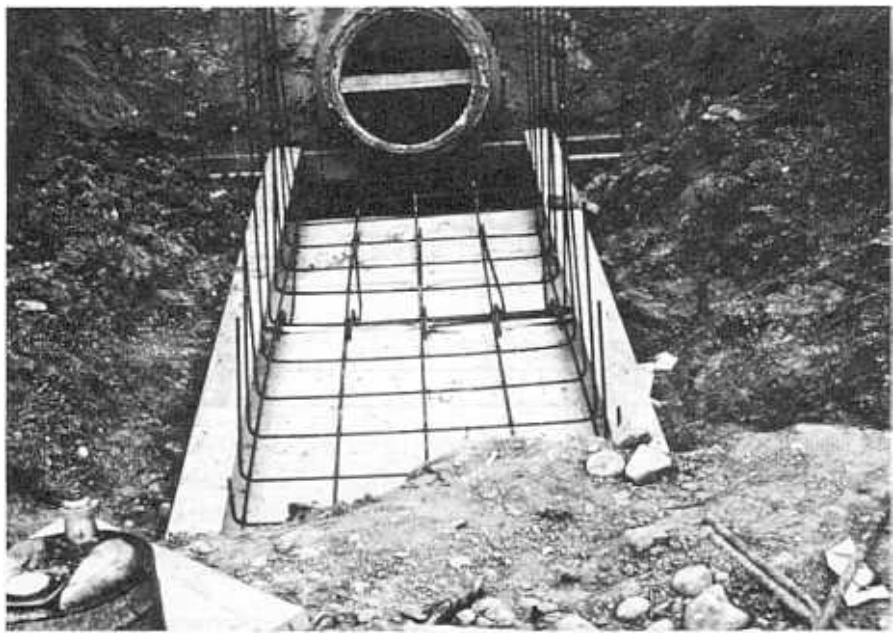
A short length of the Minot Pipeline Extension in North Dakota was insulated with Styrofoam placed between the ground surface and the pipe in a critical area. For more information, see Bureau specifications No. DC-6977.

Presently, there is extensive rehabilitation underway on the Riverton Unit in central Wyoming. About half of this $19 million rehabilitation is for land drainage and the balance is for the replacement of concrete canal structures. Many of the original structures were damaged by frost heaving. This rehabilitation, which is expected to require 5 to 7 years for completion, has afforded an excellent opportunity to study frost action on hydraulic structures and use new approaches toward controlling frost heave. From 1975 to 1978, cooperative research among personnel of the Upper Missouri Region, the Riverton Unit, and the Engineering and Research Center was performed. The research consisted of measuring frost penetration and structural heave under the soil moisture and climatic conditions at Riverton, field trials with Styrofoam insulation to reduce frost penetration, and measurement of frost penetration in the soil foundation beneath concrete canal linings.

In November 1975, the inclined rectangular drop structure No. 83 on the Lost Wells Lateral was selected as a test site [12]. This is a small concrete structure 1.37 m (4.5 ft) wide and 10.7 m (35 ft) long. The structure was built in 1946–47, and was in good condition with only a few hairline cracks. It was located in a known frost heave area with a borderline clayey foundation of fine sand-lean clay (SC-CL). The water table was about 1.5 m (5 ft) below the ground surface. Frost penetration was measured beneath the structure and in uninsulated and Styrofoam-insulated areas adjacent to the lateral. No attempt was made to insulate the structure itself. Temperature measurements were made at intervals to a 1.5-m (5-ft) depth by a thermocouple system and with methylene dye-filled plastic tubes (fig. 8).

Based on National Weather Service temperature data, the 1975–76 winter was about normal. Snowfall was light and intermittent with a maximum snow depth of about 10 cm (4 in), which was not considered enough to significantly affect frost penetration. Elevation measurements showed the structure heaved about 25 mm (1 in) rather uniformly and no new cracks were observed from this heaving. The frost penetrated about 0.6 m (2 ft) below the 152-mm (6-in) thick concrete floor of the chute. In an uninsulated area near the structure, the frost penetrated about 1 m (3.2 ft). In a comparable area with 25-mm thick Styrofoam insulation and covered with 152 mm of loose natural soil, the frost penetration was about 200 mm (8 in) below the insulation. In another area with 50-mm (2-in) thick Styrofoam insulation covered by 152 mm of soil, frost did not penetrate through the insulation. The test site was left undisturbed and was not affected by the 1976 irrigation operation. Measurements were taken at the site during the 1976–77 winter and the results were quite similar to those for the preceding winter.

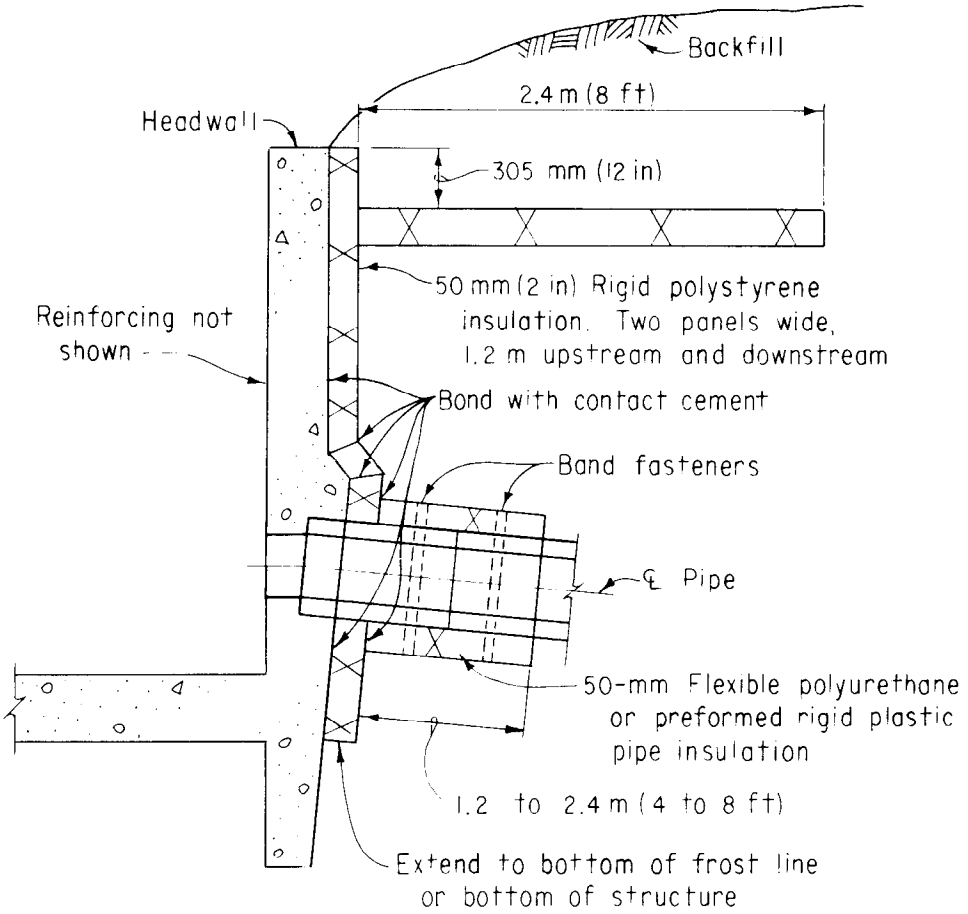
In October 1976, a second frost investigation test site was established on the Riverton Unit [13]. This site was on a reach running east and west on the small lateral No. 15.1 located about



(a) Insulation under turnout. P801-D-79865

(b) Insulation outside the wall of turnout. P801-D-79866

Figure 40.—Styrofoam insulation used on pipe chute turnout. Sand Butte B Lateral, Riverton Unit.



