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Bulletin 135

Factors Influencing Ground Freezing

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Factors Influencing Ground Freezing

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Contents

Modification of Frost-Heaving of Soils With Additives

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This paper describes a three-year search for additives to reduce the frost susceptibility of soil. Fifteen soils and about forty additives have been tested. A discussion of the theoretical considerations for the choice of additives is presented. The additives are divided into four groups: (1) void pluggers and cements, (2) aggregants, (3) dispersants, and (4) "waterproofers" — according to their action in soil.

Tests reported herein show a number of additives, especially dispersants and polyvalent cation salts, merit further laboratory evaluation. Other tests on soils treated with resins and "waterproofers" have also indicated promising results.

A small-scale field test showed a laboratory-proved dispersant to be effective under field conditions; measurements made during the second freezing cycle showed no reduction in the potency of the dispersant treatment. Four freeze-thaw cycles on four soils tested in the laboratory also had no adverse effects.

●WHEN a wet soil is subjected to a low enough temperature, the water within the soil freezes. If the soil moisture is "pore" water (i.e., water not under significant attractive forces from the soil particles) it freezes at essentially the same temperature as water in a large container. "Adsorbed" water — that water under significant attractive forces from the soil particles — freezes at a temperature lower than the freezing point of free water. Accompanying the water-to-ice phase transformation is a volume increase of approximately 10 percent. Upon freezing, therefore, saturated soil swells a minimum of 10 percent of the pore volume.

There is, unfortunately, a phenomenon which occurs in certain soils when frozen that results in a volume increase which far exceeds the minimum (e.g., see Figures 1a and 1b). This phenomenon is the movement of soil moisture to form ice lenses. Thus, freezing a soil can cause swelling, or heaving, many times greater than the amount attributable to volume change of pore water. While more heave occurs when the freezing soil has access to an outside source of water, considerable heave can occur by a redistribution of moisture within a soil. In a soil to which no moisture is added, the migrated moisture is either replaced by air, or the soil reduces in volume, or both.

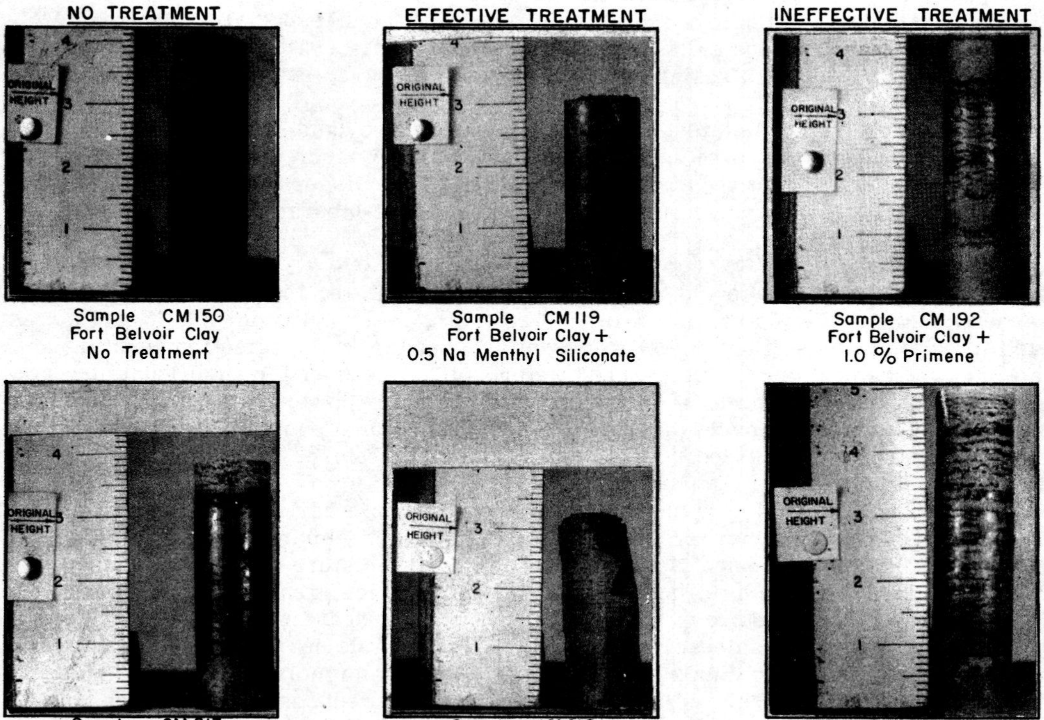
Frost-heaving in soil causes two major engineering problems. The soil expansion moves structures in contact with the soil, such as building foundations, retaining walls, and pavements. Of much more importance to the construction and maintenance of highways and airfields is the loss of soil strength upon melting. Since melting occurs from the ground surface down, and the melt water cannot easily drain downward because of the underlying ice barrier, this melt water can make the soil sloppy wet and, thereby, very weak. Many base courses and subgrades lose a major portion of their strength during the spring thaw and thus cause pavement failure.

Three conditions must exist at a site for frost-heaving to occur — a frost-susceptible soil, a freezing temperature, and a water source. Since seasonal freezing and thawing of surface soils occur in more than one-half of the land area of the Northern Hemisphere, frost is a major concern to soil engineers. On an alarming number of highway and airfield projects, the pavement design is controlled by frost considerations (4). As the supply of select granular soils is being exhausted, the construction of frost-resistant pavement is becoming extremely expensive.

Researchers in many countries, especially in Sweden and the United States, have studied frost action in soils. The Artic Construction and Frost Effects Laboratory (ACFEL), Corps of Engineers, U.S. Army, has been conducting studies on many facets of construction in freezing climates. The author and personnel of ACFEL have collaborated in a search for additives that will effectively reduce the frost susceptibility of soil. This report summarizes three years of experimentation on this continuing research.

MECHANISMS WHEREBY ADDITIVES CAN REDUCE FROST

Although the most obvious method of making a frost-susceptible soil non-frost-susceptible is to treat the soil so that moisture cannot migrate to form ice lenses; it is not easy to accomplish this since the principles of water migration are not fully understood. A coarse-grained soil (e.g., a clean sand or coarser) does not heave; a very plastic clay (e.g., sodium montmorillonite) does not heave appreciably under natural conditions. Some fines are needed to aid the movement of water by soil-water forces, probably of the type which cause capillarity. Too many fines or fines of certain mineralogical compositions can make the soil so impermeable that water for the formation of ice lenses cannot move fast enough under normal freezing rates. Therefore, there is apparently a critical range of particle sizes necessary for ice lens formation. While particle size is probably not the fundamental characteristic, no precise relation between frost susceptibility and any other soil characteristic (such as permeability, density, capillarity, composition, and specific surface) has been found.



NO TREATMENT — EFFECTIVE TREATMENT — INEFFECTIVE TREATMENT

Sample CM 150
Fort Belvoir Clay
No Treatment

Sample CM 119
Fort Belvoir Clay +
0.5 Na Menthyl Siliconate

Sample CM 192
Fort Belvoir Clay +
1.0 % Primene

Sample CM 213
Boston Clay
No Treatment

Sample CM416
Boston Clay +
1.0 % Na Tetraphosphate

Sample CM 434
Boston Clay +
0.01% Peat Fines

TYPICAL FROZEN SAMPLES

Figure 1A.

Two other ways of reducing frost heave are: (1) to prevent freezing of the soil pore water, and (2) to cement the soil particles together with a bond strong enough to resist the expansion forces from frost action.

Additives can reduce moisture migration, reduce the freezing temperature of pore water, and cement particles together. Some of the mechanisms whereby additives can perform these functions are described in the following.

Fill Soil Voids

Completely plugging the voids of a soil with a non-pervious material prevents, of course, the movement of water. Asphaltic concrete and portland cement concrete, having most of their voids filled, are not frost-susceptible even though they can contain frost-producing fines. The prevention of frost heaving in soils by this technique is uneconomical; the soil in question could more cheaply be replaced with a non-frost-susceptible gravel or crushed stone.

Cement Soil Particles

Closely related to the plugging of soil voids is the cementing of soil particles. The non-heaving of concrete is undoubtedly due to cementing in addition to low permeability. As with void-plugging, the prevention of heaving by cementing is usually uneconomical.

Alter Characteristics of Pore Fluid

The dissolution of additives in the soil water can result in a lowering of the freezing temperature. Sodium chloride and calcium chloride are used to reduce frost action by this mechanism. Lowering the freezing point reduces the depth of frost formation; it has little or no effect on the heave characteristics of freezing soil (13).

An understanding of the nature of forces involved in water migration for ice lens formation might suggest other beneficial treatments to the pore water.

The drawback to the treatment of pore fluid is its impermanence. A study (13) in Massachusetts, for example, showed the effectiveness of a calcium chloride treatment of a subgrade to be about 3 years. Since stability requirements necessitate that base course soils be free-draining, the pore fluid in these soils is probably soon leached out by the movement of ground water. The leaching of salts from fine-grained subgrade soils can take considerably more time.

Aggregate Soil

As already noted, a soil must have a minimum amount of fine particles to be frost-susceptible. Casagrande (5) set this minimum as 3 percent by weight finer than 0.02mm While soils have been encountered that possess less than this 3 percent but still are frost-susceptible, no better criterion other than one based on laboratory freezing tests has been found.

A frost-susceptible soil can be made non-frost-susceptible by removing the frost-producing fines. This principle has been employed by washing the fines out of "dirty" gravels. This washing can be a difficult and expensive operation for soils containing more than a small percentage of fines, especially where water is not readily available.

The amount of fines in a soil can be reduced with additives that cause small particles to aggregate into larger units. Either conventional cements (e.g., portland cement) or chemicals that cause flocculation by electro-chemical reactions can be used to reduce soil fines. Michaels (10) hypothesized on the various mechanisms by which aggregants, especially the synthetic polymers, flocculate soil fines. The polymers usually exist as long-chain molecules whose ends can attach themselves to the soil mineral surfaces; the particles are thus linked together by the polymer.

Synthetic polymers have been marketed as "soil conditioners" for the improvement of the agricultural properties (primarily increasing the porosity and the permeability) of soil. Even though the polymers are effective in trace quantities, in fact as low as 0.005 percent of the soil weight, their high unit cost (upward of $0.50 per pound) has greatly limited their use.

Soil aggregation can also be obtained by polyvalent cations such as Fe^{+++} and Al^{+++}. These cations act by shrinking the diffuse double layers around the soil colloids enough to permit the interparticle attractive forces to make the particles cohere. Another phenomenon, ion fixation, comes into play with certain ions to increase greatly their aggregating ability. The most notable example is ferric iron, Fe^{+++}. If Fe^{+++} is added to a fine-grained soil an ion exchange reaction can occur wherein the iron replaces some of the exchangeable cations on the soil. This reaction tends to produce flocculation because of the reduction of the interparticle repulsive charges (11). If the exchanged soil is now dried, some of the iron ions link adjacent particles together with a very strong bond that is resistant to water attack. These ions become fixed and are no longer exchangeable. When the iron is added to the soil as a chloride salt ($FeCl_3$), the formation of iron hydroxide is possible; iron hydroxide can be a weak cement.

Considerable study (e.g., see ref. (9)) has shown that natural clays which contain iron and have been dried are considerably less plastic and have only a fraction of the fines that would be expected from the mineralogical composition of the clays. For example, a clay from Jamaica had 60 percent by weight of clay mineral matter, but had only 20 percent by weight of particles finer than 0.002 mm. The 2.3 percent iron oxide (Fe_2O_3) the clay contained, effectively made silt sizes out of most of the clay minerals.

Disperse Soil

Just as there are chemicals that can aggregate soil fines, so there are other chem-

TABLE 1

ADDITIVES TRIED AS FROST MODIFIERS

ITEM NO.	TRADE NAME	DESCRIPTION	SUPPLIER
1	-	Vegetable pitch	General Mills, Inc , Minneapolis, 13, Minn.
2	-	Tall oil	General Mills, Inc., Minneapolis, 13, Minn
3	-	Vegetable residue	General Mills, Inc , Minneapolis, 13, Minn
4	-	Asphalt emulsion	American Oil Products Co , Somerville 43, Mass.
5	-	Polyamide resin	General Mills, Inc , Minneapolis 13, Minn.
6	-	Portland cement	-
7	Pozzolith	A calcium lignosulfide	Master Building Co., Waltham, Mass.
8	Flocgel	A modified starch	W.A. Scholten's Chemische, Fabrieken, Netherlands
9	CRD-197	Sodium salt of a polymer	Monsanto, Everett Station 49, Boston, Mass.
10	Guartec	Polygaloctomannan	General Mills, Inc , Minneapolis 13, Minn
11	Krilium	Maleic polymer	Mansanto, Everett Station 49, Boston, Mass
12	Agrilon	Sodium polyacrylate	American Polymer Div , Peabody, Mass
13	-	Copolymer of Styrene and methosulfate	Koppers Co , Inc., Pittsburgh 19, Penna.
14	P V A.	Polyvinyl alcohol	E.I. duPont deNemours & Co , Grasselli Chemicals Dept , Boston 10, Mass
15	Quadrafos	Sodium tetraphosphate	Rumford Chemical Works, Rumford 16, R.I
16	-	Sodium tripolyphosphate	West Vaco, New York 17, N.Y
17	-	Sodium hexametaphosphate	West Vaco, New York 17, N Y
18	Versenate	Sodium salt of ethylene diamine tetra acetic acid	Bersworth Chemical Co , Framingham, Mass.
19	Tamol 731	A sodium salt of a carboxylic acid	Rohm & Haas, Wellesley Hills, Mass
20	Daxad 11 and 21	Formaldehyde-condensed naphthalene sulfonates	Dewey & Almy, Cambridge 40, Mass
21	Marasperse N and C	Lignosulfonate salts	Marathon Corp., Rothschild, Wis.
22	Lignosol	Lignosulfonate	Marathon Corp , Rothschild, Wis.
23	SC-50	Sodium methyl siliconate	General Electric Co , Pittsfield, Mass.
24	XS-1	Sodium methyl ethyl propyl siliconate	Dow Chemical Co., Midland, Mich.
25	-	Potassium phenyl siliconate	Mansanto, Everett Station 49, Boston, Mass.
26	Triton K-60	Stearyl dimethyl benzyl ammonium chloride	Rohm & Haas, Wellesley Hills, Mass
27	Volan	Methacrylate chromic chloride	E.I duPont deNemours & Co. Grasselli Chemicals Dept., Boston 10, Mass.
28	Quilon	Stearate and chromic chloride	E.I. duPont deNemours & Co Grasselli Chemicals Dept., Boston 10, Mass.
29	Hyamine 1622 and 2389	Fatty quantenary ammonium salts	Rohm & Haas, Wellesley Hills, Mass.
30	-	Triethylene tetramine	Eastman Kodak Co., Rochester, N.Y Distributor Howe & French, Boston, Mass.
31	-	Hexamethylene diamine	Eastman Kodak Co., Rochester, N.Y. Distributor Howe & French, Boston, Mass
32	-	Di-N-butylamine	Olin Mathieson Chemical Corp , East Rutherford, N.J. Distributor Howe & French, Boston, Mass.
33	Primene 81-R	Tertiary alkyl primary amine	Rohm & Haas, Wellesley Hills, Mass.
34	Carbowax 200 and 6000	Polyethylene glycol	Carbide & Carbon Chemical Co , New York 17, N.Y
35	Arquad 2HT	Dioctadecyl dimethyl ammonium chloride	Armour Chemical Div., Chicago 9, Ill.
36	Armeen 18D	Octadecyl amine	Armour Chemical Div., Chicago 9, Ill.
37	-	Diethanol rosin amine D acetate	Hercules Powder Co , Wilmington 99, Del.
38	-	Monoethanol rosin amine D acetate	Hercules Powder Co , Wilmington 99, Del.
39	-	Peat fines	Northeastern Massachusetts

TABLE 2

SOILS USED IN COLD ROOM TESTS

LAB NO	SPECIMEN IDENT SYMBOL	SOURCE	CORPS OF ENGINEERS' UNIFIED SOIL CLASSIFICATION		GRAIN SIZE (PERCENT FINER THAN) U S STANDARD SIEVE				WEIGHT LOSS ON IGNITION (Percent)	SPECIFIC GRAVITY (Total Sample)	COMPACTION CHARACTERISTICS		ATTERBERG LIMITS[a]	
			DESCRIPTION	SYMBOL	#4 4 76 (mm)	#40 0 42 (mm)	#200 0 074 (mm)	- 0 02 (mm)			MAXIMUM DRY DENSITY (pcf)	OPTIMUM WATER CONTENT (Percent)	LIQUID LIMIT (Percent)	PLASTICITY INDEX (Percent)
	PBJ	Greenland TP-250	Sandy GRAVEL	GW	38	18	4	2 0	-	2 72	148[b]		Non-plastic	
	DFB	Dow AFB Bangor Maine B-11	Sandy GRAVEL	GW GM	42	12	5	2 4	-	2 73	139[b]	-	Non-plastic	
	DFSB	Dow AFB Bangor Maine B-18	Sandy GRAVEL	GW GP	49	18	8	3 2	-	2 69	139[b]	-	Non-plastic	
49-11	RC	Ellsworth AFB Weaver, South Dakota	Silty Sandy GRAVEL	GM	56	30	12	8	-	2 76		-	19	2
49-8	CL	Clinton County AFB Wilmington Ohio	Silty Clayey GRAVEL	GM OC	53	25	15	9	-	2 74	-	-	25	7
49-87	LST	Loring AFB Limestone Maine subgrade frost test section	Clayey Sandy GRAVEL	OC	68	52	41	30	-	2 73	137[c]	7 5	22	8
49-21	SPK	Spokane AFB, Spokane Washington	Gravelly SAND	SW SH[d]	70	15	8	4 0	-	2 82	-	-	Non-plastic	
49-102	LIN	Lincoln AFB Lincoln Nebraska	Gravelly SAND	SW SM[d]	67	24	6	4 7	-	2 65	133[e]	-	Non-plastic	
49-60	WWS	Fairchild AFB, Spokane Washington	Silty Gravelly SAND	SM	88	33	19	10	-	2 77	142[b]	-	22	3
49-54	PAFB	Portsmouth AFB Portsmouth New Hampshire	Silty Gravelly SAND	GM-SM[d]	68	45	23	14	2 1	2 71	129[b]	-	Non-plastic	
49-17	SF	Sioux Falls Airfield Sioux Falls South Dakota	Silty Clayey Gravelly SAND	GC SC[d]	72	28	15	8	-	2 74	-	-	24	6
49-9	PT	Patterson AFB Fairfield Ohio	Silty Clayey Gravelly SAND	OC-SC[d]	68	34	21	15		2 74	-		22	6
49-46	NH	Goffs Falls New Hampshire (New Hampshire silt)	Clayey SILT	ML CL	100	100	99	77	-	2 74	110[e]	14 7	24	6
49-66	FB	Fort Belvoir Virginia	Sandy CLAY	CL	97	88	82	46	-	2 73	116[c]	16 1	41	19
49-42	BC	North Cambridge Massachusetts (Boston blue clay)	CLAY	CH	100	100	100	94	-	2 78	109[c]	20 2	53	28

[a] On material passing the U S Standard No 40 sieve
[b] Providence Vibrated Density Test
[c] Corps of Engineers Modified AASHO Density Test is made by compacting soil in a 6-in diameter cylinder 1/13 cu ft in volume using 5 layers 10-lb tamper and 18-in drop, 56 blows per layer
[d] Dual symbols used are not in strict accordance with unified classification system but are modified as a result of visual examination of material
[e] The maximum dry density was obtained by compacting soil (minus No 4 sieve material only) in a 4 in diameter cylinder 1/30 cu ft in volume using 5 layers 10-lb tamper and 18-in drop, 25 blows per layer

icals that can do the reverse; namely, disperse some of the natural aggregates of soil fines. Most of these chemical dispersants are made up of a polyanionic group (e.g., phosphate or sulfonate) and a monovalent cation, usually sodium. Some of the anionic groups can remove any polyvalent cations by forming insoluble products, and others can become attached to the soil mineral surface. The sodium ions become linked to the soil, replacing the removed polyvalent exchangeable cations.

Both the cation exchange — monovalent for polyvalent — and the anion adsorption expand the diffuse double layers around the soil colloids, thus increasing interparticle repulsion. This increase of interparticle repulsion tends to disperse the soil aggregates. Particles that do not stick together can be manipulated into a more orderly and denser structure. Attendant with improved structure are higher density, lower permeability, and higher stability to water. These and other alterations of soil properties that can be effected with trace quantities of chemical dispersants have been described (8).

Since dispersants can alter soil properties that are related to frost susceptibility, they should change frost susceptibility. By decreasing the sizes of soil voids, dispersants also tend to lower the freezing temperature of soil moisture.

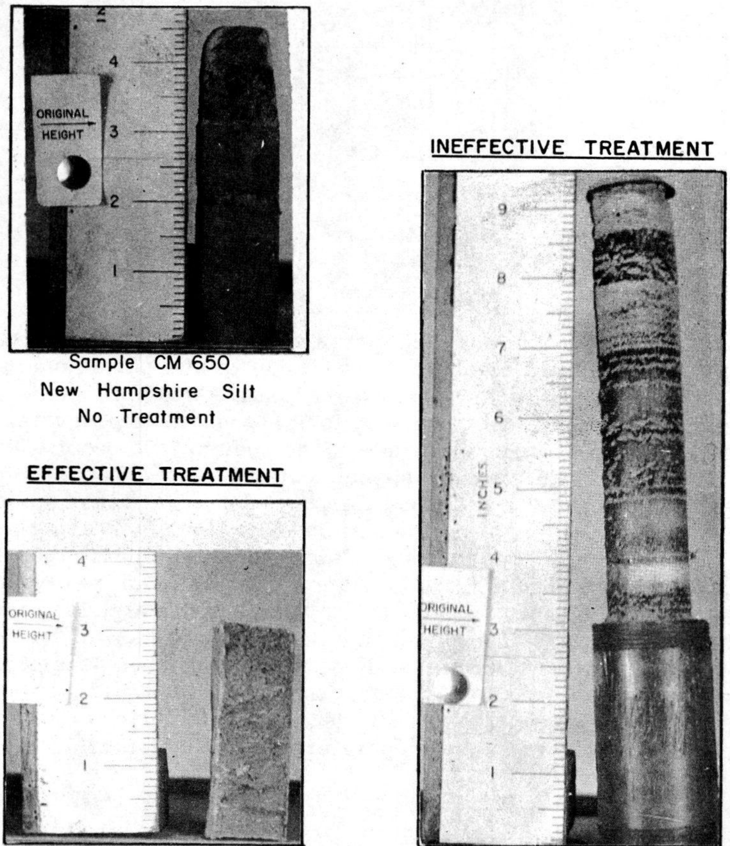
ALTER CHARACTERISTICS OF THE SURFACES OF SOIL PARTICLES

If the characteristics of the soil surfaces that contribute to the migration of soil moisture were fully known, the alteration of these characteristics could possibly be effected. Mineral surfaces can be made hydrophobic with the proper additives. This treatment can be effected in two ways: (1) by treating the soil with a substance made up of molecules one end of which is first preferentially adsorbed on the soil surface and then undergoes an irreversible reaction with the surface, the other end of the molecule is hydrophobic and thus makes the soil non-wettable with water; and (2) by treating the soil with "non-hydratable" cations that are attracted to the negatively-charged soil particles. A soil can thereby be waterproofed so it will not be "wet"; such a soil has little or no adsorbed moisture.

Coating soil with additives that have highly polar groups exposed to the soil moisture can increase the amount of moisture adsorbed by the soil and thereby lessen area available for flow. Such a treatment might reduce the permeability of a fine-grained soil enough to make it non-frost-susceptible.

Additives Studied

The preceding theoretical considerations guided the selection of chemicals evaluated as frost modifiers (Table 1 lists those evaluated). Many, in fact most, of the chemicals employ more than one of the mechanisms described; for example, to be an effective dispersant, a chemical must alter the characteristics of the surfaces of soil particles. While all of the mechanisms should, under certain circumstances, alter the frost characteristics of soil, the potency of each must be determined by actual freezing tests.



NO TREATMENT

Sample CM 650
New Hampshire Silt
No Treatment

EFFECTIVE TREATMENT

Sample CM 686
New Hampshire Silt+
1.0 % Na Tetraphosphate

INEFFECTIVE TREATMENT

Sample CM 635
New Hampshire Silt +
0.05 % Guartec

Figure 1B. Typical frozen samples.

FREEZING TEST PROCEDURE

The detailed freezing test procedures employed by the Arctic Construction and Frost Effects Laboratory (ACFEL) have been described in their official reports and in publications by personnel of the Laboratory (1, 2, 3, 6).

The soil and additive to be tested were thoroughly mixed and then compacted at a selected moisture content, usually optimum. In some instances, as noted in the results, the treated soil was dried prior to compaction. Two sample sizes were employed: a cylinder 1.25 inches in diameter and 3.108 inches long (Wilson miniature size sample), and a cylinder 5.91 inches in diameter and 6.0 inches long (standard size sample). Some of the compacted samples (i.e., those with portland cement) were cured at room temperature in a humid room for 7 days prior to freezing, but most were frozen with no curing.

The samples were saturated and then placed in a freezing chamber with a free water surface maintained approximately ⅛ inch above a porous stone at the bottom of each sample. After equilibrating for a day, the samples were frozen from the top by gradually decreasing the air temperature above the samples while the bottoms of the samples were maintained between 35 and 38F. The temperature in the test cabinet was lowered to obtain approximately ¼-inch penetration per day of the 32F isotherm into the sample, a rate of penetration typical of severe field freezing. Each sample was examined at the end of the freezing period so that the results could be based on the frozen portion of the sample. Thus, none of the beneficial effects of the additives can be attributed to lowering the freezing temperature of the pore water.

The results of freezing tests are expressed as the average rate of sample heave in millimeters per day. Since alteration in frost characteristics is needed to evaluate an additive, the average rate of heave of a treated sample is divided by the average rate of heave of an untreated sample. The value obtained is termed "heave ratio" and is a measure of heave alteration, since a ratio below one indicates improvement and a ratio above one, impairment.

ACFEL studies have shown that frost heave varies with many test conditions, such as molding moisture, compacted density, sample surcharge, and rate of freezing (1, 2, 3). In the tests described herein an attempt was made to control these variables so as to permit the effects of the additives to be isolated and studied. Since 36 miniature samples were frozen in the same cold chamber, the rate of frost penetration could actually be controlled at ¼ inch per day on only one of the 36. To minimize the differences in penetration rates as much as possible, all samples in a given run were prepared from the same soil. With each run, at least one untreated sample was frozen.

The reproducibility of results within a given tray of 36 samples is good; for example, one tray had 5 samples of untreated Fort Belvoir sandy clay with heaves of 1.49, 1.46, 1.53, 1.59, and 1.84 mm per day. However, in another tray where the freezing rate was controlled from thermocouples in New Hampshire silt, untreated Fort Belvoir clay heaved 2.74 mm per day. Where different soils were frozen in the same tray, a blank for each soil was included; the rate of heave of this blank was used to compute the heave ratios of samples of that soil in the tray.

Because of the difficulties of controlling the many variables, especially freezing rate, the heave rates are probably no better than ± 15 percent. Since the miniature tests are used only to screen the many additives studied, this reproducibility is acceptable.

The question has fairly been raised: Can the results of a laboratory freezing test ac-

TABLE 3

EFFECT OF RESIN-TYPE ADDITIVES ON FROST HEAVE

(In Heave Ratios[a])

Additive	Percent	Boston Blue Clay	New Hampshire Silt	Fort Belvoir Sandy Clay
Vegetable pitch	0.50	0.29, 0.89	0.52, 0.82	0.76, 0.52
	1.00	0.58, 0.75	0.72, 1.01	0.84, 0.39
	3.00	0.70, 0.67	0.94, 1.27	0.33, 0.29
Tall oil	0.05	0.73	0.48	0.64
	1.00	0.73	0.24	0.47
	3.00	0.64	0.25	0.37
Vegetable residue	0.50	0.73	0.34	0.78
	1.00	0.69	0.50	0.74
	3.00	0.88	0.33	0.41
Asphalt emulsion	0.50	0.34	0.43	0.23
	1.00	0.62	0.60	0.24
	3.00	0.27	0.53	0.37
Polyamide resin	0.10			1.03
	1.00			0.98
	3.00			1.22

[a] $\text{Heave ratio} = \dfrac{\text{average rate of heave of treated soil}}{\text{average rate of heave of untreated soil}}$

curately indicate the frost behavior of a soil in the field, especially since the major problem is loss of strength upon melting? The answer is thought to be generally yes. Since the more the sample heaves, the greater is the water imbibed during freezing, and the greater is the quantity of water present upon thawing, the lower is the strength of melted soil. In other words, rate of heave does give an indication of strength of melted soil. As a matter of fact, the strength of melted soil can be (and sometimes is) measured with a cone penetrometer or California Bearing Ratio apparatus.

The use of amount of heave to predict strength loss of the soil upon thawing has limitations. All soils are not equally sensitive to moisture; a soil might heave only a mod-

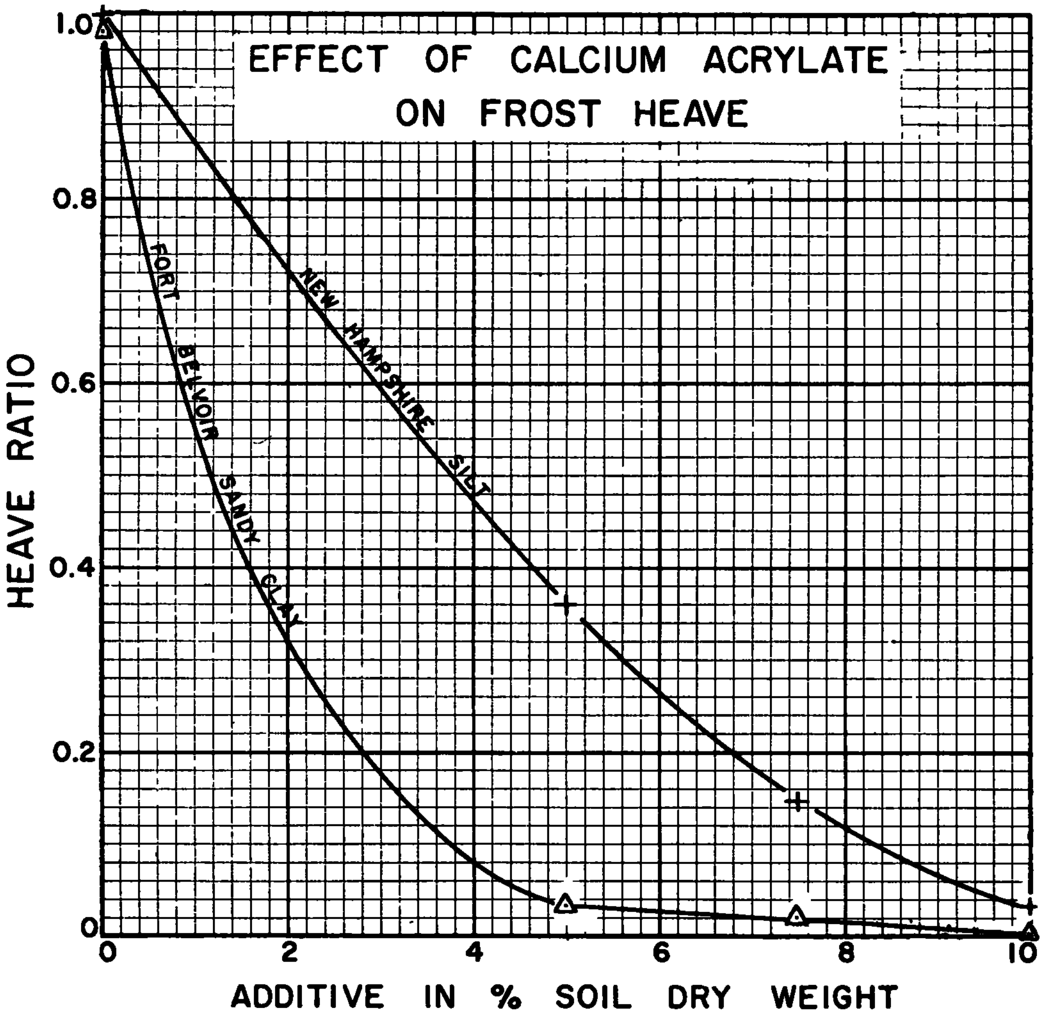


Figure 2. Effect of calcium acrylate on frost heave.

erate amount but lose considerable strength upon melting. The cumulative effects of several freeze-thaw cycles can be more serious on fine-grained soils than on coarse soils. A later section of this paper considers this point further.

The laboratory test program was employed to select promising frost modifiers from the many materials that theory suggests might be effective. Those additives shown to have promise in miniature tests are subjected to evaluation employing different soils and the standard size test sample. Field testing is the next step for worthy additives. This sequence of testing is illustrated subsequently herein with the chemical dispersants.

DESCRIPTION OF SOILS EMPLOYED IN TESTS

Table 2 gives a description of the soils employed in the cold room tests. The great majority of the miniature tests were run on 3 soils — Fort Belvoir sandy clay, Boston blue clay, and New Hampshire silt — selected to give a wide range of frost-susceptible soil types.

EFFECT OF ADDITIVES THAT FILL SOIL VOIDS OR CEMENT SOIL PARTICLES

Under this category are included those additives whose primary function is either to plug soil voids or to cement soil particles. Most of them do both, as well as employ other mechanisms already described.

To plug completely the voids in a soil would require an inordinate amount of additive; for example, the void ratios of the compacted samples of soils studied herein varied from about 0.5 to 1. To fill these voids would require an additive volume of between one-half and the full volume of soil grains. Effective sealing may, of course, be accomplished at a lower level of treatment by plugging only the larger voids.

Because of the high treatments of pluggers and cementers required, only very cheap additives offer promise. Attempts were made to increase the effectiveness of cements with trace additives. The water-sensitive void pluggers were tried because they can employ pore water to help make up the volume needed to seal.

A disadvantage of most sealers and cements is that a reaction after addition to the soil is required (e.g., hydration of portland cement, breaking of asphalt emulsion, polymerization of calcium acrylate). The samples containing these materials were, therefore, cured before freezing.

Synthetic Polymer

Research at the Massachusetts Institute of Technology (7) sponsored by the Arctic Construction and Frost Effects Laboratory, indicated that the in situ polymerization of monomers, especially calcium acrylate, effected significant changes in soil properties. Figure 2 shows that 50 percent of calcium acrylate essentially prevented heave in Fort

TABLE 4

EFFECT OF PORTLAND CEMENT ON FROST HEAVE

(In Heave Ratios)

Additives	Percent	Boston Blue Clay	New Hampshire Silt	Fort Belvoir Sandy Clay
Portland cement	1	1.35	1.74	1.04
Portland cement	2	1.36	0.63	0.58
Portland cement	3	0.46	0.46	1.08
Portland cement + Pozzolith	1 0.1	1.35	0.59	0.67
Portland cement + Pozzolith	3 0.2	0.56		0.74
Portland cement + Daxad 21	1 0.1	1.41		0.82
Portland cement + Daxad 21	2 1.5	0.68	0.76	0.10
Portland cement + Daxad 21	3 0.2	0.61		1.10
Portland cement + Daxad 21	5.0 1.0	0.37	0.47	0.26

TABLE 5

EFFECT OF AGGREGANTS ON FROST HEAVE

(In Heave Ratios)

Additives	Percent	Boston Blue Clay	New Hampshire Silt	Fort Belvoir Sandy Clay
Flocgel	0.01	0.70	1.29	0.82
	0.05	1.12	0.58	0.60
	0.10	0.68	0.38	0.55
	0.50	0.59	0.36	0.55
CRD-197	0.01	-	0.74	0.46
	0.05	-	0.70	0.48
	0.10	1.31	0.35, 1.04	0.40, 0.56
	0.50	3.26	0.58	0.32, 0.71
	1.00	1.87	0.73	0.05
Guartec	0.05	1.11	6.15	0.56
	0.10	0.88	2.16	0.73
	0.50	0.66	0.31	0.60
	1.00	0.76	0.42	0.20
Krilium (maleic polymer)	0.01	1.26	1.35	0.69
	0.05	1.24	1.36	0.59
	0.10	1.16	0.96	0.53
	0.50	0.70	0.11	0.37
Agrilon	0.01	1.18	0.79	0.70
	0.05	1.35	0.83	0.69
	0.10	1.05	0.82	0.39
	0.50	0.52	0.45	0.32
Copolymer of styrene and methosulfate	0.10	0.61	0.06	0.76
	0.50	1.23	0.28	1.42
	1.00	1.53	0.61	0.87
P.V.A. (polyvinyl alcohol)	0.01	1.60	1.08	-
	0.05	1.54	0.78	0.70
	0.10	1.46	0.74	0.43
	0.50	0.76	1.40	0.56
	1.00	-	0.78	0.49

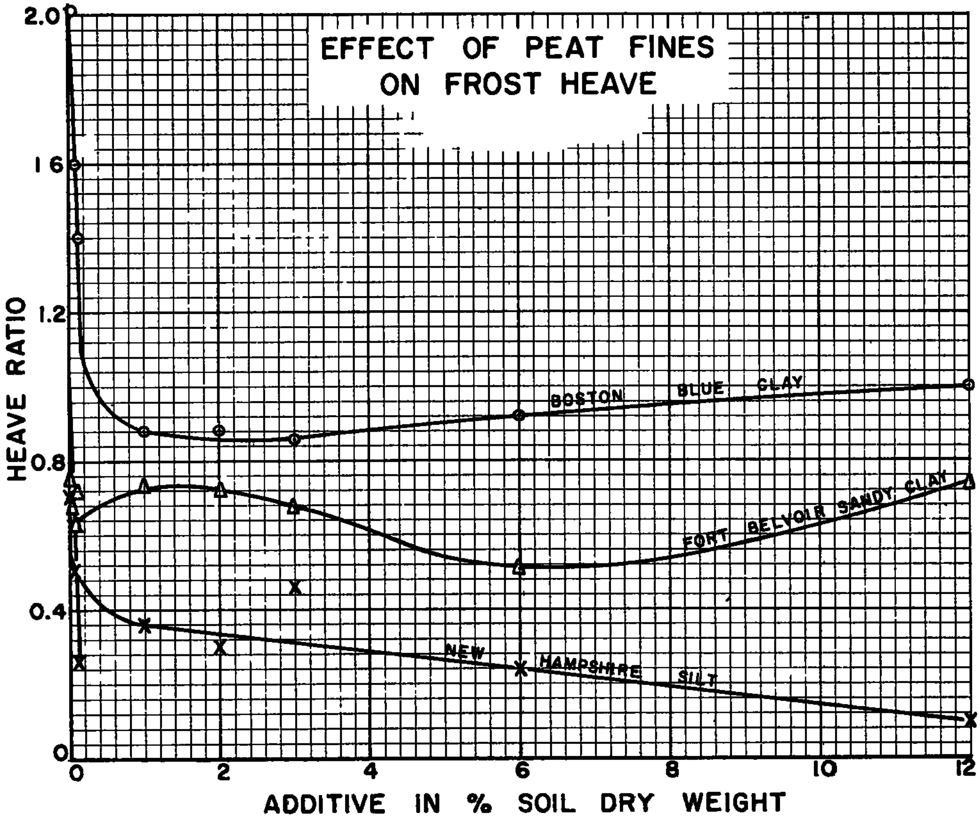
Belvoir sandy clay and that 10 percent prevented heave in New Hampshire silt.

Acrylate stabilization was developed for emergency military conditions; its cost is too great for large-scale non-emergency use at other than trace level treatments.

Resin-type Additives

Table 3 lists the heave ratios of soils treated with each of five resin-type additives. Since the four non-asphalt additives contain antistripping-type components, they do not need as much soil predrying to get good soil-additive bonding as does asphalt. This advantage does not show in Table 3, since all samples were dried before and after treatment.

The data show neither general significant reduction in frost susceptibility nor increase of effectiveness with increase in concentration. These facts suggest that the treatment was too low to obtain beneficial effects from plugging soil voids.



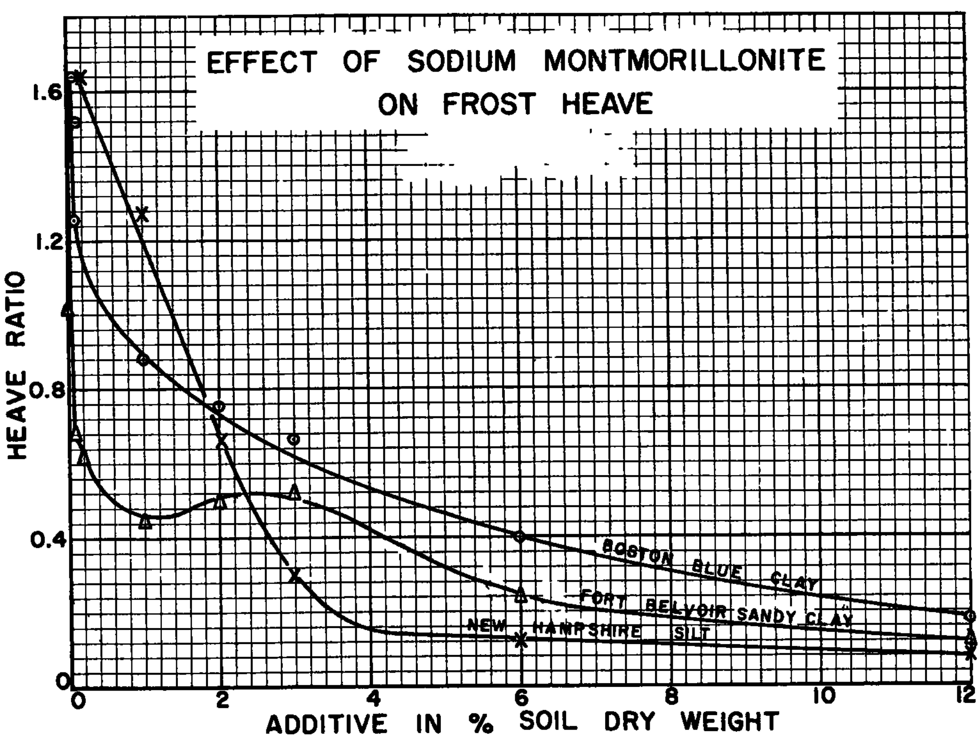


Figure 3 (above), and Figure 4 (below).

Portland Cement

Table 4, presenting results of tests on soils treated with portland cement, indicates cement is not a promising frost modifier. The data show that more than 5 percent cement is required to reduce the frost heave significantly, and that the dispersants made the cement more effective. However, a treatment of cement plus dispersant is not as effective as a treatment of dispersant without cement; for example, 1 percent dispersant reduced the heave ratio of Fort Belvoir sandy clay to 0.06 (see Table 7) compared to 0.26 for the sample with 5 percent cement plus 1 percent dispersant. The cement apparently had an adverse effect on the dispersant.

Natural Fines

Freezing tests were run on soils treated with two natural fines, sodium montmorillonite and a local peat, both of which are highly water sensitive. The peat, obtained at a depth of 4 feet below an athletic field in northwest Massachusetts, was a black fibrous material with some sand and gravel mineral particles. The portion coarser than a No. 200 sieve was removed and that portion passing through was used in the freezing tests. The organic content of the fines was measured to be 77 percent by the H_2SO_4 - $K_2Cr_2O_7$ digestion method (adapted from reference (12)). It had a specific gravity of 1.61, liquid limit of 375, and plastic limit of 260. The tests were performed both to evaluate the benefits of these materials as additives and to permit prediction of frost susceptibility of soils naturally containing either of these materials.

Figures 3 and 4 show that very low-level treatments of both additives caused an increase in heave, but that treatments usually greater than 0.1 percent and always greater than 1 percent reduced the heave. Sodium montmorillonite was more effective than peat fines.

Because both of these naturally occurring materials are water-sensitive, they are likely to have detrimental effects on soil; i.e., decrease strength and increase compressibility. Their water sensitivity would also cause problems in their field incorporation with soil.

EFFECT OF AGGREGANTS ON FROST HEAVE

Two types of aggregants — polymers and polyvalent cations — were studied. Table 5 presents the results on six widely different polymers. From this table the following observations can be made:

1. The polymers are generally not very effective.
2. The effect of concentration of polymer can be large and unpredictable.
3. The polymers can be detrimental

TABLE 6

EFFECT OF CATIONS ON FROST HEAVE

(In Heave Ratios)

Soil	Iron, $FeCl_3$	Iron, $Fe_2(SO_4)_3$	Lead, $PbAc_2$ [a]	Barium, $BaAc_2$ [b]	Potassium, KCl	Mercury, $HgCl_2$
Fort Belvoir	0 28	0 64	0.12	-	-	0 48
Sandy Clay	-	0 84	-	-	-	-
Boston Blue Clay	1.35	0.56	1 41	1.08	1 09	0.63
	0.20	0.35	-	-	-	-
	0.45	0.38	-	-	-	-
	0.40	-	-	-	-	-
New Hampshire Silt	0 48	-	0.37	-	-	0.77
Fairbanks silt	0 88	0.88	-	1.93	0.65	-
Niagara Falls Clay	0.03	-	-	0.98	2.09	-
Portsmouth sand	0.29	2 55	-	3.22	2.13	-
Loring till	0 05	0.44	-	1.20	0.52	-
Fargo clay	1.05	0 29	-	4.62	1.35	-
WASHO clay	0.12	1 32	-	0.97	0 62	-

[a] Lead Acetate
[b] Barium Acetate

TABLE 7

EFFECT OF DISPERSANTS ON FROST HEAVE

(In Heave Ratios)

Additives	Percent	Boston Blue Clay	New Hampshire Silt	Fort Belvoir Sandy Clay
Quadrafos	0.01	1.71	0.63	0.64
	0.05	2.03	0.48	0.76
		0.42, 0.74, 0.87	0.36, 1.17, 0.26	0.84, 0.83, 0.57
	0.10	0.65, 1.37, 0.48	0.85, 0.68, 1.09	0.62, 0.53, 0.53
		0.22, 0.30, 0.06	0.43, 0.36, 0.39	0.08, 0.18, 0.26
	0.50	0.29, 0.40, 0.31	0.64, 0.31, 0.32	0.18, 0.11, 0.22
	1.00	0.39	0.29	0.06
Sodium hexa-metaphosphate	0.01	1.31	1.32	0.81
	0.05	1.44	0.58	0.95
	0.10	1.32	0.40	0.69
	0.50	0.35	0.47	0.41
	1.00	0.25	0.23	0.06
Sodium tri-ployphosphate	0.10	1.20	0.50	0.50
	0.50	0.47	0.38	0.09
	1.00	0.32	0.36	0.00
Versenate	0.05	0.85	0.75	0.71
	0.10	0.69	0.71	0.18
	0.50	0.46	0.63	0.49
Tamol 731	0.01	1.13	0.98	0.71
	0.05	1.14	0.97	0.53
	0.10	0.94	0.54	0.39
	0.50	0.31	0.44	0.11
	1.00	0.26	0.38	0.00
Daxad 11	0.01	-	1.11	0.69
	0.05	-	0.83	0.68
	0.10	-	0.74	0.62
	0.50	-	0.15	0.20
	1.00	-	0.33	0.05, 0.09
Daxad 21	0.01	1.10	-	-
	0.05	0.98, 1.00	3.38	0.70
	0.10	1.06, 0.82	0.86	0.60
	0.50	0.63, 1.05	0.91	0.30
	1.00	0.44, 0.92	0.41	0.10
Marasperse N	0.05	1.57	0.40	0.78
	0.10	1.11	0.34	-
	0.50	1.36	0.43	-
Marasperse C	0.05	1.11	0.41	0.39
	0.10	1.41	2.46	0.64
	0.50	0.85	0.22	1.06
Lignosol	0.05	0.82	0.98	0.70
	0.10	0.64	1.18	0.40
	0.50	0.41	1.55	0.17
	1.00	0.51	0.57	0.17

4. The effectiveness of the polymers depends considerably on the soil treated. The unpredictable behavior of polymeric aggregants, especially the influence of aggregant concentration, is a discouraging fact which has been observed in other studies.

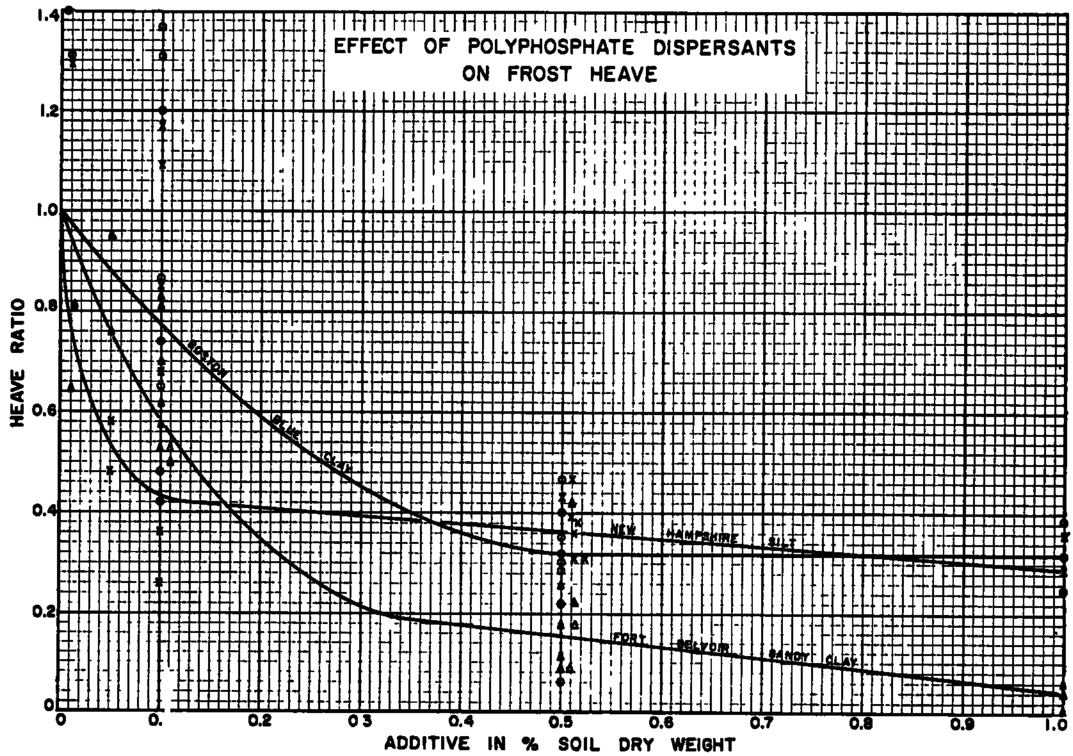


Figure 5. Effect of polyphosphate dispersants on frost heave.

As a matter of fact, many of these aggregants behave as dispersants at high-treatment levels. The modest effectiveness, importance of concentration, and high (chemical) cost combine to make the polymer aggregants unpromising as a group of additives for frost susceptibility modification.

Table 6 presents the effects of six cations on the frost heave of nine soils. Lead and mercury ions were investigated, not so much as aggregants but as "waterproofers" since they are non-hydratable ions. No treatment levels are given in Table 6; enough of each salt was added to saturate the ion exchange capacity of the soil with the salt's cations. Since all of the soils listed in Table 6 have low exchange capacities, the required treatments were low, always less than 0.5 percent. After treatment the soils were washed and dried.

The results show that some reaction in addition to ion exchange and ion fixation took place, since ferric sulfate was inferior to ferric chloride. The reasons for this difference were suggested previously where it was noted that ferric hydroxide, a potential cementing agent, could be formed from ferric chloride. Future tests on cations should consider the accompanying anions and the amount of salt used.

The main disadvantage to the use of cations, such as ferric iron, is the probable need for drying the treated soil. The importance of drying will be investigated. Certainly, the results on one-half of the soils treated with ferric chloride are most encouraging. Other studies (Massachusetts Institute of Technology Soil Stabilization Laboratory) have demonstrated the additional encouraging fact that ferric chloride has a beneficial effect on the strength characteristics of soil.

EFFECT OF DISPERSANTS ON FROST HEAVE

Table 7, presenting the results of freezing tests on miniature samples treated with various dispersants, indicates dispersants can be very effective as frost-heave reducers. The data show several important and favorable facts, as follows:

TABLE 8

EFFECT OF "WATERPROOFERS" ON FROST HEAVE

(In Heave Ratios)

Additive	Percent	Boston Blue Clay	New Hampshire Silt	Fort Belvoir Sandy Clay
SC-50	0.01	1.03	0.85	0.48
	0.05	0.96	0.51	0.36
	0.10	0.29, 0.43	0.82, 0.60	0.37, 0.47
	0.50	0.06, 0.06	0.87	0.06, 0.26
	1.00	0.05, 0.13	0.55	0.05, 0.01
XS-1	0.01	-	-	0.73
	0.05	0.63	0.78	0.65
	0.10	0.56	0.66	0.39
	0.50	0.13	0.53	0.12
	1.00	0.12	0.23	
Potassium phenyl siliconate	0.01	-	-	0.69
	0.05	0.48	1.20	0.72
	0.10	0.81	1.58	0.70
	0.50	0.33	0.45	0.60
	1.00	0.23	0.76	0.74
Triton K-60	0.10	0.87	0.86	0.74
	0.50	1.16	1.11	0.53
	1.00	1.02	0.58	0.71
Volan	0.01	0.77	0.40	0.58
	0.05	0.73	-	-
	0.10	0.58, 0.64	0.63, 1.36	0.27, 0.63
	0.50	0.00, 0.37	0.43	0.24, 0.22
	1.00	0.17, 0.08	0.13	0.11, 0.10
Quilon	0.01	-	1.31	0.53
	0.10	0.83, 0.89	0.66, 1.08	0.40, 0.65
	0.50	0.48, 0.88	0.98	0.38, 0.34
	1.00	0.55	0.04, 0.86	0.18, 0.14
Hyamine 1622	0.10	1.02	1.40	0.92
	0.50	1.31	0.84	0.42
	1.00	1.26	0.74	0.60
Hyamine 2389	0.10	1.06	1.86	0.59
	0.50	1.20	0.80	0.41
	1.00	1.24	0.56	0.42
Triethylene Tetramine	0.10	0.94	1.29	0.41
	0.50	1.16	0.57	0.42
	1.00	1.05	0.40	0.51
Hexamethylene Diamine	0.01	-	0.55	-
	0.05	-	0.78	-
	0.10	1.01	0.04, 0.96	0.49, 1.07
	0.50	2.04	0.74	0.38, 0.73
	1.00	1.99	0.66	0.16, 0.49
Di-N-butylamine	0.05	-	-	0.54
	0.10	1.83	-	0.44
	0.50	0.61	-	0.31, 0.69
	1.00	0.69	0.80	0.12, 0.25
Primene 81-R	0.01	-	0.95	-
	0.05	-	0.57	-
	0.10	2.51	0.10, 0.67	0.99
	0.50	3.54	0.50, 0.38, 0.04	0.75
	1.00	3.91	0.30, 0.05	0.81
Carbowax 200	0.01	-	-	0.55
	0.05	-	-	0.99
	0.10	1.59	1.11	0.50, 0.91
	0.50	0.70	0.65	0.12, 0.21
	1.00	0.62	0.76	0.20
Carbowax 6000	0.01	-	-	0.56
	0.05	-	-	0.61
	0.10	1.34, 0.86	1.00	0.57, 0.88
	0.50	1.23, 0.82	1.07	0.18
	1.00	0.31, 0.36	0.42	0.26
Arquad 2HT	0.05	0.29[a], 0.49	0.63[a], 1.05	0.66[a], 0.41
	0.10	(1.56, 0.39[a], 0.44)	(0.84, 0.92[a], 0.84)	0.76, 0.70[a], 0.39
Arneen 18D	0.10	1.61	1.10	0.89
Diethanol rosin amine D acetate	0.10	0.87	-	-
	0.50	1.12	-	-
	1.00	1.27	-	-
Monethanol rosin amine D acetate	0.10	0.65	-	-
	0.50	1.23	-	-

[a] Sample air-dried after chemical treatment.