Extend insulation 1.2 m (4 ft) each way from the pipe ℄ or all the way around the turnout box for small turnouts.

Figure 41.—Suggested insulation design for a pipe turnout to prevent frost heave damage.

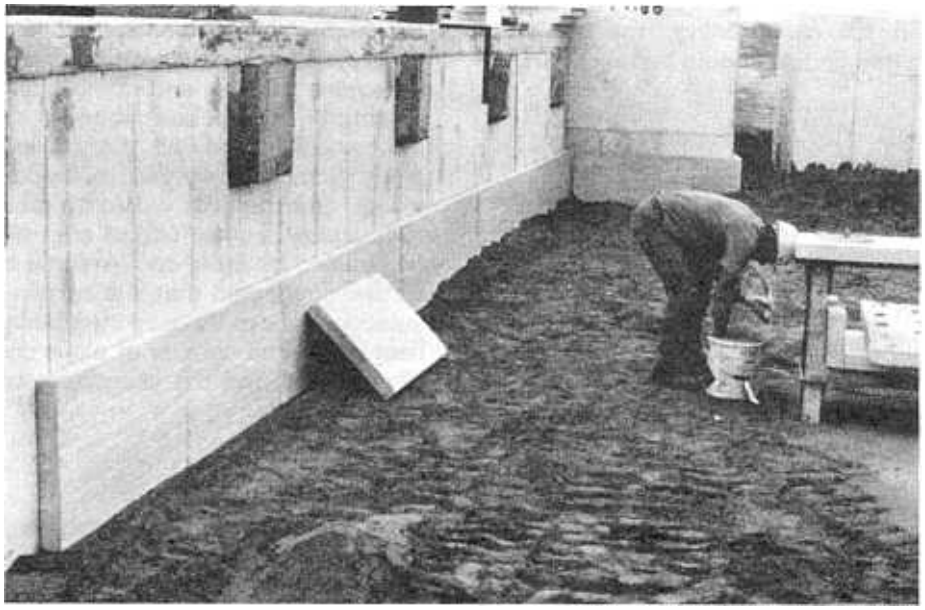


Figure 42.—Installation of styrofoam insulation on outside wall of fish and trash structure on McClusky Canal. July 27, 1976. C769-603-7343NA

1.6 km (1 mi) from Pavillion, Wyo. This was a known frost heave area where the lateral had been lined with 64 mm (2½ in) of unreinforced concrete during the spring of 1975. A thermocouple system was installed to measure temperatures beneath the lining which was insulated with 50-mm (2-in) thick closed-cell polystyrene, for comparison with an instrumented uninsulated area nearby. The insulation was fastened with a mastic on top of the already constructed lining to show what frost protection might have been afforded had it been placed beneath the concrete, which would have been the normal procedure during construction. Thermocouples beneath the insulated concrete on the north, sun-exposed slope showed that temperatures during the winter of 1976–77 reached 0 °C (32 °F). The temperatures on the south, more shaded slope reached −1 °C (31 °F) under the concrete. In contrast, on an uninsulated section of lining nearby, the minimum temperatures beneath the concrete were −7 °C (19 °F) and −9 °C (16 °F) on the north and south slopes, respectively. Adjacent to the lateral, frost depths measured with methylene dye-filled frost tubes showed the depth in a tree-shaded area was over 1.4 m (4.6 ft) (the limit of the gage) compared with 0.7 m (2.3 ft) for an unshaded area.

During the 1977–78 winter, the frost test site on lateral 15.1 was modified so a passive solar experiment could be performed [14]. A section of the concrete lining on the lateral was painted black, and temperatures were measured on the surface and in the soil beneath the lining for comparison with similar measurements on an adjacent unpainted surface. The surface painted black would show any effect of radiation energy absorption from the sun in reducing frost penetration beneath the lining. An analysis of the temperatures showed that on the sun-exposed side of the lining, frost penetrated about 0.5 m (1.6 ft) beneath the black-painted lining compared to about 0.7 m (2.4 ft) for the unpainted concrete. Thus, frost penetration was reduced by about one-third. However, for the black-painted concrete on the shaded side of the lateral, frost penetration was about 1 m (3.3 ft) compared to about 0.7 m for the unpainted concrete. This increase by about one-third was due to long-wave radiation causing excessive heat loss from the black surface at night.

Although the use of polystyrene insulation or a black surface on concrete would probably not be economical on a canal-wide basis, it could be considered in areas where frost would be expected to be a particular problem. If polystyrene

is considered for canal lining, the effect of buoyancy on thin linings should be taken into account.

Heat Pipes

The Federal Highway Administration and others have experimented with ground (earth) heat pipes to raise the temperature of highway pavements to melt ice and snow in critical areas such as on- and off-ramps and bridge decks [23,24]. A heat pipe, as used for this purpose, is a pipe about 25 mm (1 in) in diameter, 9 to 12 m (30 to 40 ft) long, and closed at both ends. A volatile fluid, such as ammonia, is placed in the pipe. The pipe is then buried in the ground with the top embedded in the pavement to be heated. The heat from the ground volatilizes the ammonia which condenses and gives off heat when it reaches the cool upper end of the pipe in the pavement. The condensed liquid flows by gravity to the lower end of the pipe to complete the cycle. The heat pipe is a simple and effective heat transfer device that has a long life and is free of maintenance.

The possibility of heat pipes to control frost around canal structures was discussed with representatives of the Gruman Aerospace Corporation which manufactures ground heat pipes. Using the characteristics of a typical canal chute, they made some rough calculations to determine the size and number of pipes required to control frost action around a canal structure. This analysis showed that a large number of pipes would be required to produce the desired effect, and the cost would be prohibitive except for possibly a small critical area which could not be handled by more conventional methods. They further suggested that if a certain amount of insulation were to be used in combination with the heat pipes, the number of pipes could be drastically reduced and the economics would be more favorable.

Other Techniques

Other techniques for supplying heat to the ground around canal structures to combat frost have been suggested. One is for a wind-powered generator to provide electricity for heating electrical resistance wires placed in the soil foundation. Another would employ a solar system with a collector to supply warm air or a winterized fluid through pipes in the foundation. The use of dark paint on structural concrete to better absorb solar radiation is another possibility. Presently, these other techniques do not seem practicable from economic and maintenance standpoints for application on a large scale.

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APPENDIX A

STRUCTURAL DAMAGE FROM FROST HEAVE

Spillway on Vallecito Dam

The radial-gate-controlled chute spillway located on the right abutment of Vallecito Dam is founded principally on morainal deposits. Spillway and outlet releases pass through a common basin, with a maximum stilling capacity of 850 m^3/s (30 000 ft^3/s), which is located about 305 m (1000 ft) downstream from the toe of the dam. The damage to the spillway basin area occurred sometime during the first 10 days in April 1973. Apparently, two panels of the left spillway wall in the basin area and the upper portions of six counterforts were jacked inward by expansive freezing action and fell into the basin. They came to rest in a nearly horizontal position on top of the basin floor baffles and sill. The bottom limit of the damage appeared to be the approximate winter water level in the basin. Below this level, the wall appeared to be plumb, and as far as could be determined, undamaged. The break progressed upward from the basin side and through the wall and counterforts at an angle of 30 to 45° from vertical. The backfill, left standing at about a 1¼:1 slope between the counterforts, was frozen and appeared to be completely saturated impervious soil. The right basin side wall was intact except for a noticeable deflection which damaged the fence above it. Contraction joints and a well developed crack pattern both showed substantial mineral deposits and seeps, indicating high backfill saturation levels.

Since the main problem appeared to be saturation of the backfill, underdrains were installed to reduce moisture in that zone. The existing backfill material was replaced with 13- to 6-mm (0.5-to 0.25-in) gravel with an underdrain pipe system. The drains outletted through the wall at an elevation below the water surface in the stilling basin to prevent freezing of the drain outlet. Another possibility would have been to extend the underdrain downstream beyond the extent of the basin. This latter solution would have been more expensive, but would have been a more permanent solution to the problem since flap valves often become ineffective because they do not close due to corrosion or accumulation of debris.

An impervious layer of soil was placed on the top of the backfill. This soil layer contained clay, silt, sand, and gravel which later developed severe shrinkage cracks that allowed surface water to flow into the backfill. An impervious soil blanket for this purpose should be carefully designed to prevent such shrinkage. An alternative to prevent the penetration of surface water would be to use a flexible membrane lining such as asphalt, or 508-μm (20-mil) polyethylene plastic sheets covered by 254 to 305 mm (10 to 12 in) of gravel instead of an earth blanket.

Spillway on Lemon Dam

The spillway inlet walls on Lemon Dam are 7-m (23-ft) high cantilevered concrete walls ending in curves. In 1963, the spillway was built and backfilled with impervious soil which later became saturated with surface water. In 1966, minor deflections at the joints were first noted. In 1967, deflections up to 140 mm (5.5 in) at the top of the left wall and 114 mm (4.5 in) on the right wall were observed (fig. A-1). Original backfill material was replaced at that time with pervious earth 305 mm (12 in) out from the wall to a depth of 2 m (6.5 ft) below the top of the wall. In December 1970, over 50 mm (2 in) of rain fell and thoroughly saturated the backfill. In January 1971, unusually low temperatures were recorded. From January 4–8, temperatures did not rise above −6 °C (21 °F) and dropped to −55 °C (−67 °F) one night. At that time, additional deflections of about 38 mm (1.5 in) were noted.

On April 9, 1973, two panels situated on opposite sides of the spillway entrance channel and immediately upstream from the free overflow crest were observed to be severely deflected. The abutment counterforted walls on the downstream side were in good condition. The base of the walls could not be examined because of snow and ice, but 1971 photographs showed compressive failure spalling at the conjunction of the wall stem with the base on the channel side. The walls were backfilled with impervious embankment which was reportedly saturated by percolating surface water. Corrective drainage features constructed in 1967 had been ineffective because deflections had doubled since their installation. It was believed that water was freezing at the drain outlet, thus blocking any further drainage of the

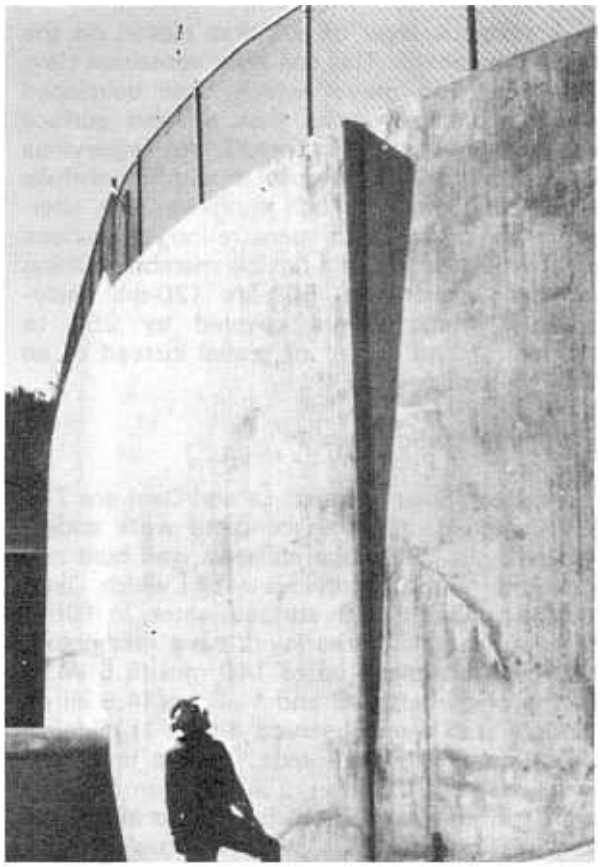

Figure A-1.—Displacement of right spillway forebay wall panel on Lemon Dam by frost action in backfill. P801-D-79867

percolating surface water. The deflection of the two walls had been progressive since completion of the structure in 1963.

In 1971, there was evidence that freezing action was jacking the walls into the channel and that failure would result if corrective measures were not taken. Because of the location of these walls adjacent to the crest of the spillway, the safety of the dam could be affected and repairs would be needed during the summer of 1973. It appears that if some of the backfill material in the top layers along the wall had been more pervious and had a larger cross-sectional area of drainage at the outlet, the effect from freezing would have been minimized through better drainage. In May 1973, the second panel on the left spillway entrance collapsed into the spillway. This failure was probably due to water collecting behind the wall because ice plugged the drain outlets. Unfrozen seepage water in the backfill became available for the formation of ice lenses. The walls had to be repaired before the next flood season. Small deflections had also occurred on the inlet walls, serving as bridge abutments, causing the top pipe on the chain link fence to fail.

Corrective drainage features included underdrains with 100-mm (4-in) clay floor pipe emptying into the stilling basin. These drains, located at 1-m (3-ft) centers, emptied through the stilling basin wall as before, but electric heaters were added to prevent freezing at the outlet. The drains were left ungated to allow seepage water to flow freely in and out of the embankment. Rigid plastic insulation, 76 mm (3 in) thick, was placed on the back of the wall to a height of 1.4 m (4.5 ft) above the upper set of pipe drains. The gravel pockets around the drain outlets measured 305 mm (12 in) radially out from a 102-mm (4-in) screen placed on the end of the pipe drain. The gravel pockets were surrounded by uncompacted pervious backfill. The backfill replacement was from 6 to 7 m (20 to 24 ft) below the top of the stilling basin wall.

The Chief of Water and Lands Division at the Bureau's project office in Durango, Colo., reported in March 1976 that drainage envelopes at both Lemon and Vallecito Dams appeared to be operating satisfactorily. Wells installed in the drains on Lemon Dam have had water depths the same as in the stilling basin; this indicates drainage in the backfill is occurring in about the same time span as the water surface fluctuation in the spillway stilling basin. The Reservoir Superintendent at Vallecito Dam reported that −9 °C (15 °F) was the minimum temperature at the spillway in the winter of 1976. Both managers confirmed that the sequence of events leading to the failures at Vallecito and Lemon appear to have been caused by extremely heavy rainfalls followed by excessively cold temperatures when the ground around the structures was still saturated. This combination of moisture in the frozen ground plus frost-susceptible backfill causes the maximum amount of frost heave damage.