1. All dispersants studied were effective.
2. There was no one dispersant significantly superior to the others.
3. The higher the treatment, the better the results, but improvement past 0.5 percent is slight

Figure 5 is a plot of heave ratio and concentration for the three polyphosphates tested. The sample molding conditions were intentionally varied for these tests from very dry to very wet optimum moisture. No trend of effectiveness varying with molding moisture could be detected; in fact, Figure 5 shows little variation in heave at any given additive concentration.

Dispersants are particularly promising as soil additives since they are effective in low concentrations, are relatively cheap, have beneficial effects on other soil prop-

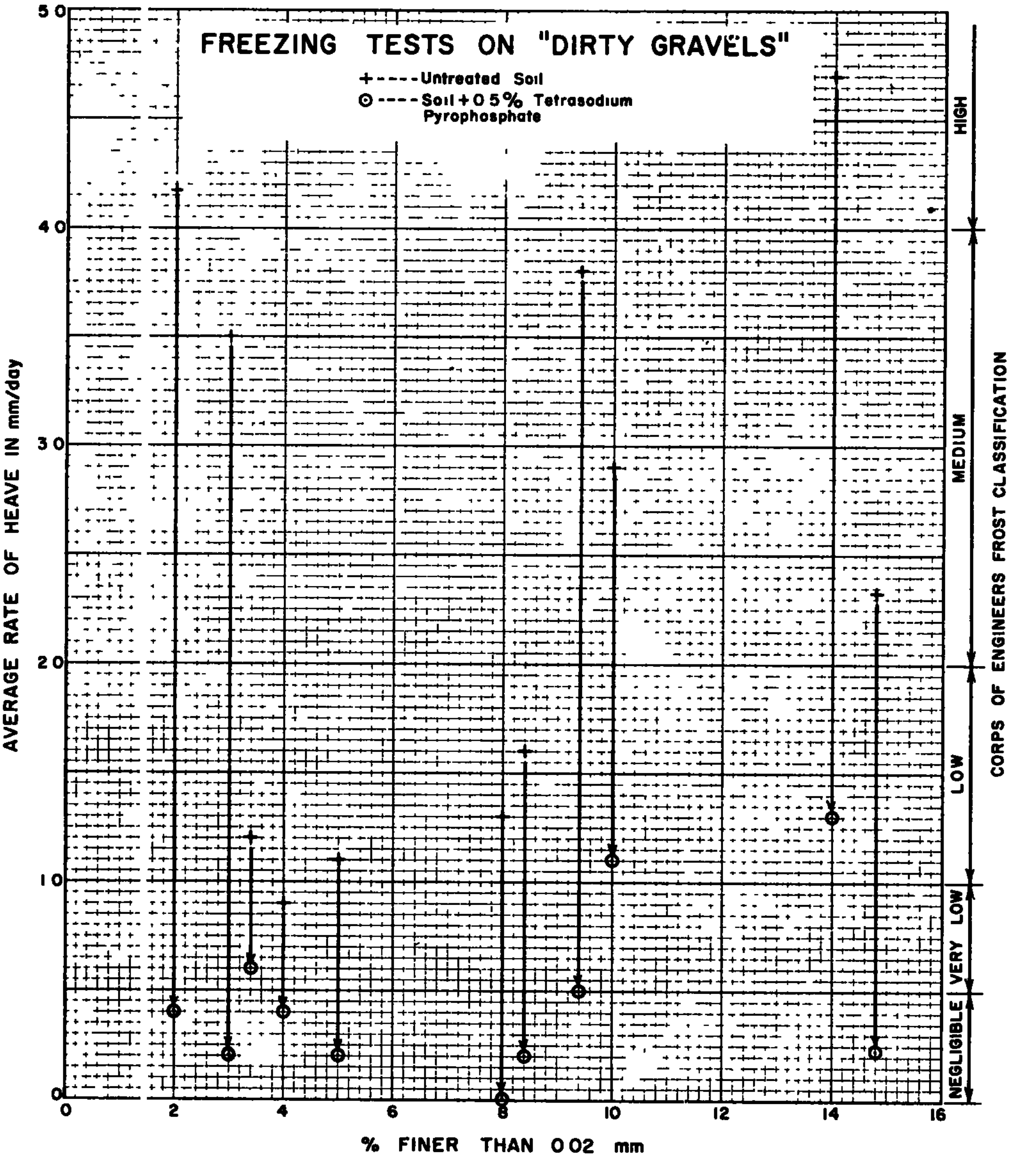


Figure 6. Freezing tests on ''Dirty Gravels.''

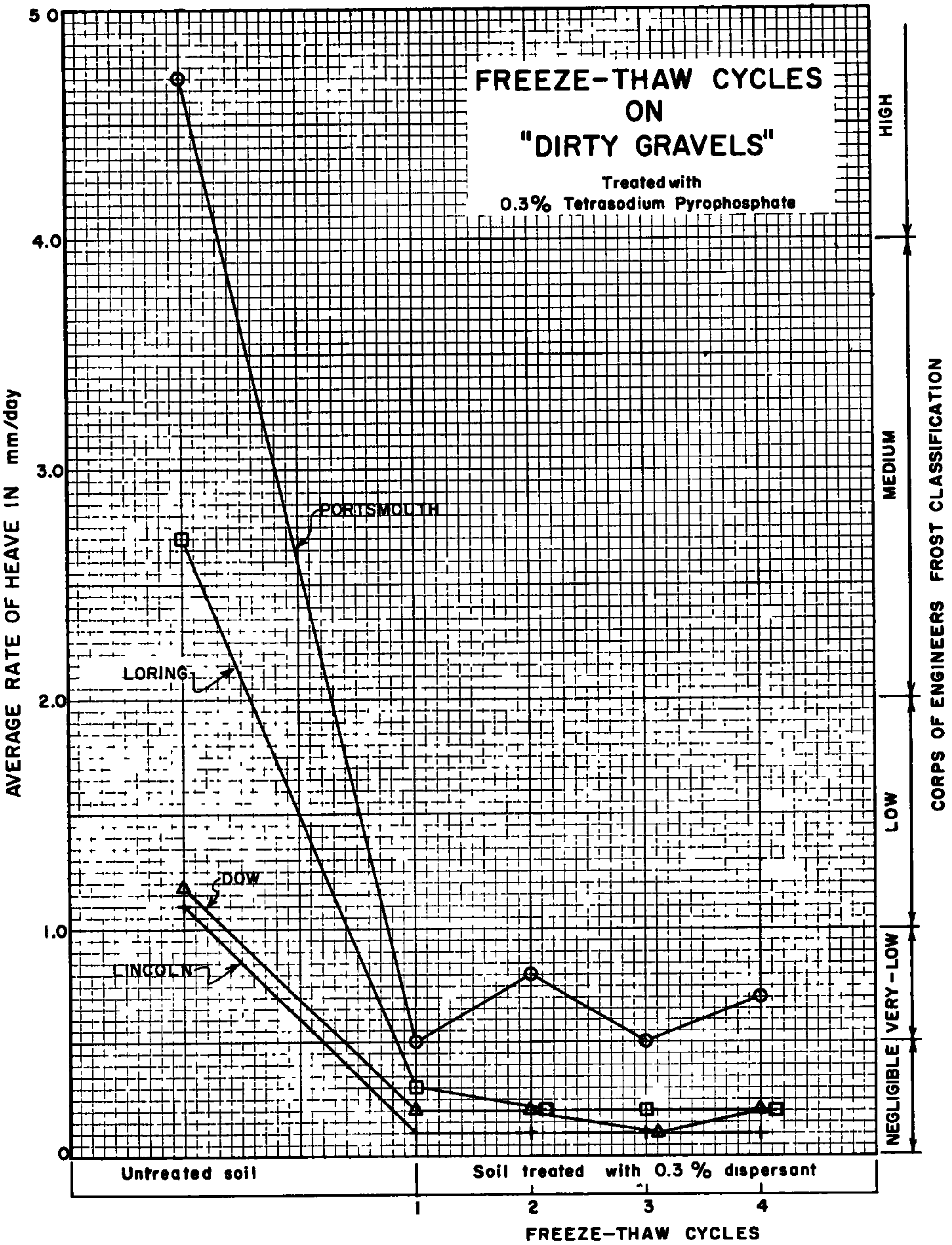


Figure 7. Freeze thaw cycles on ''Dirty Gravels.''

erties, are comparatively easy to incorporate, react instantaneously, and require no pre- or post-treatment curing (8). Based on the very favorable results obtained (Table 7), the tests described in the last part of this paper were conducted.

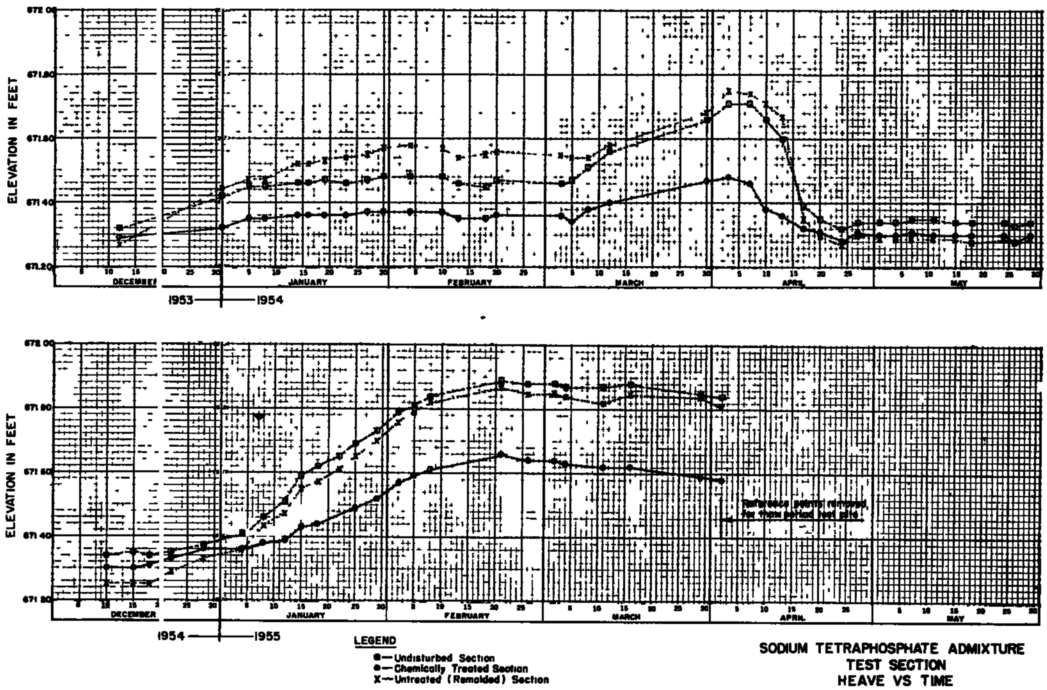


Figure 8. Sodium tetraphosphate admixture test section heave vs. time.

EFFECT OF "WATERPROOFERS" ON FROST HEAVE

Table 8 presents the results of tests to evaluate 18 "waterproofers" as frost modifiers. In all instances, the soils were dried after treatment; in some instances they were dried before the chemical was added. These necessary preparations and cure conditions are, of course, an undesirable feature of "waterproofers".

The results in Table 8 show that the effects varied from very beneficial to detrimental, with the majority being beneficial. As with polymeric aggregants, the effects of the "waterproofers" are not predictable; e.g., Primene was very beneficial with New Hampshire silt but most detrimental with Boston blue clay. As would be expected, the more "waterproofers" used, the better the results; generally, marked benefits were obtained only at treatments 0.5 percent and greater.

The high unit cost of "waterproofers" and the necessity of drying treated soils dim

TABLE 9

FREEZING TESTS ON "DIRTY" GRAVELS

Lab. No.	Source	Untreated Gravels			Gravels + 0.3 Tetrasodium Pyrophosphate			Heave Ratio
		Molding Water	Dry Unit Weight	Average Rate of Heave	Molding Water	Dry Unit Weight	Average Rate of Heave	
		Percent	pcf	mm/day	Percent	pcf	mm/day	
-	Greenland, TP-250	5.0	140.1	3.5	3.0	143.3	0.2	0.06
-	Dow AFB, Bangor, Maine, B-11	5.0	131.4	1.0	5.0	131.4	0.4	0.40
-	Dow AFB, Bangor, Maine, B-18	5 0	132.2	1.2	5 0	133 0	0.6	0.50
49-11	Ellsworth AFB, Weaver, South Dakota	6.0	137.0	1 3	5.0	137.0	0.0	0
49-8	Clinton County AFB, Wilmington, Ohio	9.0	129.0	3.8	5.0	129.4	0.5	0.13
49-21	Spokane AFB, Spokane, Washington	6.0	128.0	0.9	5.0	128 2	0.4	0.44
49-102	Lincoln AFB, Lincoln, Nebraska	7.0	132.2	1.1	4.8	134.4	0.2	0.18
49-60	Fairchild AFB, Spokane, Washington	4 5	131.3	2.9	6.3	131.3	1.1	0.38
49-54	Portsmouth AFB, Portsmouth, New Hampshire	8.5	127.0	4.7	5.0	129.8	1.3	0.27
49-17	Sioux Falls Airfield, Sioux Falls, South Dakota	4.0	131.0	1.6	11.1	128.5	0.2	0.12
49-9	Patterson AFB, Fairfield, Ohio	5.0	134.9	2.3	4.7	137.3	0.2	0.09

the prospects of these materials as frost modifiers even though they can be extremely effective.

FREEZING TESTS ON "DIRTY" GRAVELS TREATED WITH DISPERSANTS

Large percentages of the granular soils are unacceptable for fill material in the frost zone, not because of their strength characteristics under normal conditions, but because of their frost behavior. Even though a gravel with more than 3 percent by weight finer than 0.02 mm ("dirty" gravel) may have a very high California Bearing Ratio after soaking, it should not be used as a pavement base or foundation fill in the freezing zone because of frost-heaving and resultant weakening. A treatment that could make these otherwise excellent gravels non-frost-susceptible would be most useful.

To test the effectiveness of a chemical dispersant as a modifier of the frost characteristics of "dirty" gravels, 11 gravels were treated with 0.3 percent tetrasodium pyrophosphate and subjected to controlled laboratory freezing tests. The results of these tests are listed in Table 9 and plotted in Figure 6. They show that the polyphosphate reduced the rate of frost heave on all 11 gravels. The minimum reduction was to one-half of the untreated value, the maximum to essentially zero, and the average reduction was to one-fifth of the untreated value. Figure 6 shows that 75 percent of the gravels were made to fall within the Corps of Engineers' Relative Frost Susceptibility Classification lowest category, negligible (3).

In Figure 6, the results of freezing tests were plotted as a function of "percent finer than 0.02 mm" to see if the effectiveness of the dispersant was a function of fines content. The main relationship apparent is that the greater the rate of heave of the natural soil, the greater is the reduction caused by the dispersant; no relation between fines content and chemical performance is apparent.

The reason why dispersion is so potent in "dirty" gravels is not definitely known, but it may be the following: Since the overall structure of these soils is controlled by the gravel size particles, dispersion has little effect on the density (this fact can be observed from a comparison of the densities of untreated and treated samples, Table 9). Disaggregating the fines permits them to pack into a smaller space, thereby making the voids among gravel particles larger. In fact, the dispersed fines can be moved by pore water. Unpublished tests by Olsen at Massachusetts Institute of Technology which showed considerable removal of soil fines by leaching with a dispersant support this thought of "cleaning" the soil.

FREEZE-THAW CYCLES ON TREATED SOILS

For an additive to be of practical value, it must not only be an effective frost modifier, but it must also have reasonable permanence. The temporary effectiveness of salt to reduce the freezing point of pore water is, for example, a major drawback, as has already been mentioned. How permanent then are the other treatments, especially the very promising dispersants, described here?

Theoretical considerations suggest that when dispersants alter the structure of the entire soil mass, cycles of freezing and thawing may undo some of the improvements in structure. The effectiveness of a dispersant on a clay may well be gradually decreased over a number of years. On the other hand, where the structure benefits are limited to a small portion of the particles, freeze-thaw cycles probably have little, if any, influence.

A series of tests was conducted on standard-size samples (5.91 inches in diameter by 6.0 inches high) of four treated "dirty" gravels to ascertain the effect of freeze-thaw cycles. The results, presented in Figure 7, show no loss of dispersant effectiveness , during four cycles of freeze-thaw (i.e., the duration of the tests). Figure 7 also dramatically illustrates the pronounced reduction of frost action that can be obtained from dispersants.

The indicated permanence of dispersant treatment (Figure 7) is substantiated by the following field test.

FIELD TEST

To see if the laboratory tests indeed reflect field conditions, a small-scale test was

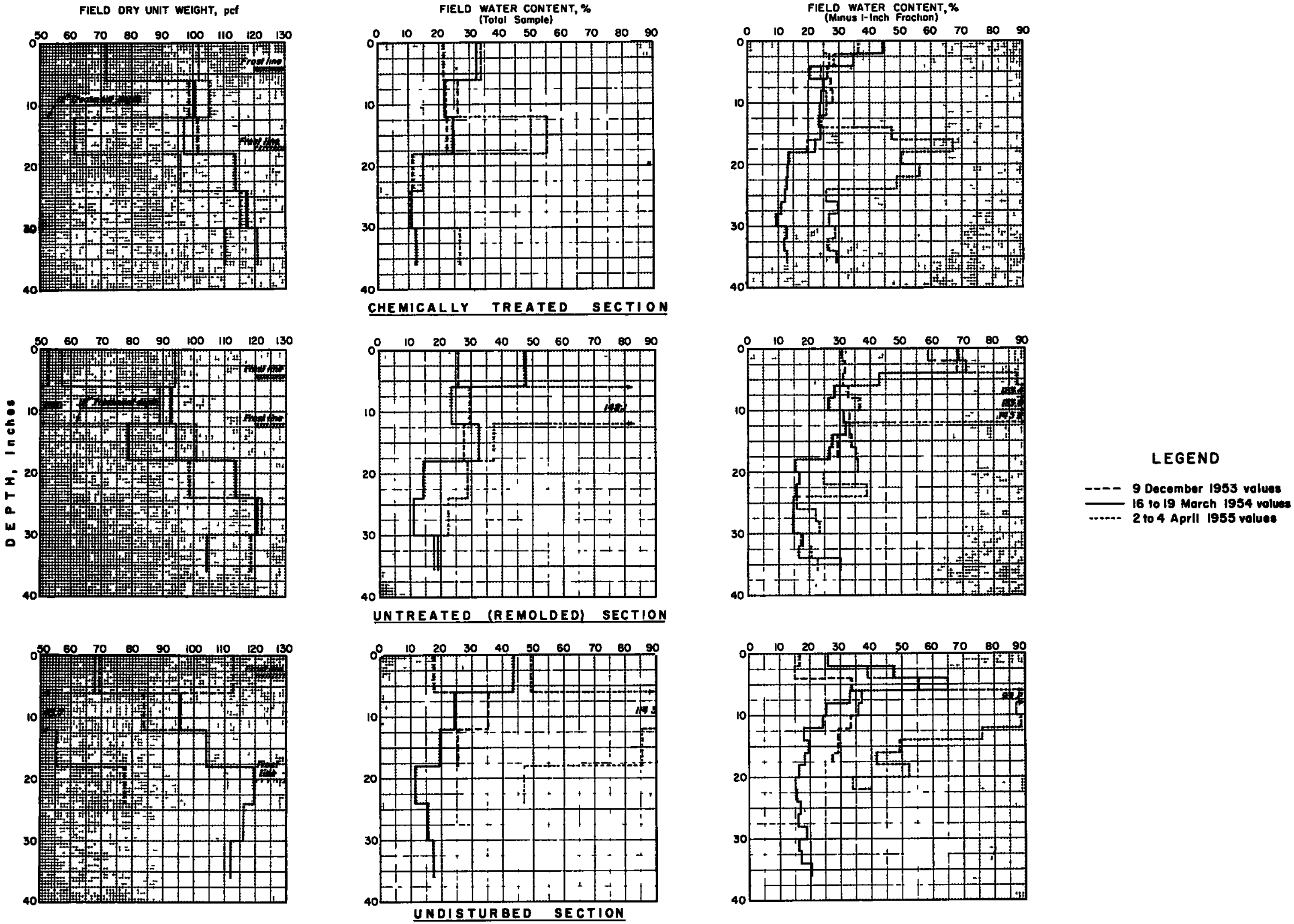


Figure 9. Sodium tetraphosphate admixture test section density and water content vs. depth.

conducted (and is still in progress) at Loring Air Force Base, Limestone, Maine. Three test sections, 4 feet by 4 feet by 1 foot deep, were prepared at a site where the water table is about 20 feet below ground surface. One section is of undisturbed soil; another of soil which had been remolded but given no chemical treatment; and a third section of soil which was hand-mixed with 0.3 percent sodium tetraphosphate, a dispersant. The soil is a clayey sandy gravel (i.e., glacial till) with a liquid limit of 22, a plasticity index of 8, and about 40 percent of its particles finer by weight than 0.02 mm.

The sections were prepared on 8 December 1953, and thus data from two freeze-thaw cycles are available. During the two frost-melting periods, the frost lines as located in test pits were as follows:

	Penetration of 32°F Isotherm (inches)		
Date	Undisturbed Untreated Section	Remolded Untreated Section	Chemically Treated Section
16-19 March 1954	3	4	4
2-4 April 1955	20	12	17

Some of the results to date are shown in Figures 8 and 9. Figure 8 presents ground surface elevation as a function of time; it shows the following:

	Heave (feet)		
Date	Undisturbed Untreated Section	Remolded Untreated Section	Chemically Treated Section
19 March 1954	0.28	0.35	0.14
5 April 1954	0.39	0.47	0.18
2 April 1955	0.50	0.56	0.28

In comparison with either of the untreated sections, the dispersant caused a significant reduction in heave. Most important is the fact that the dispersant was effective the second year; i.e., during the second freeze-thaw cycle. The heave ratios in the field test, approximately ½, compare well with those from the laboratory tests, ⅕ to ⅓.

An explanation of why the results of chemical treatment did not show up better in the field test is furnished by Figure 9. This figure shows two things: (1) the treated and untreated (remolded) soil sections were placed at very low dry unit weights of 100 and 92 pcf, respectively, caused by rain during construction (the Corps of Engineers' Modified AASHO Density Test[1] maximum is 137 pcf); and (2) most of the dry unit weight loss and moisture increase in the treated section occurred below the treated zone. In other words, a large percentage of the heave in the treated section occurred in the untreated soil underlying the treated soil. In fact, most of the 1955 heave in the treated section came from the untreated soil. Figure 9 shows that the dry unit weight loss during the 1955 thaw for the treated section was 22 pcf or 22 percent for the top 6 inches of treatment, while for the underlying untreated 6 inches of soil in the frost zone the dry unit weight loss was 40 pcf or 40 percent. These values suggest a heave ratio (0.55) which compares favorably with that obtained from the elevation observations (0.6).

This limited field test shows these encouraging results:

1. The laboratory test is indicative of field performance.
2. The chemical dispersant was effective in reducing frost heave, density loss, and moisture gain in the frost zone.
3. The effectiveness of the chemical was not adversely changed by a freeze-thaw cycle.

In future field tests, a site with a higher water table will be selected and the treatments will be extended below the frost zone.

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[1] Corps of Engineers' Modified AASHO Density Test is made by compacting soil in a 6-in. diameter cylinder, 1/10 cu ft in volume, using five layers, 10-lb tamper and 18-in. drop, 55 flows per layer.

TABLE 10

OVERALL EVALUATION OF ADDITIVES AS FROST MODIFIERS

Additive	Effectiveness[a] As Indicated By Laboratory Test	Requirements for Soil-Additive Reaction	Required Additive Concentration	Additive Cost Per Pound	Effect on Soil Properties Other Than Frost Action	Field Use	Comments	Evaluation[a] As Frost Modifier
			Percent					
Void Pluggers and Cement								
a In situ polymerization (calcium acrylate)	Excellent	Polymerization marked function of temperature	> 5	> 50¢	Beneficial increase strength density and decrease permeability	Difficult to control polymerization	Intended for emergency use	Poor
b Resins	Interesting	Drying after treatment – some require soil be predried	> 1	1 to 15¢	Beneficial	Other than cure requirements no special problems		Promising
c Portland cement	Interesting (with additives to cement)	Cure period for cement hydration	> 4	1 to 2¢ (cement) 6 to 12¢ (additives)	Beneficial	No special problems		Interesting to Poor
d Natural fines	Excellent	None	> 6	0 to 2¢	Probably detrimental to strength and permeability	Probably unusual mixing and processing problems		Interesting
Aggregants								
a Polymers	Interesting to poor	None	< 1	12¢ to $1 00	Beneficial	Moderate mixing and processing problems expected	Effectiveness unpredictable and function of concentration	Interesting to Poor
b Cations	Excellent	None for some, drying after treatment for others	< 0 5	2¢ and up	Beneficial	No special problems expected		Very Promising
Dispersants	Excellent	None	< 1	5¢ to $1 00	Beneficial	No special problems		Very Promising
'Waterproofers'	Excellent	Drying after treatment	> 0 5	25¢ to $2 00	Beneficial	Need for high degree of drying		Promising

[a]ADJECTIVE RATING SCALE poor, interesting, promising very promising, excellent

SUMMARY AND CONCLUSIONS

This report describes a search for additives to reduce the frost susceptibility of soil; the three-year search involved more than 1,000 freezing tests, using approximately 15 soils and 40 additives. The additives tried were chosen because of various theoretical reasons as to why each might reduce the moisture migration necessary for ice lens formation. The additives were divided into four groups — (1) void pluggers and cements, (2) aggregants, (3) dispersants, and (4) "waterproofers" — according to their action in soil.

Table 10 gives an overall evaluation of the additives studied. This evaluation considers effectiveness as a frost modifier as indicated by the freezing tests described herein, cost, and difficulty of field use. Any evaluation of the additives must consider the latter two practical items, even though the tests reported primarily measured additive effectiveness as a frost modifier. The evaluation in Table 10 is based on judgment as well as on quantitative test results.

The salts of polyvalent cations, especially ferric chloride, and the dispersants appear to be very promising as additives for reduction of the frost susceptibility of soil at low cost. Some of the resins and "waterproofers" show enough promise to warrant further laboratory testing.

A small-scale field test showed a laboratory-proved dispersant to be effective under field conditions; measurements made during the second freezing cycle showed no reduction in the potency of the dispersant treatment. Four freeze-thaw cycles on four soils tested in the laboratory also had no adverse effects.

While the primary objective of the test program was to screen additives, enough different soils were used to permit some important, if tentative, observations concerning effect of soil type. Well-graded soils with some coarse particles (gravel or large sand size) respond to treatment best. Uniform silts and moderately plastic clays are the least responsive. The most promising use of additives in in treatment of soils — well-graded ones with coarse particles — whose mass structure is determined by the large particles. Such "dirty" gravels, sandy clays, and silty sands can often be made essentially non-frost-heaving with additives at an economical cost; that is, at an additive cost of less than $1.00 per cubic yard of soil treated. The incorporation of additives with base and subbase soils may not necessarily be difficult, since they are usually obtained from borrow areas, placed in layers, and worked before compaction. These soils could be treated with little additional processing.

More field testing is required before the dispersants and other additives can be completely evaluated. Even though the dispersants appear to be effective in nearly all frost-susceptible soils, the characteristics of treated samples of the soil in question should be checked by laboratory freezing tests before dispersants are used in the field.

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Loss of Bearing Capacity and Vertical Displacements of New Jersey Soils

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● DURING the winter of 1954-1955 a study was made of the effects of frost action resulting from exposure to natural freeze-thaw conditions of prepared specimens of 30 New Jersey soil and subbase materials. This research was conducted by the Joint Highway Research Project, under the co-sponsorship of Rutgers University and the New Jersey State Highway Department. The materials were evaluated in relation to their frost-heaving characteristics and their relative losses of bearing capacity. Subsurface temperatures and moisture contents were studied in six of the soils.

Samples of 26 soil materials, representing approximately 75 percent of the soil areas of New Jersey, and of eight subbase materials in use in highway construction were brought to the field installation at Rutgers University. All materials were compacted in 9-square pits, the soils to a depth of 2 ft and the subbase materials to a depth of 1 ft.

Concrete slabs 4 ft square by 6 in. thick were poured on 30 of the materials, only 22 of the soils being used. Each slab was provided with brass pins at the corners for reference points in measuring elevation, and with four holes for the insertion of weighted plungers to bear upon the material beneath for the purpose of indicating loss of bearing capacity. Sufficient material was added to produce shoulders flush with the slab surfaces.

Fiberglas "soil moisture units" were installed at intervals of depth beneath the slabs and shoulders of six of the soils for the purpose of subsurface temperature and moisture content measurement. Maximum-minimum thermometers were installed for measuring air temperature. Three 20-ft deep wells were drilled for the determination of ground-water elevation and temperature.

The elevations of the slabs and plungers were measured daily with a permanently mounted wye level. All readings were made in the morning. The period of study was from December 28, 1954, to March 15, 1955.

A chart was prepared for each of the 30 materials showing the relationship between time and the following factors: (a) plunger penetration, (b) air temperature range, (c) precipitation, (d) slab displacement, (e) depth to ground-water table and, (f) ground-water temperature. An evaluation of the materials based on relative loss of bearing capacity was developed from the plunger penetration curves. The slab displacement curves were used to indicate the relative heaving characteristics of the materials.

For the six soils equipped with Fiberglas soil moisture units additional charts were prepared, showing frost penetration and moisture content beneath both the slabs and shoulders. These materials were compared according to rate and duration of frost penetration. The effect of precipitation on ground-water elevation, soil moisture content, frost heaving and loss of bearing capacity was noted.

The HRB A-1-a, A-1-b and A-3 materials in general showed the least amount of relative heave and loss of bearing capacity. Comparison between the reaction of these materials in 2-ft and 1-ft deep pits showed deterimental effects produced by the underlying material in shallow pits. The A-2-4 materials showed a wide range of relative reaction. The A-4 materials also showed a considerable range of reaction, but in general may be considered the most susceptible to frost action. The A-7-5 and A-7-6 materials showed considerably more relative loss of bearing capacity than relative heave.

The greatest depth and rate of frost penetration occurred in the granular soils as a result of their relatively high thermal conductivity. The greatest depths of frost penetration did not occur at the time of minimum air temperature nor did the maximum frost heaves occur at the time of greatest frost penetration. The fine-grained soils normally retained higher moisture contents than the coarse-grained soils. Moisture contents usually increase with depth in a soil. The effect of increased soil moisture content resulting from precipitation and percolation is modified with depth.

The dependence upon natural climatic conditions evidenced in this study shows the desirability of controlled laboratory conditions for further frost action research.

The primary objective of the frost action investigation initiated by the Joint Highway Research Project, under the co-sponsorship of Rutgers University and the New Jersey State Highway Department, was a study of the relative behavior of a selected representation of New Jersey highway subgrade and subbase materials resulting from exposure to natural freeze-thaw conditions.

CLIMATIC CONDITIONS

In relation to frost damage to pavements the winter of 1954-1955 may be classed as medium severe. Cold quantity for this winter was 285 degree-days, determined by totaling the differences between the daily mean temperatures and 32 deg. for the days that the mean was lower than 32 deg. U.S. Weather Bureau climatological data for the New Brunswick, N.J., weather station was used. This winter was the most severe since that of 1947-1948, which had a cold quantity of 528 degree days (Figure 1). The winter of 1947-1948 is a recent outstanding example of severity from the viewpoint of the extensive frost damage which resulted.

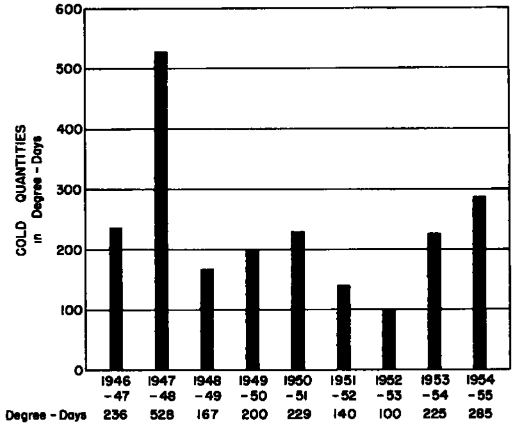


Figure 1. Cold quantities-New Brunswick Weather Station.

Requests for reports of frost damage occurring on roads in 1954-1955 were sent to all county engineers. Replies reported damage to bituminous surface treated and penetration macadam pavements. All damage reported was investigated. In nearly all cases the frost damage seemed to be the result of a combination of poor soil and inadequate drainage.

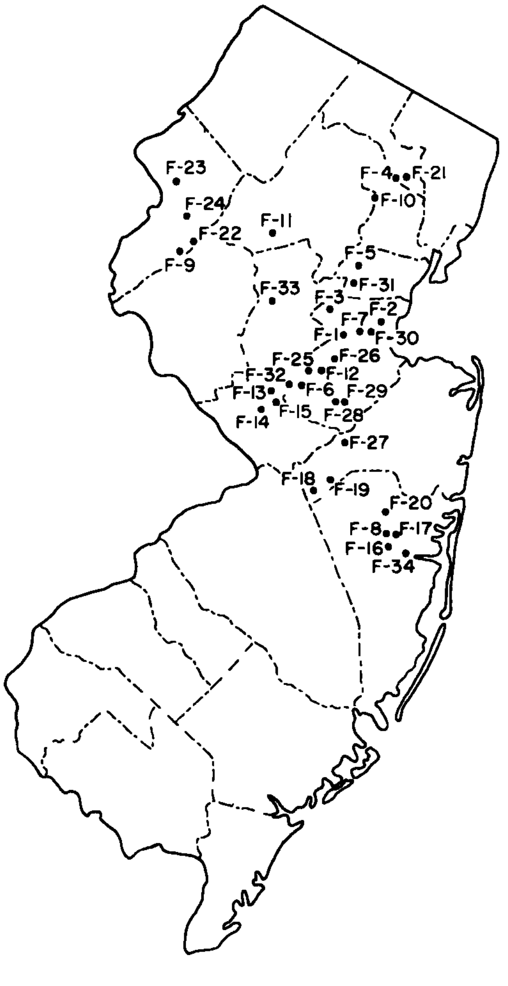


Figure 2. Map of New Jersey showing sample locations of soil and subbase materials used for frost action investigation.

DESCRIPTION OF FIELD INSTALLATION

Soil Preparation

Twenty-six soil materials, representing approximately 75 percent of the soil areas of New Jersey, were selected from various sites throughout the state. In addition,

Figure 3. Frost reaction installation, Rutgers University.

eight subbase materials in use in highway construction were suggested by the New Jersey State Highway Department (see Figure 2 and Table 1). In order that all of the materials could be studied under similar environmental conditions a sample of each was brought to the field installation at Rutgers University (Figure 3).

A representative fraction of each sample was tested in the Soil Mechanics Laboratory to determine its physical properties (Table 2). Grain size distribution curves of the materials are included in Appendix A. Existing soil at the field installation was Penn soil, a predominantly silty soil classed as A-2-4 in the Highway Research Board Classification because of a considerable percentage of soft shale fragments. Depth of soil to the parent material, Brunswick shale, was approximately 20 in.

The 26 soil materials were compacted in 6 in. layers in separate pits 9 ft square by 24 in. deep, the estimated maximum depth of frost penetration. The eight subbase materials were compacted in similar pits 12 in. deep as suggested by current construction practice. Arrangement of the pits containing the various soils at the field installation is shown in Figure 4. The field densities of the compacted materials were determined by the sand cone methods.

Concrete Slabs

The surfaces of the compacted soils were leveled and concrete slabs 4 ft square by 6 in. thick were poured on all but four of the materials (Figure 5). As reference points for determining vertical displacement ⅝-in. diameter brass pins were inserted in each corner of the slabs. During construction each slab was provided with four vertical holes by the inclusion of $2\frac{1}{16}$-in. diameter steel sleeves placed at the quarter points of the slab diagonals.

Penetration Device

Plungers having an outside diameter of 1.875 in. (contact area of 2.76 sq in.) were fabricated from steel pipe and filled with concrete. As reference points for penetration determination, the tops of the plungers were machined smooth. The plungers were inserted in the sleeves so as to bear upon the soil beneath. Heavy water-repellent grease was packed between the plungers and sleeves to exclude rain and snow. To develop contact pressures of approximately 10, 20, 30 and 40 psi, respectively, concrete weights of appropriate sizes were made to load the plungers. These contact pressures were selected after tests during the winter of 1949-1950 indicated that pressures of 50 psi or higher were not satisfactory.

TABLE 1

DESCRIPTION OF SOILS TESTED

F-1	A well-graded mixture of friable shale fragments from gravel to clay sizes Derived from Triassic shales The angular fragments might easily have been broken up by compacting them in the pits during placement of the sample
F-2	A sandy silt-clay mixture with considerable gravel Derived from glacial material of Triassic shale and sandstone origin
F-3	A silty sand with traces of gravel Derived from stratified glacial outwash, mostly Triassic rock fragments
F-4	A clay-silt-sand mixture with considerable gravel Derived from glacial drift, primarily gneiss and traprock
F-5	A sandy silt-clay mixture Derived from old glacial lake bed sediments
F-6	A silty, clayey sand with some gravel Derived from Coastal Plain sands and gravels
F-7	A gravelly sand with small amounts of silt and clay Derived from Coastal Plain sands and gravels
F-8	A fine sand with traces of silt and clay Derived from Coastal Plain sands and gravels
F-9	A clayey silt containing much sand Derived mostly from Kittatinny limestone
F-10	A mixture of coarse, medium and fine sands, containing considerable gravel and some silt and clay Derived from gneissic glacial materials which have been reworked by water
F-11	A well-graded mixture of gravel, sand, silt and clay Derived from granitoid gneiss
F-12	A silt and clay mixture containing considerable sand with traces of gravel Derived from marine clays
F-13	A sandy gravel with considerable silt and clay Derived from basalt and diabase (Gravel is large angular gragments)
F-14	A well-graded mixture of gravel, sand, silt and clay Derived from Triassic shale, sandstone and argillite
F-15	A well-graded sand-silt-clay mixture containing considerable gravel Derived from underlying Triassic shale, sandstone and argillite
F-16	A mixture of sands Derived from Coastal Plain sediments
F-17	A mixture of coarse, medium and fine sands with traces of gravel, silt and clay Derived from Coastal Plain sediments
F-18	Medium fine sand containing considerable clay and silt Derived from the glauconitic formations of the upper Coastal Plain
F-19	A gravelly, silty, clayey sand Derived from the glauconitic upper Coastal Plain deposits
F-20	Sand containing considerable gravel and some silt and clay Derived from poorly drained Coastal Plain sediments (This material had a very high organic content)
F-21	A sand, silt and clay mixture with traces of gravel Derived from glacial deposits of basalt and diabase
F-22	A sandy silt-clay mixture containing considerable gravel Derived from early glacial drift
F-23	A well-graded gravel-sand-silt-clay mixture Derived from glaciated Martinsburg shale (Gravel consists of large, flat shale fragments)
F-24	A well-graded mixture of gravel, sands, silt and clay. Derived from till containing much limestone
F-25	Coarse and medium sands containing considerable fine gravel and some silt and clay Subbase material
F-26	A medium sand with considerable gravel and some silt and clay Subbase material
F-27	Gravel containing considerable sand and some silt and clay Subbase material
F-28	A gravelly sand Subbase material
F-29	A sandy gravel Subbase material
F-30	A gravel and sand mixture containing numerous rounded shale particles Subbase material
F-31	A sandy gravel, essentially shale and sandstone Subbase material.
F-32	Traprock screenings
F-33	A sandy gravel Subbase material
F-34	A sandy gravel Subbase material

This penetration test, using weighted plungers and static loads applied continuously during the freezing season, was developed to indicate relative changes of soil bearing capacity during the test period. The procedure is not standard but was developed to meet a specific problem, as mentioned in the paragraph headed "Statement of Problem." The use of plungers to penetrate a confined soil may be compared with the field CBR test procedure. The application of a static load is, however, more nearly comparable to the plate bearing test procedure.

Facilities for Level Determination

Daily changes in elevation of the slabs and plungers were determined by means of a Gurley wye level and a Lenker L. E. Vation rod. An 8-ft by 20-ft instrument building was erected near the center of the field installation.

A 6-in. diameter steel pipe was placed vertically within the building, its lower end anchored to bedrock beneath the floor. A special head was fitted to the upper end for mounting the wye level at a convenient height. Adjustable apertures in three of the

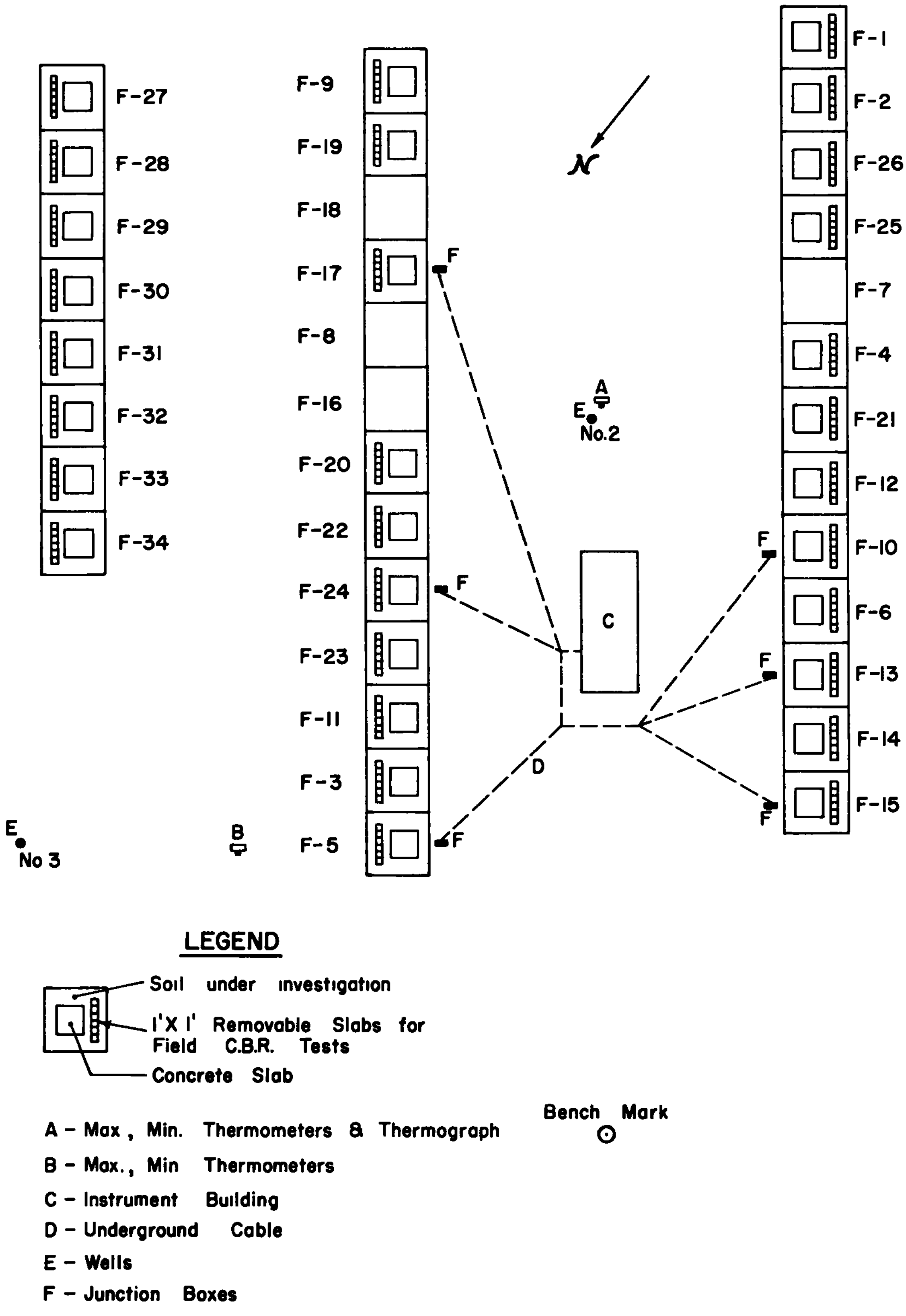


Figure 4. Field installation for frost action investigation.

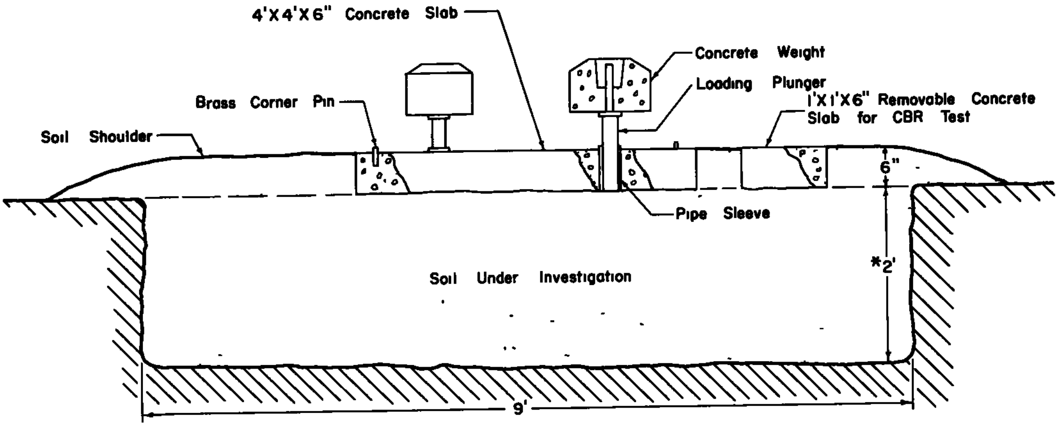


Figure 5. Soil installation.

building walls permitted unobstructed vision for level readings. For reference a permanent benchmark was established at the lower end of the field installation. A 1½-in. diameter steel pipe was anchored in bedrock well below the frost line and its upper end enclosed in a smooth tile to prevent disturbance by frost action.

Equipment for Other Data

Maximum and minimum thermometers were mounted at elevations of 1 ft and 6 ft above the ground surface to determine air temperature range. A recording thermograph mounted at a 3-ft elevation was used to obtain a continuous record of air temperature.

Subsurface soil temperatures and moisture contents were measured in six soils, F-5, F-10, F-13, F-15, F-17 and F-24, representing most of the soil types being investigated. Fiberglas soil moisture units were used. Each unit contains a thermistor for temperature measurement and Fiberglas-encased electrodes for moisture content determination. The units were installed in each of the soils beneath the concrete slabs, before pouring, at depths of 6 in., 10 in., 14 in., 18 in., 23 in. and 29 in. from the completed slab surface. Beneath the soil shoulders units were installed at depths of 0-in., 2 in., 6 in., 10 in., 14 in., 18 in., 23 in. and 29 in. (Figure 6).

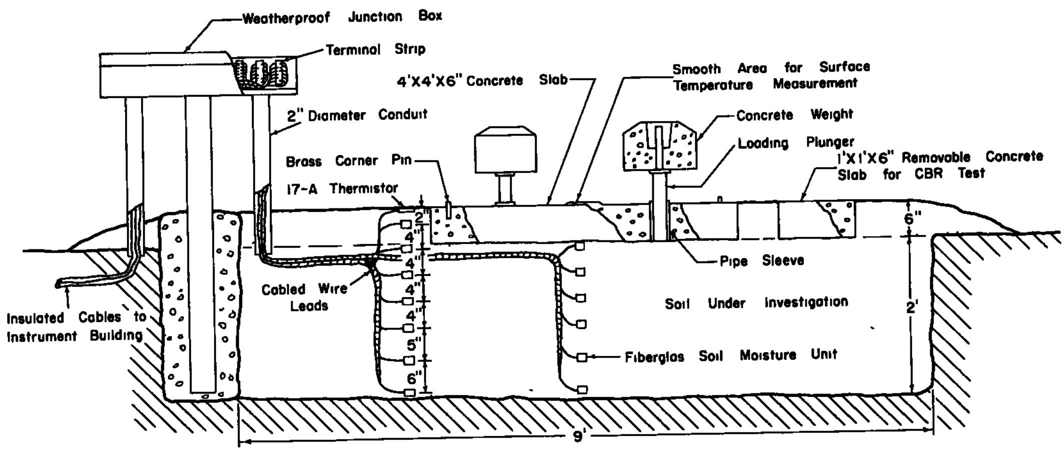


Figure 6. Soil installation equipped for subsurface soil temperature and moisture content determination.

All circuits were connected by underground cables to a selector panel and measuring instruments located in the instrument building.

Six 1 ft x 1 ft x 6 in. thick concrete slabs were poured on each material. Upon removal of these slabs during the spring and summer of 1955 field CBR tests were performed on the soil beneath as a means of determining loss and recovery of bearing capacity.

GROUND-WATER TABLE MEASUREMENTS

Three 20 ft deep wells were drilled at equal intervals on a diagonal across the field installation at positions shown in Figure 2. Steel casing was used for protection at the surface. A tape and float were assembled for measuring the elevation of the ground-water table in the wells. In order to measure ground-water temperature a thermometer was suspended in each well 15 ft from the surface by means of a cord, so that it could be drawn up for reading.

METHOD OF OBSERVATION

As soon as each slab was completed the plungers were inserted in the sleeves and sealed with grease. After 24 of the slabs had been completed a considerable delay was anticipated in the completion of the remaining six as a result of the lengthy procedure of moisture unit calibration. The initial slab elevations of the 24 were, therefore, determined and weekly elevations checked during the completion period of the remaining six.

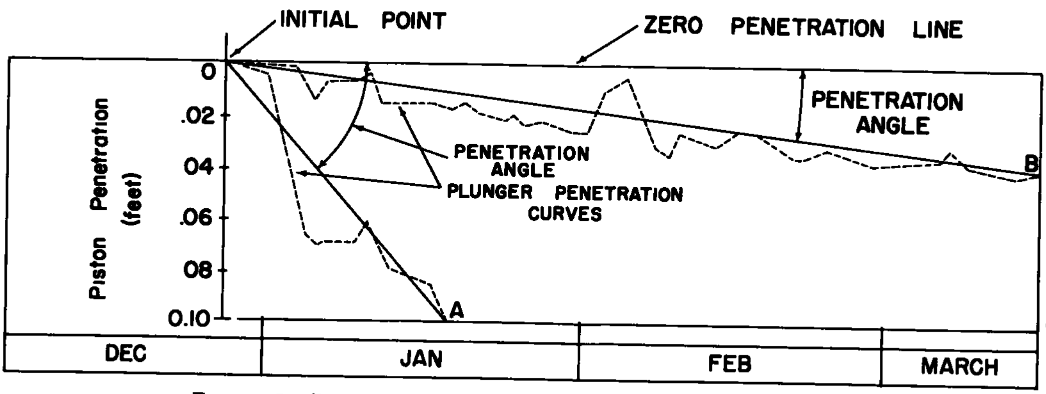


Figure 7. Determination of plunger penetration angles.

Immediately upon completion of the remaining slabs the weights were placed on all plungers, and new initial elevations were determined for all slabs and plungers. Throughout the freezing season daily level readings to the nearest 0.002 ft were taken on all slabs and plungers, with the exception of plungers that had reached maximum penetration. Snow was removed from the slabs when necessary to simulate highway conditions.

Daily air temperatures were recorded and weather and ground conditions noted. Subsurface soil temperatures and moisture contents were measured daily in the six soils equipped with moisture units. Ground-water elevation and temperatures were measured daily in the wells. Readings were stopped on March 15th, when all frost activity had ceased.

EVALUATION OF DATA

When the necessary calculations of data had been completed a graph was prepared for each of the soils investigated, showing the relationship between time and the following factors:

1. Plunger Penetration - Penetration curves for each plunger were plotted directly from field data.

2. Air Temperature Range - Maximum and minimum temperatures measured at a 1-ft elevation above the ground surface near the center of the field installation were used to determine air temperature range.