Spillway on Dickinson Dam

This spillway problem was diagnosed as a clogged drain and freezing of the filter. Inspection below station 2+81 m (9+22 ft) showed the drain to be partially blocked by soft unconsolidated clay and silt. The filter blanket in the

original construction consisted of a 457-mm (18-in) layer of a pit-run, sand-gravel mixture. Construction photographs show that the filter blanket was consolidated by an internal-type concrete vibrator while water was applied from a hose. This procedure probably caused segregation of coarse and fine particles. Analysis of the filter following the failure showed that it had picked up some fines. Obviously, the spillway panel heaving was the result of frost action in the nondraining underlaying filter blanket. The failure was described:

> "Slabs below the spillway crest and above the stilling basin raised one at a time over a 12-hour period, allowing approximately a 1-m (3-ft) depth of water to flow underneath the slabs. On April 7, 1954, during the spring thaw, the slabs became dislodged. It was impossible to shut off spillway flow during the spring runoff to minimize the damage; however, water was diverted out of the damaged area as soon as possible by using a sandbag barricade. The repair of the damaged area included the crest blocks since erosion extended slightly underneath them in the vicinity of the slab barrier. Also, under the left training wall toe slab, channels had been cut under the cutoff wall at station 2+63 m (8+64 ft)."

Extensive corrective measures were taken during the spillway repair. Thicker zoned gravel filters were used under the spillway panels, and an anchor system to tie the concrete panels down was incorporated. Cutoff and support walls were placed completely across the spillway apron under the damaged panels to increase the length of seepage paths. Station 2+48 m (8+14 ft) was at the leading edge of the spillway apron, or the upstream edge of the slabs which failed. A metal water seal was placed on each of the longitudinal joints and in the transverse joint at the leading edge of the slabs that failed. A seepage drain system was incorporated through the gravel backfill under the replaced slabs, and collector drains were located on the upstream face of each cutoff wall. Also, there were pressure relief outlets through the upper slabs (fig. A-2). The gravel envelope was taken down to a smooth hard rock surface; of key importance was the prevention of infiltration of fine soil into the envelope.

Control Structures and Canal Lining on Riverton Unit

In March 1974, two frost team members observed damage to hyraulic structures caused by frost action on the Riverton Unit. This unit is in a high valley near a range of mountains where the air can stagnate and cause long cold periods during which the daily minimum temperature may range from a high of −18 °C (0 °F) to a low of −40 °C (−40 °F). Frost penetration is common to depths in excess of 1 m (3.3 ft). The area is underlain by shale, at relatively shallow depths, which traps water and forms a high ground-

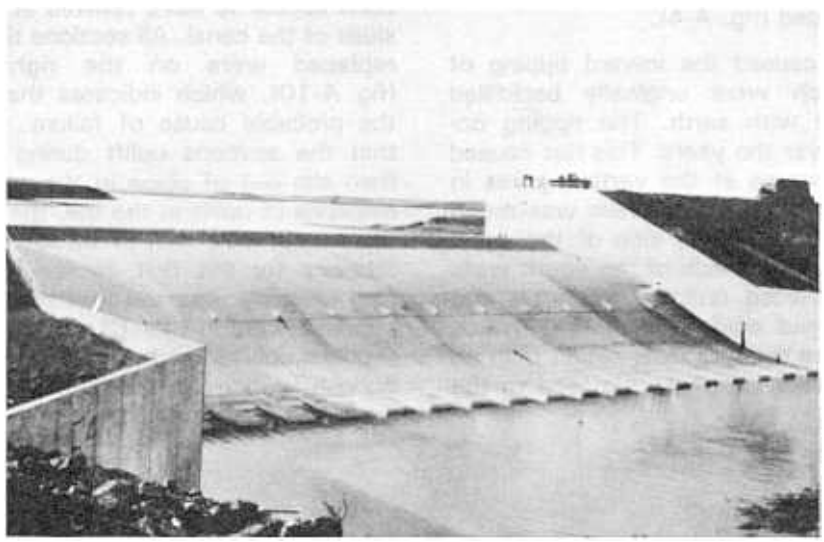

Figure A-2.—Repaired Dickinson Dam spillway showing pressure relief outlets. November 1954. 203-603-531NA

water table. High ground water is also maintained by irrigation water because of inadequate land subsurface drainage. The original canal structures were set rather deep into the ground.

Many of the structures, built in the 1940's, have been severely damaged or have failed completely from frost action. Structures built in the 1950's show frequent distress, and have required remedial measures. There is now underway a $19 million, 5- to 7-year program to rehabilitate the canal structures and install more land drainage. The following paragraphs summarize the general types of distress that have occurred and some of the corrective measures taken.

Some of the small check and drop structures appear to have been raised 305 mm (12 in) above original ground elevations by frost action. During the spring thawing periods and when water is turned into the canal, the structures settle part way back, sometimes in an unequal fashion. Watchful maintenance is required to prevent water flowing through voids beneath the structure and washing it out. On several structures there has been ice buildup in the stilling pools without having caused any significant damage over a long period (fig. A-3). The freezing of a saturated foundation may produce only a small amount of heaving, which will not necessarily affect the structure significantly. In some cases where a canal turnout pipe joins a headwall, frost had caused dislodgment between the two which produced a leak causing soil surrounding the pipe to be eroded (fig. A-4).

Frost action has caused the inward tipping of chute walls which were originally backfilled nearly to the top with earth. The tipping occurred gradually over the years. This has caused dislocation and distress at the vertical joints in the walls. Distress of the side walls was much more extensive on the south side of the canal than on the north. The inside of the south walls would be more shaded during the winter and subject to prolonged cold temperatures which would be conducive to more frost action than on the north walls which are more exposed to the sun.

Bureau personnel have tried during recent years, with varying degrees of success, to reduce the wall movements by: (1) bracing across the tops of the walls with steel (fig. A-5), wood struts, or precast concrete slabs (on small chutes, earth was piled on the slabs to reduce frost action) (fig. A-6); (2) partially removing the soil backfill behind the wall (fig. A-7); and (3) drilling weep holes through the walls to help drain water from the backfill into the canal. Gravel drains were sometimes installed behind the walls to remove ground water to points downstream in the canal.

In certain areas, frost action has heaved concrete canal lining, causing it to crack severely and deteriorate (fig. A-8). This has made it necessary to replace sections of the lining.

Concrete Lining on Steinaker Feeder Canal

Cracks first appeared in the lining of this canal of the Central Utah Project in January 1961 shortly after the lining was constructed but before it was placed in service. The cracks were routed and sealed with a mastic filler in April 1961 (fig. A-9). Further damage occurred to the lining in the summer of 1962 and repairs were made in December 1962. By 1967, 13 sections had been damaged sufficiently to need replacement.

Additional cracks have appeared in the concrete lining at different times which have resulted from uplift or heaving. Many of the sloping sections appear to have uplifted at the toe on both sides of the canal. All sections that needed to be replaced were on the right (south) side (fig. A-10), which indicates that frost action is the probable cause of failure. It was reported that the sections uplift during the winter and then slip out of place in the spring. As further evidence of uplift at the toe, the top of the lining was about 102 mm (4 in) lower than adjacent sections for the first section upstream of the pipe crossing, and the first two sections downstream at station 11 + 13 m (36 + 50 ft). All three sections uplifted and slid 102 mm into the canal. Bridging action caused two of the sections to crack half way up the slope.