TABLE 2

ENGINEERING SOIL PROPERTIES

Sample No	Agronomic Name (as mapped 1917-27)	Soil Test Results: Sieve Analysis Percent Passing ¾	4	10	40	200	Hyd Anal Silt Sizes	Hyd Anal Clay Sizes	Atterberg Test L L	Atterberg Test P I	Proctor Max Dens	Proctor Opt M C	Uniform Coef $\frac{D_{60}}{D_{10}}$	Eff Grain Size	HRB Classification Sub-grade Group	HRB Classification Group Index
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
							%	%	%	%	pcf	%				
F-1	Penn	94	76	63	46	35	16	19	31	7	106	17	850 0	002	A-2-4	0
F-2	Wethersfield	94	86	82	64	43	19	23	32	16	119	13	360 0	.001	A-6	3
F-3	Dunellen	100	98	95	76	27	--	--	16	0	120	12	33 3	--	A-2-4	0
F-4	Gloucester	100	90	86	79	56	31	21	25	6	109	16	73 3	.0015	A-4	4
F-5	Whippany	100	100	100	98	83	43	37	41	7	100	22	50 0	--	A-5	8
F-6	Sassafras	99	95	93	79	42	20	21	28	12	117	14	16 7	0015	A-6	2
F-7	Sassafras	88	67	61	28	7	--	--	NL	NP	120	12	12.0	.15	A-1-b	0
F-8	Sassafras	100	100	98	78	4	--	--	NL	NP	106	15	1 9	16	A-3	0
F-9	Hagerstown	100	99	98	92	83	40	34	43	20	101	20	51 1	--	A-7-6	13
F-10	Merrimac	100	90	77	41	11	--	--	NL	NP	125	9	11 4	07	A-1-b	0
F-11	Chester	89	74	70	55	46	26	16	33	11	109	18	30 4	023	A-6	2
F-12	Elkton	99	97	95	89	79	45	31	28	10	108	16	113 3	--	A-4	8
F-13	Montalto	91	80	58	46	28	9	19	32	9	114	17	360 0	03	A-2-4	0
F-14	Croton	97	80	73	68	64	23	27	41	21	100	21	233 3	--	A-7-6	15
F-15	Lansdale	99	87	85	69	55	21	32	41	15	95	26	283 3	--	A-7-6	6
F-16	Lakewood	100	100	100	73	1	--	--	NL	NP	102	15	2.2	16	A-3	0
F-17	Lakewood	100	99	98	64	3	--	--	NL	NP	106	14	2 8	15	A-3	0
F-18	Collington	100	100	100	80	26	10	15	32	8	105	23	166 7	0018	A-2-4	0
F-19	Collington	96	91	87	69	39	12	18	48	14	97	27	113 7	--	A-7-5	2
F-20	Portsmouth	99	87	84	56	7	--	--	NL	NP	118	10	3.7	13	A-3	0
F-21	Holyoke	99	98	96	89	60	32	20	27	12	116	14	40 0	002	A-6	6
F-22	Washington	93	88	85	76	64	25	36	31	10	104	18	130 0	--	A-4	6
F-23	Dutchess	93	84	72	61	52	26	18	31	9	110	15	22 5	.0016	A-4	3
F-24	Dover	82	72	66	54	37	20	14	31	9	112	16	400 0	.0025	A-4	0
F-25	Subbase Sand Hills	97	96	93	54	6	--	--	NL	NP	106	15	2 7	17	A-3	0
F-26	Subbase Farrington	93	86	78	36	10	--	--	NL	NP	120	12	8 8	08	A-1-b	0
F-27	Subbase Perrinville	85	48	40	24	10	3	6	NL	NP	122	12	58 3	12	A-1-a	0
F-28	Subbase Bot. Jamesburg	94	78	71	41	2	--	--	NL	NP	108	16	2 7	.26	A-1-b	0
F-29	Subbase Top Jamesburg	87	35	26	12	3	--	--	NL	NP	123	10	26 5	4	A-1-a	0
F-30	Subbase Nixon	89	66	48	17	4	--	--	NL	NP	119	13	17 5	2	A-1-a	0
F-31	Zimmerman Pit Westfield	74 5	53 6	46.4	9 3	2	--	--	NL	NP	112 3	13	26 5	4	A-1-a	0
F-32	Kingston Traprock Screening	100	97.5	84 4	40 7	15	--	--	NL	NP	131 3	10	19	--	A-1-b	0
F-33	Franklin Pit North Branch	83 9	52 3	44 4	22	3	--	--	NL	NP	122.4	12	41 7	24	A-1-a	0
F-34	Whitt Pit Toms River	97.4	55 6	39 4	8 7	1	--	--	NL	NP	115	13	14 4	.45	A-1-a	0

3. Precipitation - Precipitation data, plotted as a bar graph, were obtained from the U.S. Weather Bureau climatological data for the New Brunswick, N.J., weather station. The letter "s" above a bar in the precipitation graphs indicates snow.

4. Slab Displacement - All curves with the exception of some slab displacement curves start on December 28th, the date of the first complete readings. A comparison of initial slab elevations determined on this day and those determined several weeks previously for the slabs completed at that time showed that some minor heaving had occurred during the period of completion of the remaining slabs. It was found that all of the slabs had again reached their approximate original initial elevations by December 30th. Therefore, the displacement curves for a number of slabs were started on either December 29th or 30th, using as initial elevations the actual elevations occurring on those days. Any error in initial elevation is thus minimized and all of the curves are comparable.

5. Depth to Ground-Water Table - Ground-water elevation data produced three curves showing respective depth to the ground-water table at the elevations of the three wells.

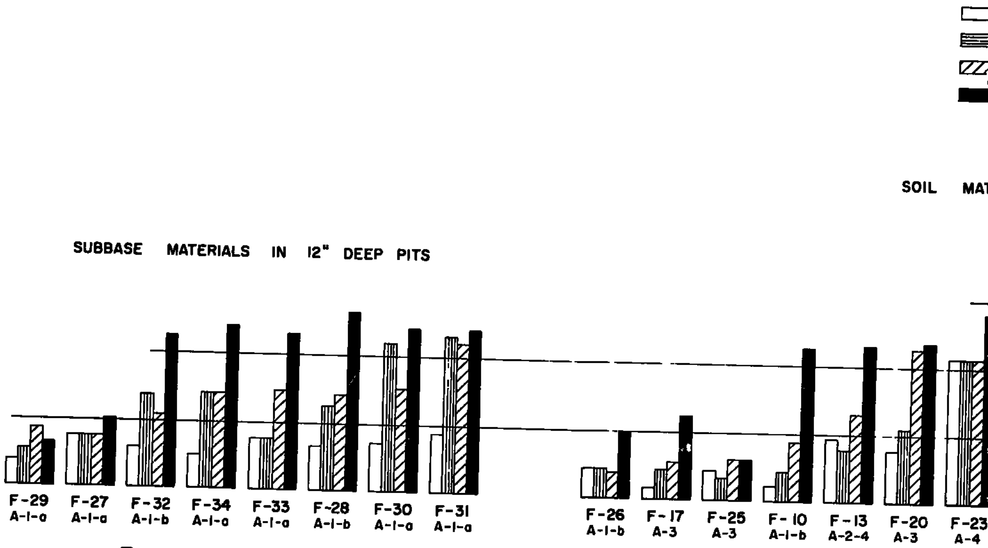


Figure 8. Relative loss of bearing capacity of 30 New Jersey subbase and
winter of

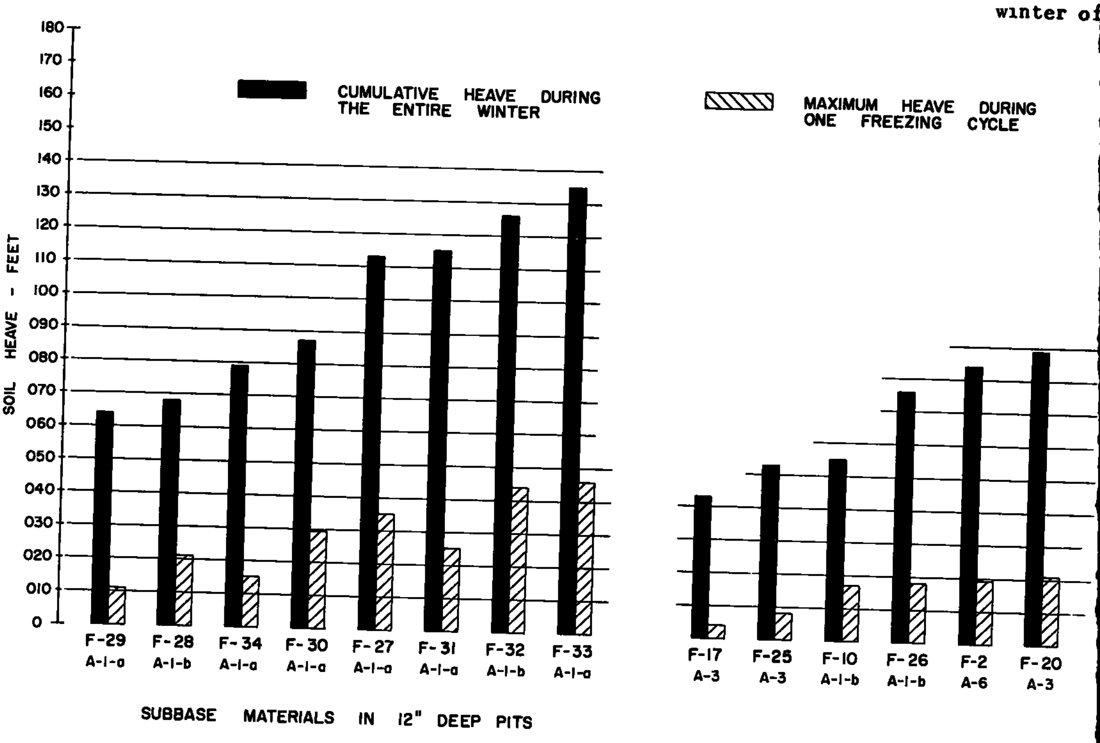
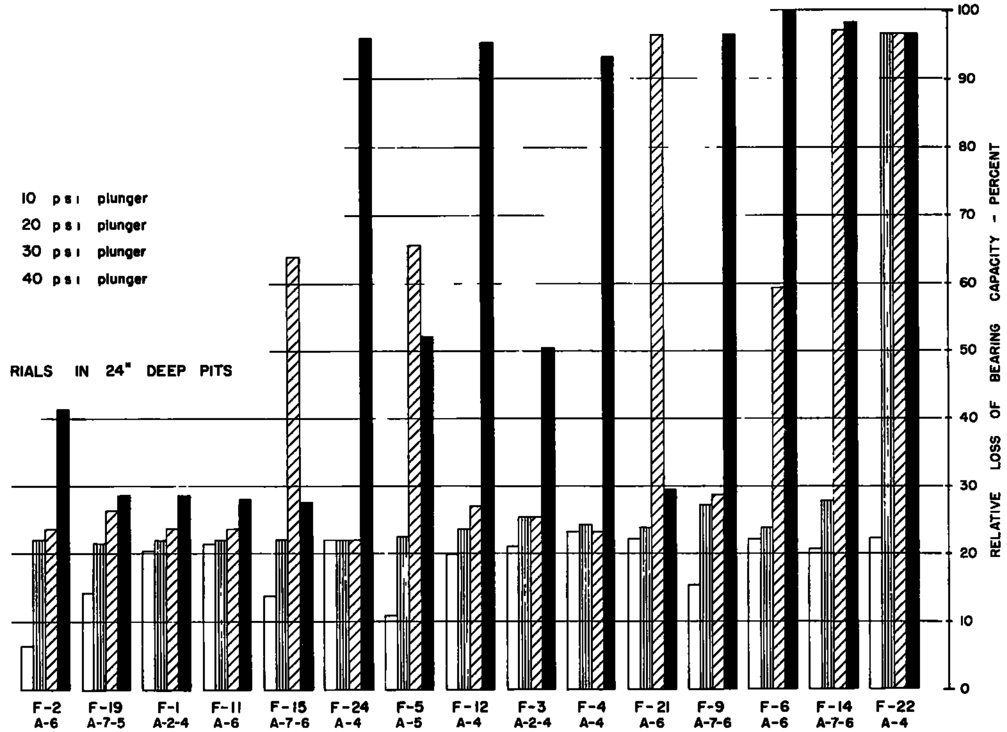
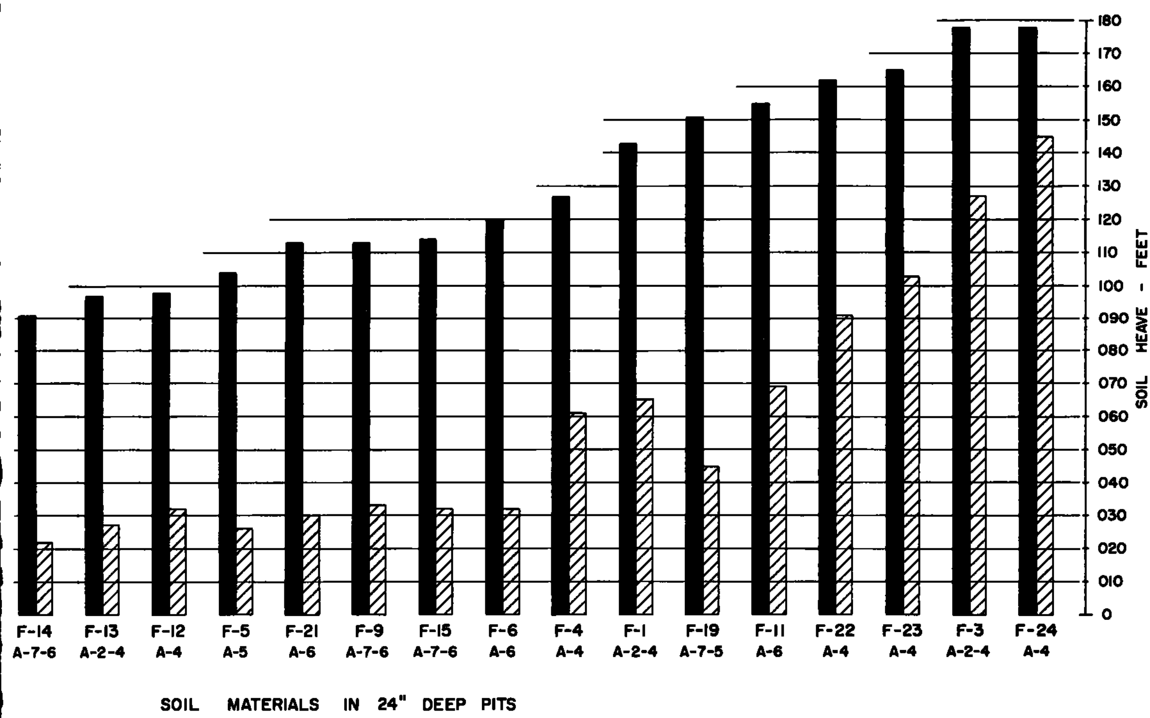


Figure 10. Frost heaving characteristics of 30 New Jersey subbase and
winter of



soil materials exposed to natural freeze-thaw conditions during the 1954-1955.



soil materials exposed to natural freeze-thaw conditions during the 1954-1955.

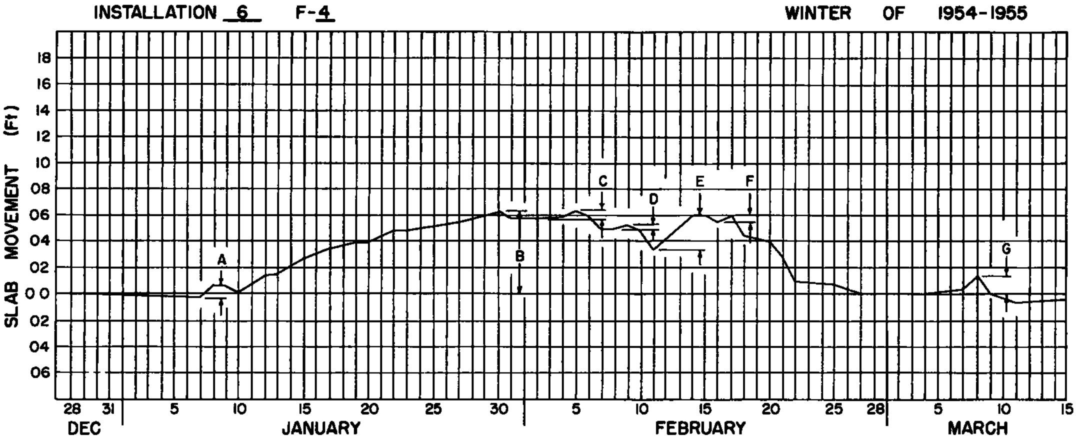


Figure 9. Determination of maximum and cumulative soil heave.

TABLE 3

RELATIVE ORDER OF MATERIALS FROM LEAST TO GREATEST LOSS OF BEARING CAPACITY AS DETERMINED BY MAGNITUDE OF PLUNGER PENETRATION ANGLE

Relative Position in Sequence	10 psi Contact Pressure	Penetration Angle	20 psi Contact Pressure	Penetration Angle	30 psi Contact Pressure	Penetration Angle	40 psi Contact Pressure	Penetration Angle
				Subbase materials in 12 in deep pits				
1	F-29	3 5	F-29	5 0	F-27	7 0	F-29	6 0
2	F-34	4 5	F-33	7 0	F-29	8 0	F-27	9 5
3	F-32	5 5	F-27	7 0	F-32	10 0	F-32	20.5
4	F-28	6 0	F-28	11.5	F-28	13 0	F-33	21 0
5	F-30	6 5	F-32	12 5	F-34	13 0	F-34	22 0
6	F-27	7 0	F-34	13 0	F-33	13 5	F-31	22 0
7	F-33	7 0	F-30	20 0	F-30	14 0	F-30	22 0
8	F-31	8 0	F-31	21 0	F-31	20 0	F-28	24 0
				Soil materials in 24 in deep pits				
1	F-17	1 5	F-25	3 0	F-26	3 5	F-25	5 5
2	F-10	2 0	F-10	4 0	F-17	5 0	F-26	9 0
3	F-25	4 0	F-17	4 0	F-25	5 5	F-17	11 5
4	F-26	4 0	F-26	4 0	F-10	8 0	F-10	20 5
5	F-2	6.0	F-13	7 0	F-13	12 0	F-13	21 0
6	F-20	7 0	F-20	10 0	F-23	19 5	F-20	21 5
7	F-13	8 5	F-23	19 5	F-24	20 0	F-15	25 0
8	F-5	10 0	F-19	19 5	F-20	20 5	F-11	25 5
9	F-15	12 5	F-1	20 0	F-4	21 0	F-23	25 5
10	F-19	13 0	F-11	20 0	F-1	21 5	F-1	26 0
11	F-9	14 0	F-24	20 0	F-11	21 5	F-19	26 0
12	F-12	18 0	F-15	20 0	F-2	21 5	F-21	26 5
13	F-1	18 5	F-2	20 0	F-3	23 0	F-2	37 5
14	F-14	18 5	F-5	20 5	F-19	24 0	F-3	45 5
15	F-3	19.0	F-6	21 5	F-12	24 5	F-5	47 0
16	F-11	19 5	F-21	21 5	F-9	26 0	F-4	84 0
17	F-23	19 5	F-12	21.5	F-6	53 5	F-12	86.0
18	F-21	20 0	F-4	22 0	F-15	57.5	F-24	86 5
19	F-6	20 0	F-3	23 0	F-5	59.0	F-9	87 0
20	F-24	20 0	F-9	24 5	F-22	87 0	F-22	87 0
21	F-22	20 0	F-14	25 0	F-21	87 0	F-14	88.5
22	F-4	21.0	F-22	87 0	F-14	87 5	F-6	90 0

TABLE 4

ORDER NUMBERS BASED ON MAGNITUDE OF PENETRATION ANGLES

Soil No.	Plunger Contact Pressures 10 psi	20 psi	30 psi	40 psi	Average Order No.
		Subbase materials in 12 in. deep pits			
F-27	6	2	1	2	2.75
F-28	4	4	4	8	5.00
F-29	1	1	2	1	1.25
F-30	5	7	7	5	6.00
F-31	8	8	8	5	7.25
F-32	3	5	3	3	3.50
F-33	6	2	6	4	4.50
F-34	2	6	4	5	4.25
		Soil materials in 24 in deep pits			
F-1	13	9	10	10	10.50
F-2	5	9	10	13	9.25
F-3	15	19	13	14	15.25
F-4	22	18	9	16	16.25
F-5	8	14	19	15	14.00
F-6	18	15	17	22	18.00
F-9	11	20	16	19	16.50
F-10	2	2	4	4	3.00
F-11	16	9	10	8	10.75
F-12	12	15	15	17	14.75
F-13	7	5	5	5	5.50
F-14	13	21	22	21	19.25
F-15	9	9	18	7	10.75
F-17	1	2	2	3	2.00
F-19	10	7	14	10	10.25
F-20	6	6	7	6	6.25
F-21	18	15	21	12	16.50
F-22	18	22	20	19	19.75
F-23	16	7	6	8	9.25
F-24	18	9	7	18	13.00
F-25	3	1	3	1	2.00
F-26	3	2	1	2	2.00

These curves were interpolated where necessary to determine depth to ground-water curves for each soil based on the respective elevation of the soil.

6. Ground-Water Temperature - The ground-water temperature curve was plotted from the average temperature determined in the three wells. During the period of observations average ground-water temperatures fluctuated from a maximum of 53 deg to a minimum of 46 deg.

EVALUATION OF PLUNGER PENETRATION

An examination of the plunger penetration curves shows that the end point of each curve is defined by either of two limits: (a) a penetration value of 0.1 ft, because rates of penetration showed considerable increase beyond this value; or (b) the March 15th date line, the point at which observations were discontinued.

For purposes of comparison it was necessary to represent each plunger reaction by a numerical value. Because of the dual limits imposed on the plunger penetration curves, representative numerical values had to be of a two-dimensional nature. Measurement of "penetration angles" was selected as the best means of determining these values.

A plunger "penetration angle" is defined as the angle between the line of zero penetration and a straight line connecting the initial and end points of a plunger penetration curve (see Figure 7). The end point of a plunger penetration curve is either that point where penetration exceeds 0.1 ft (Point A, Figure 7) or that point where the penetration curve reaches the March 15th date line (Point B, Figure 7). Any plunger penetration angle may be compared directly only with angles developed by plungers having similar contact pressures. Angles may vary from 0 deg (indicating no loss of bearing capacity during the observation period) to 90 deg (indicating an immediate complete loss of bearing capacity).

The soils and subbase materials were evaluated as separate groups because of the environmental differences induced by the 24 in. and 12 in. deep pits. A further division of materials according to the four plunger contact pressures produced eight subgroups. Within these subgroups plunger penetration angles were measured and tabulated. Within the subgroups the soils were placed in sequence, from least to greatest loss of bearing capacity. Their relative positions in sequence were determined by the magnitude of the penetration angles (Table 3).

Each material was then assigned four order numbers, determined by its position in sequence in the four contact pressure subgroups (Table 4). In cases of duplication of penetration angles within the subgroups, materials having the same angles were assigned the same order numbers. A comparison of the order numbers indicates that each of the four contact pressures produced, in general, similar sequences of materials relative to loss of bearing capacity.

The four order numbers for each material were averaged, and the materials were placed in sequence from least to greatest average order number. Where duplicate average order numbers occurred, the soil of better HRB rating was placed first.

The same sequence represents the relative order of materials from least to greatest loss of bearing capacity (Table 5). For the purpose of easily comparing the relative performance of the various materials, loss of bearing capacity in percent is plotted in Figure 8. Loss of bearing capacity has been considered proportional to plunger penetration angle. The graph was prepared by plotting penetration angles from the limits of 0° to 90° and superimposing a scale of loss of bearing capacity from 0 to 100 percent.

TABLE 5

RELATIVE ORDER OF MATERIALS FROM LEAST TO GREATEST LOSS OF BEARING CAPACITY

Relative Position in Sequence	Soil No	H R B. Class	Average Order No
Subbase materials in 12 in deep pits			
1	F-29	A-1-a	1 25
2	F-27	A-1-a	2.75
3	F-32	A-1-b	3 50
4	F-34	A-1-a	4.25
5	F-33	A-1-a	4.50
6	F-28	A-1-b	5 00
7	F-30	A-1-a	6 00
8	F-31	A-1-a	7 25
Soil materials in 24 in deep pits			
1	F-26	A-1-b	2 00
2	F-17	A-3	2 00
3	F-25	A-3	2 00
4	F-10	A-1-b	3 00
5	F-13	A-2-4	5 50
6	F-20	A-3	6 25
7	F-23	A-4	9 25
8	F-2	A-6	9 25
9	F-19	A-7-5	10 25
10	F-1	A-2-4	10.50
11	F-11	A-6	10 75
12	F-15	A-7-6	10 75
13	F-24	A-4	13 00
14	F-5	A-5	14 00
15	F-12	A-4	14 75
16	F-3	A-2-4	15.25
17	F-4	A-4	16 25
18	F-21	A-6	16.50
19	F-9	A-7-6	16 50
20	F-6	A-6	18 00
21	F-14	A-7-6	19.25
22	F-22	A-4	19 75

EVALUATION OF SLAB DISPLACEMENT

Vertical displacement of the concrete slabs indicated directly the amount of heave that each material experienced upon freezing. The effect of slab thickness upon soil heave was eliminated as far as comparison between materials was concerned as all slabs were of uniform thickness. It is reasonable to assume that the soil materials showing the greater heave can be considered to be more susceptible to frost action. To account also for the effect produced by alternate freeze-thaw conditions two methods for evaluating the materials were used; namely, (a) a comparison of maximum heaves, and (b) a comparison of cumulative heaves. The subbase materials in 12 in. deep pits and the soil materials in 24 in. deep pits were evaluated separately by each method because of the difference in their environmental field conditions.

MAXIMUM HEAVES

The maximum heave experienced by each material may be defined as the greatest total upward movement of the slab during one freeze-thaw cycle (Figure 9). After determination of the maximum heaves the materials were listed in order from least to greatest maximum heave (Table 6).

CUMULATIVE HEAVES

Cumulative heaves were determined as a means of further breakdown between those materials having maximum heaves of the same magnitude. Cumulative heave for each material is the total of the heaves occurring during each successive period of freeze and thaw (Figure 9). Cumulative heave may give a better evaluation of the reaction of the materials during the entire winter. The materials were also listed in order from least to greatest cumulative heave in Table 6.

A comparison of the relative orders of materials determined by both maximum heave and cumulative soil heave shows considerable correlation between the results obtained by the two methods. Cumulative and maximum heaves are plotted as a bar graph in Figure 10, the relative order of presentation being determined by the magnitude of cumulative heaves.

TABLE 6

RELATIVE BEHAVIOR OF SUBBASE AND SOIL MATERIALS BASED ON MAXIMUM HEAVE AND CUMULATIVE HEAVE

Relative Position in Order	Maximum Heave During One Freezing Cycle			Cumulative Heave During Entire Winter		
	Soil No	H R B Class.	Heave Ft	Soil No.	H. R. B. Class	Heave Ft.
			Subbase materials in 12 in deep pits			
1	F-29	A-1-a	011	F-29	A-1-a	064
2	F-34	A-1-a	015	F-28	A-1-b	068
3	F-28	A-1-b	021	F-34	A-1-a	079
4	F-31	A-1-a	025	F-30	A-1-a	087
5	F-30	A-1-a	029	F-27	A-1-a	113
6	F-27	A-1-a	035	F-31	A-1-a	115
7	F-32	A-1-b	044	F-32	A-1-b	126
8	F-33	A-1-a	046	F-33	A-1-a	.135
			Soil materials in 24 in deep pits			
1	F-17	A-3	004	F-17	A-3	043
2	F-25	A-3	008	F-25	A-3	053
3	F-10	A-1-b	017	F-10	A-1-b	055
4	F-26	A-1-b	018	F-26	A-1-b	076
5	F-2	A-6	019	F-2	A-6	084
6	F-20	A-3	021	F-20	A-3	.089
7	F-14	A-7-6	022	F-14	A-7-6	091
8	F-5	A-5	026	F-13	A-2-4	097
9	F-13	A-2-4	027	F-12	A-4	098
10	F-21	A-6	030	F-5	A-5	104
11	F-12	A-4	032	F-21	A-6	113
12	F-6	A-6	032	F-9	A-7-6	.113
13	F-15	A-7-6	032	F-15	A-7-6	114
14	F-9	A-7-6	033	F-6	A-6	120
15	F-19	A-7-5	045	F-4	A-4	127
16	F-4	A-4	064	F-1	A-2-4	.143
17	F-1	A-2-4	065	F-19	A-7-5	151
18	F-11	A-6	069	F-11	A-6	155
19	F-22	A-4	091	F-22	A-4	.162
20	F-23	A-4	103	F-23	A-4	.165
21	F-3	A-2-4	127	F-3	A-2-4	.178
22	F-24	A-4	.145	F-24	A-4	178

COMPARISON BETWEEN LOSS OF BEARING CAPACITY AND FROST-HEAVING CHARACTERISTICS

The relative reaction of materials to frost action as indicated by both loss of bearing capacity and cumulative frost heave is presented in Table 7. The materials are separated into groups according to HRB classification.

TABLE 7

COMPARISON OF LOSS OF BEARING CAPACITY (B) WITH CUMULATIVE FROST HEAVE (H) FOR 30 NEW JERSEY SUBBASE AND SOIL MATERIALS

Position of Material in Order	Highway Research Board Classification																		
		A-1-a		A-1-b		A-3		A-2-4		A-4		A-5		A-6		A-7-5		A-7-6	
		B	H	B	H	B	H	B	H	B	H	B	H	B	H	B	H	B	H
Subbase materials in 12 in. deep pits																			
1	Soil Numbers	29	29																
2		27			28														
3			34	32															
4		34	30																
5		33	27																
6			31	28															
7		30			32														
8		31	33																
Soil materials in 24 in. deep pits																			
1	Soil Numbers			26			17												
2						17	25												
3					10	25													
4				10	26														
5								13							2				
6						20	20												
7										23									14
8									13					2					
9											12					19			
10								1					5						
11														11	21				
12																		15	9
13										24									15
14												5			6				
15										12	4								
16								3	1										
17										4							19		
18														21	11				
19											22							9	
20											23			6					
21									3									14	
22										22	24								

OBSERVATIONS AND CONCLUSIONS

Effect of Thickness of Material

It can be seen in Figure 10 that the A-1-a and A-1-b materials placed in 12 in. deep pits experienced considerably more heaving than corresponding materials in 24 in. deep pits. Subsurface soil temperature measurements in F-10, an A-1-b material in a 24 in. deep pit, showed that frost penetration reached a maximum depth of 20 in. beneath the underside of the concrete slab. It can thus be assumed that frost penetrated through the similar 12 in. thick materials into the soil beneath. A portion of the heave measured on these materials can be attributed to the underlying soil. This soil is similar to F-1, an A-2-4 material which heaved considerably in a 24 in. deep pit.

The observed effect of the underlying soil justifies separate evaluations of the materials in 12 in. and 24 in. deep pits. This effect also demonstrates the necessity of using a sufficient thickness of frost-free subbase over frost-susceptible soils.

Climatic Conditions

During the winter of 1954-1955 the maximum heaves of all materials occurred during either of two distinctive climatic periods.

Daily air temperatures fluctuated considerably during the first period, from January 10th to February 5th (see Appendix B). The mean daily temperature was always below freezing and showed a general decline throughout the period. The maximum air temperature remained below freezing from January 24th to January 31st and February 1st to February 2nd. Thirteen degrees below zero Fahrenheit, the lowest daily minimum air temperature for the winter, was recorded on February 2nd. This entire period was characterized by an extremely low amount of precipitation. January's precipitation was the lowest ever recorded by the Weather Bureau for that month in New Jersey. During this dry period the ground-water level reached its lowest elevation for the winter.

Daily morning subsurface temperature measurements in six soils showed frost in the ground continuously during this period. Depth of frost penetration fluctuated in accordance with air temperature, but in general increased throughout the period. Maximum depth of frost penetration was recorded in all of the six soils at the end of the period.

Heavy rainfall on February 6th and 7th was accompanied by rising air temperature. An abrupt rise in ground-water level resulted. Subsurface temperature measurements indicated considerable thawing from both the surface and bottom of the frozen layer. Complete thawing, however, did not result in any of the six soils studied.

Rain on February 11th and rapidly falling air temperature initiated the second climatic period. The soil appeared to be completely saturated. Maximum air temperatures remained below freezing on February 12th and 13th. Thawing started on February 17th with rising air temperatures.

Effect on Soil Heaving Characteristics

During the first primary period of frost activity (January 10th to February 5th) 18 of the materials investigated experienced maximum heaves, but only eight reached their maximum elevations. Although air temperatures were lower near the end of the period, the general rate of heave was highest near the beginning. This illustrates the beneficial effect of the lowered ground-water table through lack of precipitation.

The maximum heaves of four of the A-4 materials and one A-2-4 extended over the entire period. As a result of minor thaws the maximum heaves of the remainder of the 18 materials were of shorter duration. An A-1-a, an A-1-b and the three A-7-6 materials experienced maximum heaves between January 10th and 22nd, the others between January 10th and 30th.

Only three A-4, two A-2-4, two A-7-6 and one A-3 material reached their maximum height above initial elevation at the end of this primary period.

During the second period of freezing activity the remaining 12 materials experienced maximum heaves. These consisted primarily of most of the A-1-a, A-1-b and A-3 materials. However, at the end of this period 22 materials reached maximum elevations.

The observed effect of climatic conditions on the heaving characteristics of soils clearly defines the best and poorest materials. The A-1-a, A-1-b and A-3 materials in general experienced maximum heaves only after complete saturation, resulting primarily from the direct effect of precipitation. The A-4 materials experienced maximum heaves at a time when the ground-water table was lowering and moisture conditions were least conducive to frost heave. It may be noted that these effects are, in part, also a result of the greater thermal conductivity and more rapid reaction to temperature change of the A-1-a, A-1-b and A-3 material. Had the second period not been of such short duration, it is possible that the poorest materials might have had even greater maximum heaves.

Effect on Loss of Bearing Capacity

It is difficult to develop separate relationships between the previously mentioned climatic periods and loss of soil bearing capacity. Materials which failed (plunger penetration exceeded 0.10 ft) before or at the end of the first period were, of course, no longer suitable for evaluation during the second period.

Nearly all of the A-7-6 materials failed before or at the end of the first period, in general showing the greatest rate of loss of bearing capacity. A considerable number of A-4 materials failed at the end of the second period. Few failures of A-1-a, A-1-b and A-3 materials occurred before the end of the second period.

GENERAL CONCLUSIONS

Having demonstrated the effect of climatic conditions on frost action, it must be concluded that had other conditions prevailed, the materials studied might have presented different reactions. Because of this fact and because of the rather limited number of materials observed, only generalized conclusions may be developed from this study.

For final evaluation the materials may be grouped according to HRB classification. Concerning rating, it may be assumed that the best materials heave the least and show the least loss of bearing capacity. The use of the term "relative" in referring to heave and bearing capacity signifies that each material may be rated only in comparison with the others investigated.

A-1-a, A-1-b and A-3 Materials

These materials of a coarsely granular nature may be considered the best in view of their resistance to heaving and of their retention of bearing capacity. Relative correlation between these two factors is good. The materials A-1-a, A-1-b and A-3 should, therefore, provide good to excellent subgrade or base and subbase courses with a minimum danger of frost damage. Good results should be expected when using these materials as subbase over poorer materials, providing a sufficient thickness is used to insure good drainage of surface, thawing or percolating flood waters.

A-2-4 Materials

The silts or clayey gravels and sands of the HRB soil classification system show a wide range of relative reaction to frost action. Each of the materials observed, however, showed a considerably greater relative heave than relative loss of bearing capacity. In relation to all of the materials investigated, one A-2-4 (F-13) was good from the viewpoint of bearing capacity and another (F-3) was very poor concerning heave. As a group it is difficult to rate the A-2-4 soils as to frost susceptibility. The favorable reaction of F-13 (Montalto) to bearing capacity studies should be particularly noted, as in some areas of the state this material is finding use as subbase.

A-4 Materials

The silty A-4 materials also experienced a considerable range of reaction. No correlation was noted between loss of bearing capacity and heaving characteristics. Of all the soils studied, F-22 showed the greatest loss of bearing capacity and F-24 the greatest heave. As a group the A-4 materials may be considered the most susceptible to frost action and in general unsuitable as subgrade

A-5 Materials

No general rating can be given to A-5 materials on the basis of only one material observed. This soil, F-5, showed a medium reaction in comparison to all materials studied.

A-6 Materials

These clayey materials in general showed considerably more relative loss of bearing capacity than relative heave. Their reactions were varied and they may be rated as fair to poor.

A-7-5 and A-7-6 Materials

Clayey A-7-5 and A-7-6 materials also showed in general considerably great relative loss of bearing capacity than relative heave. High loss of bearing capacity makes these materials unsuitable for subgrade.

Although these studies permitted a general rating of the thirty soils as to frost behavior, it is quite possible that studies of thermal properties of soils and moisture migrations into soils under thermal potential may permit obtaining a more accurate insight into the behavior of various soils. Such an insight would, in turn, afford a careful evaluation of substitute materials in areas where the most desirable soil types are not available.

SUMMARY

This report summarizes and analyzes data obtained during the year 1954-1955 on the subject of relative loss of bearing capacity and relative soil heaving characteristics of 30 New Jersey soil materials subjected to natural freeze-thaw conditions. The relative loss of soil bearing capacity was determined by measuring the penetration into the soils of weighted plungers; the soil heaving characteristics were determined by measuring relative vertical displacements of concrete slabs.

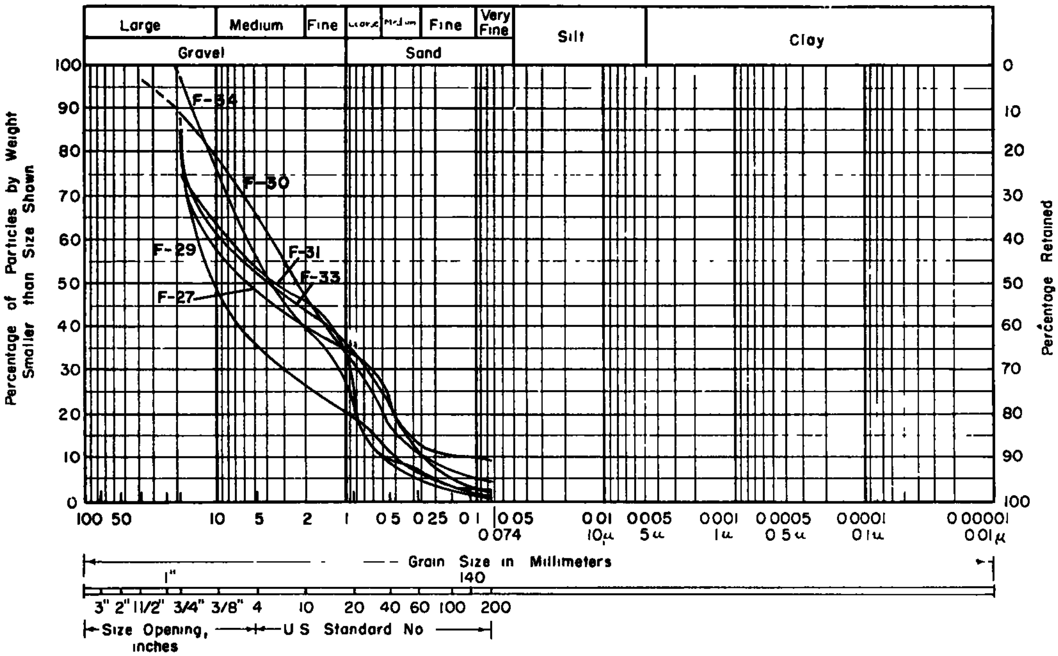
As a means of simulating pavement conditions, concrete slabs were poured over prepared specimens of the materials under investigation. Four weighted plungers were arranged so as to bear upon the soil beneath each slab. Air temperature, precipitation, and ground-water elevation and temperature were considered in the analysis. Summarization of data resulted in the materials being tabulated first in relation to their relative loss of bearing capacity and second in relation to their relative heaving characteristics. In each case the order of materials is from best to poorest.

ACKNOWLEDGMENT

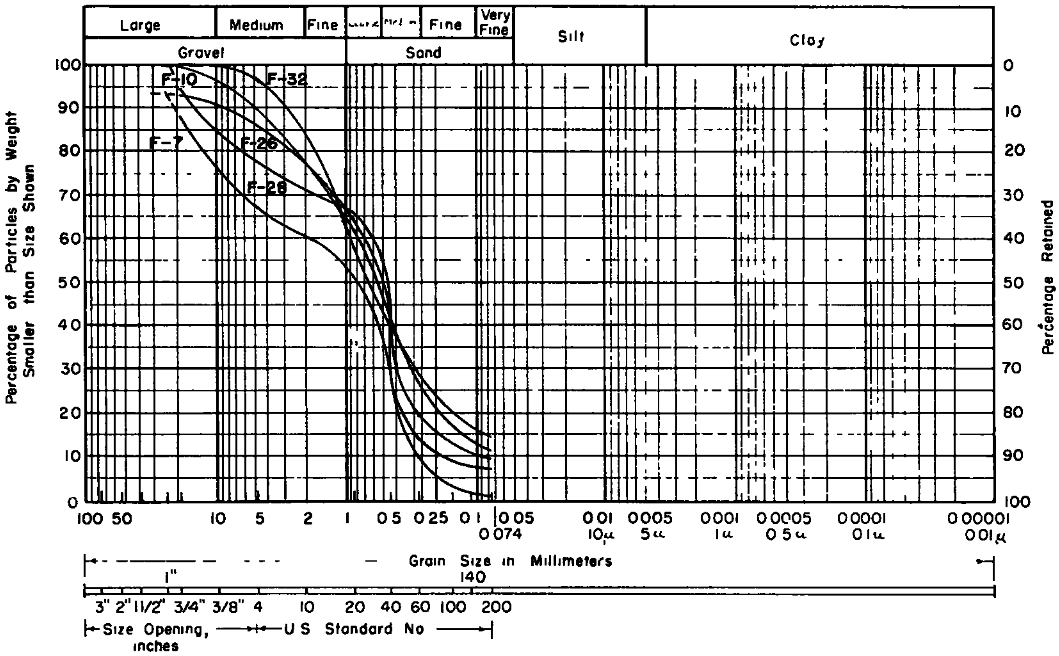
The authors wish to express their appreciation to the New Jersey State Highway Department for having initiated the project which provided the material for this report, to the Joint Highway Research Committee and in particular to Chairman Allen C. Ely for their interest and advice, and to Dr. E. C. Easton, Dean of the College of Engineering, Rutgers University, for his support of a project of such potential value to the state as a whole.

Appendix A

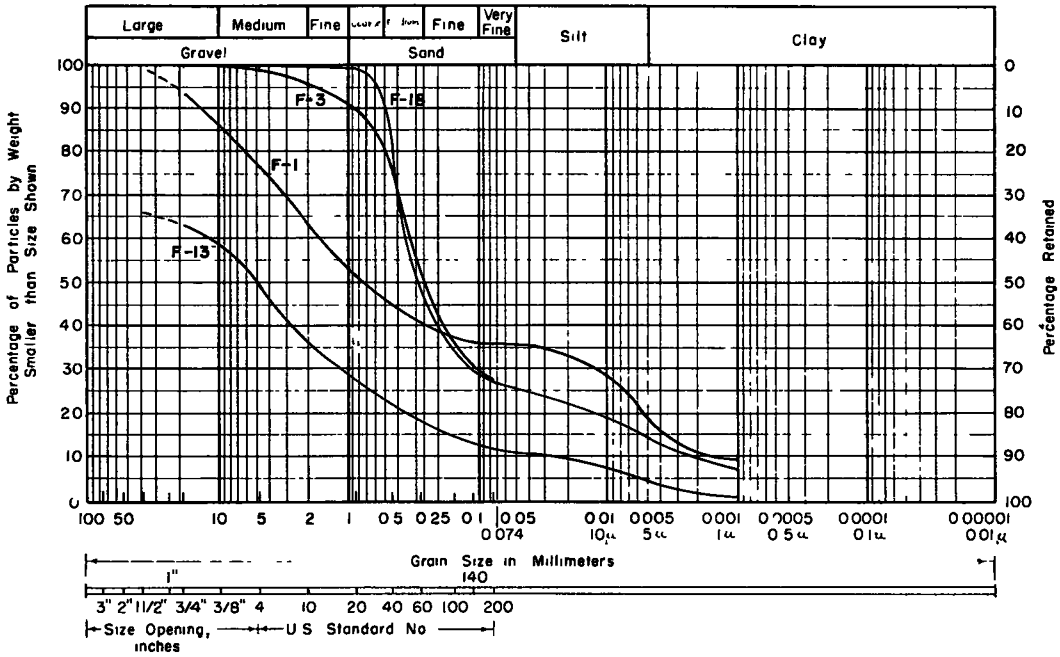
Loss of Bearing Capacity and Vertical Displacements



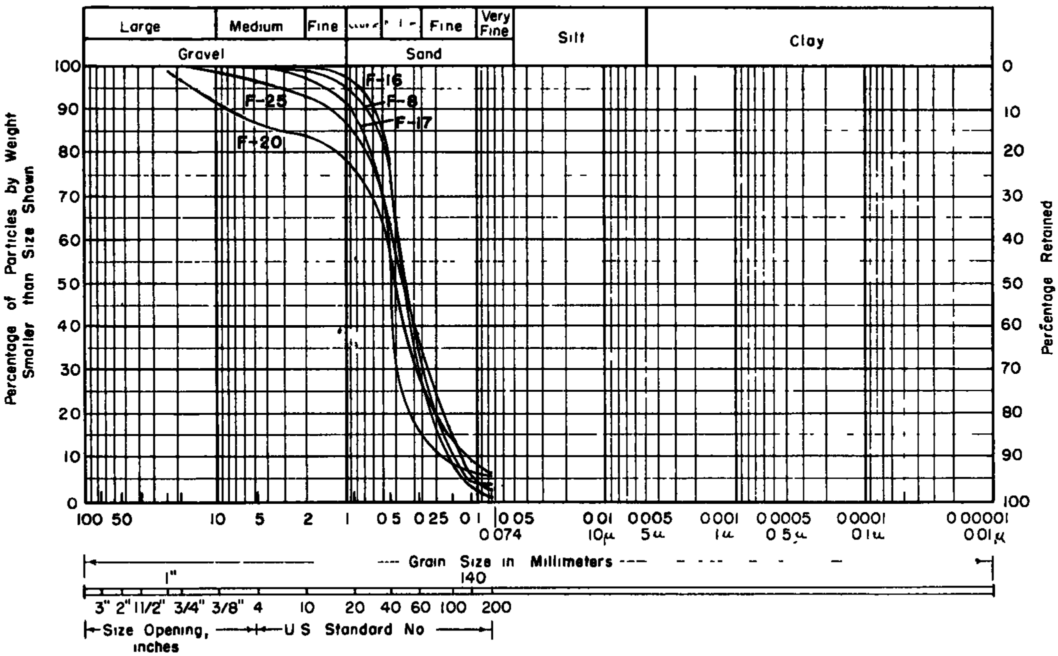
HRB - A-1-a.



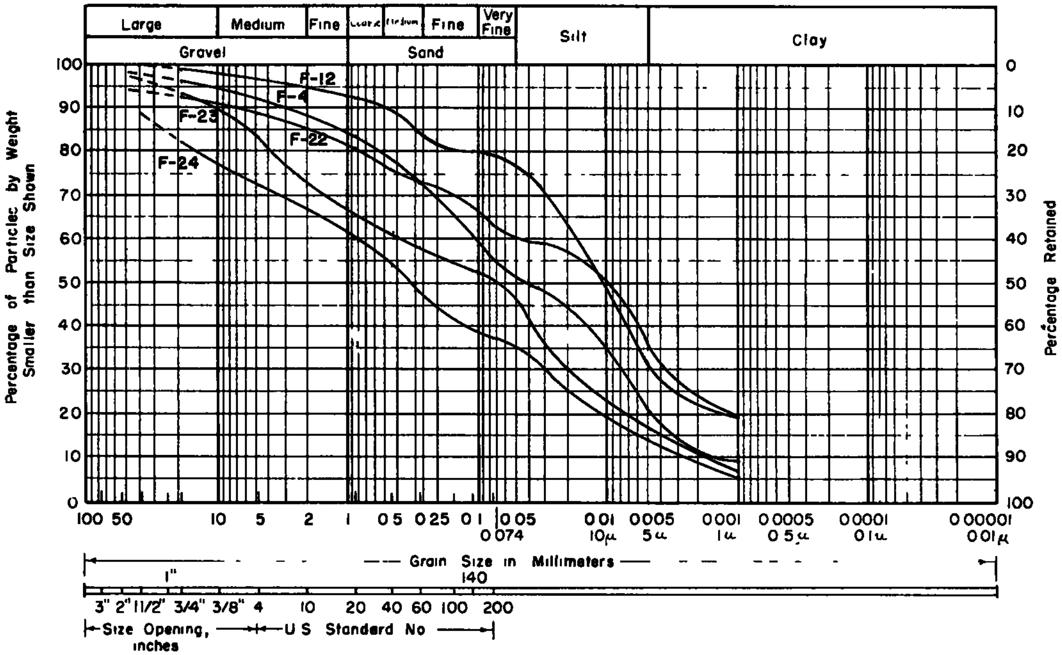
HRB - A-1-b.



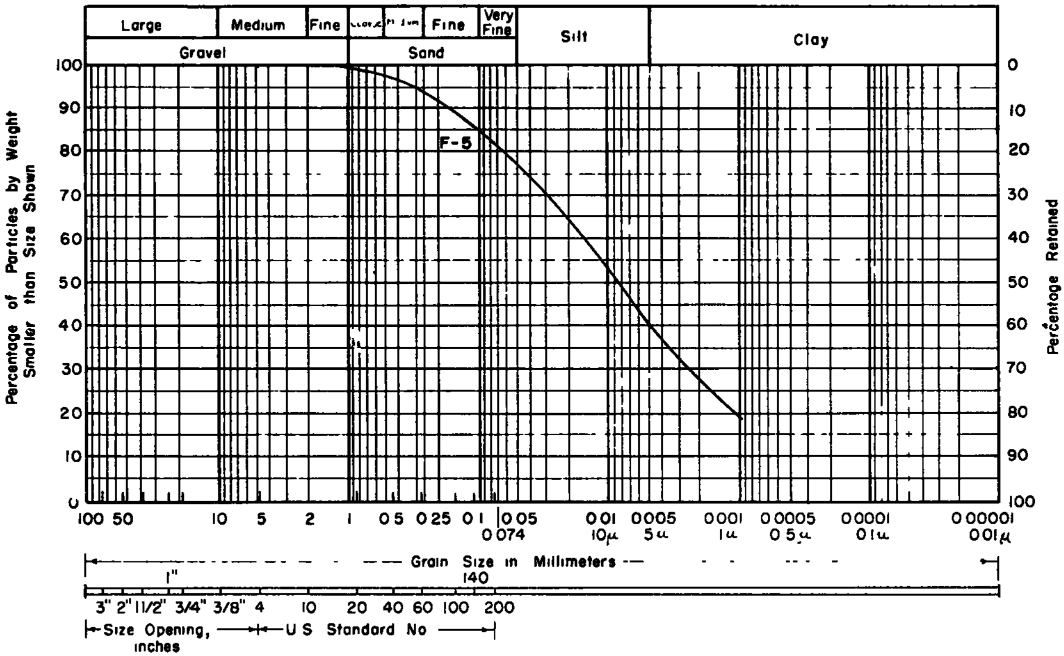
HRB - A-2-4.



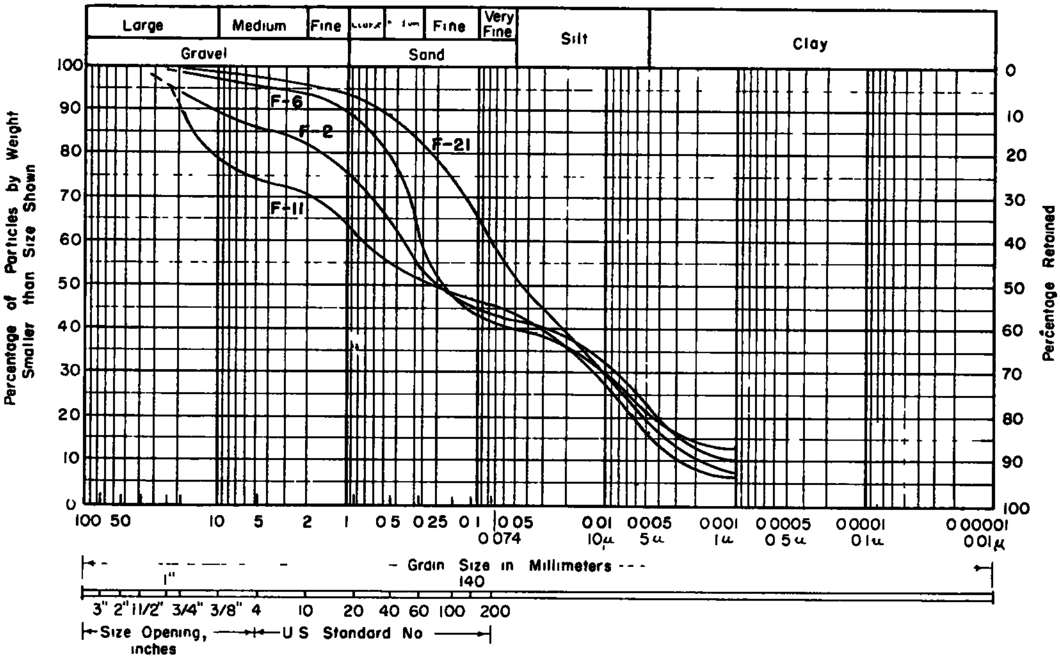
HRB - A-3.


Large
Medium
Fine
Fine
Very Fine
Silt
Clay
Gravel
Sand
F-12
F-4
F-23
F-22
F-24
Percentage of Particles by Weight Smaller than Size Shown
Percentage Retained
100 50 10 5 2 1 0.5 0.25 0.1 0.05 0.01 0.005 0.001 0.0005 0.0001 0.00001
0.074
10μ 5μ 1μ 0.5μ 0.1μ 0.01μ
Grain Size in Millimeters
1"
140
3" 2" 1 1/2" 3/4" 3/8" 4 10 20 40 60 100 200
Size Opening, inches
U S Standard No

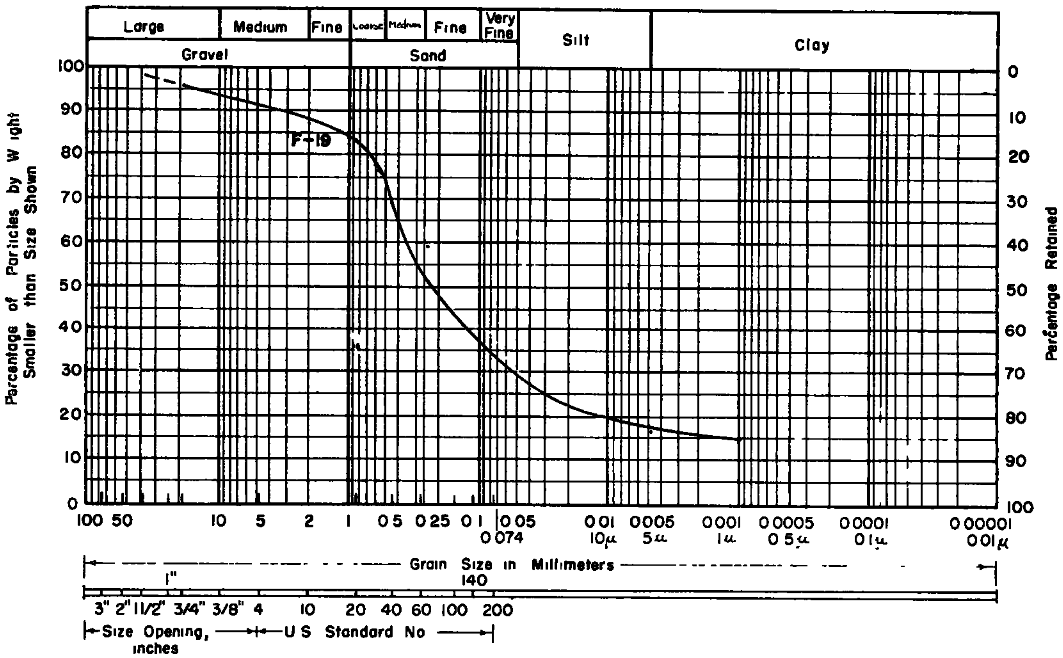
HRB - A-4.