Figure A-3.—Lower end of a small canal structure nearly enveloped with ice. Sand Gulch Lateral 3.0A, Riverton Unit, March 1974. P801-D-79868

Figure A-4.—Sinkhole caused by waterflow through a break resulting from frost action at joint between headwall and pipe turnout. Pavillion Main Lateral, Riverton Unit. P801-D-79869

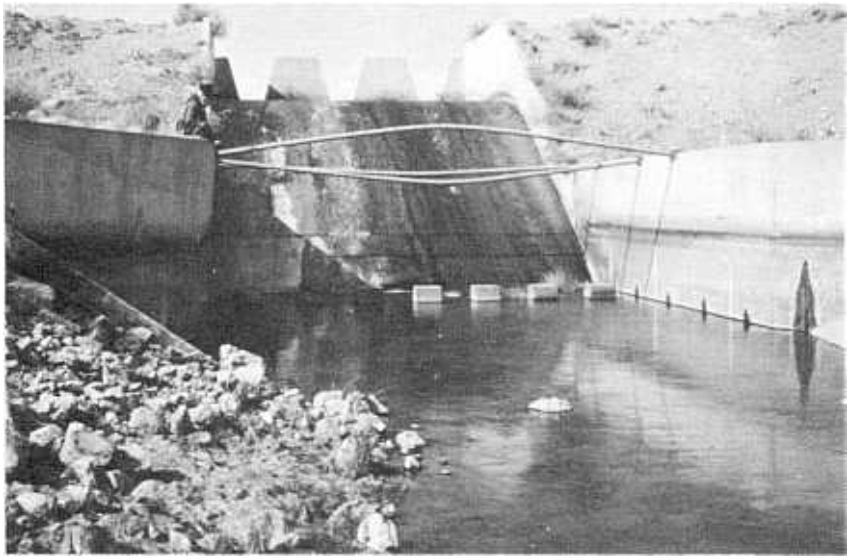

Figure A-5.—Drop structure on east-west oriented Wyoming Canal showing steel wall bracing. Wall on shaded south side has been tipped inward by frost action, and concrete chute floor has cracked from heaving. Riverton Unit. P801-D-79870

Figure A-6.—Precast concrete sections and soil placed on top of chute to prevent collapse of walls. Weep holes have been added to drain soil behind end walls. Lateral W44.13, Riverton Unit. P801-D-79871

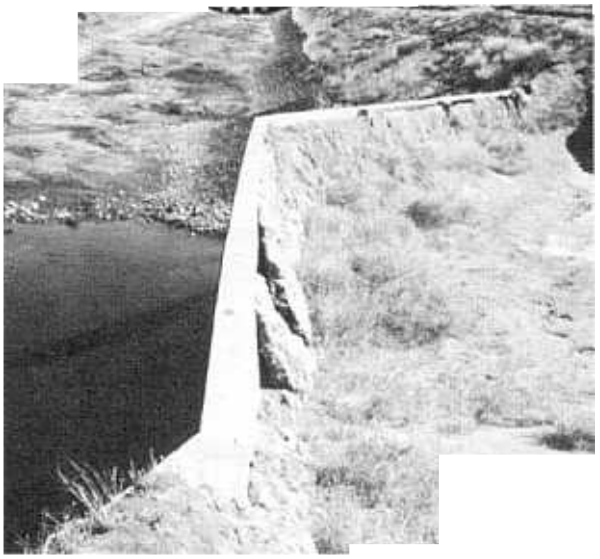

Figure A-7.—Backfill partially removed behind wall to reduce pressure. Wyoming Canal, Riverton Unit. P801-D-79872

Figure A-8.—Concrete canal lining broken by frost action Wyoming Canal, Riverton Unit. P801-D-79873

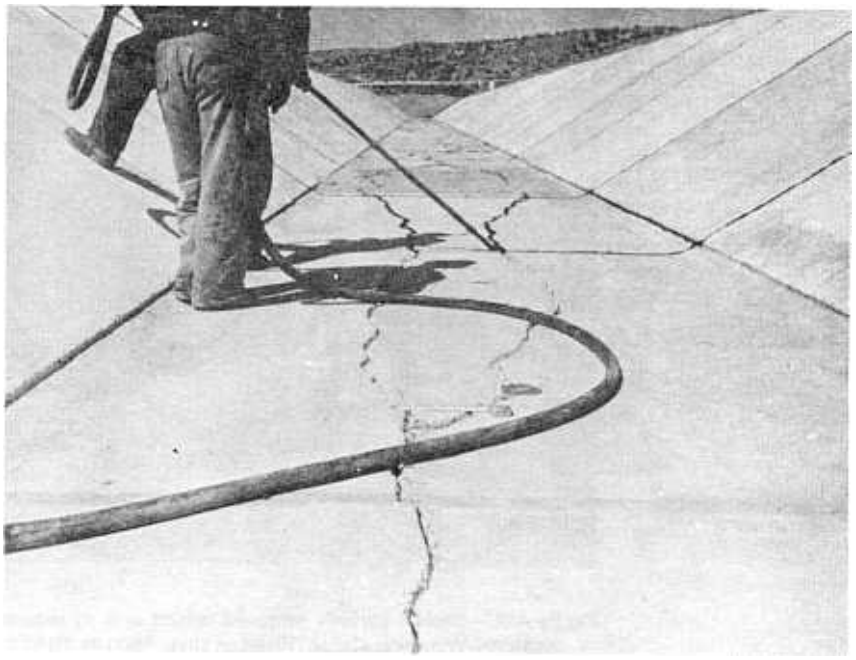

Figure A-9.—Mastic filler being placed in routed cracks in canal lining. Steinaker Feeder Canal, Central Utah Project, 1961. P325-418-494NA

Figure A-10.—Heaved and cracked concrete lining on south (shaded) side of Steinaker Feeder Canal. Central Utah Project. 325-418-1291NA

APPENDIX B

SYNTHETIC INSULATION

During the 1960's, numerous field tests were conducted on extruded, expanded, rigid polystyrene insulation to prevent frost heave of highway and airfield pavements. This plastic type of insulation is produced in rigid sheets 2.4 m (8 ft) long and either 0.6 m (2 ft) or 1.2 m (4 ft) wide, and in various thicknesses from 20 to 76 mm (¾ to 3 in). The thermal and physical properties are given in table A-4 of reference [12]. Currently, the product is designated by Dow Chemical Co. as Styrofoam SM or HI, which have similar properties; the HI is produced to have either a compressive strength of 207 or 414 kPa (30 or 60 lb/in²). Early research on polystyrene insulation was conducted by several investigators, including Purdue University [25]. There have been subsequent test reports primarily by State highway departments and by agencies in foreign countries. A report by the manufacturer summarizes the experience with this type of insulation up to 1970 [26].

Insulation in Airfield Construction

The Alaskan International Airport at Anchorage, Alaska installed Styrofoam HI insulation for testing under the airfield pavement in 1967. The test included two conventional sections of pavement each containing a 3-m (10-ft) thickness of non-frost-susceptible soil and two insulated sections, each with a different thickness and both with about 0.9 m (3 ft) of cover. The sections were installed in a taxiway, and instrumentation was installed to monitor temperatures throughout the winter [27].

The two insulated test sections consisted of 51-and 102-mm (2- and 4-in) thick Styrofoam placed beneath a 0.9 m (3 ft) pavement section. Two 51-mm layers made up the 102-mm thickness. This material was found to have a maximum water absorption of 0.25 percent by volume. The thermal conductivity was 0.03 W/(m·K), or 0.2 Btu·in/(h·ft²·°F), and the density was 40 kg/m³ (2.5 lb/ft³). The insulation boards, 0.6 by 2.4 m (2 by 8 ft), were placed over the existing silt subgrade. Each board was attached to the subgrade with 6-mm (0.25-in) diameter by 152-mm (6-in) long skewers. The joints were lapped in the section with the two 51-mm layers, with the second layer tightly butted against and connected to the first layer. The first lift of gravel subbase was bladed over the insulation in a 152-mm (6-in) lift, and each lift was compacted to 95 percent of Proctor density. One section had a crushed stone base which was compacted to 100 percent maximum density at optimum moisture. The other section was covered with an asphalt-stabilized base course. Prior to placement of the insulation, the subgrade for each test section was excavated to a depth of 3 m (10 ft) and backfilled about 2.1 m (7 ft) with gravel covered with an asphalt-stabilized base course in one quadrant and crushed stone in the other. The four pavement samples were instrumented with 65 copper-constantan thermocouple temperature measurement points. Six or seven thermocouples were located to a depth of 3 m (10 ft). The 102-mm insulation layer was buried under only 1 m (39 in) of material for a reduction in overexcavation and backfill of about 2.1 m (7 ft) depth. The Styrofoam for this installation cost between $68 and $80 per cubic meter ($0.16 and $0.19 per board foot) in place. The minimum temperature underneath the 51-mm insulation was −2 °C (28 °F). The 102-mm insulation held subgrade temperatures to 0 °C (32 °F) just underneath the Styrofoam. The test sections proved so successful that in 1969 two insulated taxiways using 76-mm (3-in) insulation beneath a 0.9-m (3-ft) pavement section were installed as part of a regular contract at the airport.

In 1969, about 610 m (2000 ft) of runway located on frozen peat was insulated with Styrofoam HI at an airport in Kotzebue, Alaska. The insulation was placed about 0.9 m beneath the existing grade of the gravel surface. This placement was designed to prevent or at least retard the degradation of the underlying frozen peat which might result from the increased absorption of solar energy when the runway was surfaced with asphaltic concrete. The construction process consisted of excavating the existing gravel embankment to a depth of about 0.9 m, placing the insulation, and backfilling above the insulation with the same material that was removed. The construction process was continuous, leaving a finished runway in its wake.

Future studies will include an analysis of the insulation under airfield operations to determine the effect of high vibrational loading imposed by jet aircraft. Also to be included is a subgrade test to determine the effect of the insulation on transfer of this vibrational loading to the subgrade soils and the determination of associated improvements or adverse effects on pavement performance.

Insulation in Highway Construction

Insulation under highway pavements has been extensively tested in the United States. Experimental sections have been placed in Iowa, Maine, Colorado, Minnesota, South Dakota, Michigan, Wisconsin, Alaska, and Idaho. A heat-flow computer mathematical model involving finite differences has been developed [28].

In 1962, the Manitoba Highways Branch installed a highway test section near Winnipeg, Canada using polystyrene insulation. The highway subgrade was a clay, and the polystyrene was covered with 305 mm (12 in) of gravel base and a 102-mm (4-in) bituminous surface. Data from thermocouples indicated that a polystyrene thickness of 51 mm (2 in) reduced frost penetration to less than 100 mm (a few inches) in the subgrade, and that 82 mm (3.25 in) of insulation would essentially prevent any penetration. The control section showed that the rigorous climate, which generally experiences a freezing index of 1944 °C·d (3500 °F·d), resulted in frost penetration of 2.1 m (7 ft) when the insulation was not used.

Purdue University has cooperated with Dow Chemical in testing Styrofoam as an insulator for highways. One test section was in Michigan on a lacustrine lake bed deposit where the soil was a heavy brown clay. Both 25- and 50-mm (1-and 2-in) thicknesses of insulating layers were used in the test section. Thermocouples were used during the winter to measure thermal performance. The insulation was beneath 64 mm (2.5 in) of asphaltic concrete surfacing, a 203-mm (8-in) base of crushed gravel, and a 457-mm (18-in) subbase of pervious soil. A section with a 660-mm (26-in) subbase was used as a control section. The following conclusions were drawn from this test:

1. A 25-mm (1-in) thickness of Styrofoam insulation effectively prevented frost action in the subgrade of a flexible pavement in an area where the freezing index reached 889 °C·d (1600 °F·d).

2. A width of insulation some distance beyond the pavement edge was required to prevent frost formation beneath the pavement.

3. Frost penetrations beneath a plastic insulating layer to a depth of 152 mm (6 in) for a short period did not appear to be detrimental.

4. The excess moisture that collected in the subbase during freezing was responsible for some of the loss of bearing capacity during spring breakup.

5. The presence of an insulating layer that was essentially impermeable to moisture did not cause moisture concentrations in the subgrade over 2 years time.

6. The insulating layer showed negligible moisture increase, and therefore no significant change in thermal conductivity after two winters of effective performance.

7. The plastic insulating layer reduced the thickness of the flexible pavement structure normally required for satisfactory performance in frost areas.

In 1963, a test installation was placed on U.S. Highway No. 18 in northeastern Iowa [29]. The original concrete pavement was badly cracked and, in certain areas, had experienced perennial frost heaves. In these areas, the conventional treatment was to remove the subgrade soil for a depth of 0.9 m (3 ft) and replace it with a material not susceptible to frost action. In one of these treatment areas, a Styrofoam insulating layer 38 mm (1.5 in) thick was used in lieu of the 0.9 m of non-frost-susceptible material for a 76-m (250-ft) length of the total 137-m (450-ft) treatment length. The freezing index for the 1963–64 winter at Waterloo, Iowa, about 64 km (40 mi) southeast of the test installation, was 676 °C·d (1217 °F·d) with a freezing duration of over 130 days. From this test it was found that the 38-mm insulating layer placed directly on the frost-susceptible subgrade prevented frost penetration beneath the rigid pavement during a normal winter. Based on temperatures recorded in the subgrade of both frost action treatments and on observations of comparative performance, the 38-mm insulating layer effectively replaced more than 0.9 m of granular material.

In 1963, the Minnesota Department of Highways placed a test section of Styrofoam on a highway about 64 km (40 mi) west of the Twin Cities [30]. The highway had been constructed in the early 1930's, and contained 152 mm (6 in) of bituminous surfacing and 254 mm

(10 in) of a loamy sand and gravel base. Each year a number of frost heave cycles had been observed. In the summer of 1963, two of the most severe frost-heave areas were to be excavated and the upper 1.1 m (3.5 ft) of the subgrade soil replaced with gravel; however, it was decided to excavate these areas to a much shallower depth and insulate the subgrade with 50-mm (2-in) thick boards of Styrofoam. Subgrade soils at the location were clay loam and silt loam. Evaluation of the Styrofoam installations was accomplished by periodically reading thermocouples installed during construction, taking cross sections and profiles of the surface, and performing plate-bearing tests. Observations and measurements taken during the 4-year period following installation resulted in the following findings:

1. No below freezing temperatures were recorded beneath the Styrofoam 0.6 m (2 ft) in from the edge of the installation where all boards were placed horizontally.

2. No below freezing temperatures were recorded beneath the Styrofoam boards placed horizontally where a 0.9-m (3-ft) wide board was attached to the edge of the horizontal boards at a 1:1.5 slope.