Large
Medium
Fine
Fine
Very Fine
Silt
Clay
Gravel
Sand
F-5
Percentage of Particles by Weight Smaller than Size Shown
Percentage Retained
100 50 10 5 2 1 0.5 0.25 0.1 0.05 0.01 0.005 0.001 0.0005 0.0001 0.00001
0.074
10μ 5μ 1μ 0.5μ 0.1μ 0.01μ
Grain Size in Millimeters
1"
140
3" 2" 1 1/2" 3/4" 3/8" 4 10 20 40 60 100 200
Size Opening, inches
U S Standard No

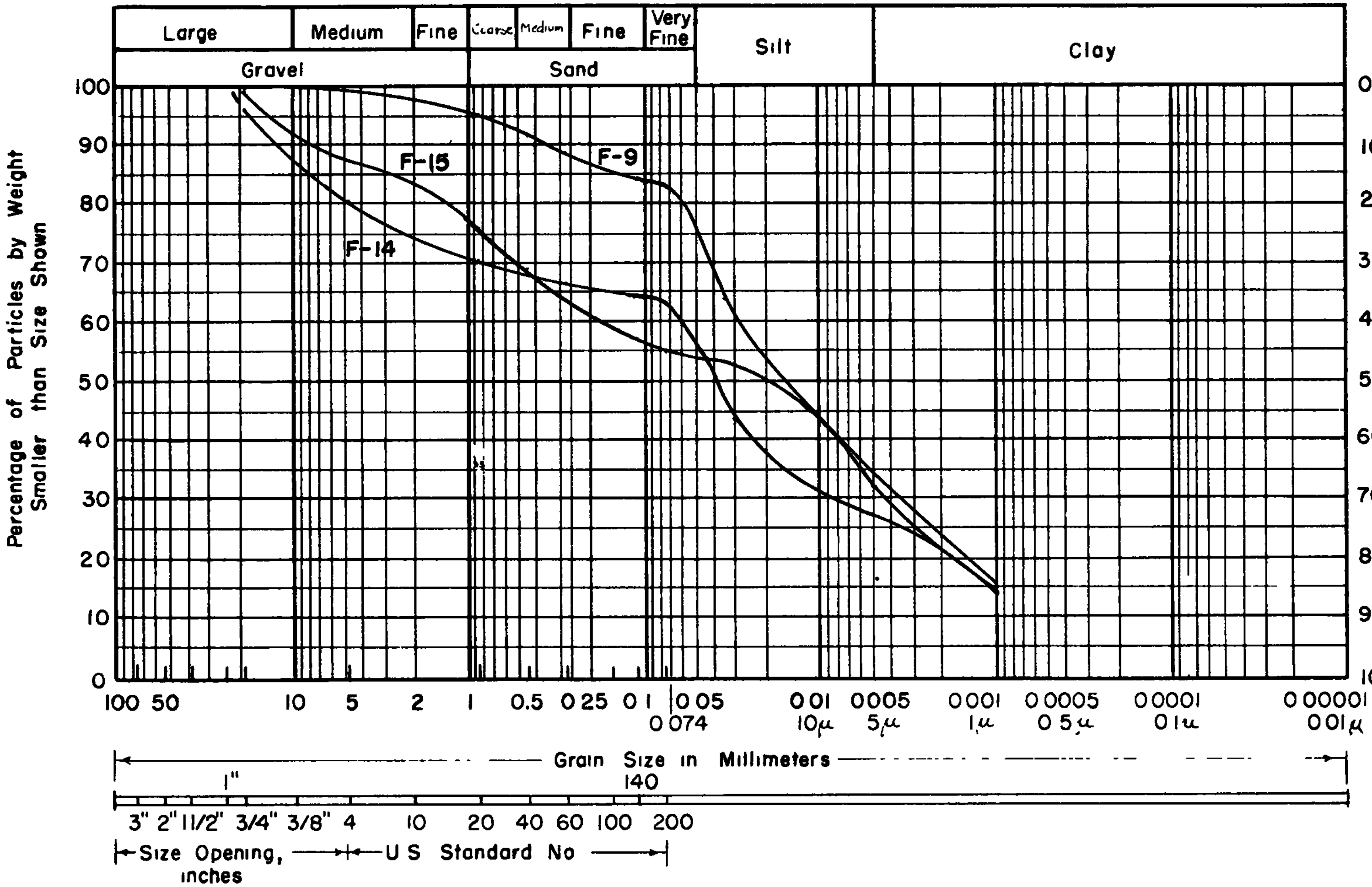
HRB - A-5.



HRB - A-6.



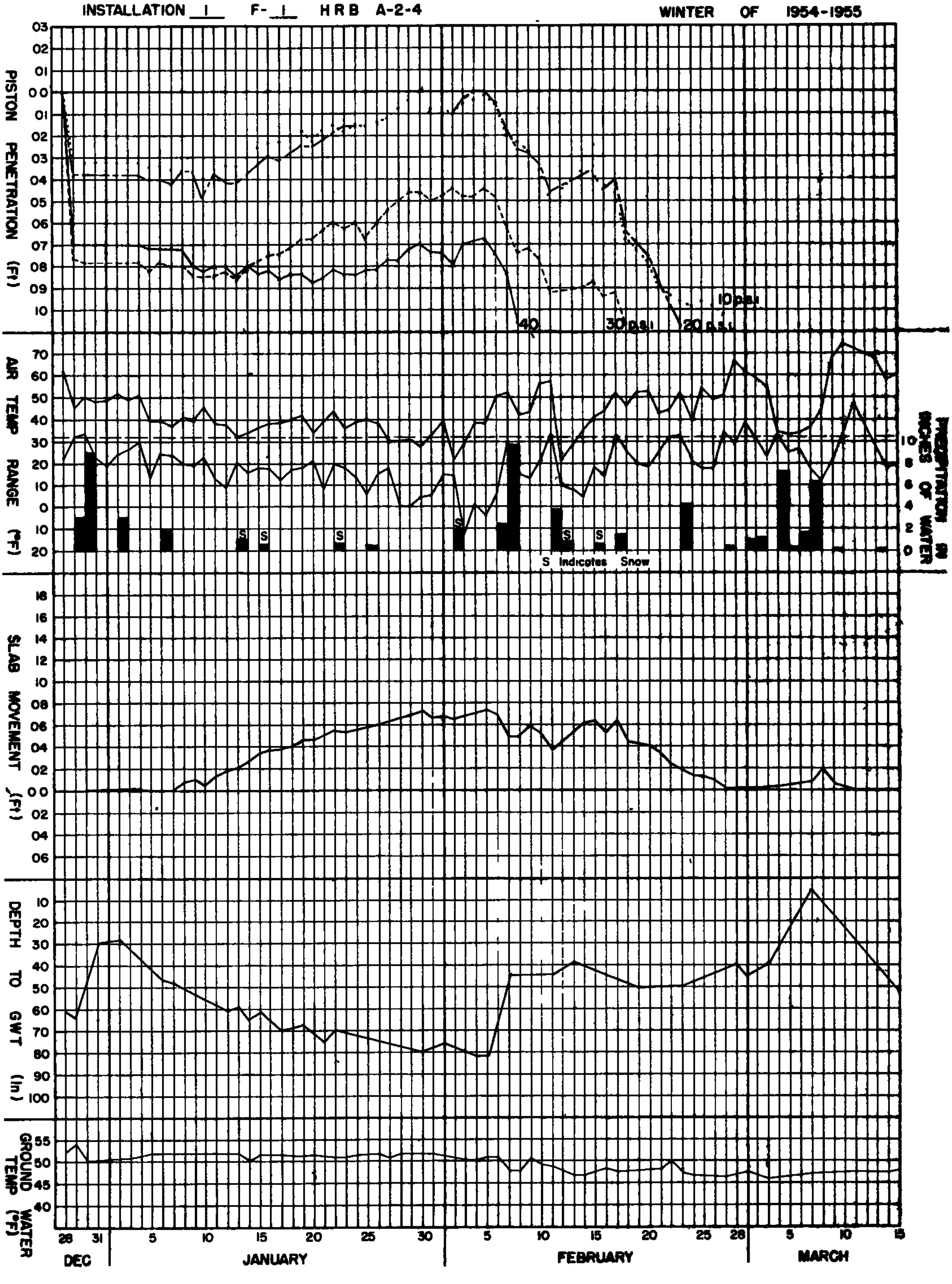
HRB - A-7-5.

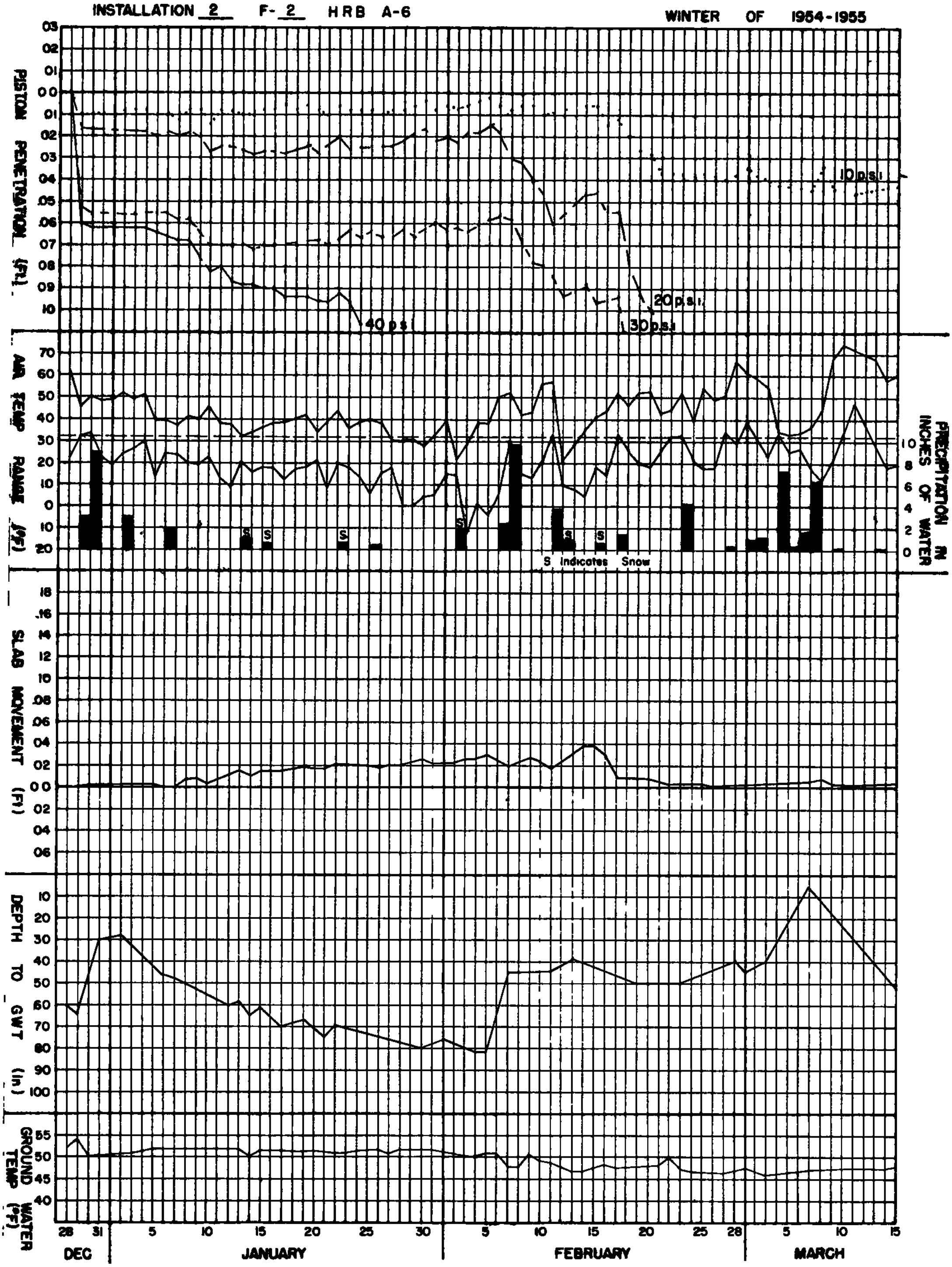

Large
Medium
Fine
Coarse
Medium
Fine
Very Fine
Gravel
Sand
Silt
Clay
F-15
F-9
F-14
Percentage of Particles by Weight Smaller than Size Shown
Percentage Retained
100 50 10 5 2 1 0.5 0 25 0 1 0 05 0 074 0 01 10μ 0005 5μ 0 001 1μ 0 0005 0 5μ 0 0001 0 1μ 0 00001 0 01μ
Grain Size in Millimeters
1"
140
3" 2" 1 1/2" 3/4" 3/8" 4 10 20 40 60 100 200
Size Opening, inches
U S Standard No

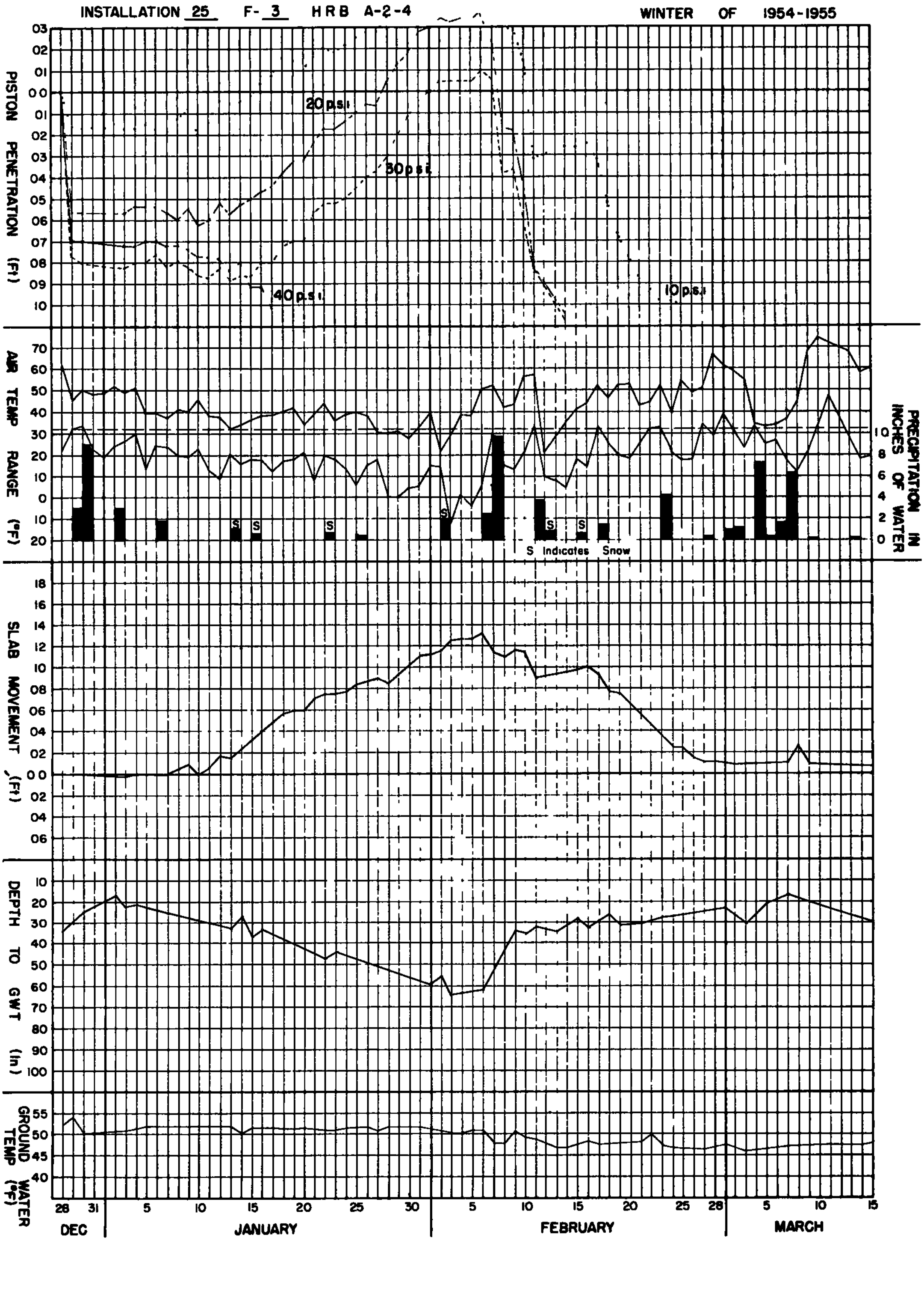
HRB - A-7-6.

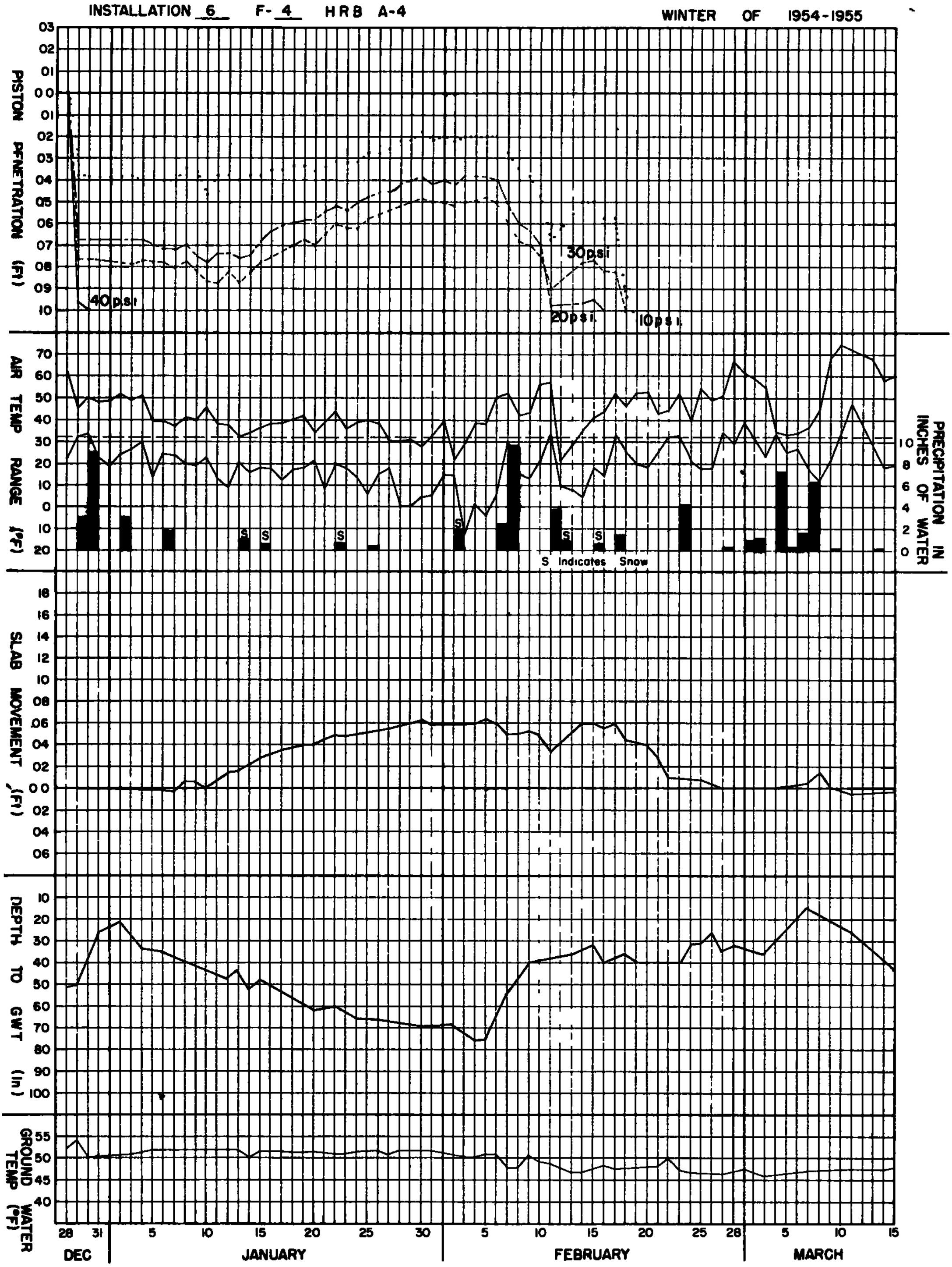
Appendix B

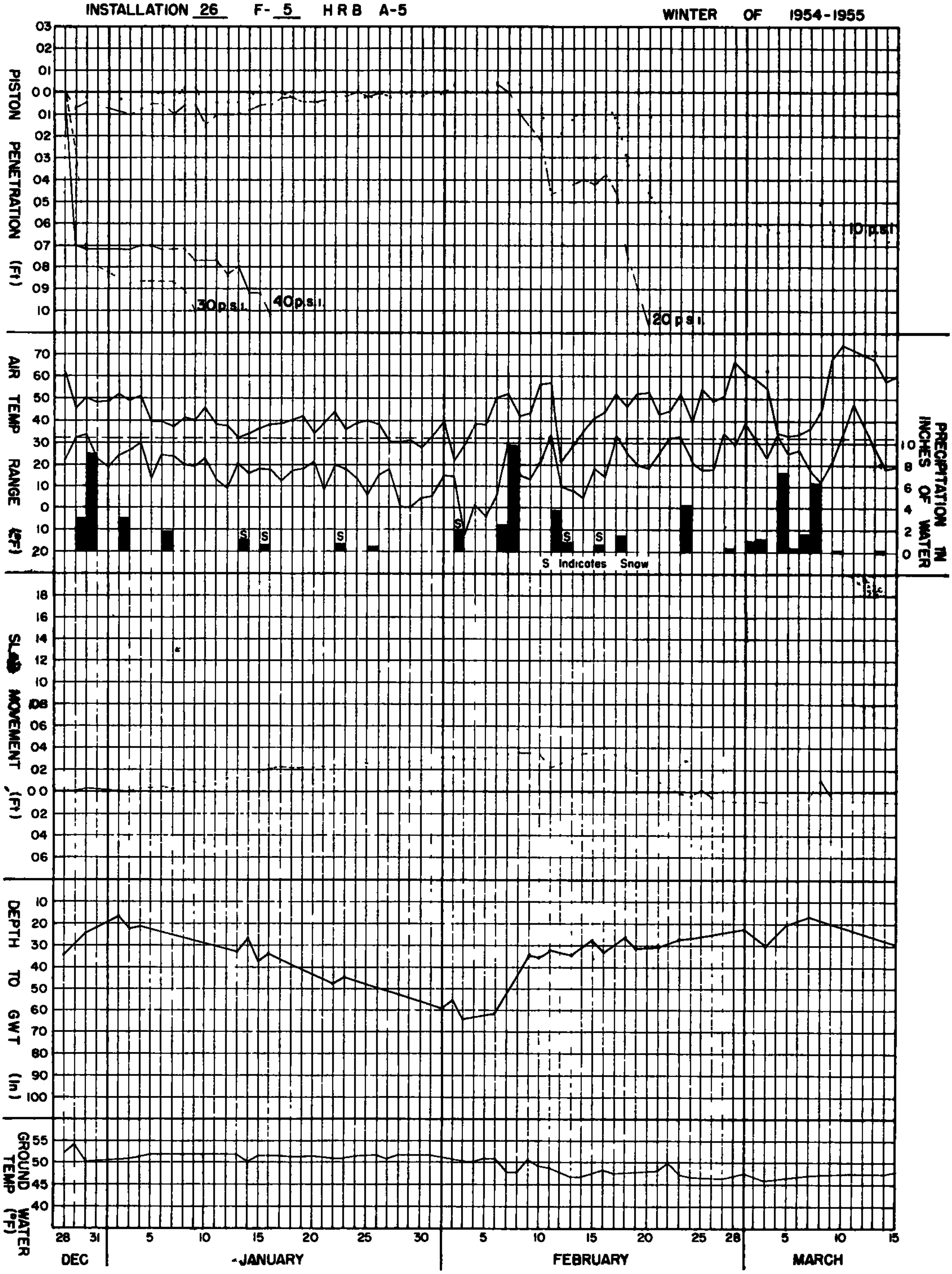
INSTALLATIONS 1-34

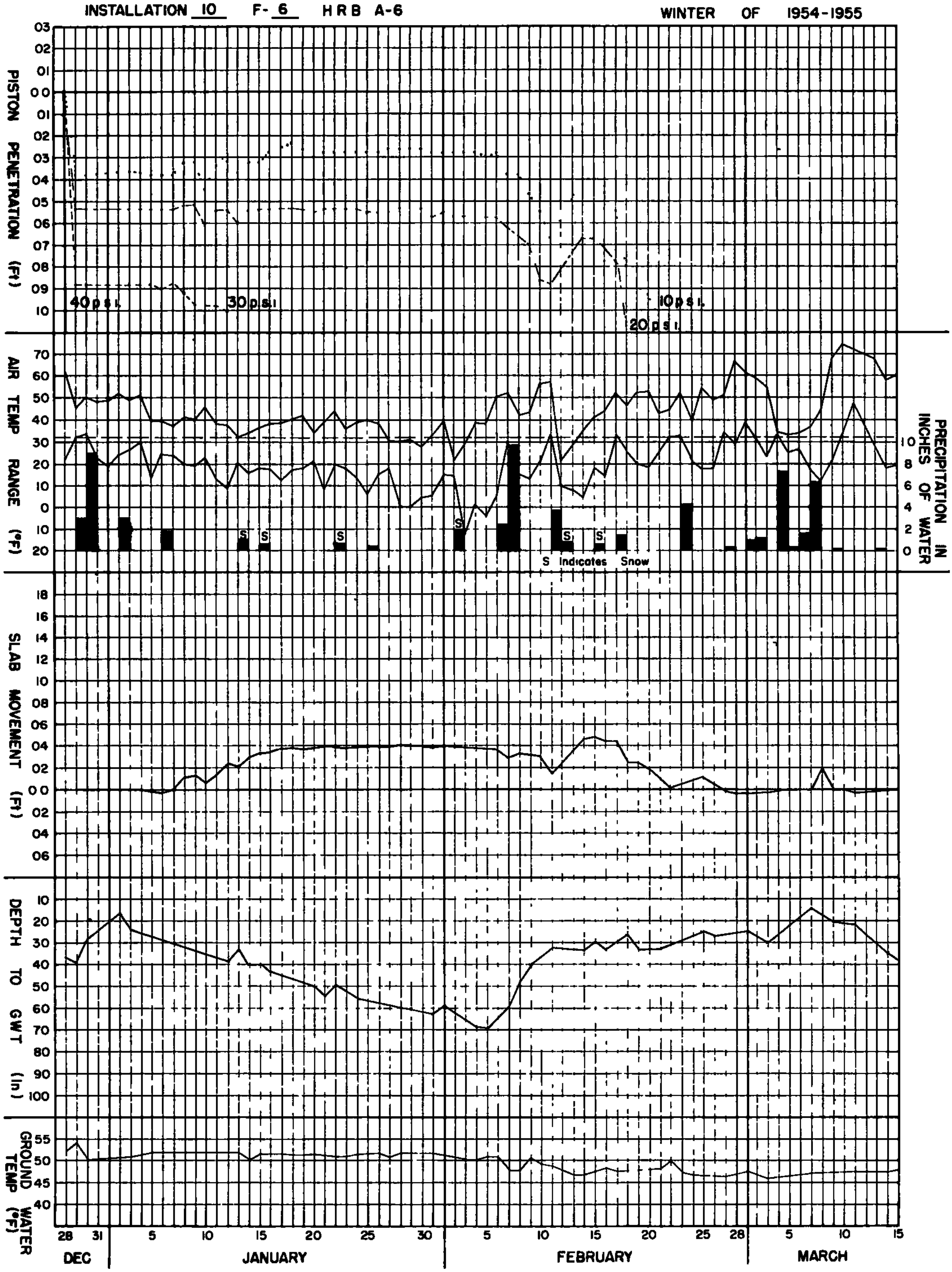


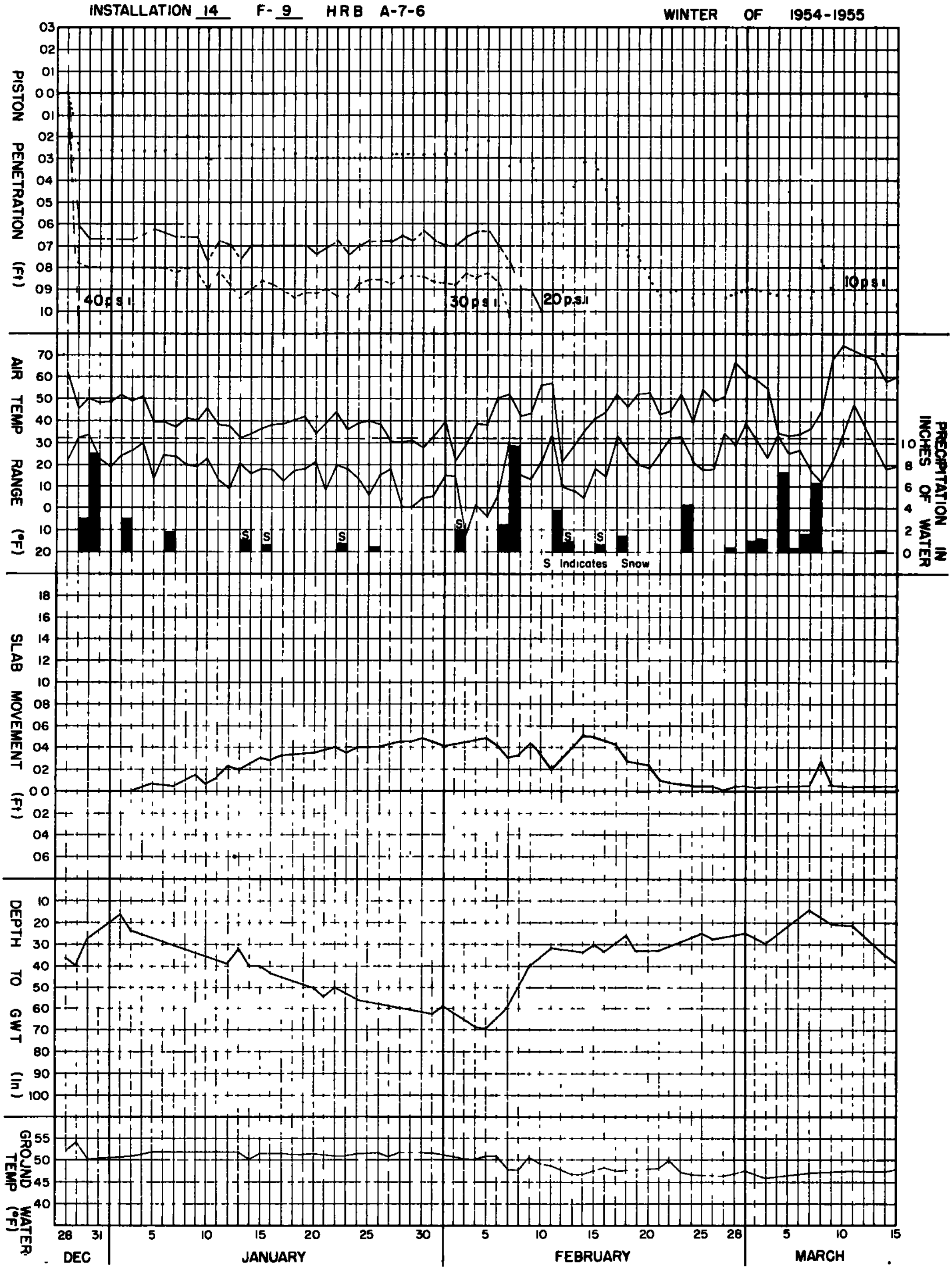

INSTALLATION 2 F- 2 HRB A-6
WINTER OF 1954-1955
PISTON PENETRATION (Ft.)
10 p.s.i.
20 p.s.i.
30 p.s.i.
40 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

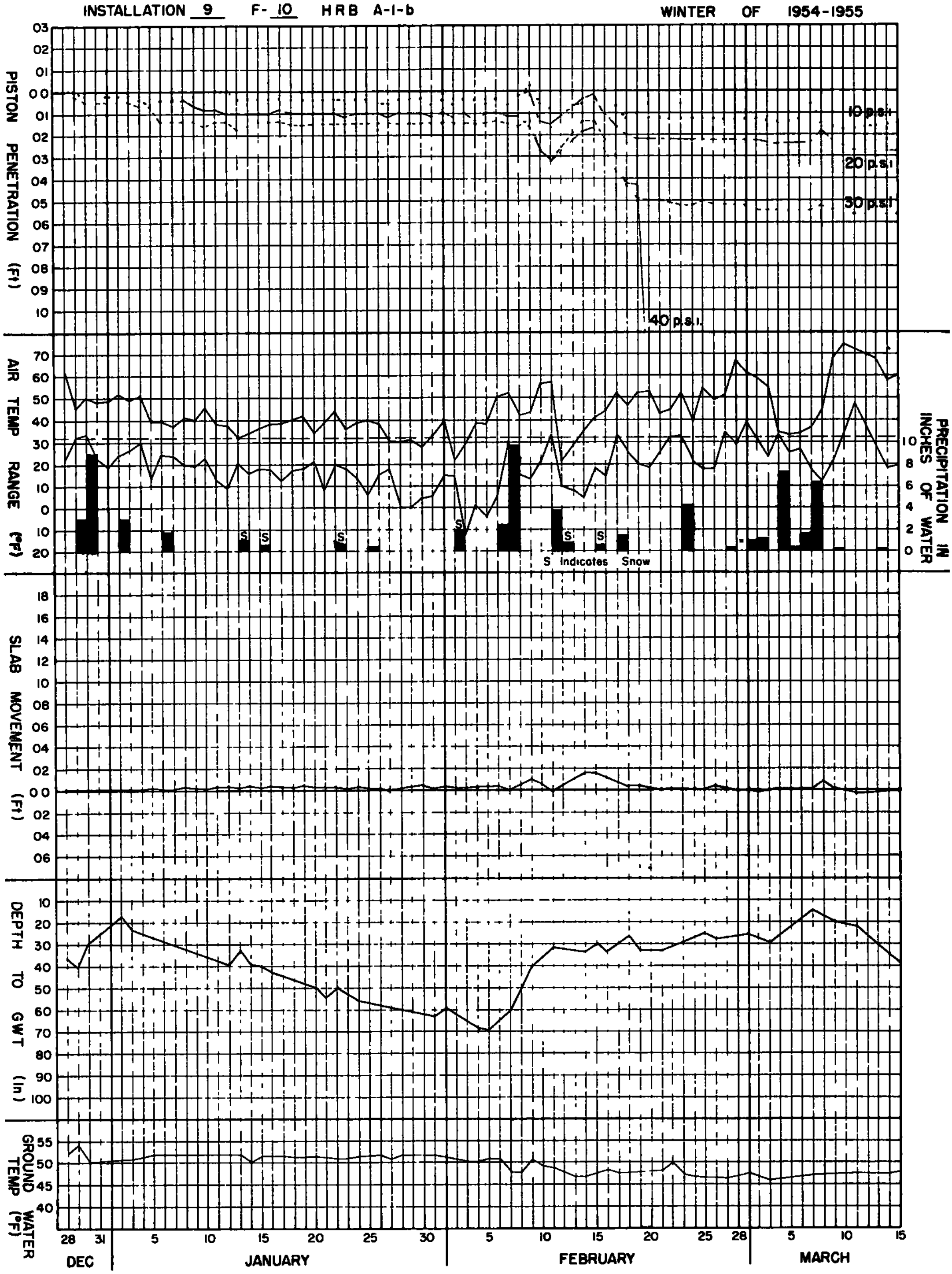

INSTALLATION 25 F- 3 HRB A-2-4
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
20 p.s.i
30 p.s.i
40 p.s.i
10 p.s.i
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH


INSTALLATION 6 F- 4 HRB A-4
WINTER OF 1954-1955
PISTON PENETRATION (FT)
40psi
30psi
20psi
10psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (FT)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH


INSTALLATION 26 F- 5 HRB A-5
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
30 p.s.i.
40 p.s.i.
20 p.s.i.
10 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

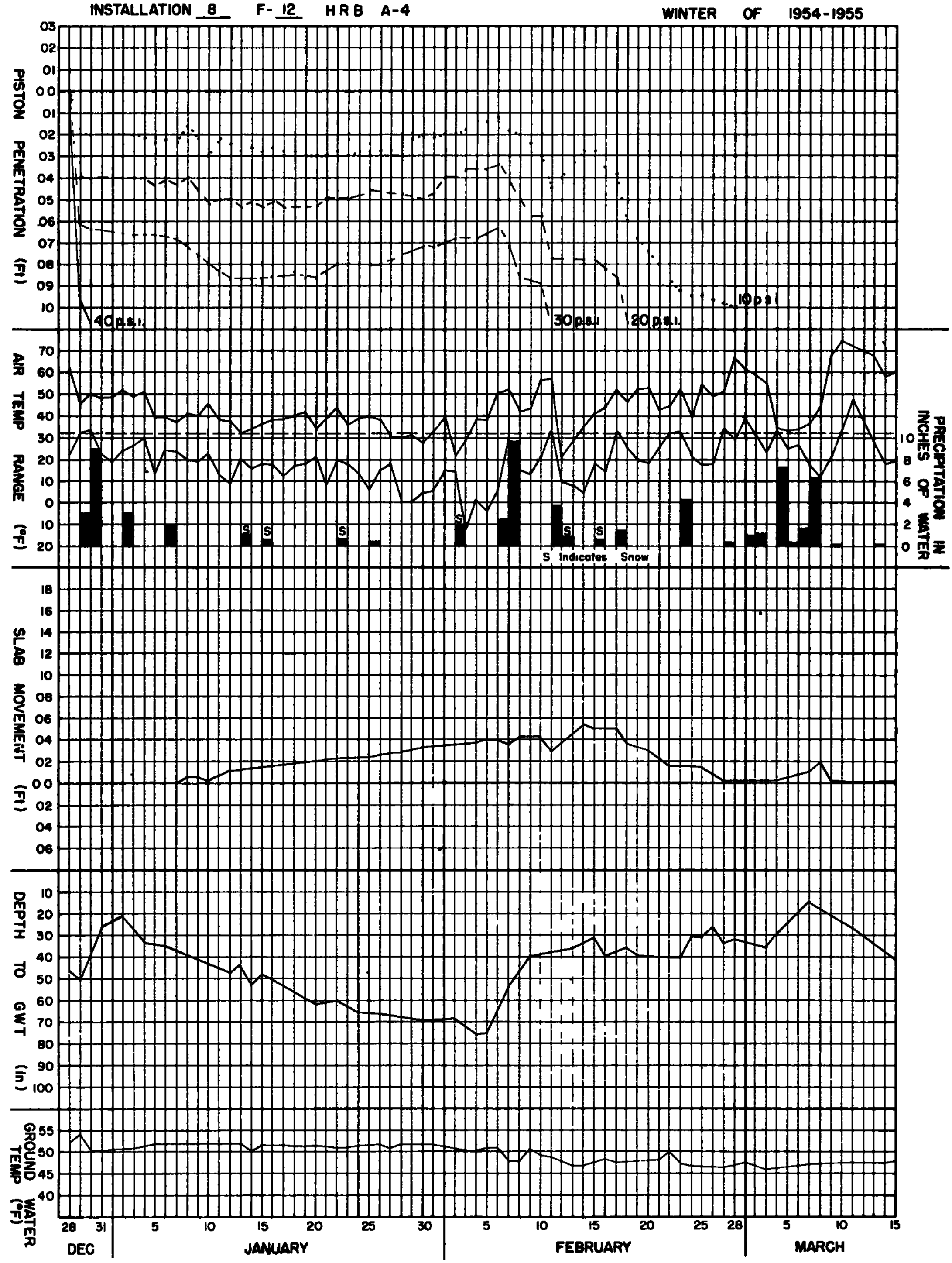

INSTALLATION 10 F- 6 HRB A-6
WINTER OF 1954-1955
PISTON PENETRATION (ft)
40 p.s.i.
30 p.s.i.
10 p.s.i.
20 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH


INSTALLATION 14 F- 9 HRB A-7-6
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
40psi
30psi
20psi
10psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

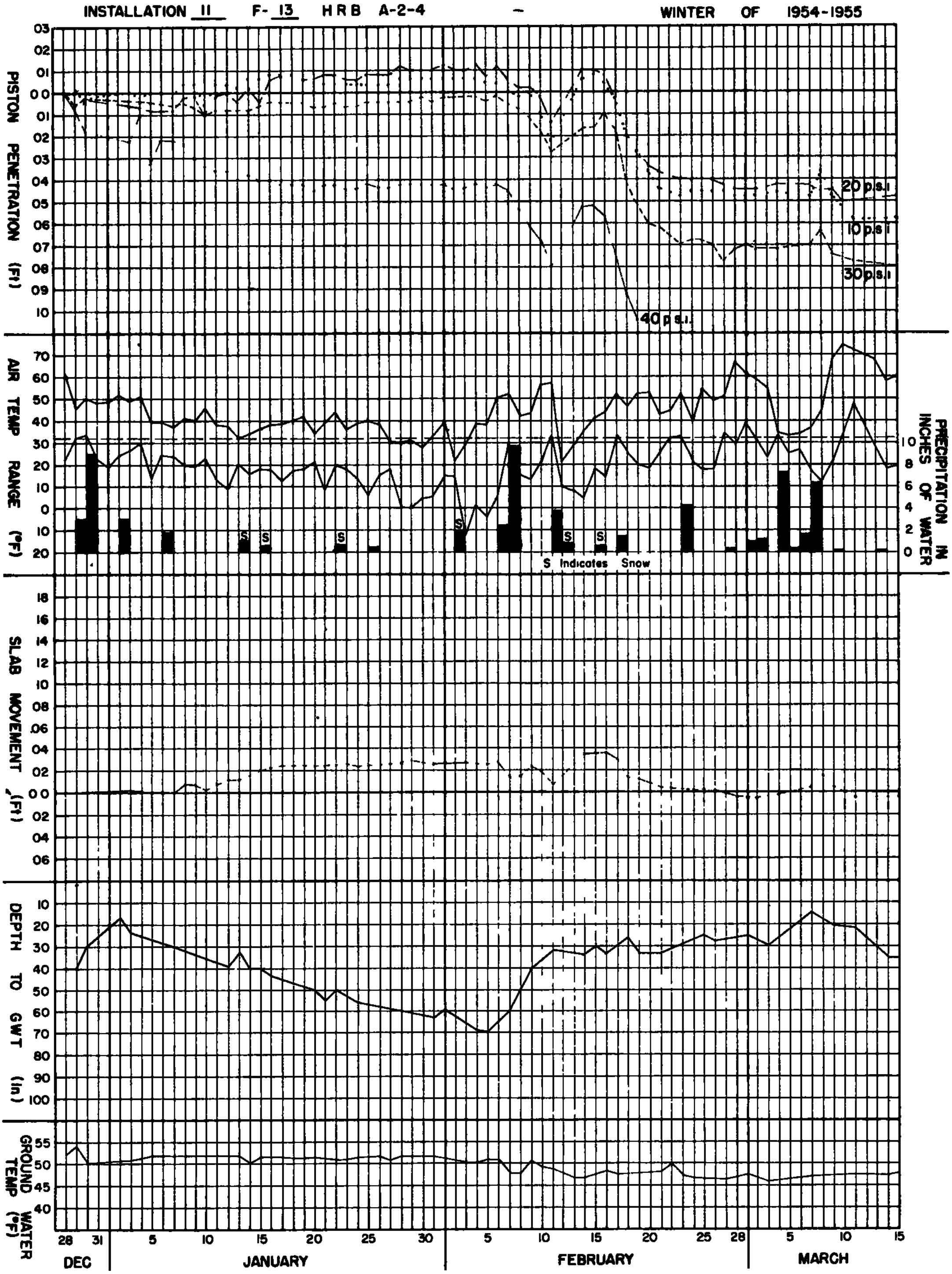


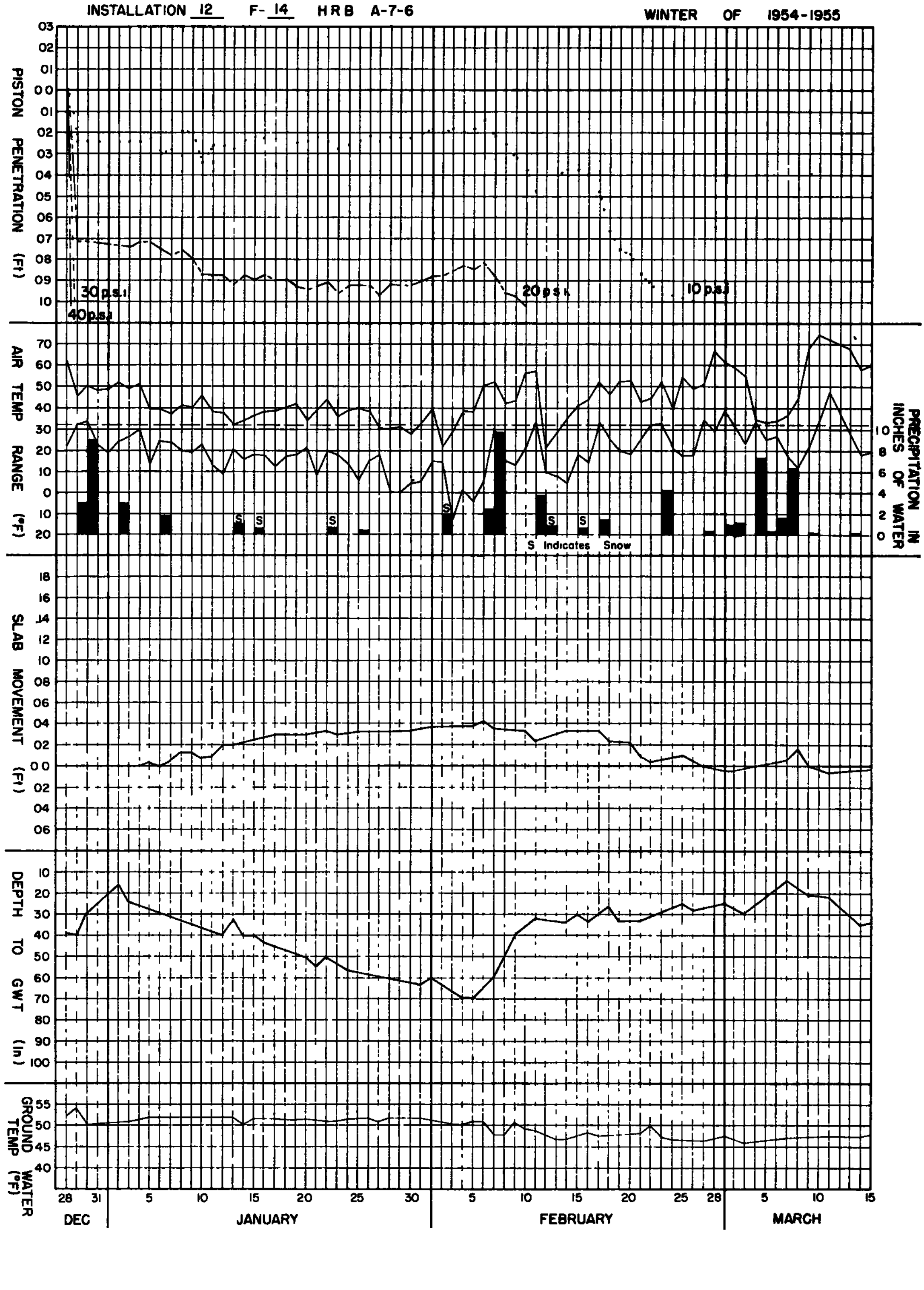
INSTALLATION 9 F- 10 HRB A-1-b
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
10 p.s.i
20 p.s.i
30 p.s.i
40 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

INSTALLATION 24 F- 11 HRB A-6
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
40 psi
30 psi
20 psi
10 psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

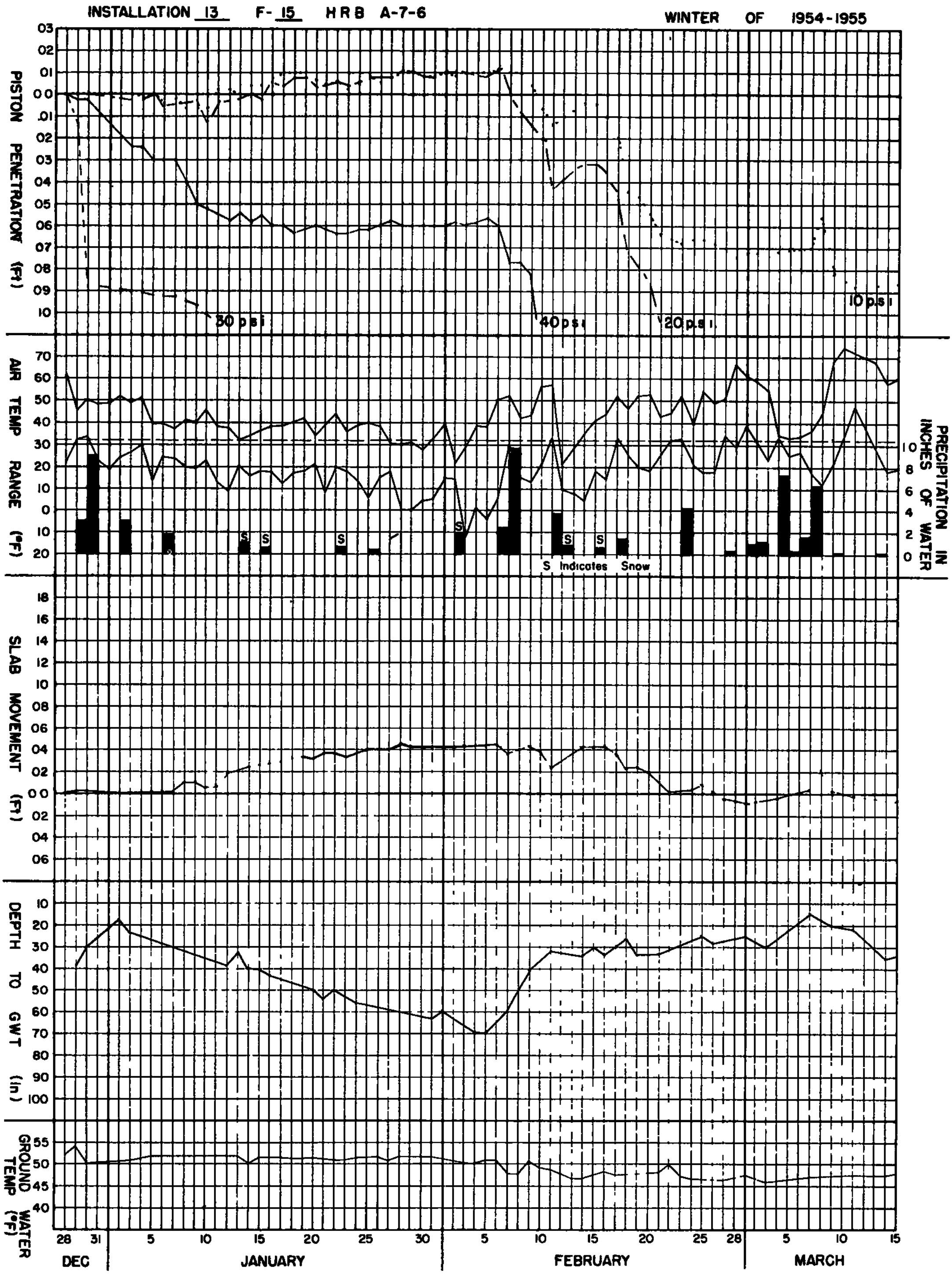


INSTALLATION 8 F- 12 HRB A-4
WINTER OF 1954-1955
PISTON PENETRATION (FT)
40 p.s.i.
30 p.s.i.
20 p.s.i.
10 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (FT)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

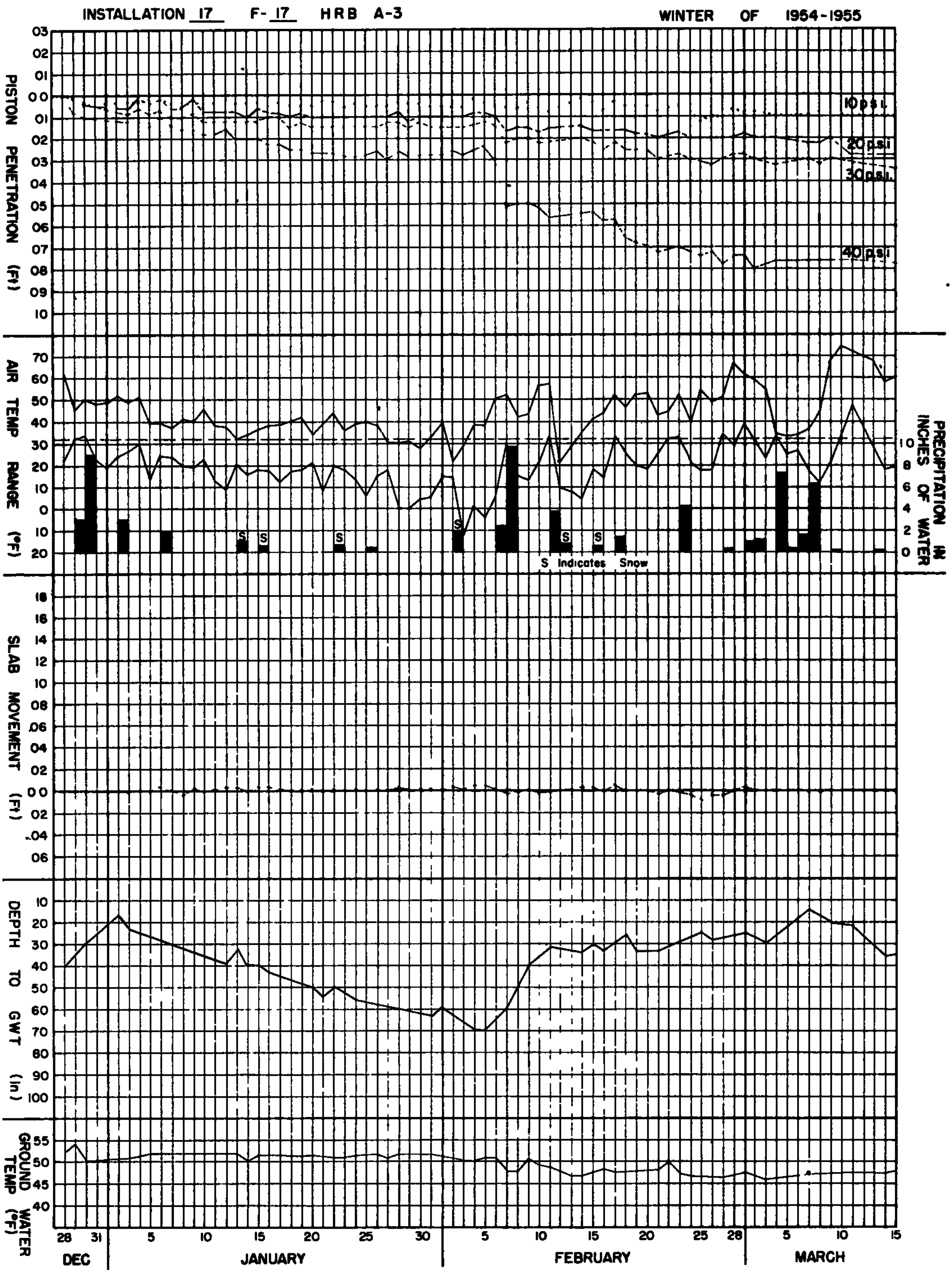

INSTALLATION 11 F- 13 HRB A-2-4 - WINTER OF 1954-1955
PISTON PENETRATION (Ft)
20 p.s.i
10 p.s.i
30 p.s.i
40 p.s.i
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH



INSTALLATION 12 F- 14 HRB A-7-6
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
30 p.s.i
40 p.s.i
20 p.s.i.
10 p.s.i
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

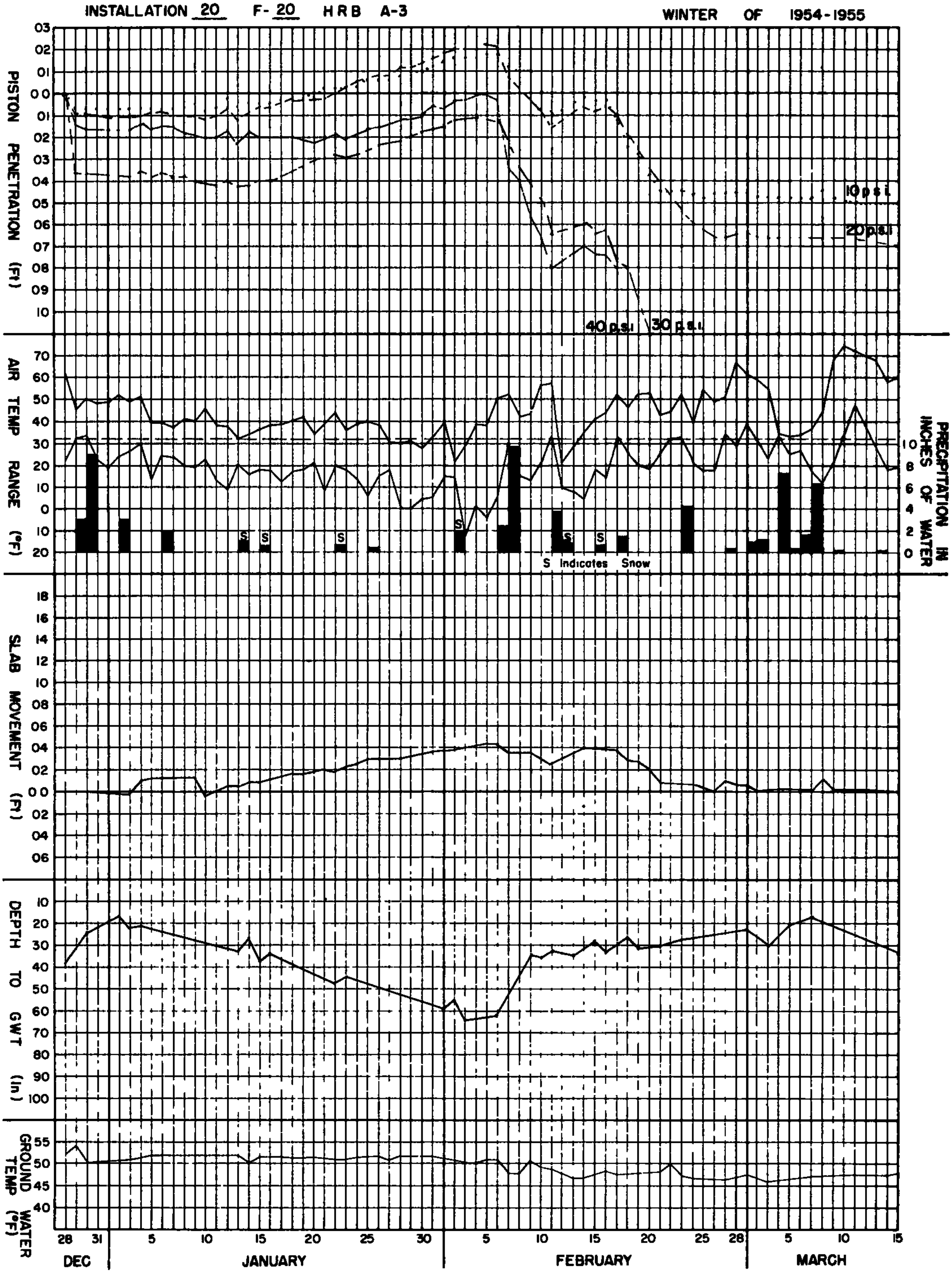


INSTALLATION 13 F- 15 HRB A-7-6
WINTER OF 1954-1955
PISTON PENETRATION (ft)
30 psi
40 psi
20 psi
10 psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

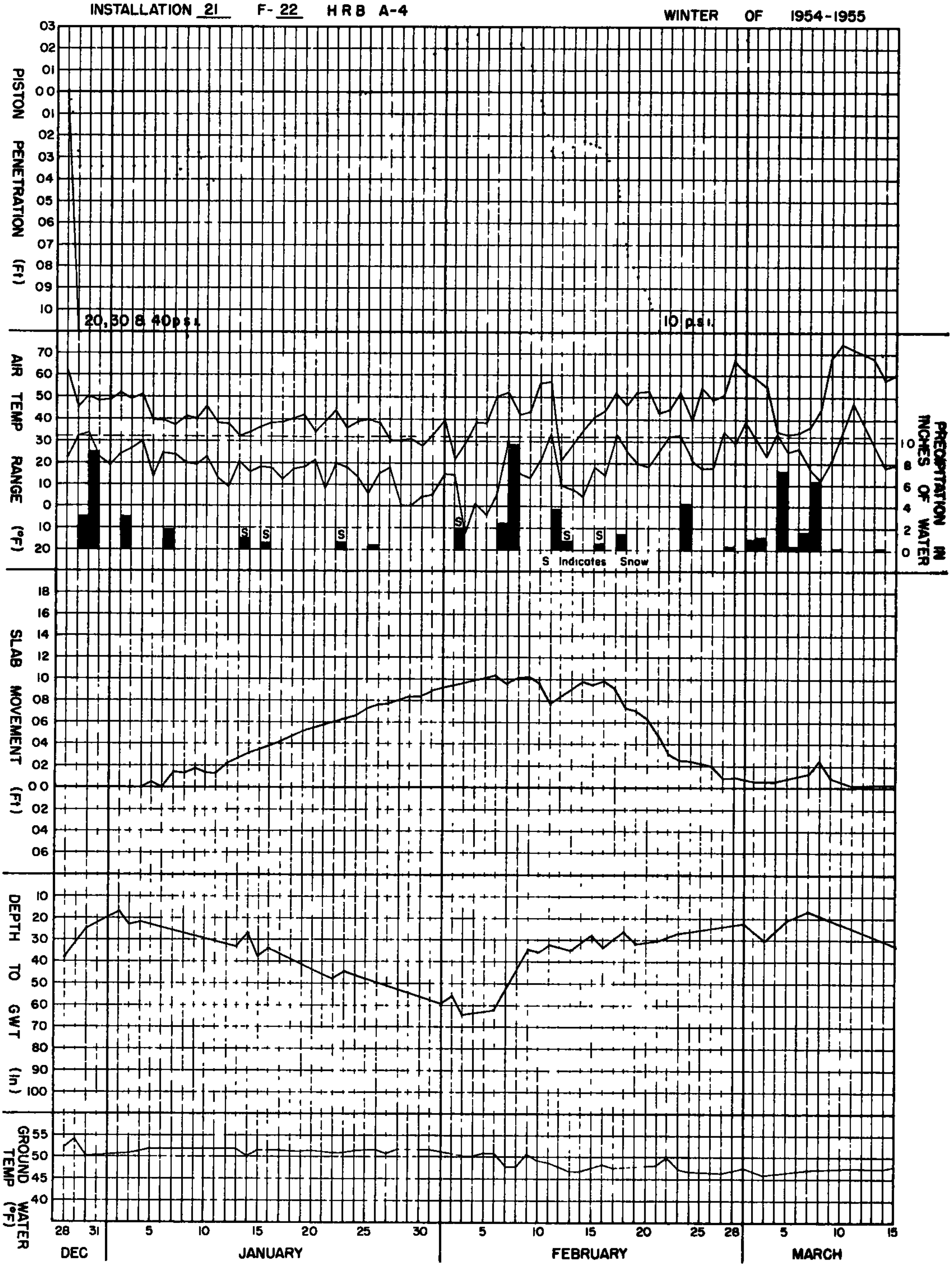


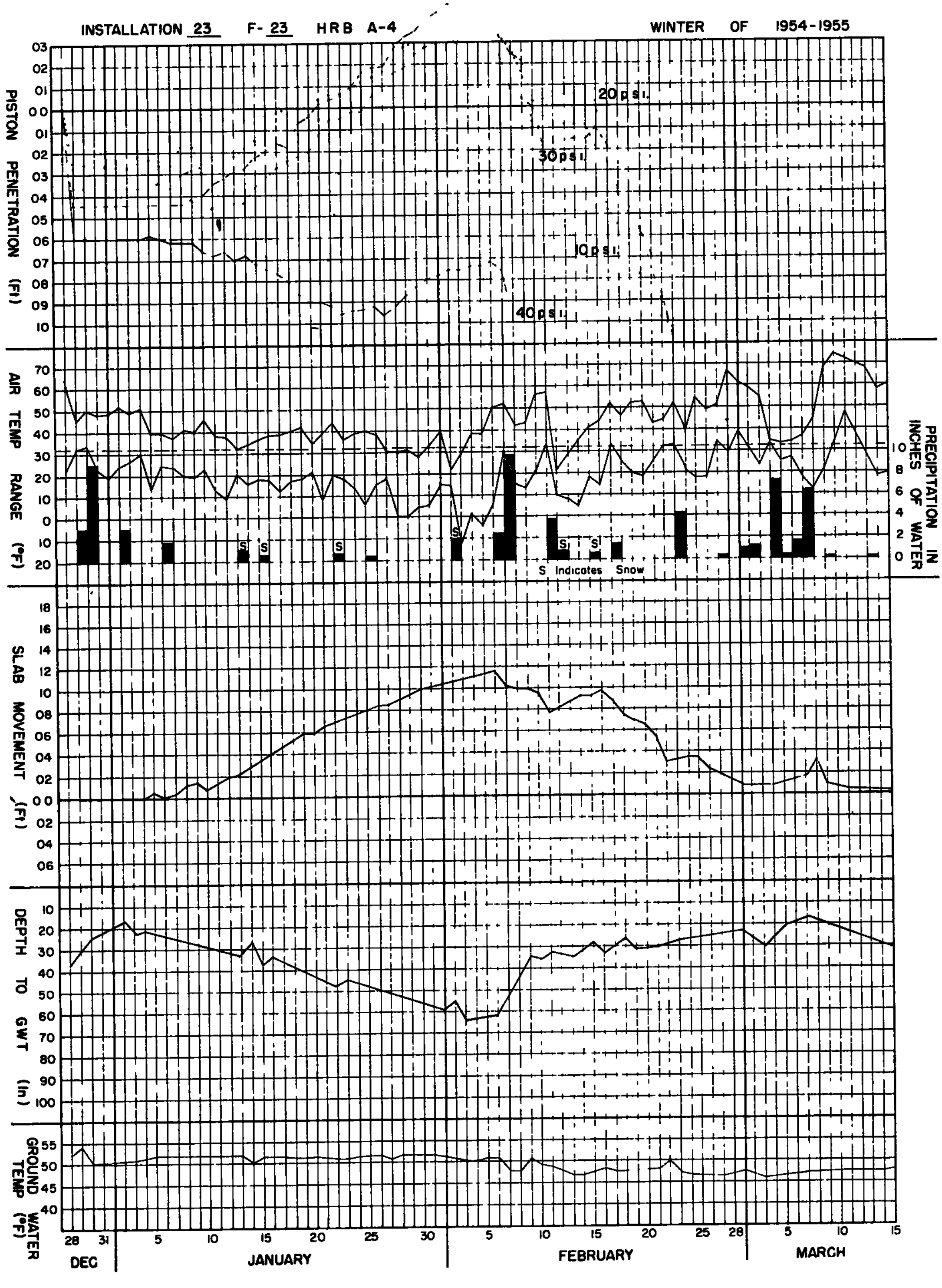
INSTALLATION 17 F- 17 HRB A-3
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
10 psi.
20 psi
30 psi.
40 psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH


INSTALLATION 15 F-19 HRB A-7-5
WINTER OF 1954-1955
PISTON PENETRATION (ft)
40 p.s.i.
30 p.s.i.
20 p.s.i.
10 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

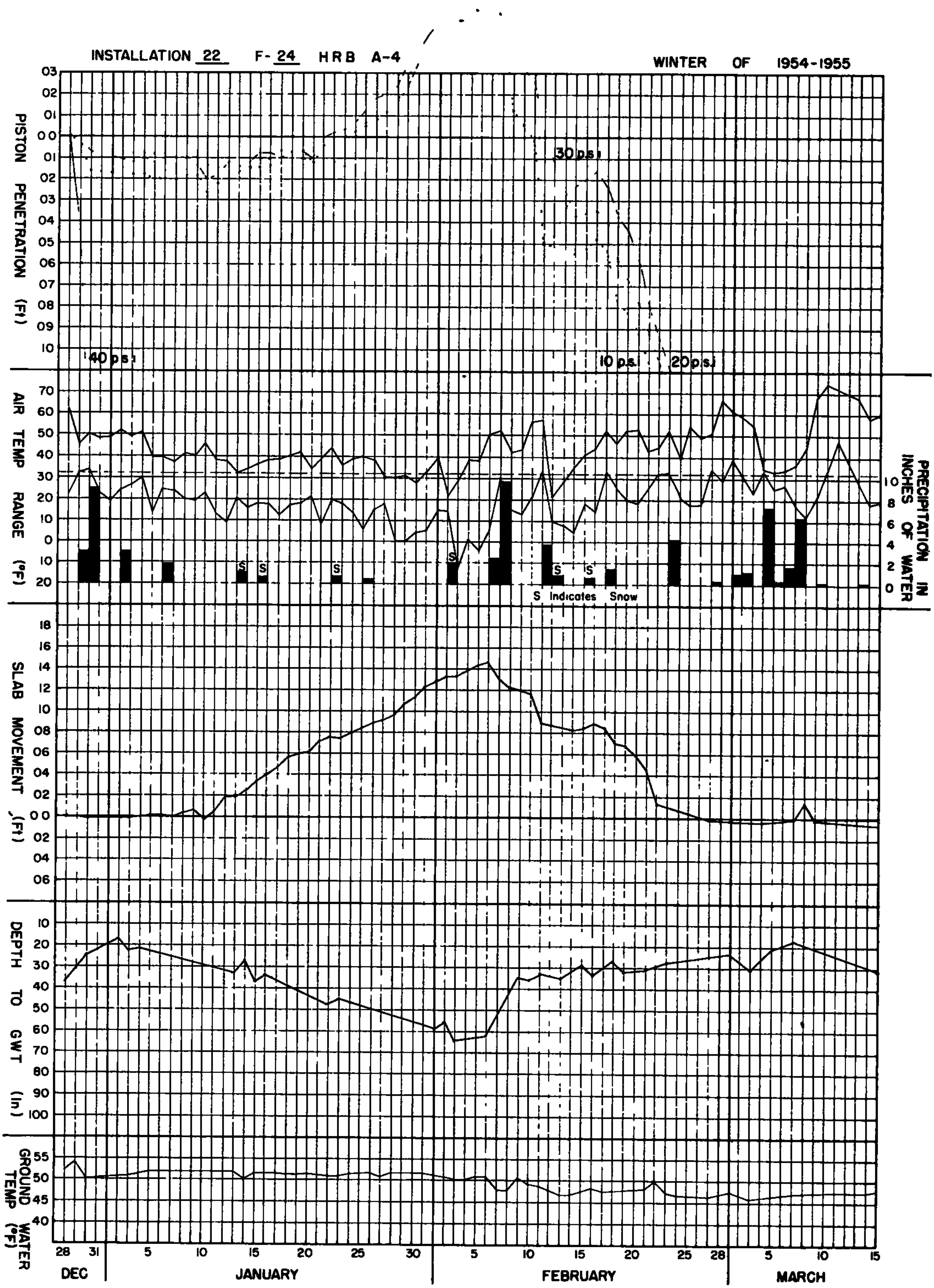

INSTALLATION 20 F- 20 HRB A-3
WINTER OF 1954-1955
PISTON PENETRATION (FT)
10 p.s.i.
20 p.s.i.
40 p.s.i.
30 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (FT)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

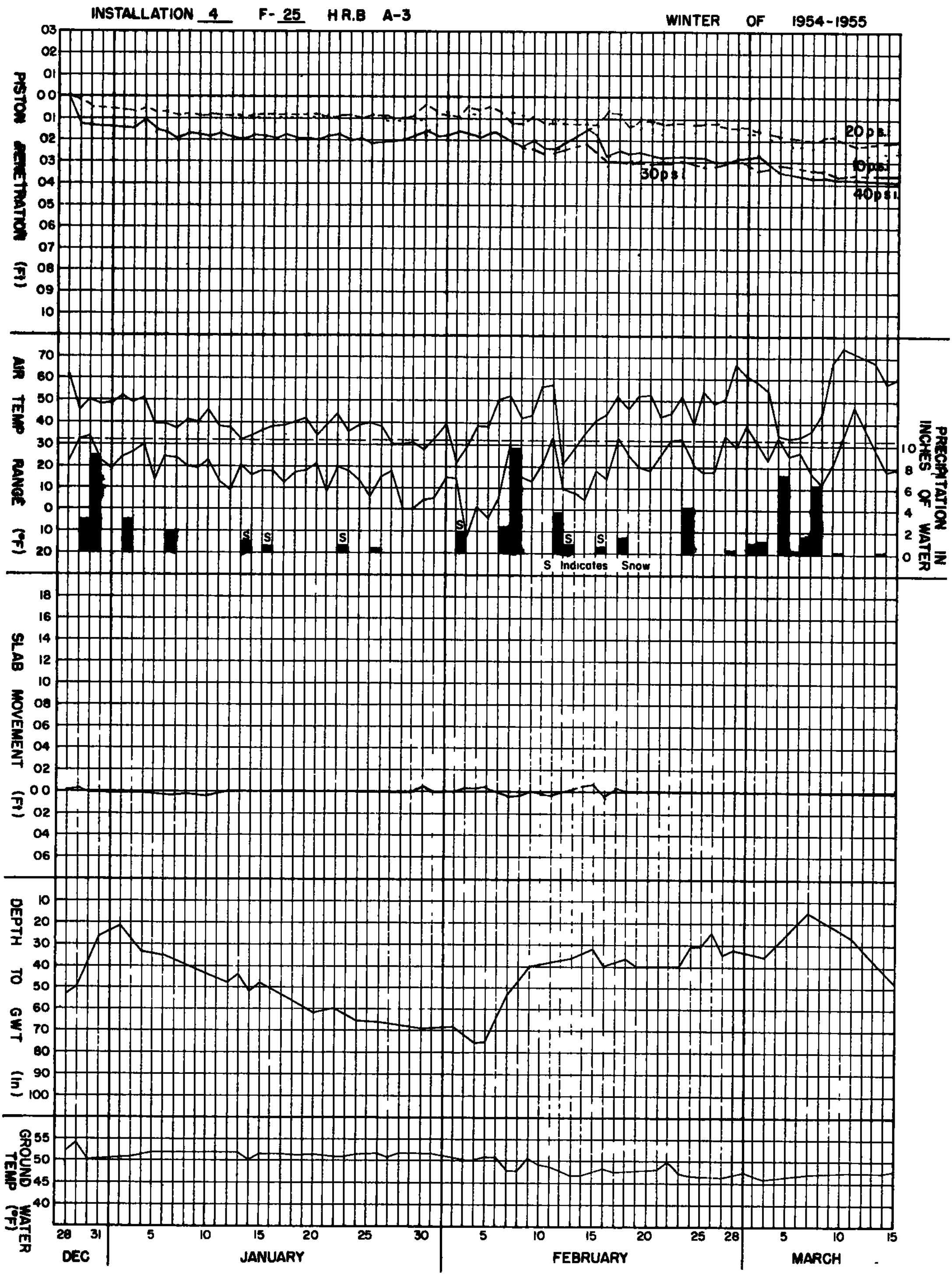

INSTALLATION 7 F- 21 HRB A-6
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
30 psi
40 psi
20 psi
10 psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH


INSTALLATION 21 F- 22 HRB A-4
WINTER OF 1954-1955
PISTON PENETRATION (ft)
20,30 & 40 psi.
10 psi.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

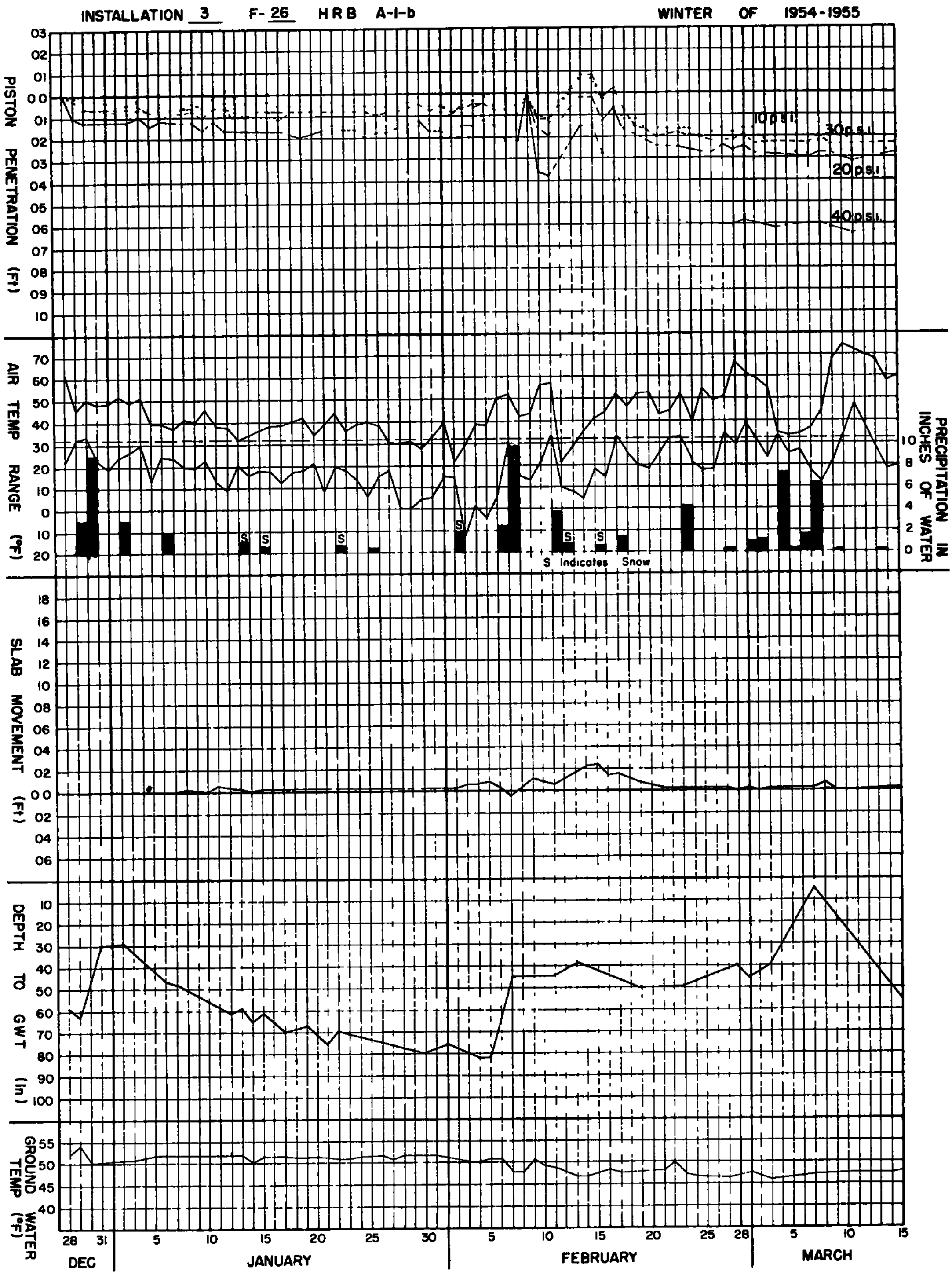


INSTALLATION 23 F- 23 HRB A-4
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
20 psi.
30 psi.
10 psi.
40 psi.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

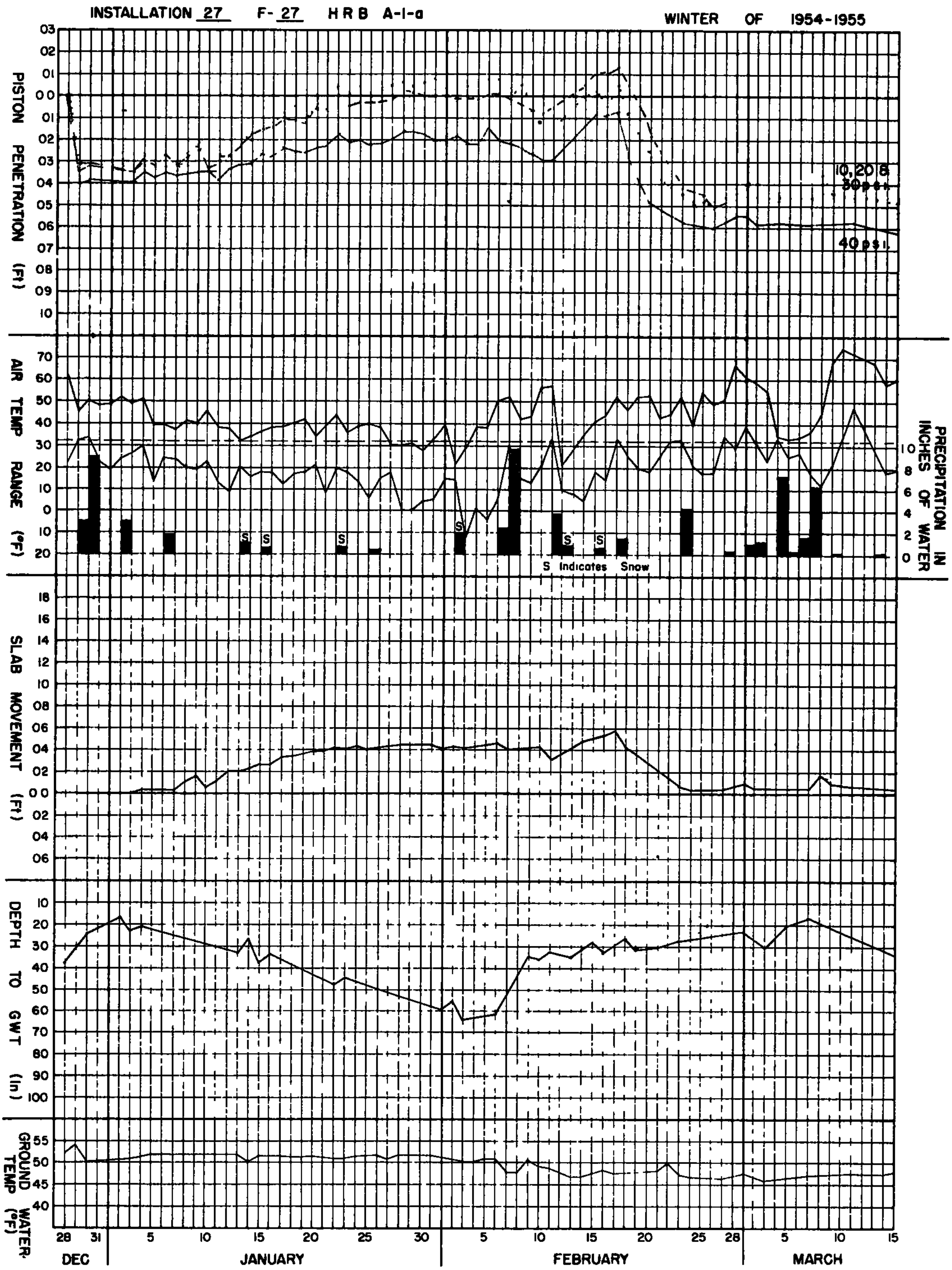

INSTALLATION 22 F-24 HRB A-4
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
30 p.s.i
40 p.s.i
10 p.s.
20 p.s.i
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

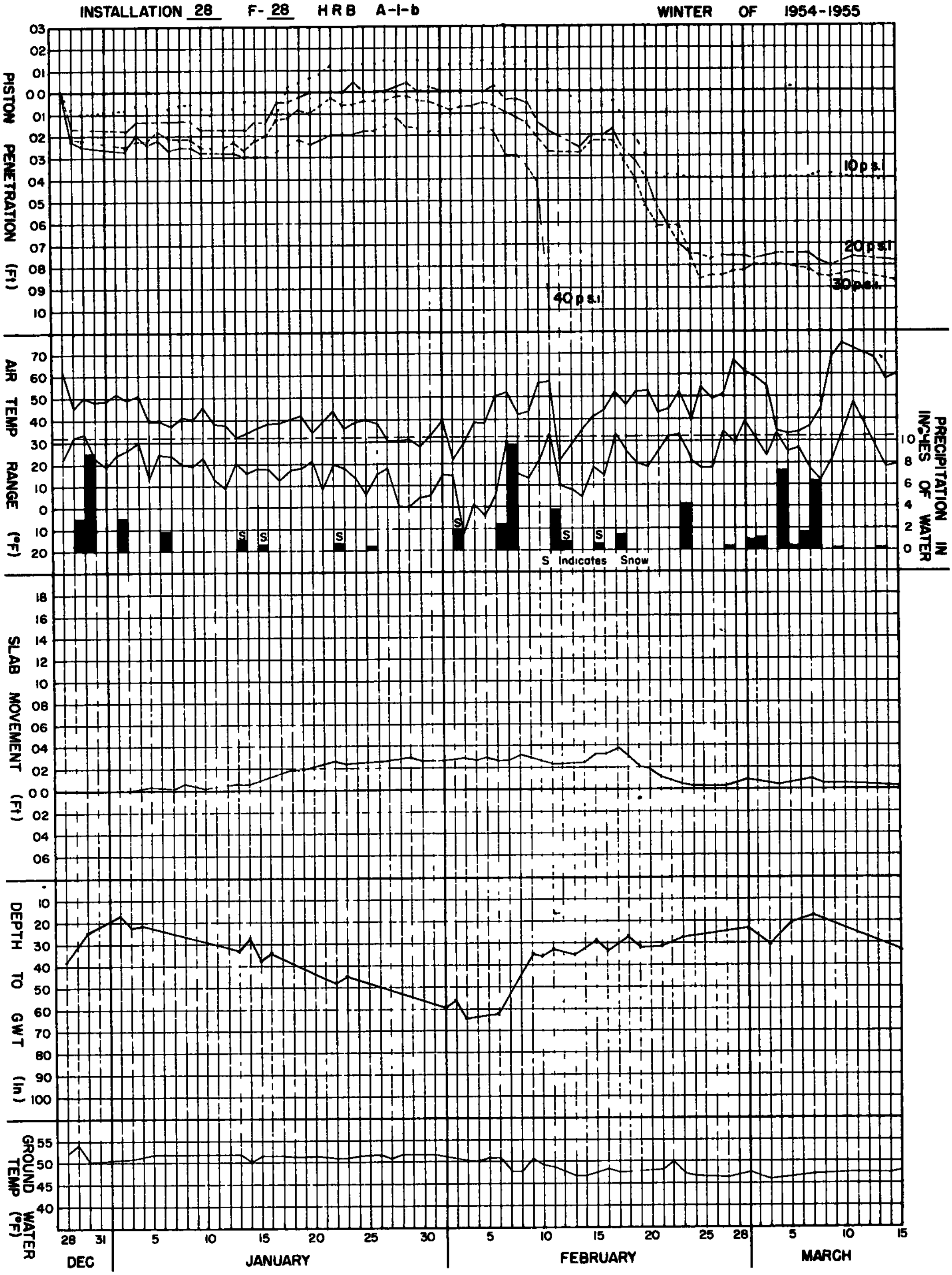


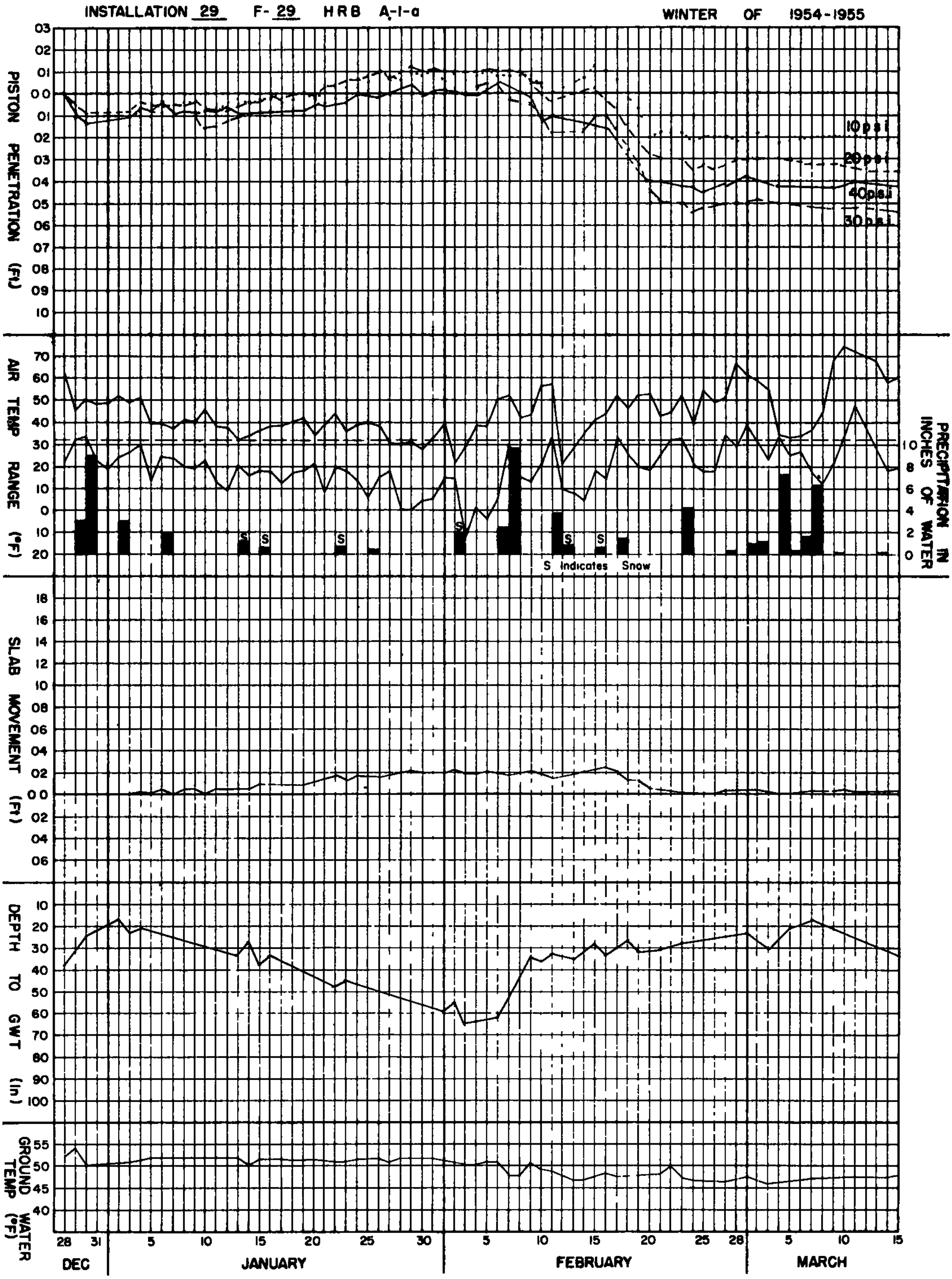
INSTALLATION 4 F-25 H.R.B A-3
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
20 p.s.i.
10 p.s.i.
30 p.s.i.
40 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH



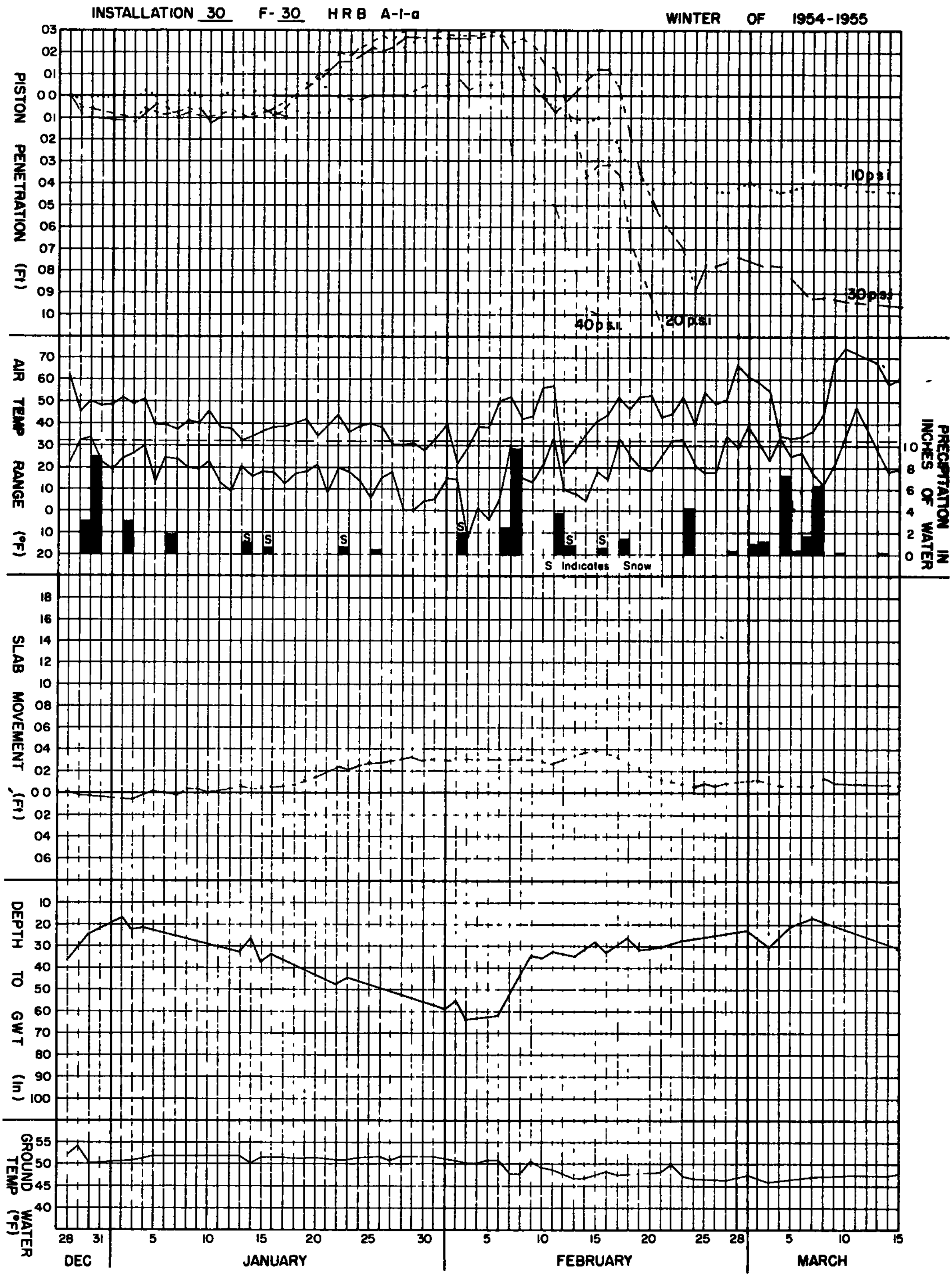
INSTALLATION 3 F-26 HRB A-1-b
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
10 p.s.i.
30 p.s.i.
20 p.s.i.
40 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH


INSTALLATION 27 F-27 HRB A-1-a
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
10,20 & 30 psi
40 psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH


INSTALLATION 28 F- 28 HRB A-1-b
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
10 p.s.i.
20 p.s.i.
30 p.s.i.
40 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

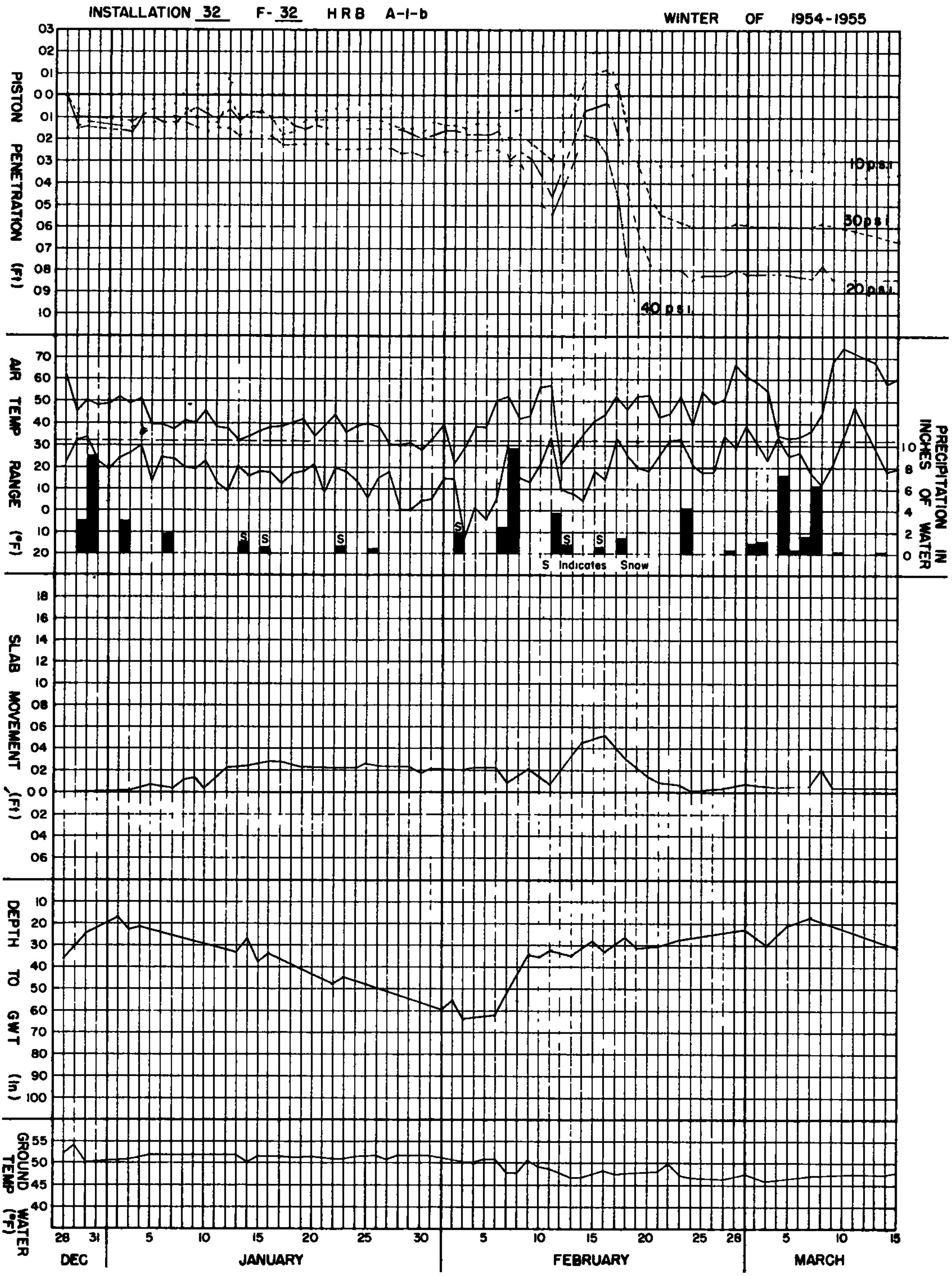


INSTALLATION 29 F- 29 HRB A-1-a
WINTER OF 1954-1955
PISTON PENETRATION (Ft.)
10 psi
20 psi
40 psi
30 psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

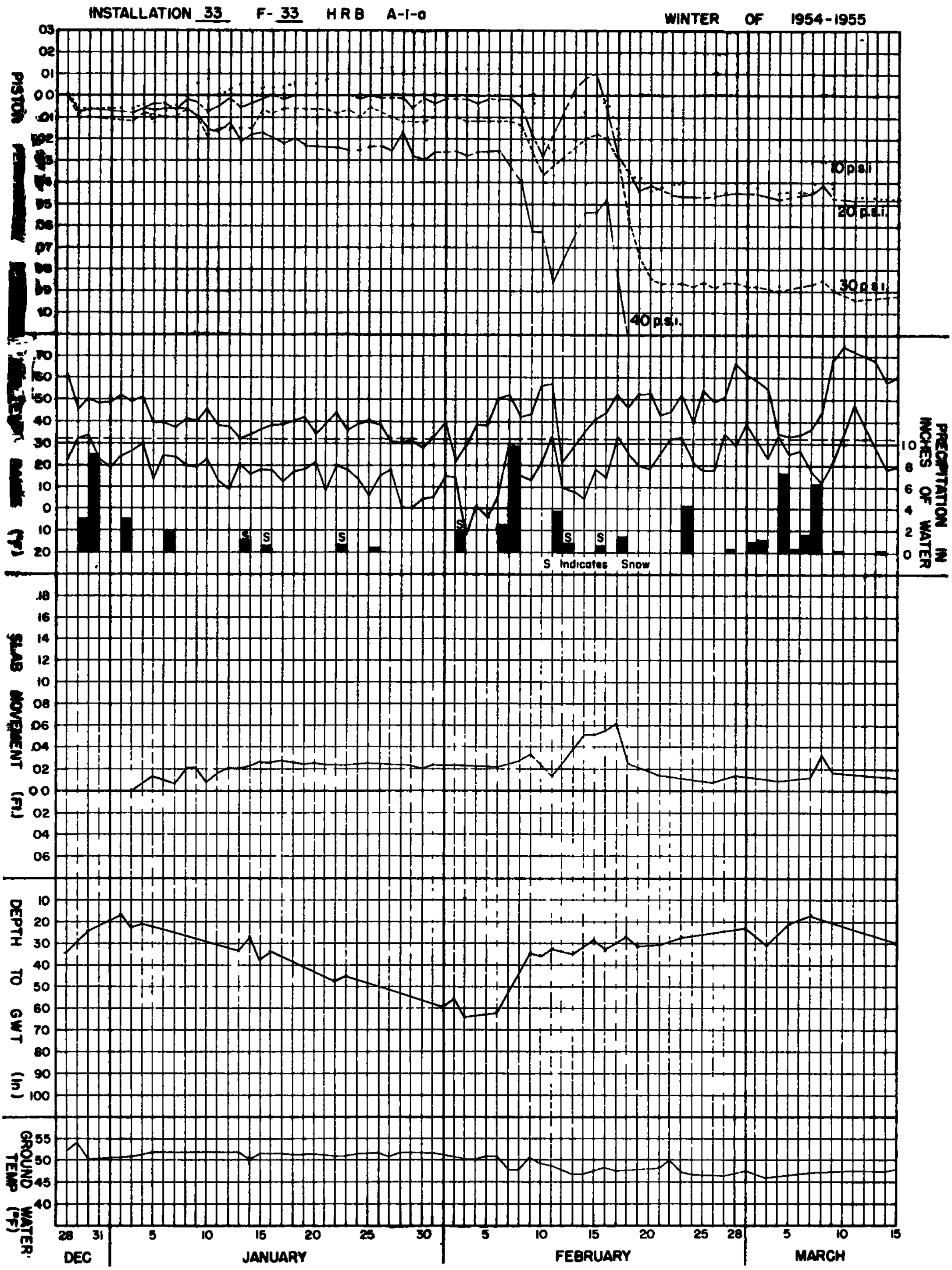


INSTALLATION 30 F-30 HRB A-1-a
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
10psi
30psi
40psi
20psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH

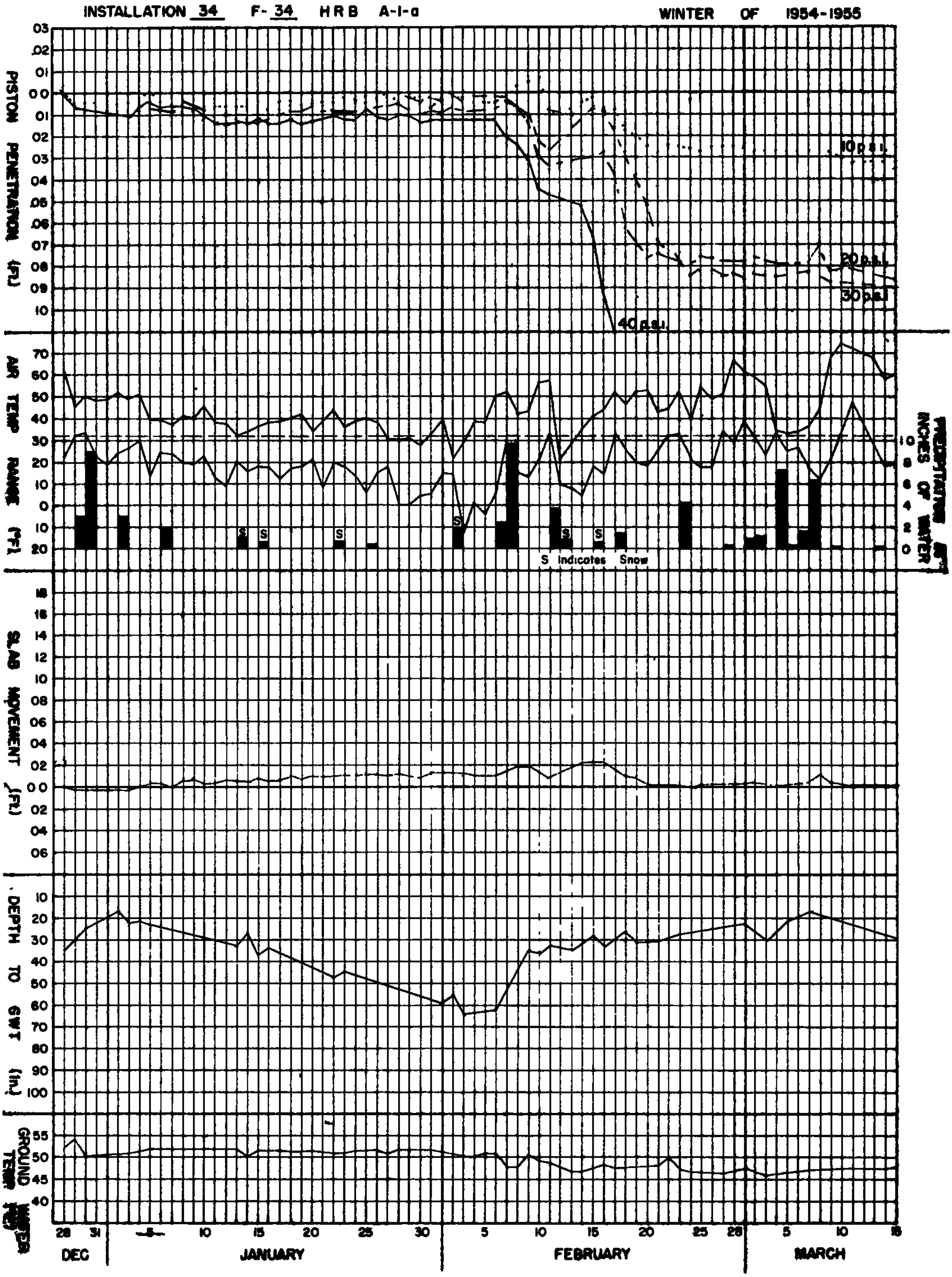

INSTALLATION 31 F-31 HRB A-1-a
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
10psi
30psi
40psi
20psi
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (in)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH



INSTALLATION 32 F-32 HRB A-1-b
WINTER OF 1954-1955
PISTON PENETRATION (Ft)
10 p.s.i.
30 p.s.i.
20 p.s.i.
40 p.s.i.
AIR TEMP RANGE (°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft)
DEPTH TO GWT (In)
GROUND WATER TEMP (°F)
DEC
JANUARY
FEBRUARY
MARCH



INSTALLATION 33 F- 33 HRB A-1-a
WINTER OF 1954-1955
PISTON
10 p.s.i.
20 p.s.i.
30 p.s.i.
40 p.s.i.
(°F)
PRECIPITATION IN INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (Ft.)
DEPTH TO GWT (In.)
GROUND WATER TEMP. (°F)
DEC
JANUARY
FEBRUARY
MARCH


INSTALLATION 34 F-34 HRB A-I-a
WINTER OF 1954-1955
PISTON PENETRATION (FT.)
10 p.s.i.
20 p.s.i.
30 p.s.i.
40 p.s.i.
AIR TEMP RANGE (°F.)
PRECIPITATION INCHES OF WATER
S Indicates Snow
SLAB MOVEMENT (FT.)
DEPTH TO G W T (in.)
GROUND TEMP
DEC
JANUARY
FEBRUARY
MARCH

Subsurface Temperatures and Moisture Contents In Six New Jersey Soils, 1954-1955

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ALFREDS R. JUMIKIS, Professor of Civil Engineering
Rutgers University

● SUBSURFACE temperatures and moisture contents were measured in six New Jersey soils subjected to natural freeze-thaw conditions during the winter of 1954-1955. These soils, compacted in 9-foot square by 2-foot deep pits, were among a group of 30 soils being used for frost heave and bearing capacity loss studies at the Joint Highway Research Project, Rutgers University. For this purpose a 4-foot square by 6-inch thick concrete slab was poured on each soil. In the six soils considered in this report temperatures and moisture contents were measured daily at selected intervals of depth beneath the centers of the concrete slabs and beneath the soil shoulders at the edge of the slabs. Air temperature, ground-water temperature, ground-water elevation, and the relative vertical displacement of the concrete slabs resulting from frost heave were also measured. This report presents an analysis of the data obtained.

SOILS

The six soils used for the temperature and moisture study were selected from the available soils to represent a range of HRB classification groups, and, for convenience in instrumentation, to present a compact grouping in their existing positions at the field installation at Rutgers University. The soils, listed according to their assigned sample numbers, may be described thus: F-5 a sandy silt-clay mixture, derived from old glacial lake bed sediments; F-10 a mixture of coarse, medium and fine sands containing considerable gravel and some silt and clay—derived from gneissic glacial materials which have been reworked by water; F-13 a sandy gravel with considerable silt and clay, derived from basalt and diabase (gravel size is large; fragments are angular); F-15 a silt-clay mixture, derived primarily from underlying Triassic argillite; F-17 a mixture of coarse, medium and fine sands, derived from Coastal Plain sediments; F-24 a well-graded mixture of gravel, sands, silt and clay, derived from till containing much limestone.

A representative fraction of each sample was tested in the Soil Mechanics Laboratory for the determination of engineering properties. These are presented in Table 1. Grain size distribution curves are shown in Figure 1.

Several years previously the soils had been placed in the 9-foot square by 2-foot deep pits at the field installation. They were originally compacted in 6-inch layers by means of an air hammer. Recompaction of the soil surface developed field densities shown in Table 2. Measurement was by the sand cone method.

INSTRUMENTATION

For the measurement of subsurface soil temperatures and moisture contents the Fiberglas Soil-Moisture Instrument was selected (1, 2). This instrument, developed by E. A. Coleman at the California Forest and Range Experiment Station, consists of two basic parts: soil moisture units which are placed in the soil being studied and a meter unit used to measure the electrical resistance of the soil moisture units. The instrument was supplied by the Berkeley Division of Beckman Instruments, Inc.

Soil Moisture Unit

A fiberglas soil-moisture unit consists of two monel screen electrodes separated by and encased in a wrapping of fiberglas fabric. This electrode "sandwich" is mounted within a rigid monel case which is perforated to allow contact between the fiberglas and the soil. A Western Electric 7A thermistor is also enclosed in the case. Details of the

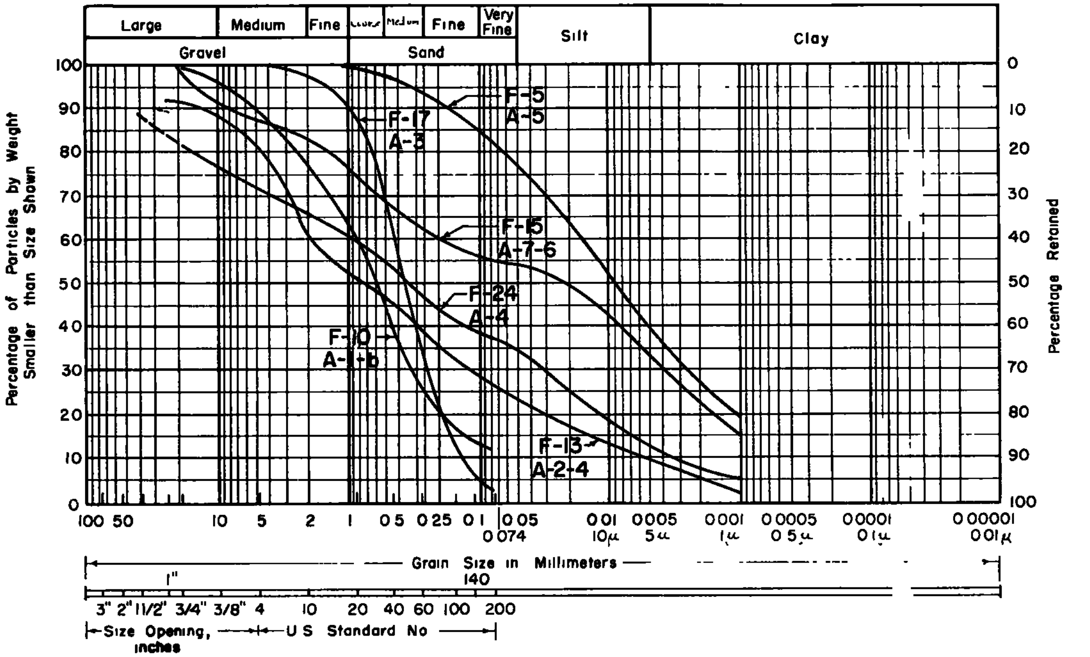

Figure 1. Grain size distribution curves.

soil moisture unit are shown in Figure 2.

When the unit is placed in soil the fiberglas readily absorbs or gives up moisture, so as to remain in a constant equilibrium of moisture content with the soil. The electrical resistance between the electrodes varies with the moisture content of the fiberglas. Thus, after proper calibration the measured electrical resistance of the soil moisture unit indicates the moisture content of the soil in which it is placed. The resistance of the unit is also dependent upon temperature, hence the inclusion of a thermistor so that temperature can be measured and a proper correction applied to the indicated resistance.

Meter Unit

The instrument used for measuring soil moisture unit resistance is a self-powered alternating current ohmmeter. Alternating current of 90 cps is generated in the vacuum tube circuit of the instrument, passed through the soil moisture unit or thermistor and rectified for indication on a d-c microammeter.

Selection of Unit Spacing

As a result of previous subsurface temperature measurements in the six soils used for this study, underground cables and junction boxes were already in place at the field

TABLE 1

ENGINEERING SOIL PROPERTIES

Sample No	Agronomic Name (as mapped 1917-27)	Soil Test Results: Sieve Analysis Percent Passing					Hyd Anal: Silt Sizes	Hyd Anal: Clay Sizes	Atterberg Test: LL	Atterberg Test: PI	Proctor: Max Dens	Proctor: Opt MC	Uniform Coef D60/D10	Eff Grain Size	HRB Classification: Subgrade Group	HRB Classification: Group Index
		3/4	4	10	40	200	%	%	%	%	pcf	%				
F-5	Whippany	100	100	100	98	83	43	37	41	7	100	22	50 0	--	A-5	8
F-10	Merrimac	100	90	77	41	11	--	--	NL	NP	125	9	11 4	07	A-1-b	0
F-13	Montalto	91	80	58	46	28	9	19	32	9	114	17	360 0	03	A-2-4	0
F-15	Lansdale	99	87	85	69	55	21	32	41	15	95	26	283 3	--	A-7-6	6
F-17	Lakewood	100	99	98	64	3	--	--	NL	NP	106	14	2 8	15	A-3	0
F-24	Dover	82	72	66	54	37	20	14	31	9	112	16	400 0	0025	A-4	0

installation. This equipment provided electrical circuits sufficient for placing 13 soil moisture units in each of the soils. Selected spacing of the units was at depths of 6, 10, 14, 18, 23 and 29 inches beneath the completed top surface of the concrete slab, and at depths of 2, 6, 14, 18, 23 and 29 inches beneath the soil shoulder which is flushed with the slab surface.

TABLE 2

FIELD DENSITIES

Soil No	Dry Density	Percent of Proctor Density
	pcf	
F-5	98 8	98 8
F-10	111 9	89 6
F-13	101 2	88 9
F-15	88 9	93 5
F-17	104 0	98 2
F-24	104 3	93 3

Calibration

Thermistor resistance varies with temperature. The resistance of each thermistor falls within ± 10 percent of a standard resistance-temperature curve (Figure 3). Thermistor calibration is accomplished by measuring resistance at a known temperature. A thermistor coefficient is determined by dividing the standard resistance at this temperature by the measured resistance. When performing temperature measurements with a thermistor the measured resistance is multiplied by the coefficient and the corrected resistance referred to the standard curve for temperature determination. For convenience, a table was prepared showing the range of standard thermistor resistance for each 0.1 deg F. Soil moisture units were supplied with calibrated thermistors, the coefficients being engraved on the cases by the manufacturer.

As the electrode "sandwich" of the soil moisture unit is not standardized, it was necessary to calibrate each unit in the soil in which it was to be placed.

The laboratory method of calibration suggested by the manufacturer was followed with several modifications (2). Briefly, this method consists of packing each soil moisture unit with a small amount of its respective soil into a small container. The soil was saturated and dried slowly at room temperature, its moisture content being determined at intervals by weighing. Measurement of soil moisture unit resistance at respective intervals provided a calibration curve.

The manufacturer's instructions suggest several drying cycles before calibration and several during calibration. As anticipated, each drying cycle proved to be a lengthy process. As this study was dependent upon natural climatic conditions, the time available for calibration of the soil moisture units before installation in the field was limited. It was thus possible to perform only two drying cycles, both of which were used for calibration.

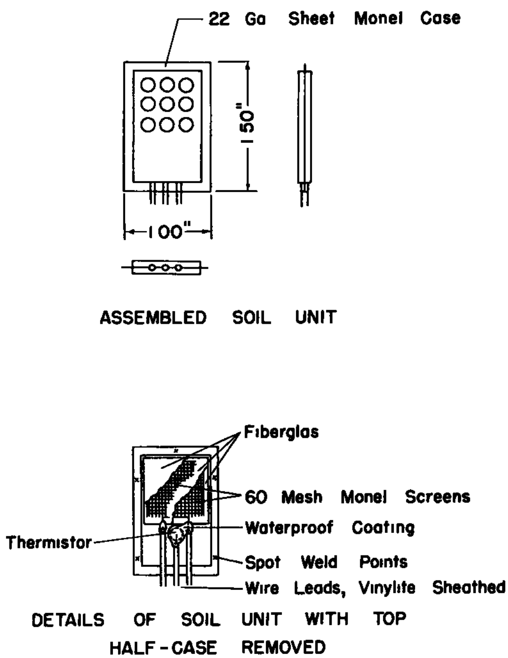

Figure 2. Fiberglas soil moisture unit.

Calibration Details

Seventy-eight calibration boxes were made to allow simultaneous calibration of all soil units. Monel was used for resistance to corrosion and ease of fabrication. Details of a calibration box are shown in Figure 4. Each soil moisture unit and its calibration box were assigned code numbers and their weights recorded. Thirteen units were assigned to each soil.

Samples of each soil were prepared for calibration by removing all material retained on the No. 10 sieve and oven-drying. A small specimen of each soil was mixed with water to achieve respective optimum moisture content. Each specimen was then compacted in an empty calibration box by tamping with the end of a short length of

½-in. square rod. To prevent distortion of the screened sides, the box was supported between the upright legs of pieces of 2- by 2-in. angle clamped to a steel plate. Density determination of the compacted specimens showed that reasonable approximations of Proctor density could be achieved by this means of compaction.

The preparation of each soil moisture unit for calibration was achieved as follows: A 150-g specimen of the required soil was mixed thoroughly with sufficient water to bring it to optimum moisture content. The soil moisture unit was placed vertically in its respective calibration box, the wires at the top, and the soil compacted around it. The box was filled to within ¼ in. of its top. The remainder of the soil specimen was then oven-dried and weighed. Subtracting this weight from 150 g gave the weight of dry soil in the calibration box.

Six humidity chambers were prepared for the calibration process. Clear plastic cake covers 11 inches in diameter by 5 inches high were placed on plywood bases. Slots were provided in the bases to allow passage of the soil moisture unit wires. In order to maintain high humidity, a jar containing a wick and filled with a saturated solution of lead nitrate, $Pb(NO_3)_2$, was placed in each chamber.

Upon completion of preparation of the soil moisture units in the calibration boxes they were placed in shallow pans and sufficient distilled water added to cover the soil surface. After saturating overnight the boxes were allowed to drain briefly and placed in the humidity chambers for a 48-hour period. The covers of the chambers were then removed for a five-hour period to allow for evaporation of soil moisture. The covers were then replaced. At the end of a 19-hour period the electrical resistances of each soil moisture unit and thermistor were measured and the boxes removed from the humidity chambers and weighed. The calibration cycle of five hours' evaporation and 19 hours in the humidity chamber was continued until the soils no longer showed loss of moisture. This cycle was described by Kelley (3). His experiments show that for most soils a 19-hour period in the humidity chamber is sufficient to allow the moisture content throughout the soil specimen to reach an equilibrium. The average moisture con-

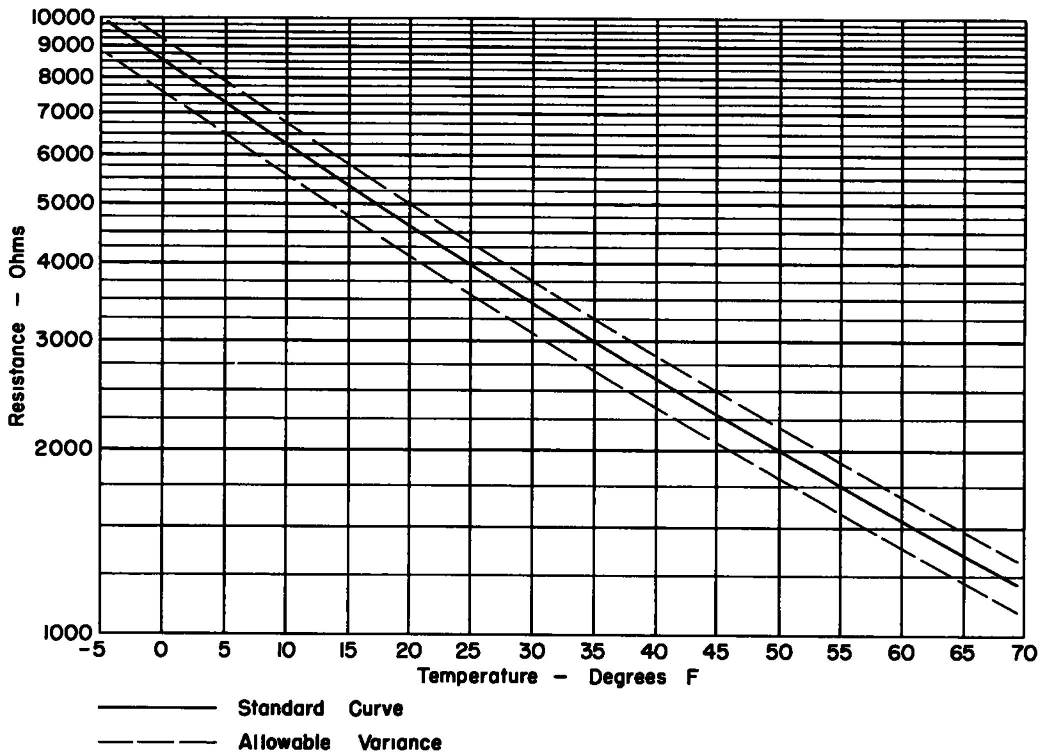

Figure 3. Temperature-resistance curve for 7-A and 17-A thermistors.

tent of the specimen determined by weighing is essentially the same as that of the soil at the center of the specimen in contact with the soil moisture unit.

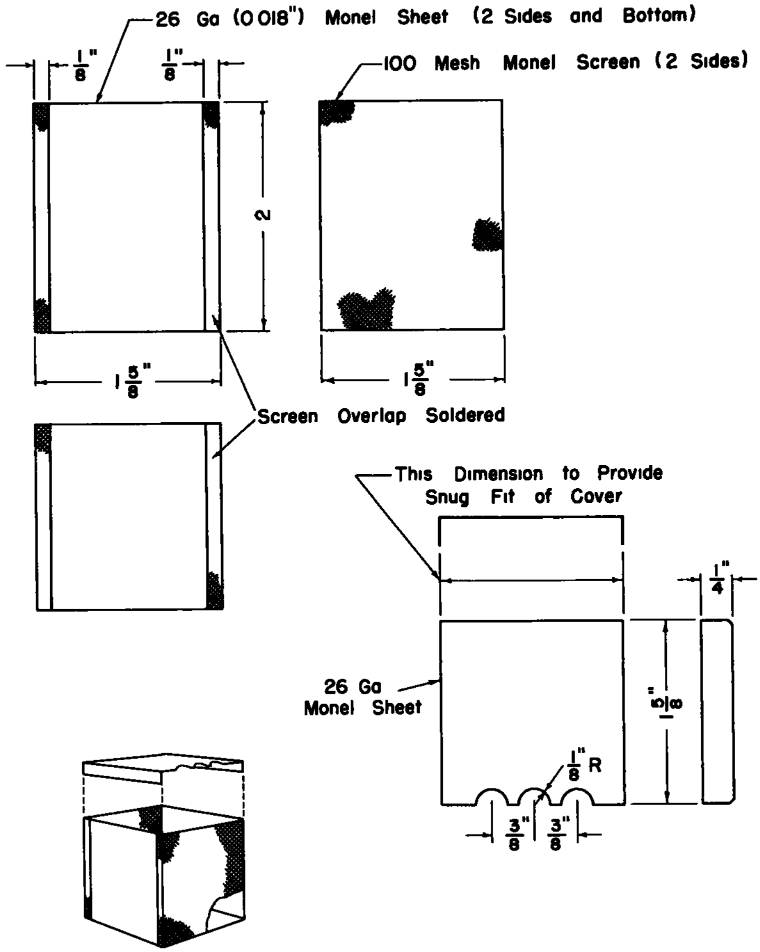

Figure 4. Calibration box for soil moisture unit.

Soil temperature was calculated from thermistor resistance and soil moisture unit resistance corrected to 60 deg F by means of a chart furnished by the manufacturer.

Weighing during each calibration interval determines the total amount of water in both the soil specimen and the soil moisture unit. The water within the soil moisture unit does not contribute to the moisture content of the soil specimen. Hence, the weight of this water must be determined and subtracted from the total in order to determine the moisture content of the soil specimen.

Calibration to determine the relationship between soil moisture unit resistance and the weight of water in the unit was performed on five units. The units were saturated with distilled water and left exposed for evaporation. Resistance and weight were determined at one-hour intervals. Resistance was corrected to 60 deg F. A plot of resistance vs. water in unit for each of the five soil moisture units indicated that the characteristics of the units were sufficiently similar to allow the use of an average curve based on these five units for the purpose of determining the weight of water in each of the other 78 soil moisture units during calibration.

Though distilled water was also used in the calibration of the 78 soil moisture units it was felt that materials from the soils might be dissolved with varied effects upon the resistance of the units. Investigation of these effects proceeded as follows: A small amount of one of the soils was stirred into distilled water and allowed to settle. The clear water containing any solutes from the soil was poured off and used to saturate the five soil moisture units. The calibration process was repeated. The average curve of resistance vs water in moisture unit which resulted was decidedly different from that produced by the use of distilled water alone, resistance values being relatively lower.

The five soil moisture units were then washed thoroughly by flushing with distilled water to remove the soil solutes. The process was then repeated, using solutions from the remaining soils. An average calibration curve of soil moisture unit resistance vs weight of water in the soil moisture unit was then produced for each of the six soils. An example is shown in Figure 5.

During the calibration of the 78 soil moisture units these average curves were used in calculating the weight of water within each soil specimen for the purpose of moisture content determination. As previously mentioned, two drying cycles were performed.

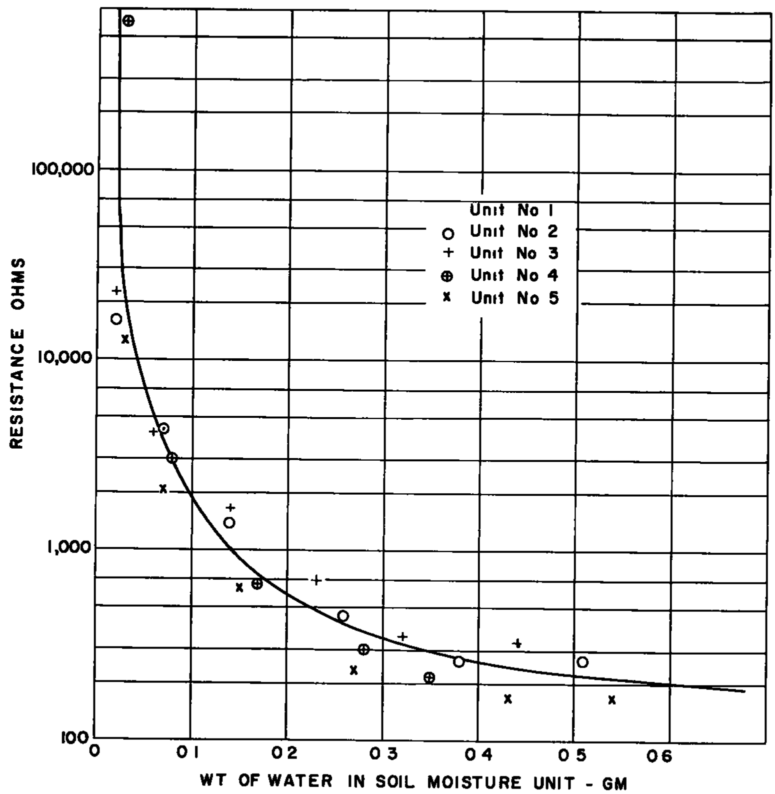

Figure 5. Relationship between soil moisture unit resistance and weight of water in unit. Resistance corrected to 60 deg. Units saturated with a solution of soil F-15.

Plots of resistance vs moisture content for each of the soil moisture units showed rather close correlation between the points determined during each cycle. Similarity was noted among the 13 curves developed for each soil. Considerable contrast was apparent, however, when the curves of each soil were compared with those of the others. Examples of calibration curves are shown in Figures 6 and 7.

Thermistor Calibration

Sensitivity of the a-c ohmmeter when measuring thermistor resistance was not sufficient to allow temperature determination to the desired accuracy of 0.1 deg F. During previous subsurface temperature studies a d-c Wheatstone bridge was found satisfactory for thermistor resistance measurement. The soil moisture unit thermistors were recalibrated with this instrument. The units were immersed in circulating ice water at 32.0 deg F and their thermistor resistances measured. The resulting thermistor coefficients were thus determined to a greater accuracy than those supplied by the manufacturer, using the a-c ohmmeter.

Thermistor Preparation

Measurement of moisture content at the surface of the soil shoulders was considered impractical. To allow soil temperature measurement along at these points Western Electric 17-A thermistors were used. Their characteristics are similar to those of the 7-A thermistors, the same standard curve and method of calibration being applicable. The 17-A thermistors were prepared for field use as shown in Figure 8, a method which proved satisfactory in previous subsurface temperature studies.

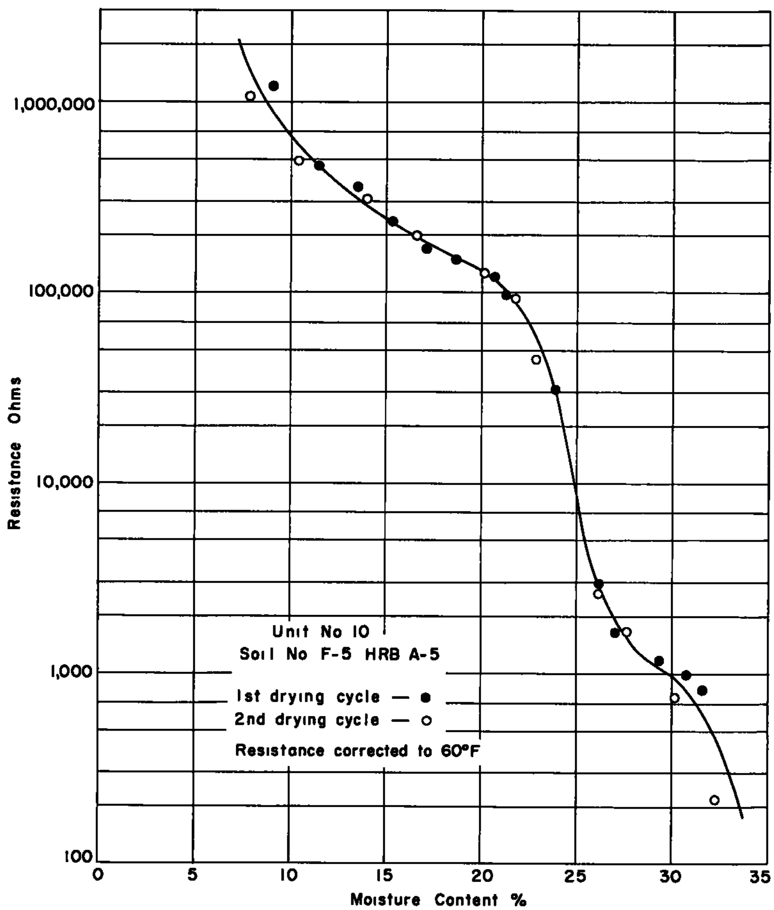

Figure 6. Soil moisture unit calibration curve.

Three groups of 17-A thermistors were also prepared for use in soil F-10. Two groups were intended for installation beneath the concrete slab and one beneath the soil shoulder at depths corresponding with the soil moisture units. These groups of thermistors and soil moisture units were all arranged in a single vertical plane for the purpose of determining the profile of frost penetration beneath the shoulder and slab.

INSTALLATION OF SOIL MOISTURE UNITS AND THERMISTORS

The soil moisture units and thermistors were arranged in groups according to the

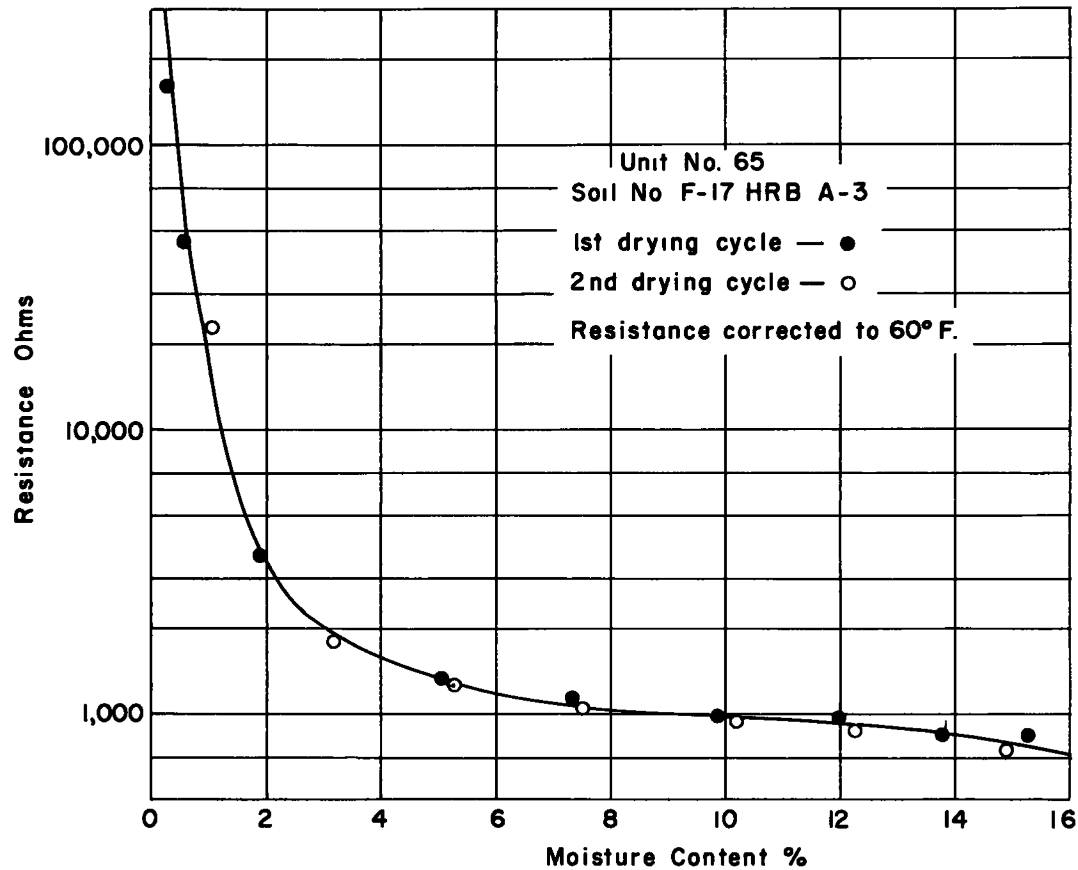

Figure 7. Soil moisture unit calibration curve.

soils in which they were calibrated. Their wire leads were cabled with nylon cord, the units being spaced according to their required depth of placement. The wire leads were lengthened where necessary to reach the existing junction boxes. All splices were carefully insulated with varnish and tape.

After leveling and compaction of the soils in the field, 8-inch diameter auger holes were placed at the proper positions for installation of the soil moisture units and thermistors. Shallow trenches were formed to carry the wire leads beneath the soil surface (Figure 9).

The prepared groups of soil moisture units and thermistors were then lowered into their respective holes and the soil replaced, each unit being carefully compacted in place at its respective depth. The wire leads were pulled into the junction boxes, the trenches filled and the soil surfaces smoothed. The soil moisture units and thermistors intended for installation in the top six inches of the soil shoulders were left exposed. Forms were set and the concrete slabs poured. Additional soil was then put in and compacted, building up the shoulders flush with the slab surfaces. The remaining soil moisture units and thermistors were placed at their respective depths. Completed soil installations are shown in Figures 10 and 11. After a two-day period to allow the soil moisture units to reach

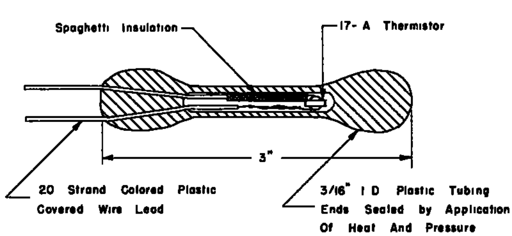

Figure 8. Section through a sealed thermistor unit.

equilibrium with the soil, measurements were started.

INSTRUMENTS

The instruments for measuring soil moisture unit and thermistor resistance were conveniently located in a small heated building. Permanent underground cables were used for the electrical circuits between the building and the soil installations (Figure 12). Within the building, circuits were connected to panel-mounted rotary selector switchs (Figure 13). By this means each soil moisture unit and thermistor could be connected in turn to its respective instrument. The resistances of the moisture elements of the soil moisture units were measured by means of the a-c ohmmeter. Resistances of all thermistors were measured with a Leeds and Northrup d-c Wheatstone bridge. For convenience in balancing the bridge a Brown Electronik Null Indicator was used. Power for this instrument and for lighting was supplied by a 600-watt gasoline generator set.

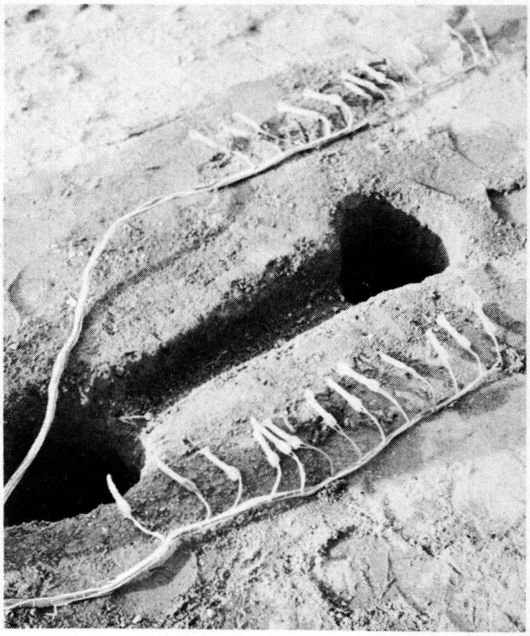

Figure 9. Groups of thermistors before installation in the soil.

CABLE RESISTANCE

The d-c resistances of all cable circuits were measured. The highest resistance determined for the longest cable, when added to the resistance of a thermistor, would change the temperature indication less than 0.1 deg F. Thus no correction was necessary.

When using the a-c ohmmeter for measuring soil moisture unit resistance, the apparent resistance of the cable resulting from the capacitance of the parallel wires must be considered. As suggested by the manufacturer, all cable circuits were opened at the junction boxes and their apparent resistances measured with the a-c ohmmeter. Six

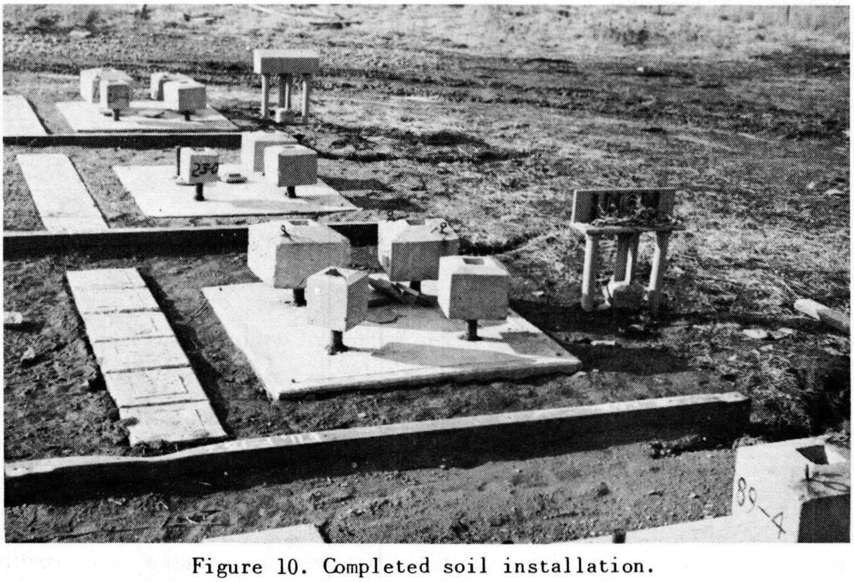

Figure 10. Completed soil installation.

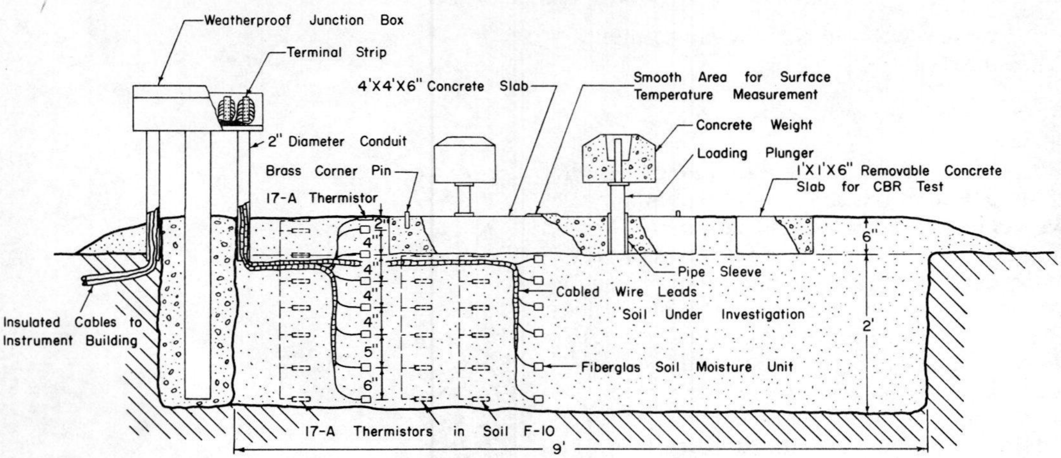

Figure 11. Soil installation equipped for subsurface soil temperature and moisture content determination.

correction curves were calculated to cover all circuits by means of the following expression (2):

$$\frac{1}{R(\text{soil unit})} = \frac{1}{R(\text{measured})} - \frac{1}{R(\text{cable})}$$

where R is resistance (Figure 14). The apparent resistance (reactance) of the cable is considered in parallel with the combined reactance and resistance of the soil moisture unit. Each measured soil moisture unit resistance was to be corrected by the use of

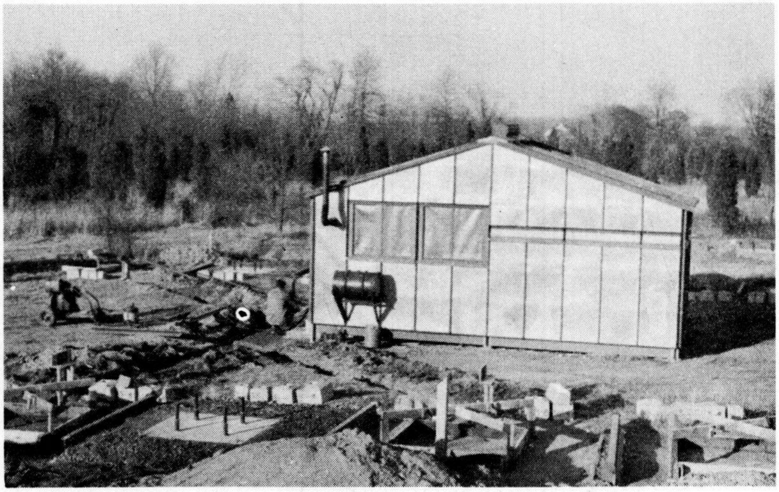

Figure 12. Preparation of soil installations. Wooden forms, completed slabs, installation of underground cables and the instrument building are shown.

the appropriate correction curve. During the period of winter study, however, it was found that all soil moisture resistances were so low, as a result of high soil moisture contents, that no corrections were necessary.

EQUIPMENT FOR OTHER DATA

Air Temperature

Maximum and minimum thermometers were mounted at elevations of 1 foot and 6 feet above the ground surface to determine daily air temperature range. A recording ther-

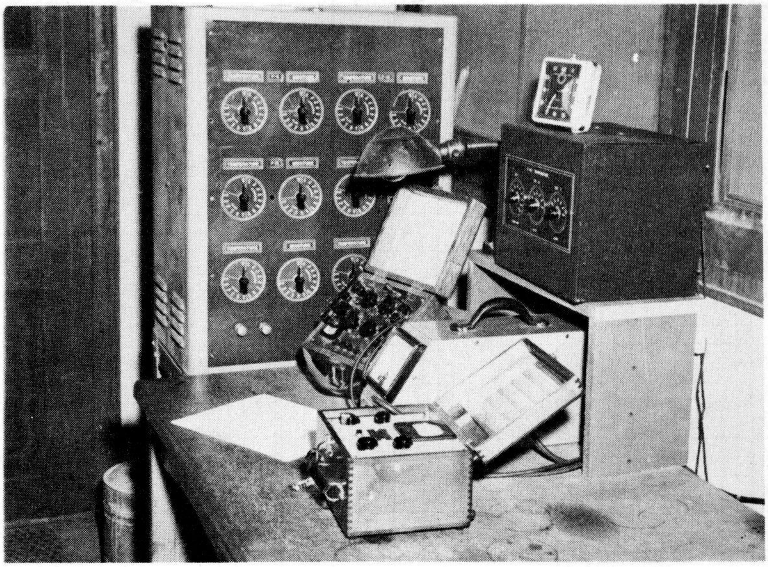

Figure 13. Instrument grouping for measuring subsurface soil temperatures and moisture contents.

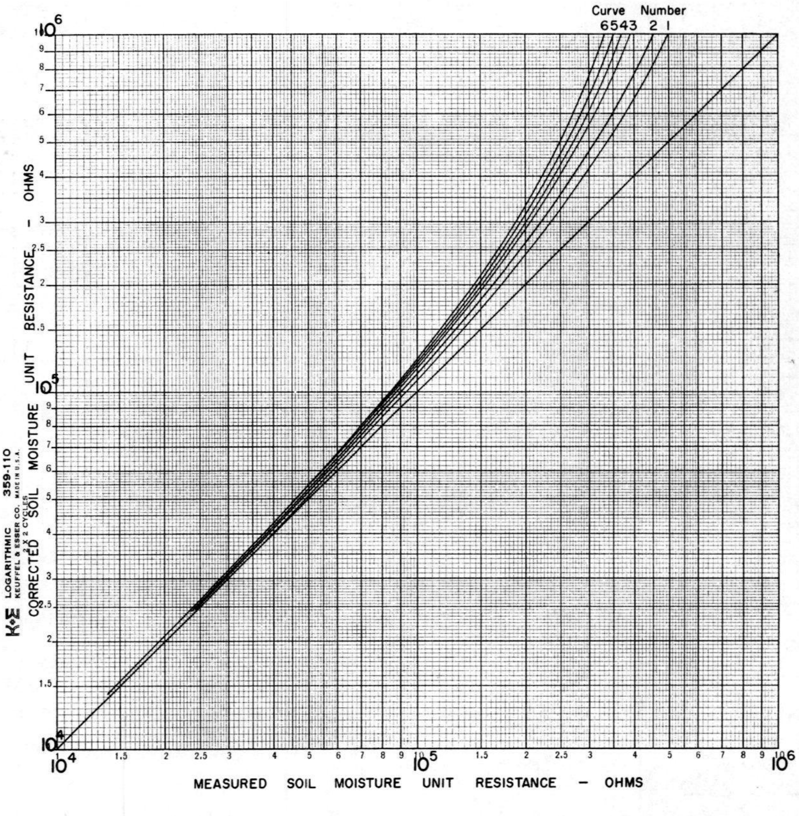

Figure 14. Correction curves for apparent cable resistance.

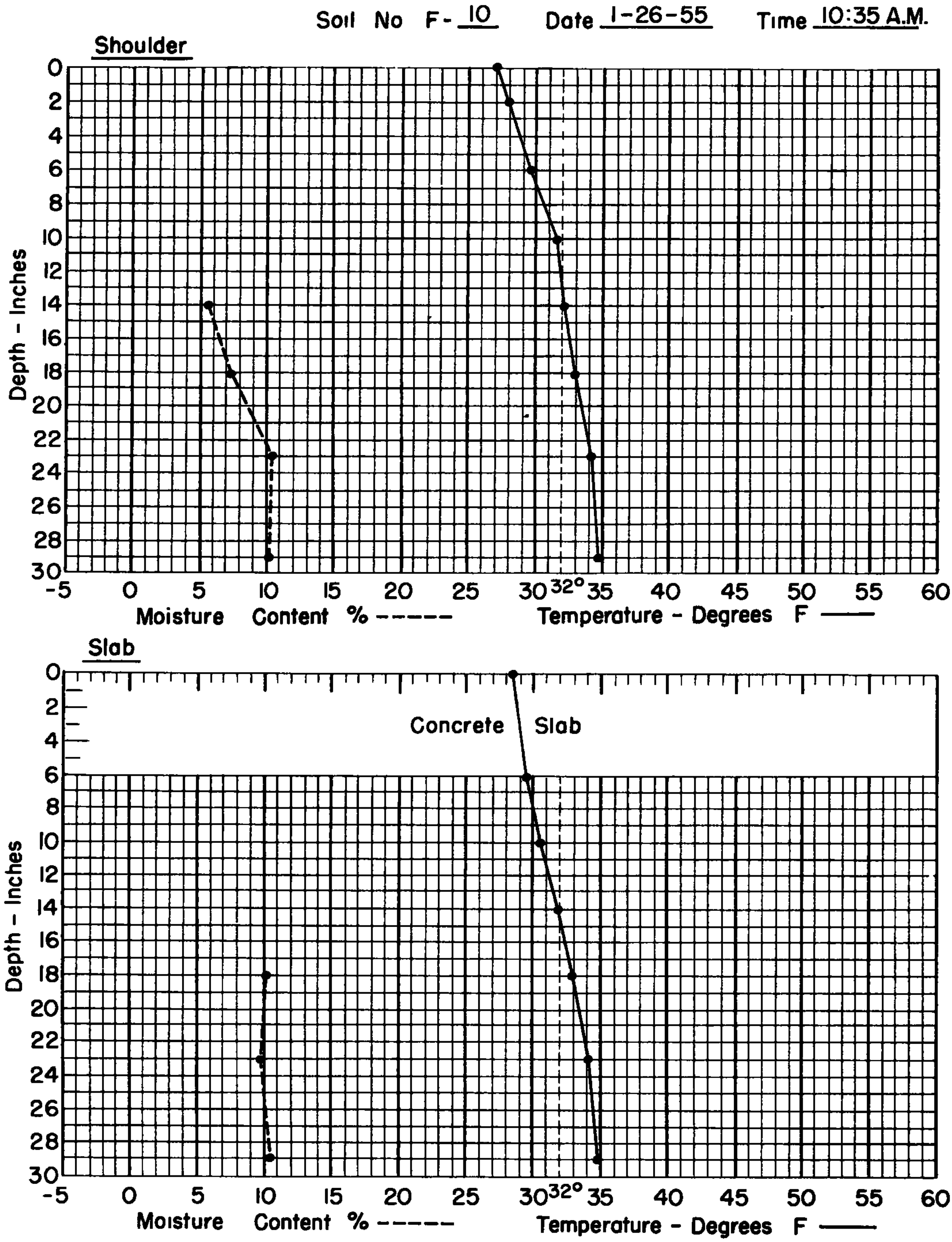

Figure 15. Subsurface soil temperature and moisture content.

mograph mounted at a 3-foot elevation was used to obtain a continuous record of air temperature.

Ground-Water Table

Three 20-foot deep wells were drilled at the field installation for the purpose of daily determination of ground-water elevation and temperature. For protection and to afford a means of capping, the wells were lined with steel casing at the ground surface. Ground-water elevation was measured with a cloth tape and float. Each well was provided with a thermometer suspended 15 feet below the ground surface by a cord. The thermometers were drawn up for reading temperatures.

Soil Heave

The amount of soil heave upon freezing was indicated by fluctuations in elevation of the concrete slabs. Each slab was provided with brass pins at its corners for reference points in determining its elevation. Slab elevations were measured daily by means of a Lenker L. E. Vation rod and a Gurley Wye level permanently mounted within the instrument building.

METHODS OF OBSERVATION

Daily measurements of air temperature and ground water were started on December 1, 1954. After completion of the installation of the soil moisture units, thermistors and

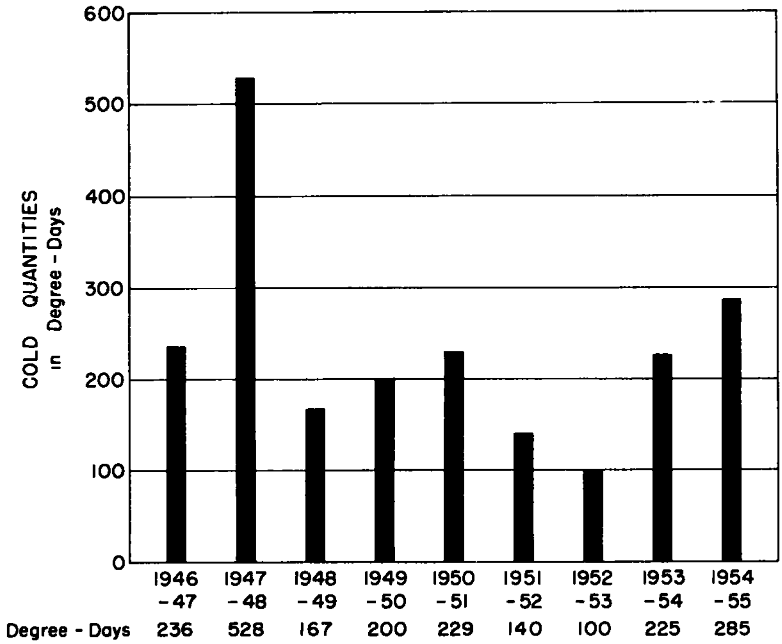

Figure 16. Cold quantities - New Brunswick Weather Station.

concrete slabs, daily subsurface temperature and moisture measurements and slab elevation measurements to the nearest 0.002 foot were started on December 28. Readings were made during the period from 9:00 to 11:00 a.m. each morning. All readings were continued until March 15, when frost activity had ceased. During the period of study the concrete slabs were cleared of snow when necessary to simulate highway conditions. Data were recorded on forms prepared for convenience in performing the calculations necessary for the determination of temperature, moisture content and relative soil heave.

ASSEMBLY OF DATA

Daily charts were prepared for each soil showing the relationship between soil depth, moisture content and temperature. An example is shown in Figure 15. From these charts the daily depth of frost penetration beneath the slabs and shoulders was interpolated for each soil. For this purpose it was assumed that soil moisture freezes at 32 deg F and the depth of frost penetration coincides with the depth of the 32 deg isotherm.

For each of the six soils studied master charts were prepared for both the soil beneath the slab and the soil shoulder. These charts (Appendix A) show the relationship between time and the following factors:

Precipitation. Precipitation data, plotted as a bar graph, were obtained from the United States climatological data for the New Brunswick, N.J., weather station. The letter "s" above a bar in the precipitation graphs indicates snow.

Air Temperature Range. Maximum and minimum temperatures measured at the 1 foot elevation above the ground surface were used to determine daily air temperature range.

Slab Displacement. Daily slab displacements were plotted using initial slab elevations determined on December 29, 1954, as zero reference points.

Frost Penetration. The 32 deg isotherm was plotted as the depth of frost penetration, the cross-hatched area on the chart indicating frozen soil.

Soil Moisture Content. The soil moisture content indicated by each soil moisture unit was plotted as a separate curve. The discontinuity of these curves results from the soil moisture units freezing and becoming inoperative.

Depth of Ground-Water Table. Ground-water elevation data produced three curves showing respective depth to the ground-water table at the surface elevations of the three wells. These curves were interpolated where necessary to determine depth to ground-water curves for each soil, based on the respective elevation of the soil.

Ground-Water Temperature. The ground-water temperature curve was plotted from the average temperature determined in the three wells.

OBSERVATIONS

Climatic Conditions

A general evaluation, based upon temperature, of the severity of the winter of 1954-1955 was made by means of cold quantity determined by a degree-day method. To allow comparison with previous winters, U.S. Weather Bureau climatological data for the New Brunswick, N.J. weather station was used. The differences between the daily mean temperature and 32 deg F for the days that the mean was lower than 32 deg F was totaled for the period from September, 1954, to April, 1955. A cold quantity of 285 degree-days was obtained. By comparison, the winter of 1947-1948, a recent outstanding example

TABLE 3

MAJOR PERIOD OF FROST PENETRATION

Soil	Position	Date of Initial Frost Penetration	Date of Complete Thaw	Number of Days Frost Was in Soil
F-5	Slab	Jan 12	Feb 23	42
HRB A-5	Shoulder	Jan 7	Feb 26	50
F-10	Slab	Jan 7	Feb 21	45
HRB A-1-b	Shoulder	Jan 5	Feb 20	46
F-13	Slab	Jan 14	Feb 23	40
HRB A-2-4	Shoulder	Jan 8	Feb 28	51
F-15	Slab	Jan 15	Feb 26	42
HRB A-7-6	Shoulder	Jan 8	Feb 27	50
F-17	Slab	Jan 8	Feb 21	42
HRB A-3	Shoulder	Jan 7	Feb 20	44
F-24	Slab	Jan 14	Feb 26	43
HRB A-4	Shoulder	Jan 7	Feb 25	49

TABLE 4

MAXIMUM DEPTH OF FROST PENETRATION

Soil	Position	Max Depth of Frost Penetration		Date of Max Frost Penetration	
		Inches			
F-5	Slab	22		Feb 6	
HRB A-5	Shoulder		22		Feb 7
F-10	Slab	26		Feb 6	
HRB A-1-6	Shoulder		25.5		Feb 5
F-13	Slab	17		Feb 5	
HRB A-2-4	Shoulder		20		Feb 6
F-15	Slab	22		Feb 6	
HRB A-7-6	Shoulder		21		Feb 5,6
F-17	Slab	29		Feb 5,6,7	
HRB A-3	Shoulder		23		Feb 6
F-24	Slab	17		Feb 6	
HRB A-4	Shoulder		15.5		Feb 6

of severity from the viewpoint of resulting extensive frost damage to pavements, had a cold quantity of 528 degree-days (Figure 16). The winter of 1954-1955 may be considered medium severe.

During the entire month of January and first few days of February the mean daily air temperature remained below 32 deg F. The maximum daily air temperature remained below freezing from January 27 to January 31 and February 2 to February 3rd. Thirteen degrees below zero F, the lowest air temperature of the winter, was recorded on February 3rd. This entire period was characterized by an abnormally low amount of precipitation. January's precipitation was the lowest ever recorded by the weather bureau for that month in New Jersey.

Heavy rainfall on February 6 and 7 was accompanied by rising air temperatures. After more rain on February 11th the soils at the surfaces were thawed and appeared to be completely saturated. Air temperatures fell rapidly on February 12th, the maximum for that day and the following day remaining below freezing.

A general period of thaw began on February 17, the mean air temperature remaining above freezing until March 5th. During several days of this period the minimum air temperature was above 32 deg F. The final brief cold spell of the winter occurred between March 5 and 8th.

Ground-Water

An examination of precipitation and ground-water data shows that the primary factor controlling the elevation of the ground-water table is the presence or lack of precipitation. Each period of precipitation produced an immediate rise in ground-water elevation. A decided difference was noted between the effects of precipitation occuring as rain or as snow. The amounts of precipitation during all of the precipitation periods for the winter, when compared with resulting rises in ground-water elevation, indicated the following:

The average rise of ground-water elevation per tenth of an inch of rain was 1.7 inches A tenth of an inch of precipitation occuring as snow, however, produced an average ground-water rise of 6 inches. Apparently the melting of the snow allowed considerably more percolation and less runoff than would have resulted from an equal amount of rain. This statement, of course, does not consider the complex hydrologic conditions involved.

Slight rises in ground-water elevation were also noted during periods of thawing without the presence of precipitation. Precipitation, during periods when frost was in the

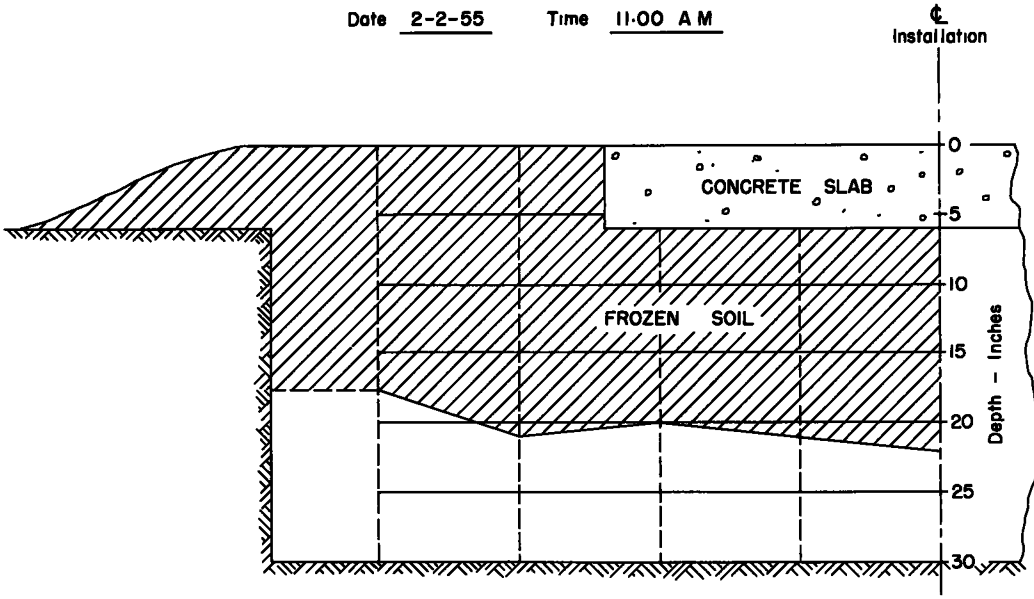

Figure 17. Profile of frost penetration in soil F-10, HRB A-1-b.

soil, still raised the ground-water elevation. Apparently the presence of frost did not prevent all percolation through the relatively porous structure of the surrounding soil at the field installation. It should be noted that the ground-water under consideration occurs in joints and fractured zones of shale.