3. No changes in elevation occurred in the roadway surface above the Styrofoam. The surface adjacent to these areas usually heaved several inches during the winter.

4. During late winter, bumps developed at the ends of the Styrofoam installations.

5. Plate-bearing tests taken on the roadway surface indicated no strength loss had taken place in the roadway structure where Styrofoam was used.

6. The horizontal placement of Styrofoam boards at one point resulted in an estimated savings of about $0.90 per square meter ($0.75 per square yard) compared to the conventional method of replacing the upper 1.1 m (3.5 ft) of the subrade with gravel.

In the summer of 1964, the Minnesota Department of Highways placed 50-mm (2-in) Styrofoam HI above a 30-m (100-ft) section of 150-mm (6-in) diameter sanitary sewer where the depth of soil cover was only 0.8 m (2.5 ft), and only 1.1 m (3.5 ft) under the roadway surface [31]. The cost of furnishing this material was about $53 per cubic meter ($0.125 per board foot). In a summary of findings, the Department of Highways stated:

1. During the winter of 1964–65, there was a freezing index of 1256 °C·d (2261 °F·d).

2. No below freezing temperatures were recorded beneath the Styrofoam along the centerline of the 2.4-m (8-ft) wide installation.

3. No below freezing temperatures were recorded beneath the Styrofoam at a point 0.6 m (2 ft) from the edge of the installation except on one occasion during the latter part of the winter when a temperature of 0 °C (31.5 °F) was noted at the bottom surface of the Styrofoam.

Based on the results of this study, 50-mm (2-in) thick Styrofoam placed below 0.6 m (2 ft) of soil cover was considered to provide adequate insulation for utilities in areas where the accumulated number of degree-days does not exceed approximately 1222 °C·d (2200 °F·d). The insulation should extend at least 0.6 m beyond both sides of the pipe being protected. This procedure is recommended as an alternative to relocating a utility where the depth of cover is reduced to a thickness inadequate to protect the utility from freezing.

In 1966, the Maine State Highway Commission placed an experimental highway insulation with Styrofoam near Hampden, Maine [32]. For this installation, a 38-mm (1.5-in) thickness of Styrofoam was placed beneath the following layers: 152-mm (6-in) sand base, 178-mm (7-in) gravel base, 102-mm (4-in) asphalt stabilized base, and a 76-mm (3-in) bituminous concrete surface. For a freezing index of 713 °C·d (128 °F·d), the temperature under the center of the Styrofoam remained above 0 °C (32 °F). Horizontal penetration of the 0 °C isotherm was found in one test section where the Styrofoam was buried at a depth of 533 mm (21 in), but no penetration was found in a second section where the Styrofoam was 750 mm (29.5 in) below the surface. Visual inspection of the pavement indicated more cracks in all sections than were found in previous years before installation;

however, there were fewer cracks in the insulated sections than in the uninsulated sections. Vertical movement data show less heaving occurring on the insulated sections.

In 1965, the Maine State Highway Commission used a Dylite expandable polystyrene insulation manufactured by the Sinclair-Coppers Co. and made into boards by the Zonolite Division of the W. R. Grace Co. [33]. The insulation was installed on a reconstructed highway 37 km (23 mi) northwest of Portland, Maine in an area having a design freezing index of 833 °C·d (1500 °F·d) and a design frost penetration of about 1.3 m (4.2 ft). During the winter of 1965–66, the freezing index at a weather station 21 km (13 mi) northeast of the project at Bridgton, Maine was 405 °C·d (729 °F·d). The main portion of the project consisted of 64-mm (2.5-in) thick insulating boards. At the end of the test section, a transition zone 9.1-m (30-ft) long and 14.6-m (48-ft) wide was installed using 25-mm (1-in) thick insulating board. The insulation was placed on a shaped surface of the original gravel base and held in place by wooden pegs. Sand base was placed directly on the insulation to a thickness of 178 mm (7 in) on the 64-mm (2.5-in) thick portion, and to a thickness of 216 mm (8.5 in) on the 25-mm portion. Then, 127 mm (5 in) of crushed gravel base was placed on the sand base and the roadway was finished with 76 mm (3 in) of bituminous concrete pavement. The 64-mm thick insulation was effective; the temperature of the subsoil never dropped below 0.4 °C (32.7 °F). The temperature record at the centerline of the 64-mm board was, in general, warmer than temperatures under both shoulders. The 25-mm thick board permitted the subgrade temperature to drop below 0 °C (32 °F) on three occasions. Greater vertical movement was noted on the insulated section than on the adjacent conventional uninsulated roadway. The explanation for this phenomenon is not known. The low water content values of the soil do not account for the vertical movement. Greater deflections occurred on the insulated section than on the conventional roadway apparently due, in part, to possible damage to the insulation during construction; however, the pavement appears to remain in good condition.

In July 1965, the Wyoming State Highway Commission installed 14.6 m (48 ft) of Styrofoam, 9.1 m (30 ft) wide and 50 mm (2 in) thick, over a perennial frost heave area located about 8 km (5 mi) east of Togwatee Pass on U.S. Highway No. 26–287. The material was placed 0.8 m (2.5 ft) below the top of the pavement. By April 1968, no frost heaving had occurred although temperatures had dropped to −32 °C (−25 °F). In June 1966, the Commission installed 11.6 m (38 ft) of Styrofoam, 8.5 m (28 ft) wide and 25 mm (1 in) thick, over a frost heave area on State Secondary Highway 1401 about 9.7 km (6 mi) west of U.S. Highway No. 85 on the Lance Creek Road. This insulation was installed 457 mm (18 in) below the top of the pavement. In April 1968, no further frost heaving had taken place. Plans were then made to install more of the Styrofoam on U.S. Highway No. 16 about 16 km (10 mi) west of Ten Sleep. This was to be a section 229 m (750 ft) long, 15.9 m (52 ft) wide, and 25 mm (1 in) thick placed 0.6 m (2 ft) below the permanent grade on new construction immediately adjacent to an old highway where severe frost heaving had occurred each winter.

Retention of Permafrost

Assuming proper design and satisfactory material performance, several advantages may be derived from the use of polystyrene insulation in embankments above permafrost to prevent degradation: (1) favorable effects due to a change in embankment geometry, (2) retardation if not complete prevention of permafrost degradation, and (3) reduction of the large temperature gradient across the insulation layer which might increase vapor-driving forces.

Permafrost is of such a sensitive nature that several years of data from a prototype installation will be required to determine both cumulative effects and equilibrium conditions. Thickness of insulation will also be used as a variable. Theoretical analysis will be developed and a suitable correlation between field and theoretical analysis established before long-term cumulative effects predictions can be made. It appears that the two-dimensional or edge effect may be one of the most critical factors in the design of permafrost installations.

General

In the review of installations of rigid insulation, no mention is made of securing the edges with contact cement. All installations reviewed used wooden skewers to secure the insulation to the subgrade. Backfilling above the insulation was

carried out by end-dumping with the loaded trucks backing over previously dumped material. A form of repair includes an overlay procedure whereby insulation is placed directly above the deteriorated pavement and construction of a new pavement section is placed above the insulation. In this manner, damaged highways become the subgrade for new pavement. Also, a construction period shorter than for pavement replacement is required for this type of installation causing less disruption of traffic. A limitation of the overlay application is that the gradeline must be raised about 0.6 m (2 ft).