The outstanding feature of ground-water conditions was the steady decline of elevation during the dry period of January and early February. By the end of this period the ground-water table had reached a depth of 69 inches below the ground surface, its maximum for the winter. A minimum depth of 14 inches was recorded following heavy rain in early March. Except during January the general depth to the ground-water table fluctuated between 20 and 30 inches.

During the winter the ground-water temperature fluctuated between 45 deg F and 53 deg F. In general, a steady decline occurred throughout the winter as a result of the cooling effect of freezing air temperatures. Precipitation produced noticeable variations in ground-water temperature, depending upon whether the temperature of the percolating water was higher or lower than that of the ground-water. Usually after rain or melting snow the ground-water temperature was lowered. An occasional increase was noted, however.

Frost Penetration

Observations indicated that prior to the completion of the soil installations there had been some frost in the soil. As a complete thaw was noted by December 29, 1954, this frost was not considered for evaluation.

The reaction of all of the soils was generally characterized by frost entering the soil early in January and remaining until late in February. Frost penetration proceeded at a reasonably steady rate in most of the soils during the extended cold period of January and early February. Superficial thawing was initiated by rising temperature and rain on February 7th. Varying amounts of frost remained continuously in all of the soils throughout several more freeze-thaw cycles until complete thawing in late February ended this major period of frost. A minor period of frost penetration was noted for all of the soils in early March. Frost occurred primarily in the soil shoulders at this time. The lengths of time of the major frost period for each soil are presented in Table 3.

It is noted that frost penetration occurred first in the soil shoulders. Soils F-10 and F-17, the granular materials, were the first to freeze both in the shoulders and beneath the concrete slabs. Complete thawing occurred first beneath the slabs. Again soils

TABLE 5

FROST PENETRATION DURING PERIOD FROM JANUARY 27 TO JANUARY 31

Soil	Position	Depth of Frost Penetration (Inches) Start of Period	End of Period	Amount of Penetration During Period Inches
F-5	Slab	14	18	4
HRB A-5	Shoulder	15	18.5	3.5
F-10	Slab	14	22	8
HRB A-1-b	Shoulder	13	21.5	8.5
F-13	Slab	8.5	13.8	5.3
HRB A-2-4	Shoulder	13.2	17.3	4.1
F-15	Slab	11	14	3
HRB A-7-6	Shoulder	13	16.5	3.5
F-17	Slab	17	25	8
HRB A-3	Shoulder	12	20.4	8.4
F-24	Slab	8.6	13.4	4.8
HRB A-4	Shoulder	8	13.4	5.4

F-10 and F-17 were the first to react. In each of the soils, frost was present in the shoulders longer than beneath the slabs. For soils F-10 and F-17 the difference between the lengths of these periods was considerably less than for the other soils.

The maximum depths of frost penetration for each of the soils are listed in Table 4. These were measured from the upper surfaces of the concrete slabs, which were flush with the soil shoulders.

It is noted that the maximum depth of frost penetration in all cases occurred just prior to the February 7 thaw, several days after the date of minimum air temperature. With the exception of F-13, all soils showed an equal or greater depth of frost penetration beneath the concrete slabs than in the shoulders, indicating that the thermal conductivity of a slab is greater than that of an equal thickness of soil. This may also be illustrated by an example of a frost profile determined by means of the additional thermistors installed in soil F-10 (Fig. 17).

The greatest depth of frost penetration occurred in the granular soils, F-17 and F-10, indicating a greater thermal conductivity than that of the finer-grained soils.

During the five-day period of continuous subfreezing temperatures from January 27 to January 31 all of the soils presented steady rates of frost penetration. The amounts of penetration during this period are listed in Table 5.

It can be observed from the frost penetration charts (Appendix A) that thawing occurs at both the top and bottom of the frozen soil layer.

As a means of comparing rates of thawing for the soils, depths of thaw from the surface to the remaining frozen soil on February 11 are listed in Table 6. Thawing on this date was the result of rising air temperature and rain. The frozen soils thawed primarily from the surface.

The rates of frost penetration and thaw-

TABLE 6

DEPTH OF THAWING INTO FROZEN SOIL ON FEBRUARY 11th

Soil	Depth of Thaw Beneath Slab	Beneath Shoulder
F-5 HRB A-5	9 5	5 5
F-10 HRB A-1-b	13 5	9 5
F-13 HRB A-2 4	11	6
F-15 HRB A-7-6	--[a]	5 5
F-17 HRB A-3	14 5	13.5
F-24 HRB A-4	8 5	6

[a] Thawing did not proceed beneath the base of the slab

ing presented in Tables 5 and 6 further demonstrated the higher thermal conductivity of the granular soils. Of particular note is the observation that thawing in the soils proceeds more rapidly beneath the concrete slabs than beneath the shoulders.

Frost Heaving

For each soil the difference between its initial slab elevation and its maximum slab elevation reached during the winter was considered the maximum frost heave. These heaves are listed in Table 7. For convenience some frost penetration data are repeated.

The granular soils F-17 and F-10 experienced little heaving. The slight fluctuations of elevation of soil F-17 as shown by the frost heave charts (Appendix A) are insignificant. Soil F-10 also experienced insignificant heaving during the cold but dry period of January and early February. Slight heaving occurred during the cold spell starting on February 11th after heavy rains had raised the water table and moisture content of the soil.

Soils F-5, F-13 and F-15 experienced relatively moderate heaves. The rate of heave of each of these soils decreased steadily toward the end of January, whereupon the soils maintained a reasonably constant elevation until the thaws on February 10th and 11th. Rapid heaving was then noted in each of these soils during the cold spell starting on February 11th.

Soil F-24, a silty material, heaved the most of all 30 soils being studied for this purpose. It showed a relatively large and steady rate of heave until the February 7 thaw. The heaving which recurred on February 11 was of minor consequence when compared with the over-all reaction of this soil.

It is noted that maximum frost heave does not necessarily occur simultaneously with maximum depth of frost penetration. Undoubtedly the effect of increased moisture content following the early February rains caused some of the soils to experience maximum heaves at that time rather than at the time at which they were frozen to the greatest depth. Had the cold spell following these rains been of longer duration it is possible that the other soils might have experienced greater heaves at this time.

Soil Moisture Contents

In most instances operation of the fiberglas soil moisture units was considered to be satisfactory. It should be expected that, when present, the variance of soil moisture content with depth should be reasonably uniform, assuming, of course, that the soil is homogenous. An examination of the moisture content curve (Appendix A) shows in some instances erratic fluctuations of soil moisture content with depth rather than steady variations. It is suspected that this is a result of the placement of the soil moisture units in the soil. It is difficult to restore each unit to exactly the same conditions of soil contact and soil density under which it was calibrated. Soil moisture units placed in soil

TABLE 7

MAXIMUM FROST HEAVES

Soil	Max Frost Heave	Date of Max Heave	Date of Max Frost Penetration	Frost Heave at Time of Max Frost Penetration	Max Frost Penetration	Frost Penetration at Time of Max Heave
	feet			feet	inches	inches
F-5 HRB A-5	041	Feb 16	Feb 6	034	22	18
F-10 HRB A-1-b	016	Feb 14 15	Feb 6	002	26	17
F-13 HRB A-2-4	035	Feb 14 15, 16	Feb 5	027	17	12
F-15 HRB A-7-6	045	Feb 6	Feb 6	045	22	22
F-17 HRB A-3	004	Feb 4, 5	Feb 5, 6, 7	004	29	29
F-24 HRB A-4	144	Feb 6	Feb 6	144	17	17

F-17 (HRB A-3) seemed to be the worst offenders, probably because of the relatively small proportion of fines in this soil to afford good contact with the fiberglas element.

In general, it is estimated that the soil moisture units as used in this study have indicated soil moisture contents to an approximate accuracy of several percent.

When a soil moisture unit freezes its resistance increases sharply. An examination of moisture unit resistance and soil temperature data showed that the actual freezing temperatures of the soils were slightly lower than 32 deg F. Measurements were not taken at sufficiently close time intervals, however, for an accurate determination of their freezing points.

As the soil moisture units could not indicate the moisture content of frozen soil each moisture content curve is discontinuous during the periods when the soil at the corresponding depth was frozen.

In both soil F-10 and soil F-15 the shoulder moisture contents were approximately the same as the moisture contents beneath the slabs. In soil F-5 the slab moisture contents were significantly higher than the shoulder moisture contents. Soil F-13 showed slightly higher slab moisture contents than shoulder contents. In soils F-17 and F-24 shoulder moisture contents were slightly higher than slab moisture contents.

Four of the six soils showed increases of moisture content with depth. The approximate ranges are as follows:

Soil	Approximate Moisture Content Range	
F-5	33-34 percent beneath slab	25-20 percent beneath shoulder
F-10	8-12	
F-13	17-22	
F-15	27-30	

The moisture content of soil F-17 varied little with depth except in the proximity of the ground-water table when present. Normal approximate moisture contents were 3 to 5 percent. Moisture content of this soil at its lower depths when affected by the ground-water table was 16 percent.

Soil F-24 presented a rather uniform moisture content with depth, ranging from 21 to 23 percent.

During the dry period of January and early February moisture contents of all of the soils were reduced, the rate of reduction increasing toward the end of the period. In the early part of the period the greatest reduction in moisture content appeared near the soil surfaces in soils F-5, F-13, F-15 and F-24. Of course, it was possible to follow the moisture content trend in the upper soils only until they were frozen. F-10 and F-17, having relatively lower moisture contents than the other soils, showed less reduction of moisture content near the surface. Toward the end of the dry period the soil moisture contents at greater depths showed reduction. Of the finer-grained materials, F-13 showed the greatest reduction, approximately 5 percent. The moisture content of F-10 was reduced a relatively small amount. The moisture content at greater depths of F-17, however, experienced a sharp reduction of over 10 percent as the effect of the ground-water table was removed.

In general, immediately after the early February rains all of the measurable soil moisture contents returned to their approximate initial values. As thawing proceeded the remaining moisture contents were found to be at or slightly above initial values. A possible increase of soil moisture content as a result of thawing was masked by the simultaneous occurrence of rain.

All of the soils showed some fluctuation of moisture content resulting from precipitation. This effect was most noted near the surface. The sharp increase of moisture content for all soils at the beginning of the period of study is believed more a result of the replaced soil and soil moisture units reaching an equilibrium than a result of precipitation. The moisture content of soil F-17 showed the greatest sensitivity to precipitation. Considerable increases in moisture contents were apparent after rains. By means of the moisture content curves it was possible to trace this increase with depth as percolation resulted. Rapid decreases of moisture content shortly after rain show the free-draining properties of this material.

CONCLUSIONS

1. The Fiberglas soil moisture units as used in this study indicated soil moisture contents to an apparent accuracy of several percent. Soil moisture units placed in coarse-grained soils presented the most erratic performance.

2. Thermistors are reliable and convenient for measuring subsoil temperatures. They can be easily calibrated to an accuracy of 0.1 deg F.

3. The winter of 1954-1955 may be regarded as medium severe for New Jersey on a comparative basis of temperature. Coincident with the coldest part of the winter, however, was an extended period of abnormally low precipitation.

4. The elevation of the ground-water table shows a direct relationship to precipitation. Percolation from melting snow may be greater than from an equivalent amount of rain. Even when frost has penetrated to a considerable depth percolation may still be possible, probably as a result of porous soil structure. The ground-water temperature is controlled primarily by the effect of air temperature. Immediate minor fluctuations may result from percolation.

5. The greatest depth and rate of frost penetration occurs in the granular soils as a result of their relatively high thermal conductivity as compared with that of fine-grained soils. Because the thermal conductivity of a concrete slab is greater than that of an equal depth of soil, frost usually penetrates deeper beneath the slabs than through the shoulders. For the same reason, the soils thaw faster beneath the slabs. As a frozen layer of soil thaws from both top and bottom, this is of particular importance because the greater rate of thawing beneath pavement slabs may produce "pockets" of thawed soil at high moisture content from which the water cannot drain. Consequently, the soil bearing capacity is reduced because of the saturated state of the soil. This observation implies a drainage problem concerning base and subbase courses to be solved with particular reference to drainage from underneath the road pavement of water produced by the melting of frozen soil moisture.

6. The greatest depth of frost penetration does not necessarily occur at the same time as minimum air temperature.

7. The silty A-4 soils are most susceptible to frost heaving. The granular A-1-b and A-3 soils heave the least.

8. Maximum frost heaving does not necessarily occur simultaneously with the maximum depth of frost penetration.

9. Frost heaving is dependent upon soil moisture content resulting from precipitation or proximity to ground-water as well as upon the amount of frost penetration resulting from freezing temperatures.

10. Soil freezes at temperatures slightly lower than 32 deg F.

11. Fine-grained soils normally retain higher moisture contents than coarse-grained soils. Moisture contents usually increase with depth in a soil. The effect of increased soil moisture content resulting from precipitation and percolation is modified with depth.

12. There is a great need for the investigation of thermal properties of New Jersey soils.

13. There is also a great need for studying moisture migration in soils upon freezing.

ACKNOWLEDGMENT

The authors wish to express their appreciation to the New Jersey State Highway Department for having initiated the project which provided the material for this report, to the Joint Highway Research Committee and, in particular, to Chairman Allen C. Ely for their interest and advice, and to Dr. E.C. Easton, Dean of the College of Engineering, Rutgers University, for his support of a project of such potential value to the state as a whole.

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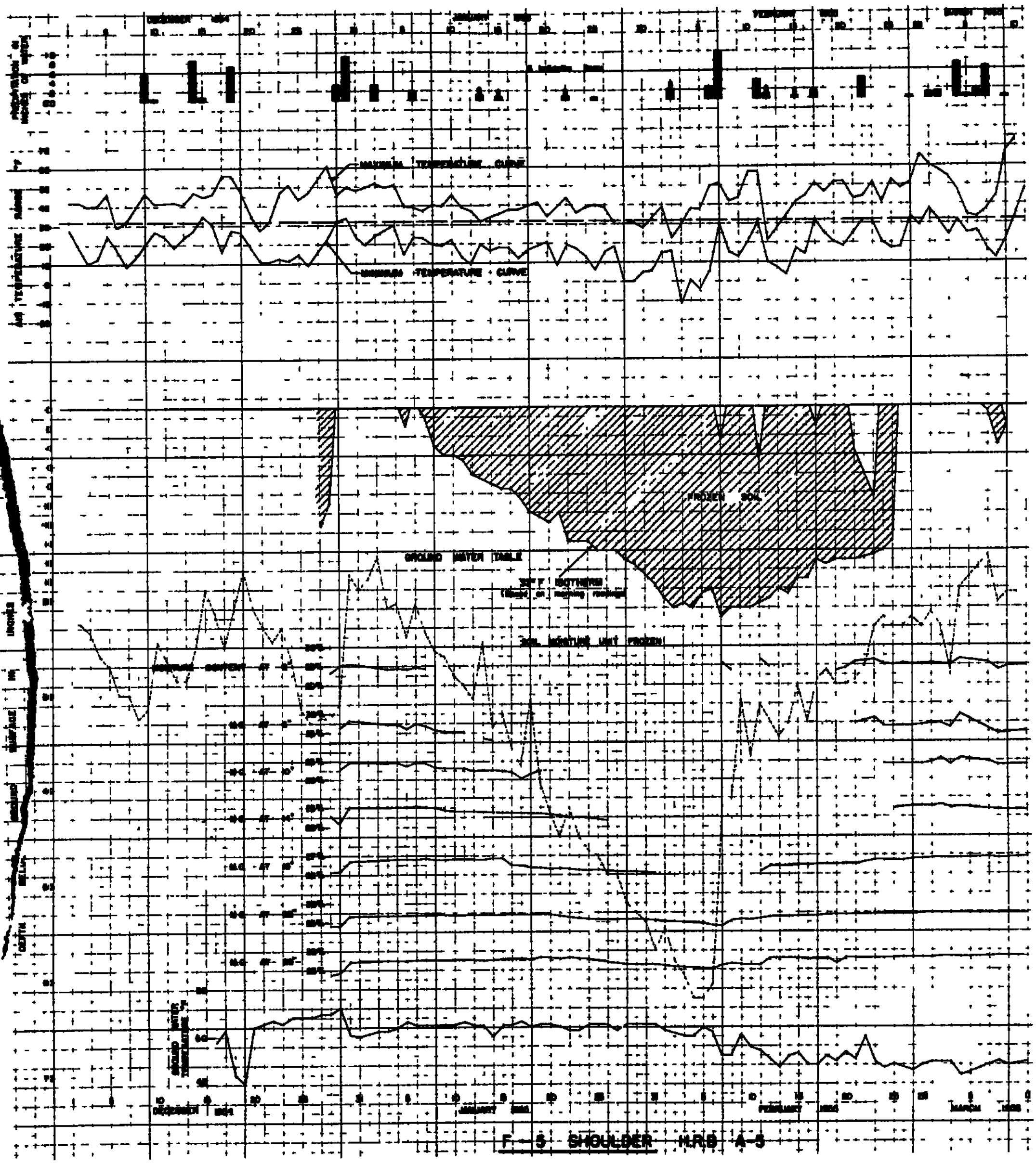
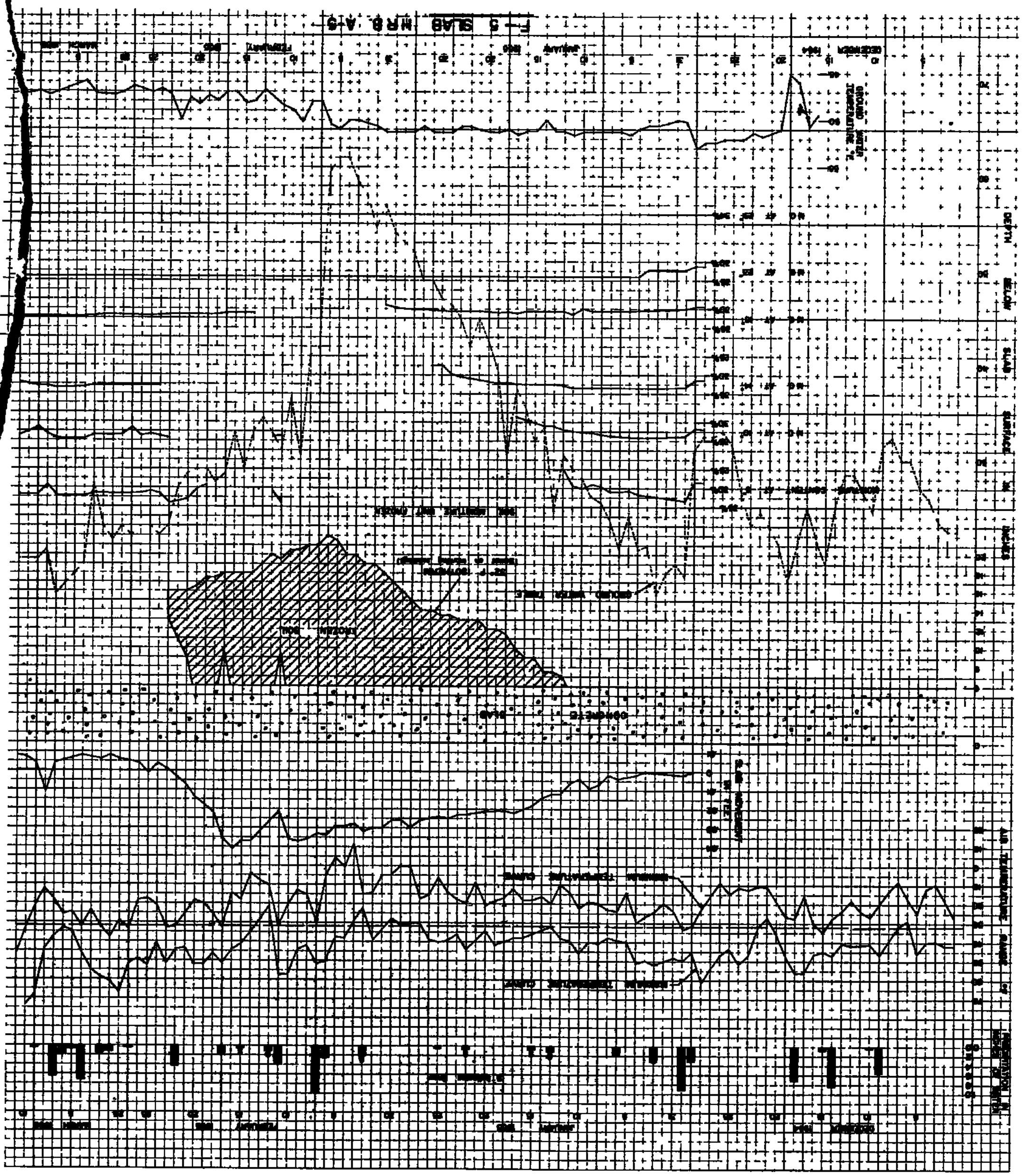
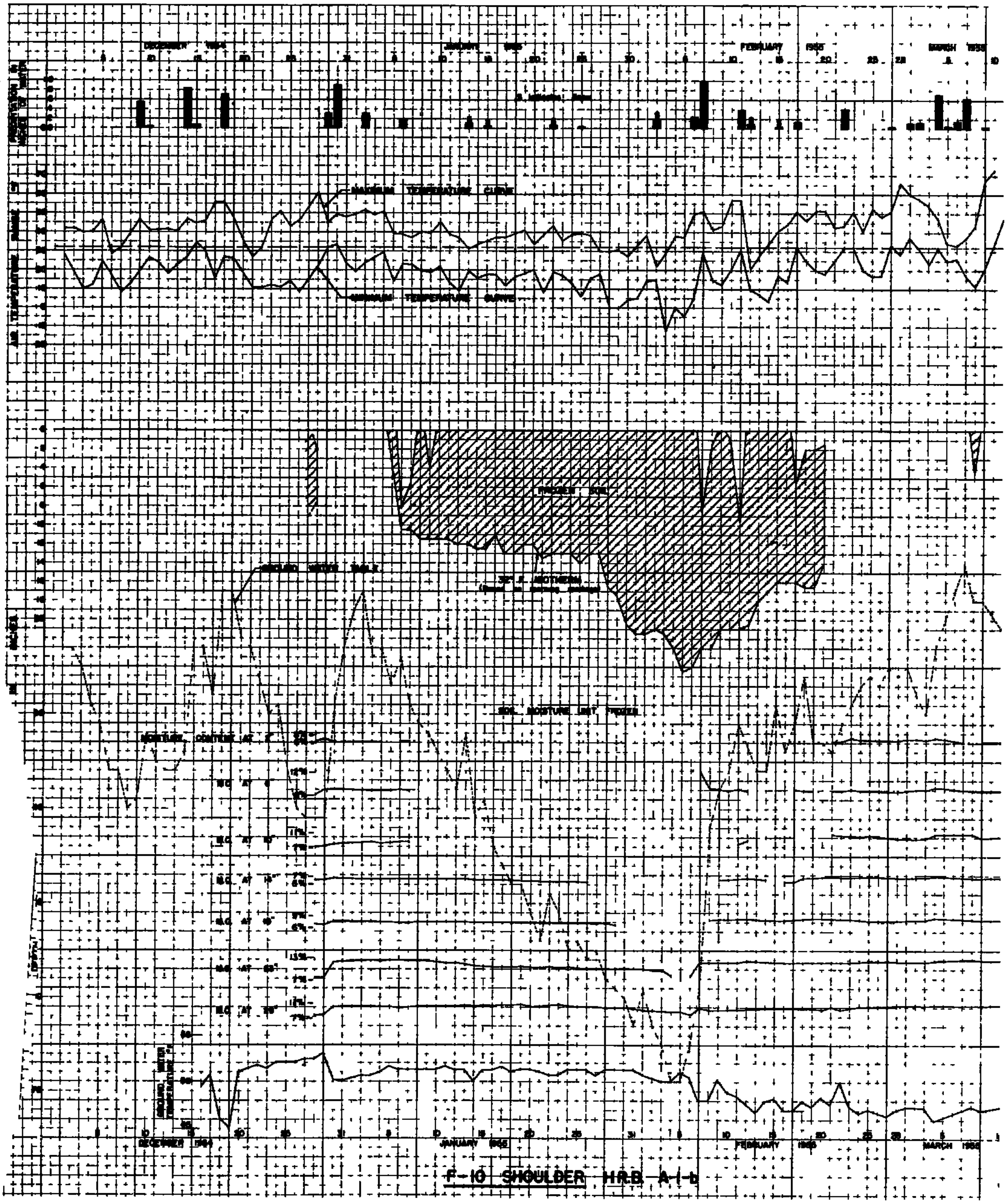
Appendix A

Curves showing precipitation, air temperature range, slab displacement (heave), frost penetration, soil moisture content, ground-water level and ground-water temperature for six New Jersey soils exposed to natural freeze-thaw conditions during the winter of 1954-1955.

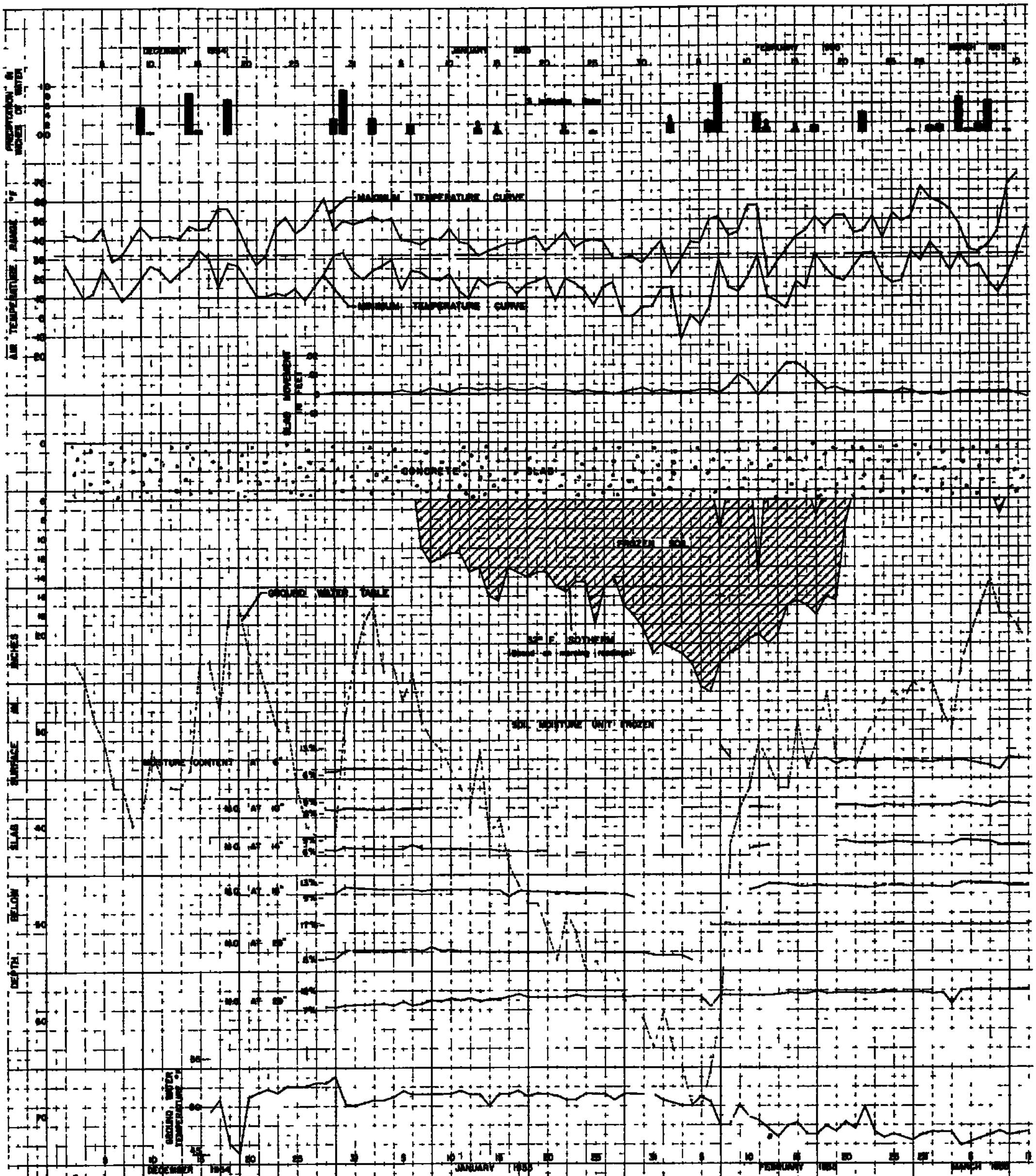




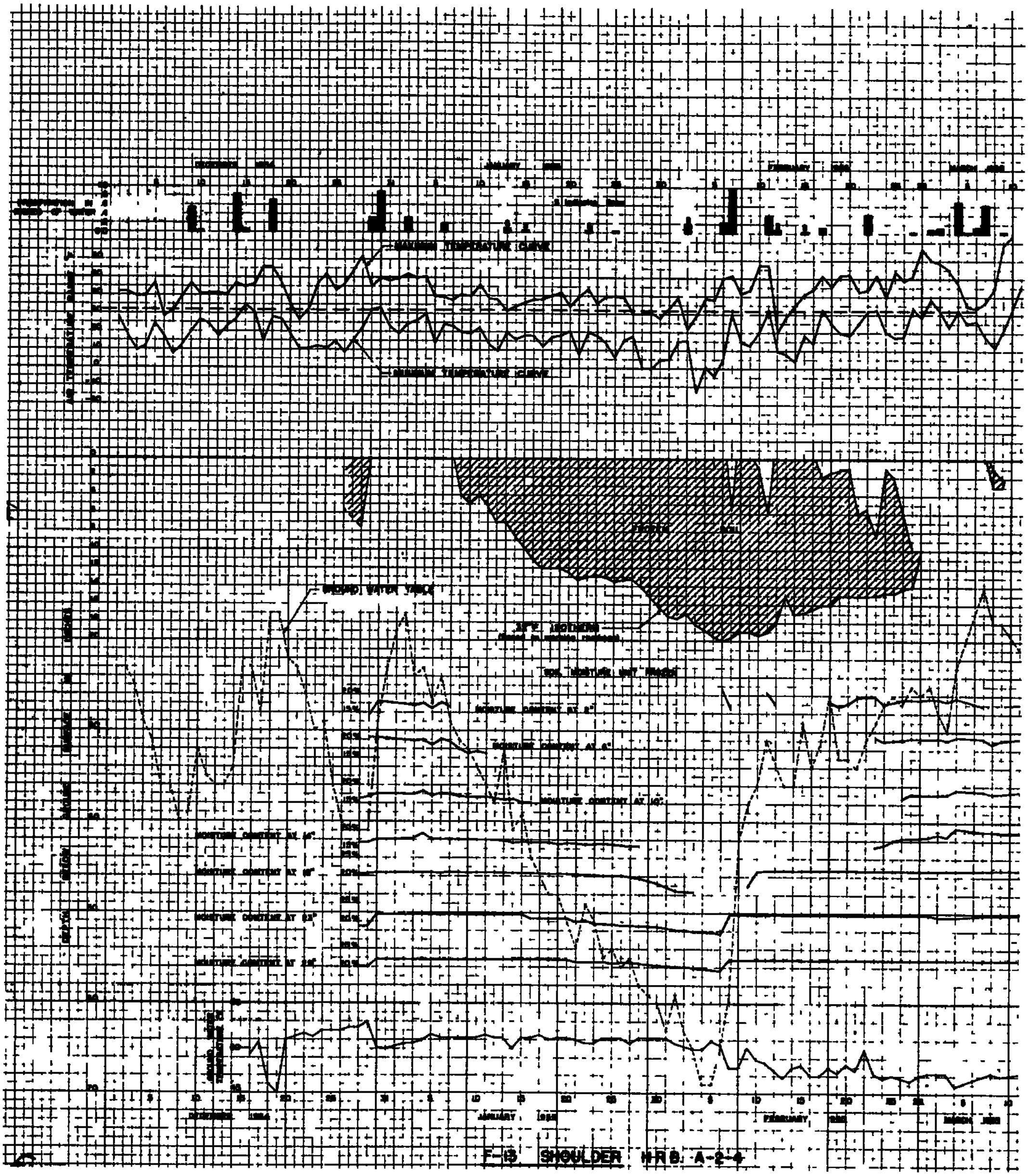
F-5 SLAB NRB A-5
DECEMBER 1964
JANUARY 1965
FEBRUARY 1965
MARCH 1965
PRECIPITATION IN INCHES OF WATER
AIR TEMPERATURE RANGE °F
MAXIMUM TEMPERATURE CURVE
MINIMUM TEMPERATURE CURVE
SLAB MOVEMENT IN FEET
CONCRETE SLAB
FROZEN SOIL
GROUND WATER TABLE
SOIL MOISTURE NOT FROZEN
MOISTURE CONTENT
DEPTH BELOW SLAB SURFACE IN INCHES
GROUND WATER TEMPERATURE °F



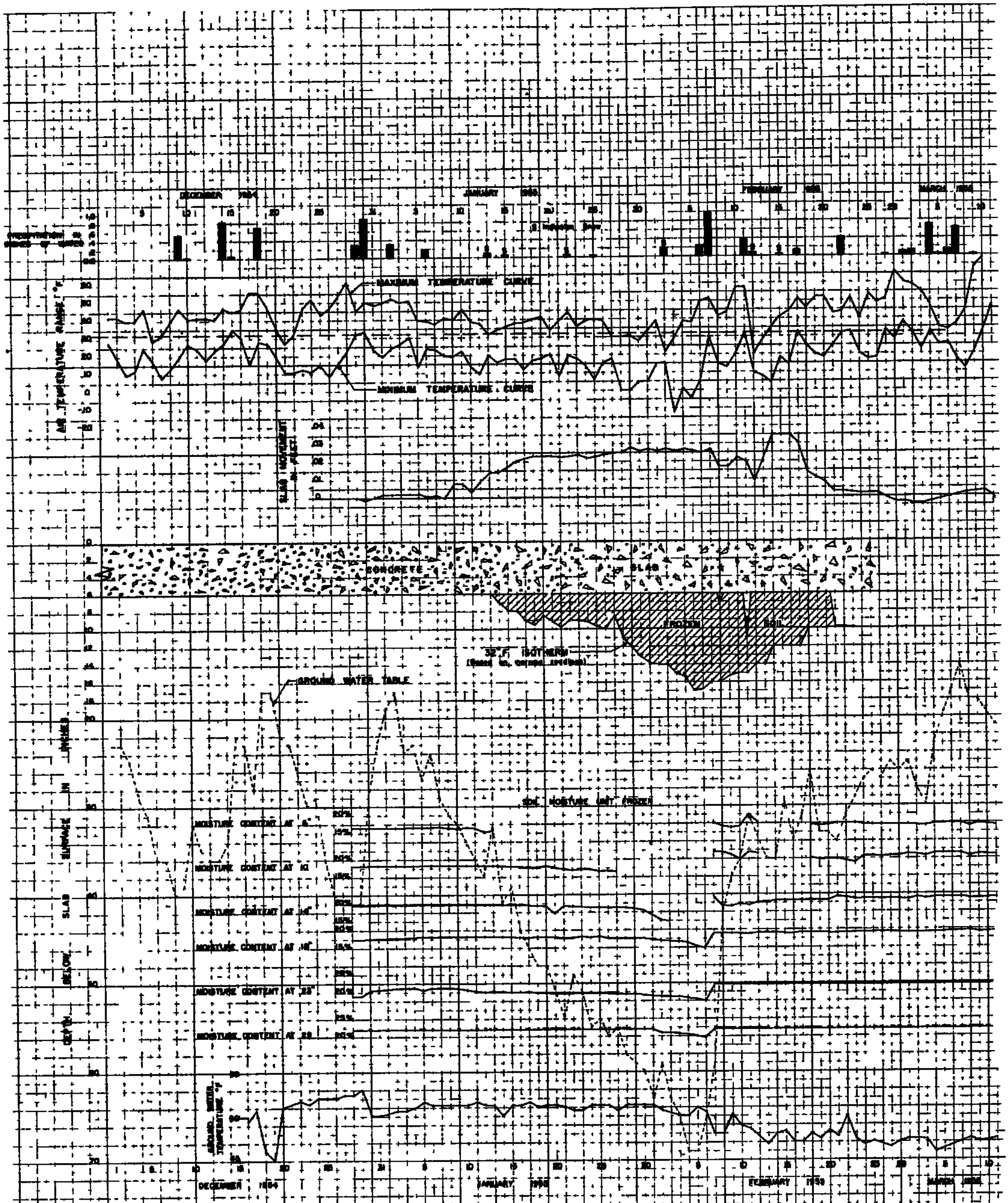
F-10 SHOULDER H.R.B. A-1-b



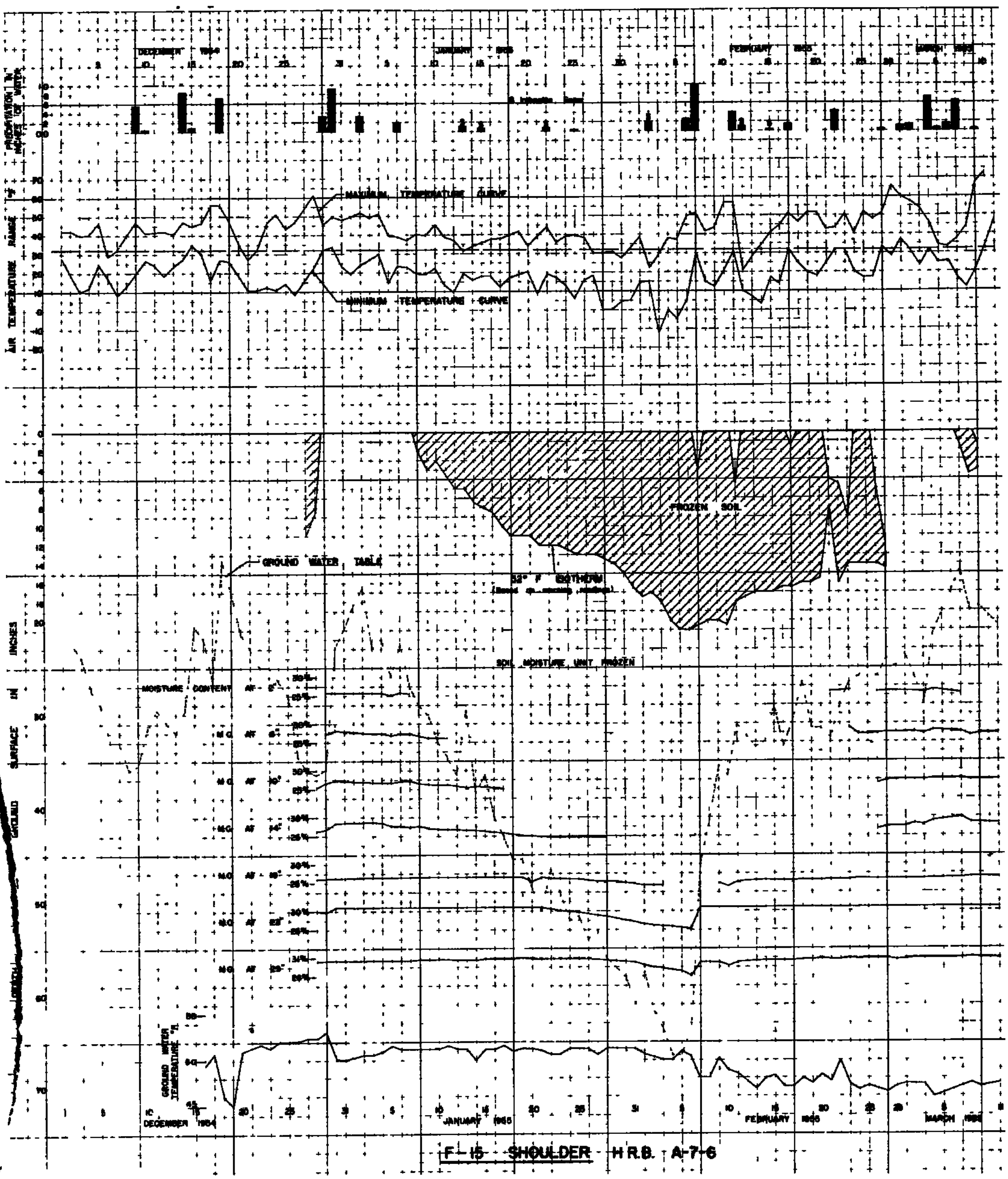
F-10 SLAB HRB A-1-b



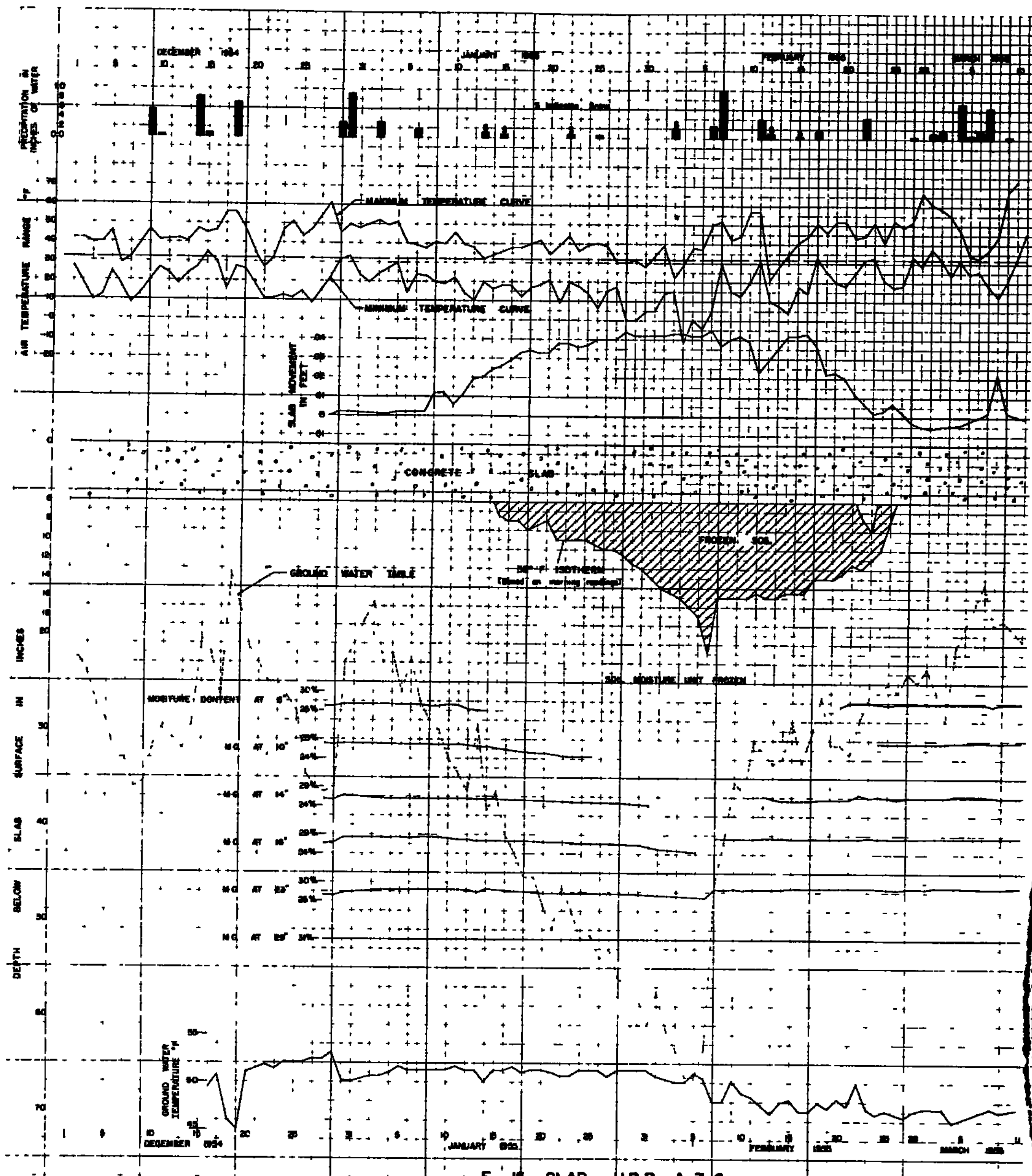
F-13 SHOULDER H.R.B. A-2-4



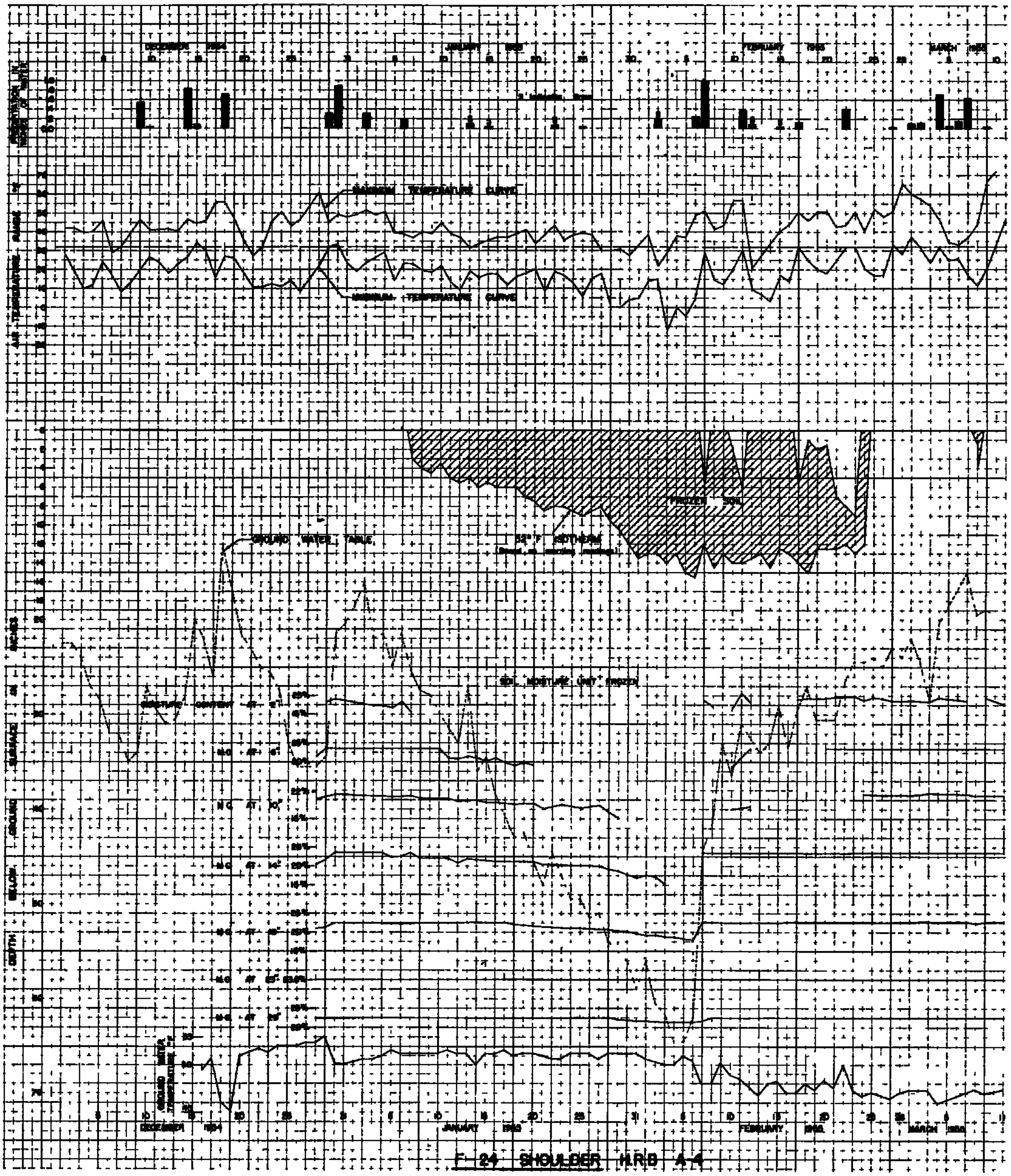
F-15 SLAB HRB A-2-4



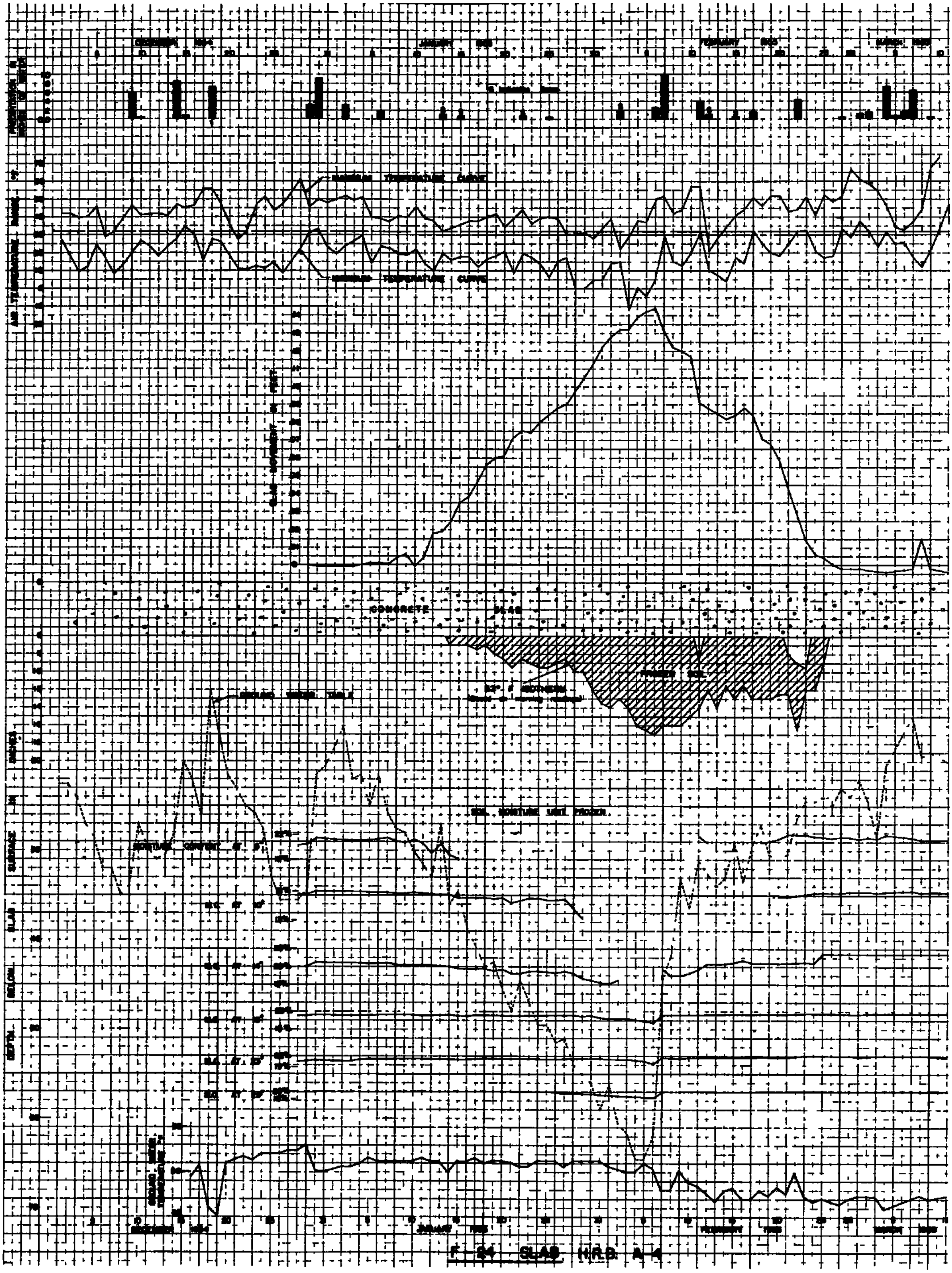
F-15 SHOULDER H.R.B. A-7-6



F-15 SLAB H.R.B. A-7-6



F-24 SHOULDER H.R.B. A-4


CONCRETE SLAB
F 24 SLAB H.R.B. A-4

Soil Moisture Movement During Ice Segregation

EDWARD PENNER, Assistant Research Officer
Division of Building Research, National Research Council of Canada

This paper reports the initial results of a continuing research study of frost action processes in soil undertaken recently by the Division of Building Research of the National Research Council of Canada.

Moisture flow to the freezing zone of small soil specimens has been measured under controlled laboratory conditions. The experimental findings agree with the now generally accepted concept of frost heaving outlined by Taber and Beskow.

In soil systems where a free water surface exists near but below the freezing zone, the results indicate that moisture flow is dependent on the unsaturated permeability and soil moisture tension characteristics. Both these soil properties integrate the effect of grain size structure, clay composition, and exchange ions among others.

Critical desaturation beneath the frost line in some soils appears to act as a barrier to liquid moisture transmission. Since in the heavier-textured soils, rapid moisture transmission continues to much higher tension values, greater heave rates are observed.

●SOME soils when subjected to naturally occurring freezing temperatures display certain features undesirable from an engineering standpoint. Uniform and differential heaving due to ice segregation and loss of strength upon thawing are perhaps the most notable. This behavior often results in the deterioration of road and airport surfaces, destructive action on railroad grades, and bad effects upon building foundations.

Much of the knowledge about frost action processes stems from the early work of Taber (1) and Beskow (2). More recently Haley et al (3), Ruckli (4) and Jumikis (5, 6) have made notable contributions. In his review of literature, Johnson (7) has drawn attention to the mass of useful information in allied fields, particularly soil science, available to the student of frost action.

The general concept of the frost action phenomenon in soils appears to be well established and is available in the literature. To a large extent frost action criteria now used in engineering have developed from this general concept. In view of this, it appears that if any improvement in frost action criteria is to be expected, more detailed information on the relationship between the physico-chemical properties of the soil and destructive frost action is required.

As part of a long-term study of frost action in soils by the Division of Building Research of the National Research Council of Canada, some initial laboratory studies are concerned with the nature of the moisture flow to the freezing zone. Soil moisture potentials and subsequent moisture flow are known to result from the phase change of soil moisture during ice segregation. In nature, this process is complicated by the heterogeneity of the soil, the complex heat flow pattern, and the unknown status of the water supply. Except in its simplest form, it does not lend itself to easy simulation in the laboratory.