APPENDIX C

MEASUREMENT OF FROST PENETRATION IN SOIL AND COLLECTION OF RELATED DATA

Frost Tube

The frost penetration tube (fig. 8) is a plastic tube filled with a solution of methylene blue dye in distilled water (0.25 gram per liter). When the liquid freezes, it becomes clear ice. The tube is placed inside a section of PVC pipe which is installed in the ground:

1. Auger a hole (diameter as small as practicable) deep enough to be below the bottom of the frost tube casing. Save the soil for refilling around casing.

2. Place casing such that it is against the soil at the wall of the hole so the frost tube will more nearly register undisturbed ground temperatures. Insert the casing at a depth so the top of the liquid in the frost tube will be even with the ground surface. Wrap a piece of tape around the tube at the ground surface as a measuring point.

3. Refill hole around casing with the same soil removed from the hole, having the refill in about the same condition as the surrounding undisturbed soil (as near as can be estimated).

   a. If hole is shallow enough, the soil can be placed in the hole and compacted with a tamping rod. Try to have the soil at about the same moisture and density as the surrounding soil. If the inplace soil is relatively pervious, the native soil can be made into a slurry for backfill; in this case the extra water in the slurry would be expected to dissipate into the surrounding soil.

   b. As an alternative to using soil from the hole, the hole can be backfilled around the frost tube casing with medium to fine dry sand with light tamping. The sand used for field density tests would be suitable, although the gradation need not be exactly within the limits specified for density sand.

4. Place a capped section of white PVC conduit extending above the ground to protect the top of the tube. About 0.3 m (1 ft) of 38-mm (1.5-in) conduit recommended.

5. To measure frost penetration at intervals of about once a week if possible during the winter, remove the frost tube and measure the length of clear ice from the tape placed on the tube level with the ground surface. In the spring, there will be thawing downward from the ground surface as well as from the maximum frost depth upward. Therefore, measure and record the distance from the ground surface to the top of the ice in the tube as well as from the ground surface to the bottom of the ice.

For example, a measurement recorded as:

$$\frac{203 \text{ mm}}{991 \text{ mm}} \text{ or } \frac{8 \text{ in}}{39 \text{ in}}$$

would indicate 203 mm (8 in) thawing from the top, and a depth of frost penetration of 991 mm (39 in). Figure C-1 shows a typical record form.

Soil Density and Moisture

When frost penetrates from the ground surface downward, the freezing causes moisture to rise toward the ground surface. At the same time, the density of the soil near the surface decreases and the soil at some depth may increase in density. More information is needed on these changes in different soil conditions to prepare future designs. It would be desirable to obtain soil moisture and density tests at 0.3-m (1-ft) intervals to a depth about 0.6 or 0.9 m (2 or 3 ft) below estimated frost penetration both in the fall before freezing and in the spring after thawing.

Temperature

Daily maximum and minimum temperatures obtained near and at the same general elevation as where frost penetration is being measured are useful for correlation with frost penetration. These temperatures can be averaged and used to plot a cumulative degree-day curve. From this curve, a freezing index and a freezing duration can be obtained, which are then correlated with

Record of Frost and Snow Depths

Frost Tube No._____ Station and Offset_____ Elev. _____

Date	Time	Frost depth	Snow depth	Recorder	Notes

Figure C-1.—Frost tube measurement form.

frost penetration. The freezing duration is the measure of the "degree of coldness" for an area.

Snow Depth

Since snow is an insulator and affects frost penetration, the average snow depth within a radius of about 4 m (12 ft) of the frost tube should be recorded each time the tube is read. Disturb the snow around the tube as little as possible. Also, a daily record of precipitation (including snowfall) during the winter will allow a cumulative curve of snow-on-the-ground to be plotted. This is helpful in relating to frost penetration. Figure C-2 is a current National Weather Service form that can be used for recording pertinent weather and ground cover data.

Ground Surface Elevation

Ground surface elevations should be obtained to the nearest 0.003 m (0.01 ft) in the general area of frost tubes to note any heaving due to frost action in the soil. A minimum record of elevations to be obtained would be: (1) in fall before freezing, (2) just before thawing, and (3) in spring after thawing. For an elevation point, a small nonmetallic, light-colored pad (plastic, wood, etc.) should be set on the ground surface with a slight projection into the ground to hold the pad in place.

WS FORM E-15
(10-77)

U. S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
NATIONAL WEATHER SERVICE

RECORD OF RIVER AND CLIMATOLOGICAL OBSERVATIONS

STATION (Climatological) | (River Station, if different) | MONTH 19 ____

RIVER | TYPE OF RIVER GAGE | COUNTY

TIME (local) OF OBSERVATION RIVER | TEMPERATURE | PRECIPITATION | STANDARD TIME IN USE

STATE | ELEVATION OF RIVER GAGE ZERO Ft. | FLOOD STAGE Ft. | NORMAL POOL STAGE Ft.

DATE	RIVER STAGE: CONDITION	RIVER STAGE: GAGE READING AT ___ A.M.	RIVER STAGE: TENDENCY	TEMPERATURE °F.: 24 HRS. ENDING AT OBSERVATION: MAX.	TEMPERATURE °F.: 24 HRS. ENDING AT OBSERVATION: MIN.	TEMPERATURE °F.: AT OBSN.	PRECIPITATION (Draw a straight line (——) through hours precipitation was observed, and a waved line (~~~) through hours precipitation probably occurred unobserved. A.M. NOON P.M.)	24-HR AMOUNTS: Rain, melted snow, etc. (Ins. & hundredths)	24-HR AMOUNTS: Snow, ice pellets, hail (Ins. & tenths)	At Ob.: Snow, ice pellets, hail, ice on ground (Ins.)	WEATHER (Calendar Day) MARK 'X' FOR ALL TYPES OCCURRING EACH DAY: FOG	ICE PELLETS	GLAZE	THUNDER	HAIL	DAMAGING WINDS	Time of Obsn. if different from above
1																	
2																	
3																	
4																	
5																	
6																	
7																	
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SUM																	

CHECK BAR (For wire-weight) NORMAL CK BAR

READING | DATE

CONDITION OF RIVER AT GAGE
A. Obstructed by rough ice.
B. Frozen, but open at gage.
C. Upper surface of smooth ice.
D. Ice gorge above gage.
E. Ice gorge below gage.
F. Shore ice.
G. Floating ice.
H. Pool stage.

SUM | Greatest | Fog | Ice Pel. | Glaze | Thun-der | Hail | Damag winds

SUPERVISING OFFICE | STATION INDEX NO.

SPECIAL OBSERVATIONS OF PRECIPITATION & RIVER STAGES

DATE	TIME OF OBSERVATION	PRECIPITATION SINCE 7 A.M.	RIVER: STAGE	RIVER: TENDENCY	CREST STAGE, DATE AND TIME, DEPTH OF SNOW OR ICE, STATE OF WEATHER AT TIME OF OBSERVATION

IF MORE SPACE IS NEEDED, USE ADDITIONAL FORM

REMARKS

OBSERVER

Figure C-2.—Weather and ground cover record. (Courtesy of National Weather Service).

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Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

A free pamphlet is available from the Bureau entitled, "Publications for Sale". It describes some of the technical publications currently available, their cost, and how to order them. The pamphlet can be obtained upon request from the Bureau of Reclamation, Attn D-922, P O Box 25007, Denver Federal Center, Denver CO 80225-0007.