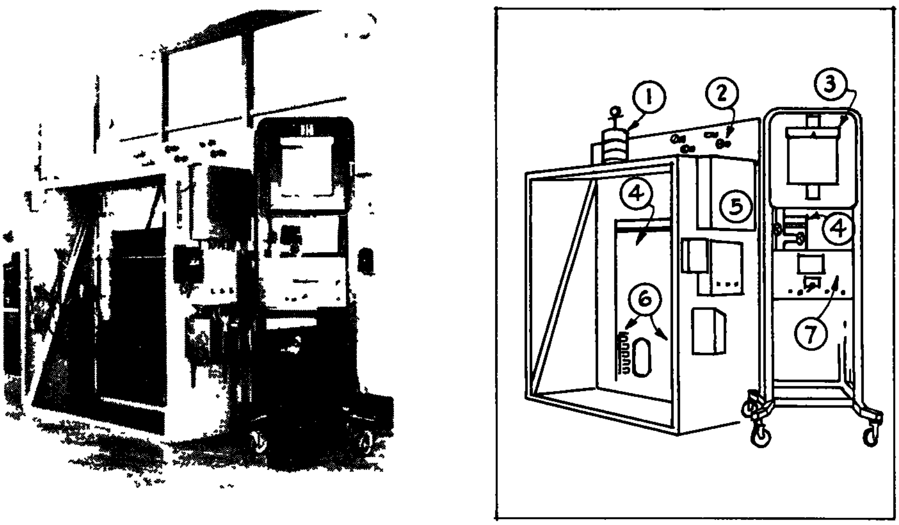
The amount of moisture available for ice segregation may be "limited" and this is commonly referred to in the literature as a "closed" system. In the "open" system water is supplied continuously from a shallow water table to the freezing zone by the mechanism of "unsaturated" liquid flow. It is with the latter condition, that is, the open system, under which extremely serious heaving may occur, that this paper is concerned.

The term "unsaturated" is defined in a number of ways in the literature. As used here it does not refer to the completeness with which the soil pores are filled with water but rather denotes that a state of soil water tension exists. At "saturation" the tension is zero, although in the case of light textured soils the soil pores may be partially filled with air. Heavy clay soils may remain completely filled with water, even when subjected to relatively high tensions, but are accordingly "unsaturated". To avoid any ambiguity, the degree to which the soil pores are filled with air may be expressed in terms of air-void volume (8). Although "saturation" is not used here in the usual engineering sense it is necessary to base the definition on conditions of soil water tension in order to explain water movements by energy concepts.

METHODS AND MATERIALS

Description of Frost Action Apparatus

The apparatus shown in Figure 1 consists of two insulated metal tanks which are independently mounted on portable frames. Each tank contains approximately 25 gallons of ethylene-glycol water mixture which is pumped at a constant rate to the headers on the frost cell frame. A series of adjustable valves on the headers permits the conditioning liquid to serve four independent frost cells. The rate of liquid flow through the



(1) FROST CELL
(2) HEADERS
(3) MILLIVOLT RECORDER
(4) CONSTANT TEMPERATURE TANK
(5) CONTROL PANEL
(6) CONDENSING UNIT
(7) PREAMPLIFIER

Figure 1. Frost cell apparatus.

headers far exceeds the withdrawal for the frost cells. Thus additional frost cells may be mounted without greatly affecting the flow through others in which tests are already in progress. In addition, the rapid flow acts as a self-stirrer in the liquid bath. A separate condensing unit mounted beneath the tank is capable of keeping the liquid mix below the operating temperature. The desired liquid temperature in the tanks is achieved by heating the chilled liquid. The amount of reheat required (a function of the ambient temperature and the number of frost cells withdrawing conditioned liquid from the system) is controlled by an electronic temperature controller. The main thermostat, consisting of two 500-ohm sensing elements, is placed in the flow path at the outlet from the tank. One 500-ohm thermostat is mounted after the heater to prevent overshooting of the operating temperature. Any unbalance in the electronic controller due to a temperature change of the liquid causes the operation of a modutrol motor which is attached to a variac for control of the electrical current supply to the immersion heater. With this arrangement it has been possible to maintain the desired temperature within ± 0.10 deg C for long periods. For short periods of twelve hours the temperatures can easily be maintained within ± 0.05 deg C.

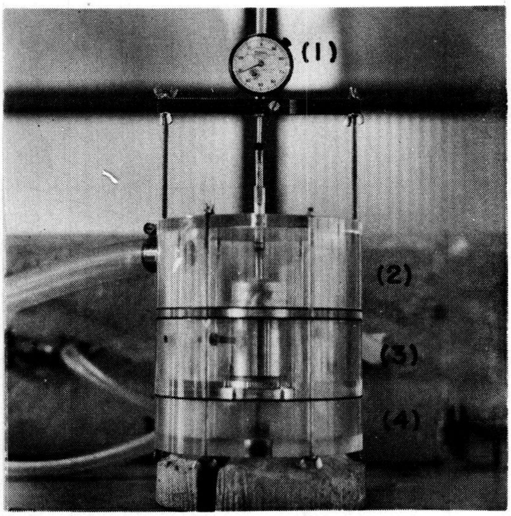


Figure 2. Frost cell in operation, (1) extensometer, (2) compartment 1, (3) compartment 2, (4) compartment 3.

The frost cell shown in Figure 2 and diagrammatically in Figure 3, which contains the soil specimen, consists essentially of three compartments. The liquid in compartment 1 conditions the upper one-third of the soil specimen and is the cold section of the specimen. Compartments 2 and 3 condition the lower two-thirds of the specimen and make up the warm section. The sample rests on a porous ceramic plate which permits control of the water tension in the specimen. A horizontal conduit through compartment 2 permits thermocouple placement in the specimen below the freezing zone. For temperature measurement in the frozen portion, thermocouples enter compartment 1 through the stem of the brass plunger. An extensometer is mounted on the cell that measures the relative movement of the plunger during ice lens growth. The frost cell is made of lucite except for the brass plate which holds the porous plate and the brass inner wall supporting the lower portion of the specimen. The inlets and outlets to the compartment are set oblique to the periphery of the cell but parallel to each other to facilitate even liquid circulation and good heat exchange.

A continuous temperature record was obtained with a specially built, high precision millivolt recorder. Copper-constantan thermocouples and ice water reference junctions were used in these experiments. The accuracy of the temperature measurements with the above arrangement was much greater than the degree of temperature control over the conditioning fluid. The frost cell was designed to give a sharp temperature gradient across the freezing zone with a constant temperature spanning the main body of the specimen. This was desirable if flow rates caused only by tension gradients were to be evaluated.

Soils and Specimen Preparation

The soils used in these experiments consisted of artificially blended mixes. Leda clay, a local marine deposit, was modified with silt obtained from natural deposits at Whitehorse and Uranium City. Leda clay and Whitehorse silt were air-dried, crushed, passed through a 200 sieve and mixed in arbitrary proportions for samples 1 and 2. Samples 3, 4 and 5 were mixtures of Leda clay (minus 200) and Uranium City silt (minus 325) pretreated in the same way. Sample 6 was Uranium City silt (minus 325).

The frost cell was designed to hold a cylindrical specimen $1\frac{5}{16}$ inch in diameter and 2.816 inches long. This is the size of a sample when molded in the Miniature Harvard Compaction Apparatus (9). Some difficulty was encountered however, with satisfactory thermocouple placement in a premolded specimen. In practice the soil specimens were compacted in the frost cell in $\frac{1}{2}$-inch layers to a density of 1.33 gm/cm^3. A thermocouple was placed between each layer. After placing the first two layers, a silicone-coated thin lucite sleeve, $1\frac{1}{4}$ inch inside diameter and 2 inches long, was lowered into

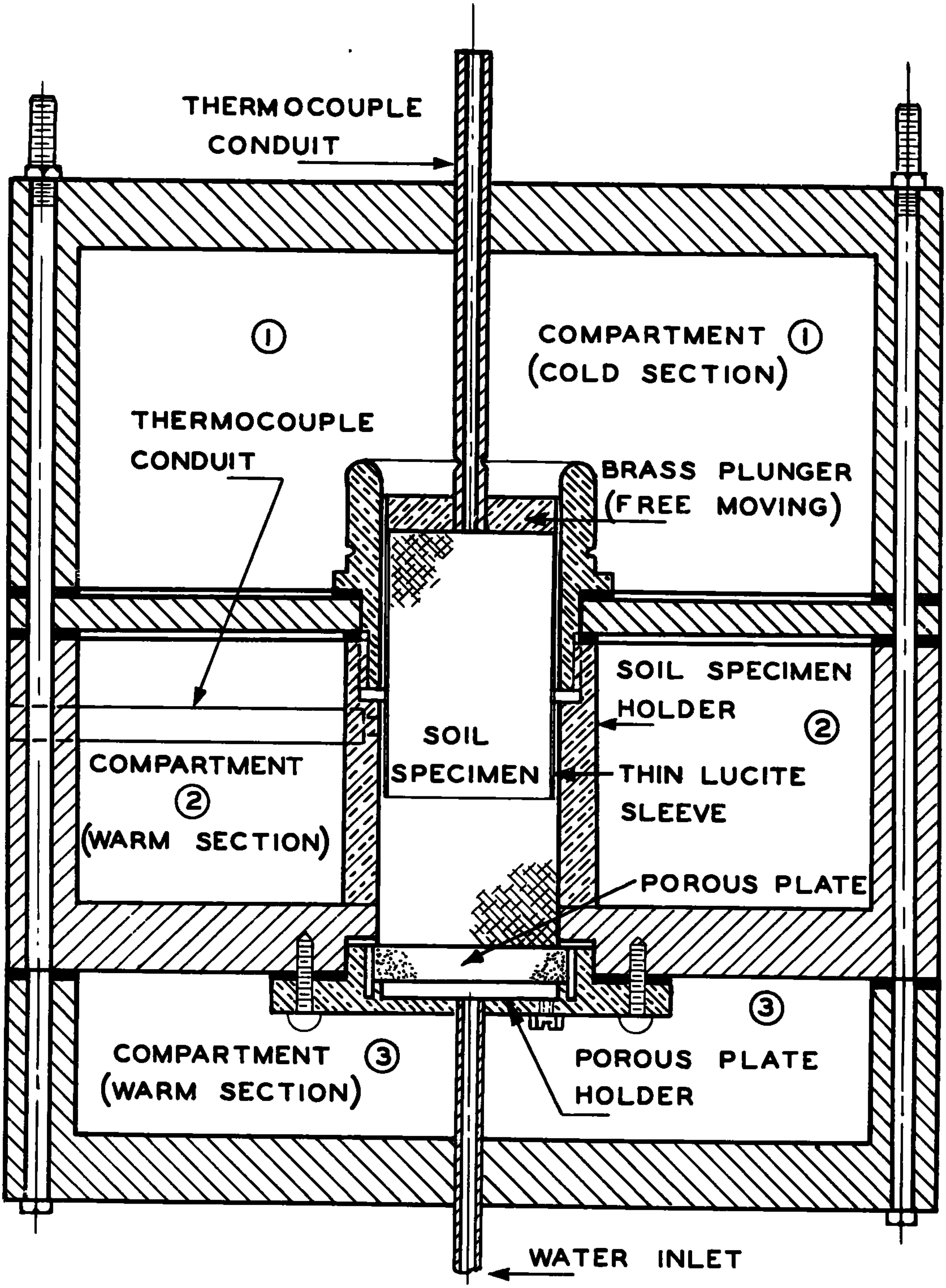


Figure 3. Section through frost cell.

the specimen holder. The soil specimen was permitted to freeze to this sleeve but heaving was restricted only by the friction between the outside walls of the sleeve and the inside wall of the specimen holder. The friction plus the load of the extensometer stem and brass plunger amounted to approximately 30 gm/cm^2 and was constant for all tests. The remaining four layers of soil were placed inside the sleeve to bring the specimen height to 3 inches.

In these experiments distilled water was allowed to enter the base of the specimen at room temperature and zero tension from a constant head device before freezing was begun. When no further water was withdrawn, it was considered that the equilibrium condition had been attained. Normally this took about twelve hours for the heavier-textured soils.

Soil Freezing Procedure

Upon completion of the saturating process, the conditioning fluid was circulated through the two compartments that condition the lower two-thirds of the specimen until an equilibrium temperature was reached. For these experiments an arbitrary temperature of approximately + 1.3 deg C was used which took some two hours to attain. Circulation of the cold liquid at a temperature of -6 deg C was then begun through compartment 1, conditioning the top inch of the 3-inch specimen. The temperature of the conditioning liquids was held constant (within the limits of the temperature control system) throughout the experiment. Supercooling was always evident but particularly in the heavier-textured soils. In the first few experiments crystallization was permitted to begin naturally but better reproducibility was attained by inducing crystallization at a fixed temperature. As stated before, a continuous temperature record made it possible to follow the progress of the freezing zone in the specimen. The amount of moisture flow into the sample and extent of heaving were measured at frequent intervals. In all the experiments reported here a free water surface was maintained level with the base of the specimen. The small hydraulic head loss caused by the porous plate at the base of the specimen in these experiments was considered to be negligible.

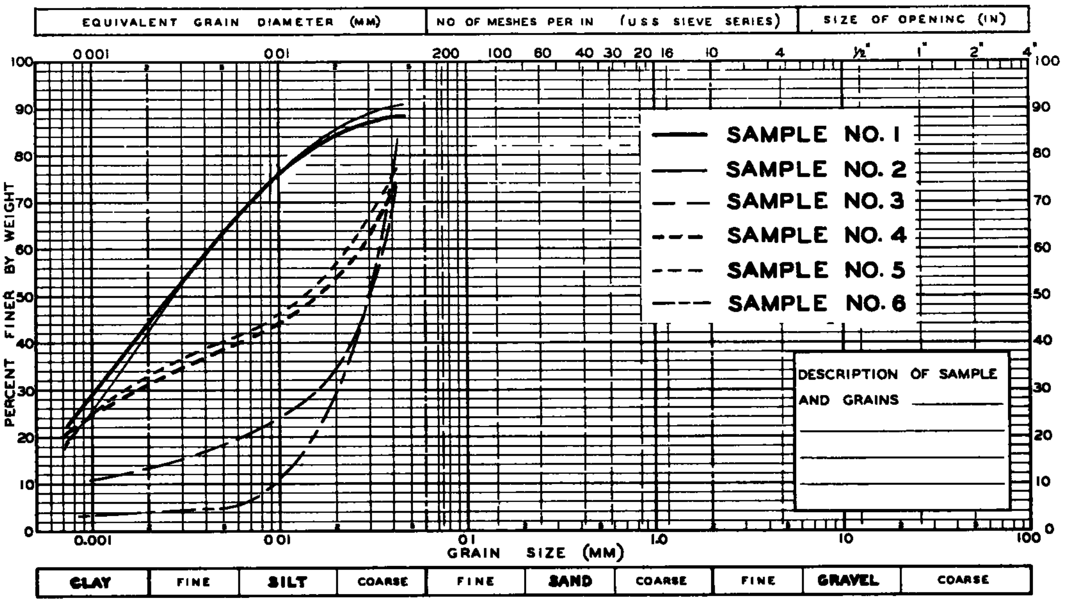


Figure 4. Mechanical analysis of soils.

Unsaturated Permeability and Moisture-Content/Tension Determinations

The general characteristics of the relationship between moisture content and moisture suction or tension for soils and their significance are well known. It has been shown in

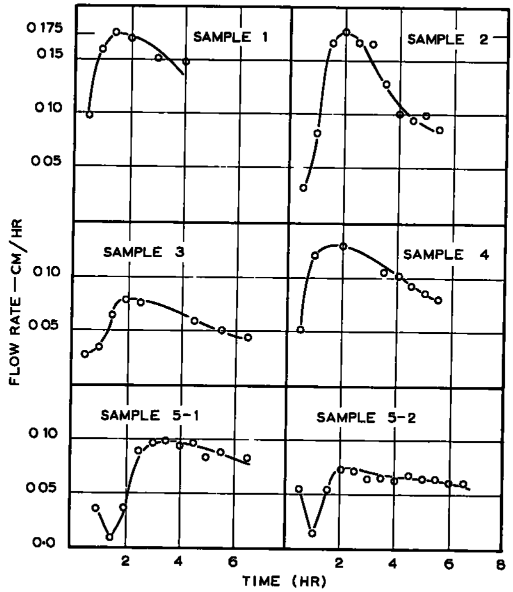


Figure 5. Rate of moisture flow induced by ice segregation as a function of time.

the past that there is hysteresis between the wetting and drying conditions, the sample being drier at a given suction when that suction is approached from the wet condition than when it is approached from the dry condition.

The freezing process, carried out according to the method described above, is essentially a drying process. Therefore it was necessary to determine only the drying moisture-content/tension relationship. This was done by the usual porous plate method. A soil is first permitted to wet against zero water tension on a porous ceramic plate. The plate is then conditioned to various tensions using air pressure. Moisture contents were determined by oven drying at 110 deg F after equilibrium conditions had been attained at the desired moisture tensions.

A modified method of the technique described by Richards (10) was used to determine the unsaturated permeability of the soil. Briefly the procedure is to create known moisture-tension gradients in a small soil specimen and to measure the flow rates under steady-state conditions. Darcy's law, v = ki, which has been shown to apply also to unsaturated flow, permits the calculation of saturated permeability coefficients k_u[1]. These are usually presented graphically as a function of either tension or moisture content.

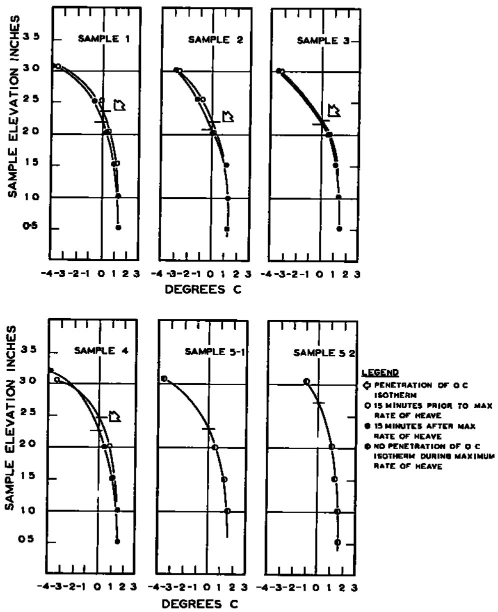


Figure 6. Temperature distribution in the soil 15 minutes prior to and 15 minutes after the maximum heave rate.

EXPERIMENTAL RESULTS AND DISCUSSION

The hydrometer analyses for the soils are shown in Figure 4. The grain-size distribution curves are fairly similar for samples 1 and 2 and for samples 4 and 5. These curves illustrate the reproducibility of sample pretreatment such as crushing, sieving, and mixing.

Figure 5 gives the rate of moisture movement into the sample versus time in hours induced by ice segregation for samples 1 to 5. No heaving occurred in sample 6. The curves all have the same general form although the starting times

[1] $k_u = v\left(\dfrac{L}{S_1 - S_2}\right)$

v = linear velocity of flow in cm/hr.
L = length of sample in cm.
S_1 and S_2 = the respective tensions at the two faces of the specimen expressed in cm of water
k_u = unsaturated permeability coefficient in cm/hr.

vary. This is largely due to the lack of control over the amount of supercooling and the initiation of crystallization. The reduction in rate of heave with time is caused by many factors, some of them not fully understood. One of these is the reduction in heat flow as the zero isotherm penetrates the soil specimen. Temperature distributions in the soil specimens that existed 15 minutes prior to and 15 minutes following the maximum heave rate are shown in Figure 6. These graphs also show the penetration (if any) of the 0 deg C isotherm during this time.

A comparison of the grain-size distribution curves with the maximum rate of heave shows that the heavy textured soils heaved at the greatest rate. In heavy soils such as sample 5 the limiting factor appears to be the rate of heat removal. Further evidence was obtained on this point when sample 5-2 was treated similarly in all respects to sample 5-1 with the exception of the temperature on the cold side. This was increased from -6 deg to -3 deg C on the sample 5-2 which reduced the maximum heaving rate from0. 1 to 0. 75 cm/hr (Figure 5).

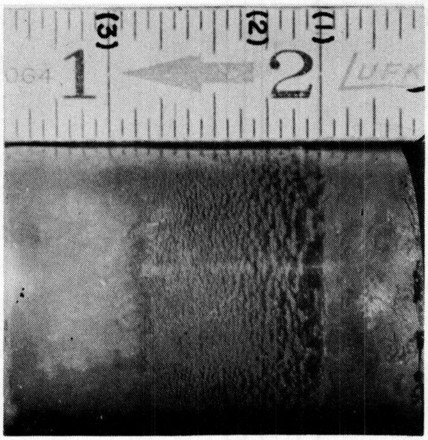


Figure 7. Sample 2, showing ice segregation beginning with no evidence of soil separation and progressing to almost continuous ice. (Total heave for 12 hour period was 1/2 in.) (1) top of sample, (2) point of maximum rate of heave, and (3) final frost line.

Examination of the frozen samples showed that for all samples no discrete ice lensing was evident at the position of the zero isotherm during the maximum heaving rate. As the frost line penetrated the sample, the separation of soil particles increased, gradually blending into solid ice. At the stage where a clear ice lens was growing, the rate did not become completely constant but progressed at a decreasing rate. The general nature of the distribution of ice is shown in Figure 7.

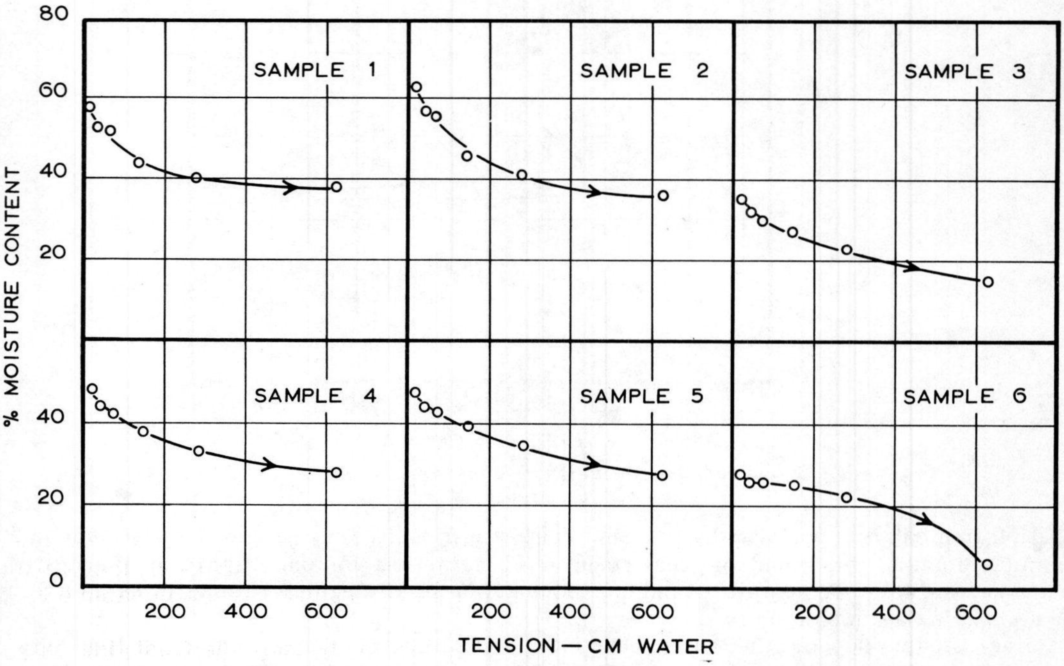


Figure 8. Relationship between moisture tension and moisture content for all samples.

The question arises why no heaving occurred in sample 6. Figure 8, which gives the moisture-content/tension curves for all soils, shows that the moisture content of sample 6 was approximately 30 percent when freezing was initiated. The unsaturated permeability for this soil, shown in Figure 9, demonstrates its ability to pass water at low tensions. In fact a comparison of the unsaturated permeability coefficients at low tensions with the flow rates in the freezing experiments shows that an inverse correlation exists for the soils studied. These experiments suggest that the crystallization of the soil moisture is able to create a sufficient tension to cause a high degree of desat-

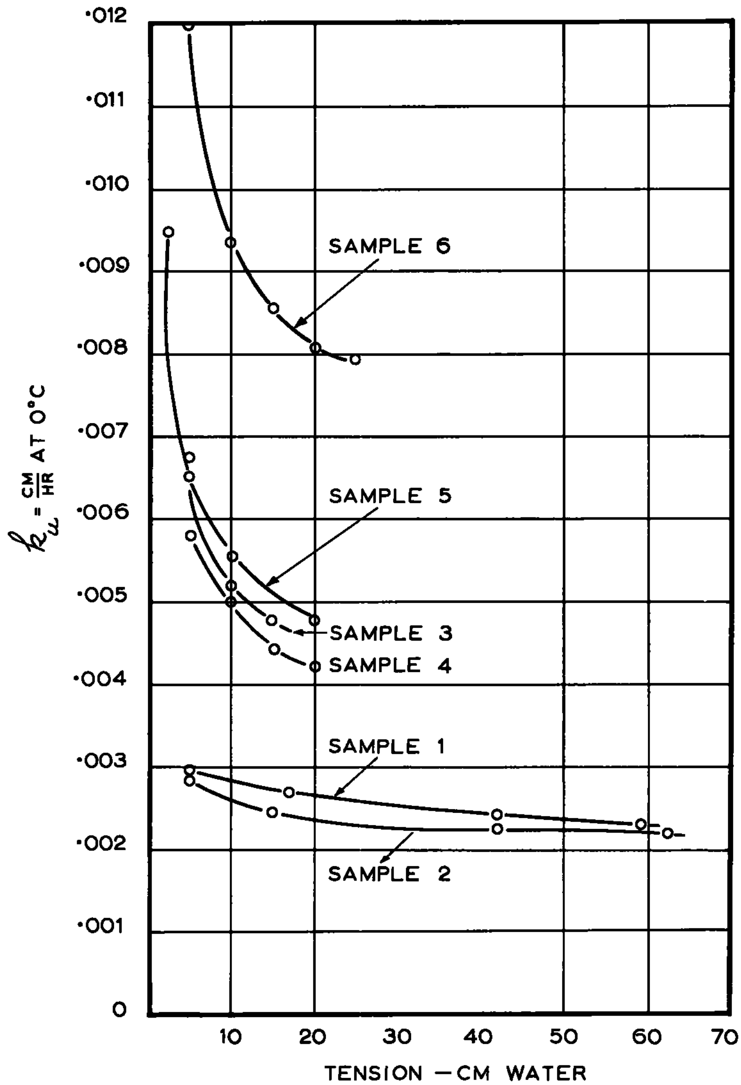


Figure 9. Unsaturated permeability coefficients as a function of tension.

uration immediately beneath the freezing zone. For the lighter textured soils (such as samples 3 and 6) this means a great reduction in moisture content (Figure 8), therefore, a low unsaturated permeability, and consequently a small heaving rate as in sample 3 or no heaving as in sample 6.

Other studies support the belief that critical desaturation beneath the frost line may act as a barrier to moisture movement in lighter textured soils. Research workers in soil science have shown the inverse relationship of unsaturated permeability coefficients and soil moisture tension (10). In moisture flow studies of the Division of Building Research, the soil moisture at one face of a soil specimen was held at zero tension(S_2) and the rate of flow was measured with increasing tension up to 1000 cm water on the opposite face (S_1). Figure 10 shows that for a predominantly silty soil there is an increase of flow with tension which rises to a maximum and then rapidly reduces to a low value. For clays the maximum flow rate is lower but the ability to transmit water extends to higher tension values. This appears to be the explanation for the higher rate of heave observed in these experiments for clayey soils as compared to coarser grained soils. It also explains how a no-heaving condition may occur even though high initial moisture contents exist.

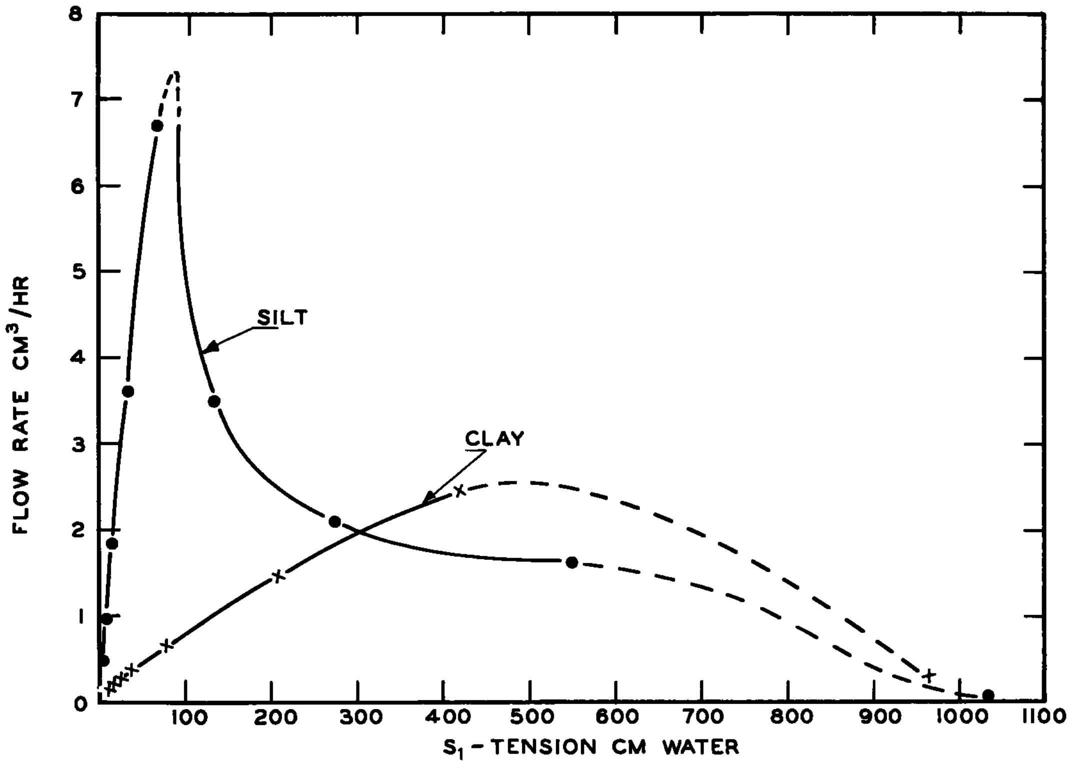


Figure 10. Flow rate as a function of S_1 with S_2 held at zero.

CONCLUSION

This paper is not a report on a completed study but rather describes the general approach and gives the initial experimental results.

The freezing experiments described were designed to measure moisture flow rates to the freezing zone for a number of soils under similar but arbitrary conditions. The results can best be understood in terms of unsaturated permeability characteristics and moisture-content/tension relationships.

The use of these concepts has the advantage of integrating all the factors, such as grain size, structure, clay composition, and exchange ions, which together result in a heaving rate peculiar to a given soil.

ACKNOWLEDGMENTS

The author gratefully acknowledges the assistance of K. N. Burn and K. R. Solvason in designing the apparatus and to C. B. Crawford and Margaret Gerard for their valuable criticism of the manuscript. Sincere thanks are due to the Director, R. F. Legget and the Assistant Director, N. B. Hutcheon, of the Division of Building Research, National

Research Council of Canada, for their constant encouragement and continuing interest in frost action research. This paper is a contribution from the Division of Building Research, National Research Council of Canada and is published with the Approval of the Director.

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Cold Quantities in New Jersey, 1901-1955

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● THE purpose of this study is to provide data for characterizing the severity of winters in New Jersey, to illustrate the distribution, magnitudes and frequencies of cold quantities, and to indicate what can be inferred from the cold quantity chart and map which accompany this report. These show the existence of the frost problem relative to New Jersey highways.

Beyond a certain degree of climatic severity, frost action in highway soils affects adversely the strength of the soil and performance of the road, particularly during thawing seasons.

In order to characterize the severity of a cold winter, climatologic data from the United States Weather Bureau for New Jersey, particularly temperatures, were studied for the past 55 years, beginning with the year 1901-02. There are no complete data available before that date.

The studies were confined to three reporting weather stations, namely, Layton (elevation 480 feet), New Brunswick (elevation 80 feet) and Indian Mills (elevation 100 feet), representing the northern part of New Jersey, central New Jersey and southern New Jersey, respectively. These weather stations were found to be the coldest in each of these three regions during the severe winter of 1947-1948. In some instances Sussex weather station data were substituted for incomplete Layton data, and Moorestown data for incomplete Indian Mills data.

Cumulative cold quantities for each year were computed in the form of degree days, tabulated in Table 1 and represented on a bar chart (Figure 1) for the three weather stations. Cumulative degree days for each winter were computed by adding the differences between the mean temperature and 32 deg F for each day that the mean was lower than 32 deg F.

Observation of this cold quantity chart indicates that: (1) severe winters in New Jersey repeat periodically with a fair degree of accuracy; (2) the recurrence of severe winters varies over a period of from 12 to 16 years, or, on the average, 14 years; (3) the order of magnitude of severe winters for the three regions of New Jersey is from 500 to 1430 degree days; (4) the first order of magnitude of a severe winter for the 55 years studied is characterized by the following cold quantity ranges: (a) in northern New Jersey, between 1000 and 1430 degree days; (b) in central New Jersey, between 500 and 800 degree days; (c) in southern New Jersey, between 500 and 730 degree days.

Everyone still remembers well the severe winter of 1947-48 and the tremendous damage to roads by frost it occasioned in this general area as well as in neighboring states. The subsequent repairs to these roads cost huge sums of money and much time.

Further observation of the chart indicates that there occurs, although in not too pronounced a fashion, a winter of medium severity (that is, of the second order of magnitude) after every four, five or six relatively mild winters. Thus, for example, six years following the severe winter of the first order in 1947-48, a medium-severe winter of the second order, with 285 degree days, occurred in central New Jersey in the freezing season of 1954-55. This caused enough damage to roads to become apparent to the highway user and owner, and to arouse concern.

From the preceding facts there may temporarily be inferred an estimate of the number of degree days at which damage to roads by frost is imminent. For the three sections of New Jersey the approximate numbers of damaging degree days would be: (1) 500 in the northern part; (2) 250 in the central part, and (3) 300 in the southern part.

It is, of course, needless to say that factors other than freezing temperatures may affect the severity of a winter, such as precipitation, elevation, winds and the orientation of mountain ranges.

From the periods or intervals of recurrence of a severe winter of the first or second order it may be inferred that in this general area: (1) there always has been, now is and always will be a frost problem in New Jersey relative to highways; (2) after several "mild winters" a severe winter will occur; (3) highways should be designed for the most

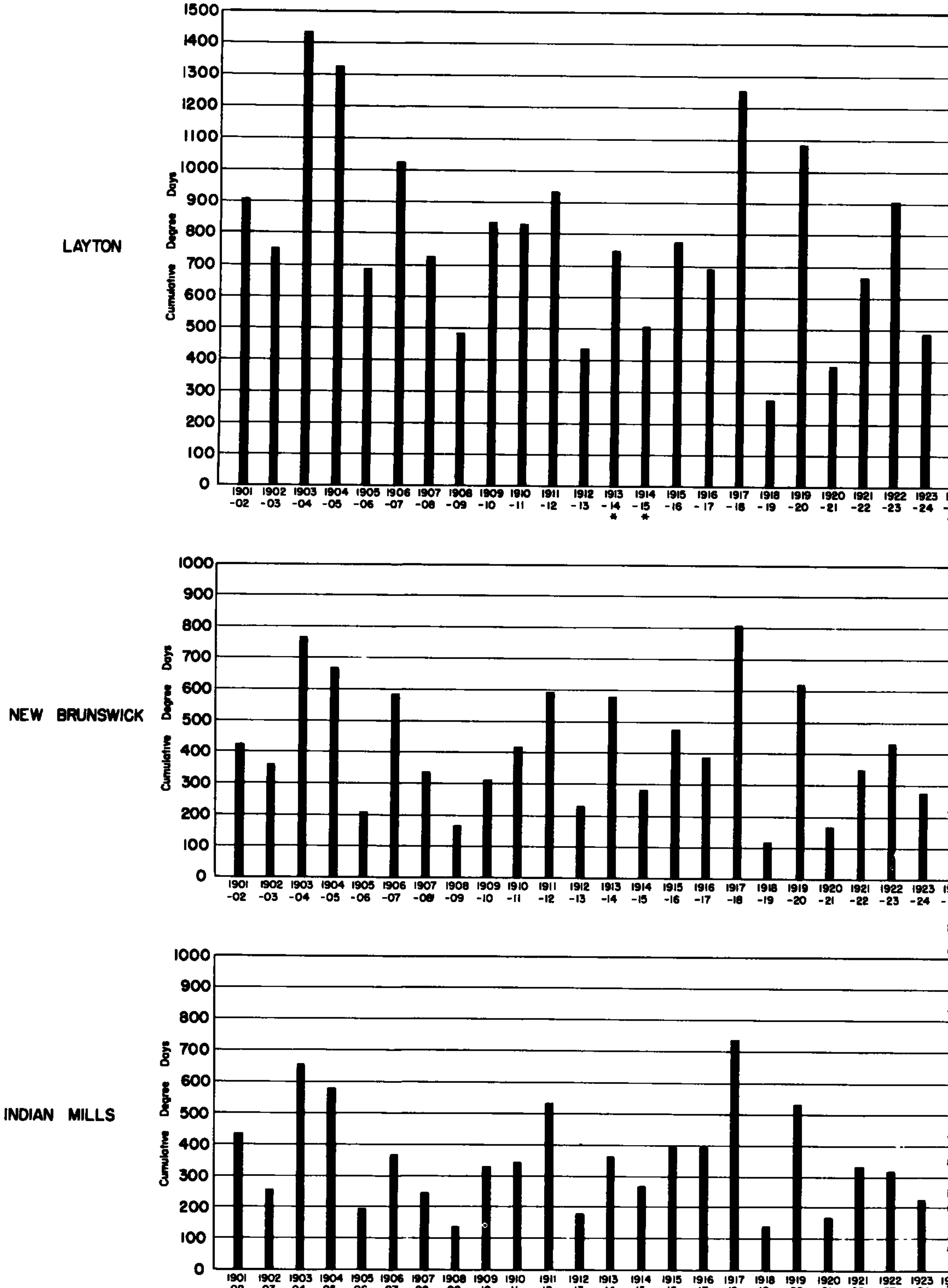

Figure 1. Cold quantities in cumulative degree days. Cumulati
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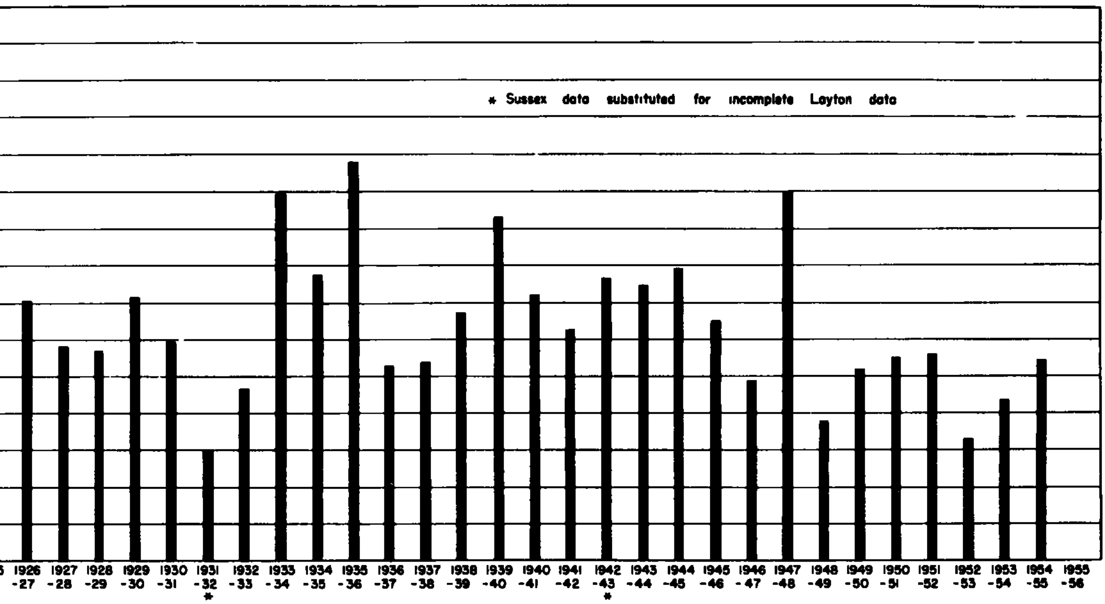

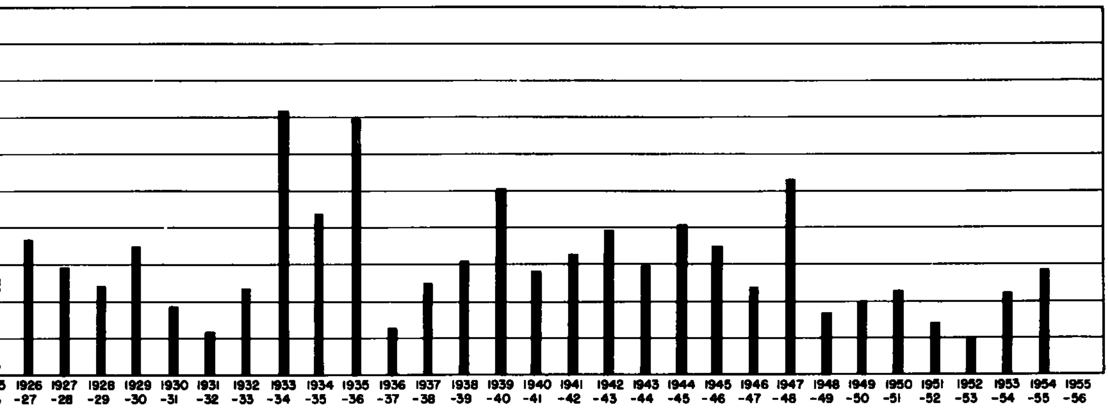

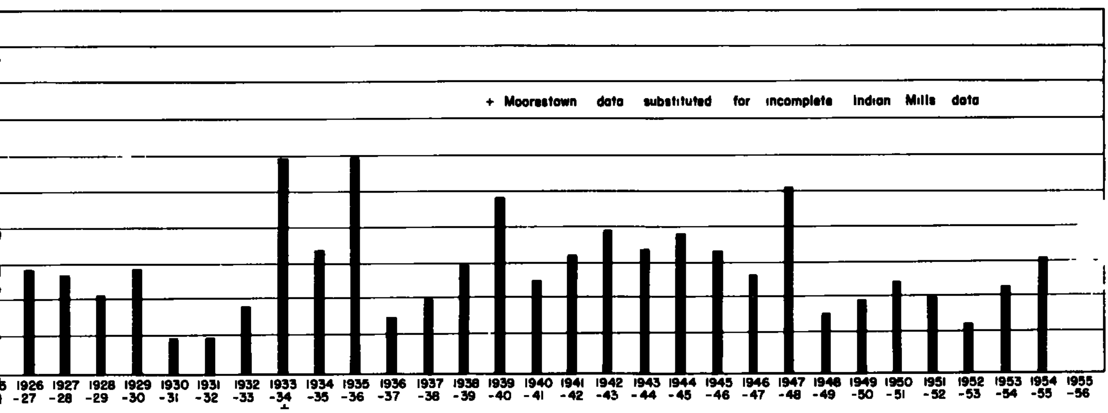

gree days for each winter computed by adding differences
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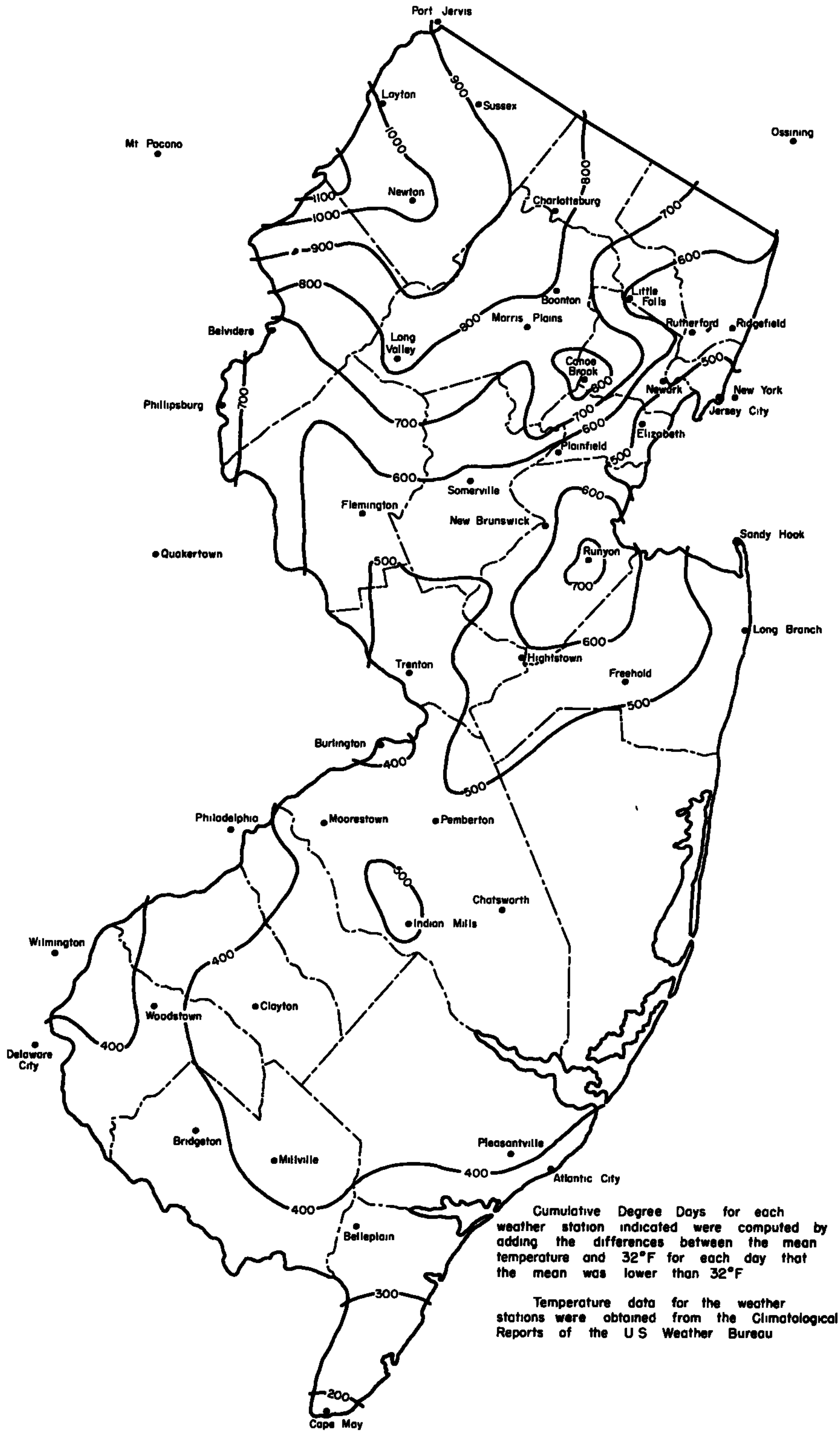

Figure 2. Cold quantities in cumulative degree days for New Jersey. Winter of 1947-1948.

TABLE 1

DEGREE DAYS IN NEW JERSEY FROM 1901-1955

(Also see Figure 1)

Year	Degree Days			Year	Degree Days		
	Layton, North, Elev 480	New Brunswick, Central, Elev 80	Indian Mills, South, Elev 100		Layton, North, Elev 480	New Brunswick, Central, Elev 80	Indian Mills, South, Elev 100
1901-02	904 0	432 0	433 0	1928-29	564.5	241 0	208 5
1902-03	750 0	356 0	257 5	1929-30	712 5	347 5	284 5
1903-04[a]	1431 5	760 5	652 0	1930-31	596 0	185 0	193 0
1904-05	1325 5	663 5	578 0	1931-32	300 0	114 5	96 0
1905-06	688 5	208 0	190 5	1932-33	465 5	232 5	175 0
1906-07	1021 0	584 5	363 5	1933-34	996 0	712 5	591.5
1907-08	726 0	334 0	240 0	1934-35	772.5	436 5	338 0
1908-09	481 5	163 0	133 0	1935-36[a]	1080 0	695 5	596 5
1909-10	835 5	307 5	327 5	1936-37	528 0	125 0	144 0
1910-11	830 0	415 5	341.5	1937-38	535 5	248 5	198 5
1911-12[b]	932 5	597 5	533 0	1938-39	676 5	309 5	294 0
1912-13	434 0	230 5	173 0	1939-40[b]	930 5	500 5	482.0
1913-14	745 0	579 0	362 0	1940-41	720.0	279 5	242 5
1914-15	503 0	281 5	265 5	1941-42	621 0	323 5	320 0
1915-16	773 5	475 5	396 0	1942-43	765 0	391 0	390 0
1916-17	692 0	384 5	396 5	1943-44	747 5	297 0	339 0
1917-18[a]	1253.5	804 5	735 5	1944-45	792 5	400 5	378 5
1918-19	273 5	114.5	133 0	1945-46	650 0	349 5	330 5
1919-20	1086 5	619 5	530.5	1946-47	483 0	236 0	268 0
1920-21	385 0	166 0	166 0	1947-48[a]	996 0	528 0	507 5
1921-22	664.0	346 0	332 0	1948-49	375 5	167 0	148 5
1922-23	905 5	430 5	317 0	1949-50	516 0	199 5	185 0
1923-24	488 5	275 5	222 5	1950-51	549 0	228 5	239 0
1924-25	686 5	355 5	221 0	1951-52	560 0	139 5	197 5
1925-26	690 0	390 5	301 5	1952-53	329 5	99 5	120 0
1926-27[b]	701 0	363.0	283 0	1953-54	436 5	224 5	220 5
1927-28	580 0	292 0	268 5	1954-55[b]	543 5	284 5	307 5

[a] Indicates severe winters
[b] Indicates medium severe winters

dangerous and disadvantageous conditions not only in relation to traffic but also in relation to frost action; (4) the maximum cold quantity on record in New Jersey is 1430 degree days; (5) it is certainly now known that when the 285 degree-days point is reached, frost damage to roads is marked, and (6) the existence of the frost problem in New Jersey, in its turn, poses many problems relative to the performance of soils under freeze-thaw conditions, with the correlative implication that much research should be done in this field.

A map of New Jersey showing degree-days for the severe winter of 1947-1948 was also prepared (Figure 2). It is realized that the degree-day lines are only as representative as the temperature data of each weather station is representative of the surrounding area in which the station is located.

ACKNOWLEDGMENT

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Frost Penetration Below Highway And Airfield Pavements

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Fundamentals related to the penetration of below freezing temperatures into the ground are outlined and a rational method of computing the maximum depth of frost penetration below highway and airfield pavements is presented.

Methods for making quick preliminary predictions of frost penetration are presented as well as "design" procedures based on thermal properties of the soil and weather conditions at the site. Of particular interest is the modified Berggren formula which was derived for the Corps of Engineers. Furthermore, reference to refined procedures, numerical and analogue computer solutions, are given for those cases where economic and engineering considerations dictate an "exact" solution.

Status of current research related to heat transfer between the air-pavement interface by radiation and convection-conduction is discussed.

● A NEWS item in the Christian Science Monitor on February 24, 1954 read: "Early warm weather in New Hampshire has made is necessary to put more than 100 highway trucks and crews to work repairing frost-buckled pavements. Further, the Highway Department says, unless cold weather comes soon, it may be necessary to keep trucks heavier than 20,000 pounds off New Hampshire roads."

In 1948 New Hampshire spent over $500,000 repairing roads damaged by frost action. Kentucky spent over $5,000,000 resurfacing roads that were damaged after the freezing and quick thawing that same year.

Frost action below pavements has assumed increased importance as wheel loadings, traffic frequency, and cost of pavement construction and maintenance have increased. Thermal problems in soil are receiving additional attention as a result of construction activity in permafrost areas. Furthermore, the potential of using the ground as a heat source or sink for heating and air-conditioning by heat pumps adds importance to this phase of soil engineering.

It is generally recognized that three requirements must be met simultaneously for significant ice-segregation and frost heave to occur in a foundation soil: (1) most obviously, below freezing temperatures must penetrate into the ground, (2) the soil must meet certain grain-size requirements to be "frost susceptible" and (3) a source of water must be available. This paper deals with the first requirement only, namely the prediction of the maximum depth of frost penetration with special reference to highway and airfield pavements. It is written primarily for practicing engineers and the author's soil engineering students.

After a presentation of the thermal properties of soil and their fundamental relationships, frost penetration formulas based on idealized assumptions are presented. The derivation of a rational formula, given the name modified Berggren formula, is presented. The effect of varying surface temperature, heat transfer between pavement surface and air, non-uniform soil, ice segregation and a depressed freezing point on the depth of frost penetration is discussed. The status of current research utilizing analogue computers to improve prediction techniques is presented.

The Arctic Construction and Frost Effects Laboratory of the New England Division, Corps of Engineers, U.S. Army, has been largely responsible for sponsoring the work described in this paper. Research is continuing in all areas of the problem, particularly frost penetration through non-uniform soil and air-pavement temperature relationships which are being studied with the aid of a hydraulic analogue computer.

NOTATION

In this paper the following symbols are used throughout:

NOTATION

a = thermal diffusivity ($= k/C$), in sq ft per hr;
A = area, in sq ft;
c = specific heat, in Btu per lb per deg F;
C = volumetric heat, in Btu per cu ft deg F;
d = thickness of soil layer in non-uniform, profile, ft;
F = surface freezing index, deg days F;
i = thermal gradient, deg F per ft
k = thermal conductivity, Btu per hr per ft per deg F;
l = length, ft;
L = latent heat, in Btu per cu ft;
q = rate of heat flow per unit area, in Btu per hr per sq ft;
Q = Rate of heat flow, Btu per hr;
t = time, hr;
t = duration of freezing period, days;
u = thermal energy, Btu per cu ft;
v = temperature, deg F;
v_o = temperature by which mean annual temperature exceeds freezing point of soil moisture, deg F;
v_s = temperature by which effective surface temperature is less than freezing point of soil moisture during freezing period ($= F/t$), in deg F;
w = water content, percent
x = coordinate direction measured vertically downward from ground surface, ft;
X = depth of frost penetration, in ft;
∂ = thermal ratio, dimensionless;
γ_d = dry density, lb per cu ft;
λ = correction coefficient, dimensionless; and
μ = fusion parameter, dimensionless.

Subscripts:

u = unfrozen soil; and
f = frozen soil.

FUNDAMENTAL PRINCIPLES

Certain thermal properties of soil and fundamental relationships among these properties are common to frost penetration problems. Indeed, these concepts are fundamental to all heat transfer problems.

Thermal Properties of Soil

Thermal conductivity, volumetric heat and latent heat are common names for the three thermal properties of soil. They are primarily dependent on the dry unit weight (dry density) and water content of the soil.

Thermal conductivity, k, expresses the rate of heat flow through a unit of area under a unit thermal gradient. Thus, the units of k are Btu per hr per sq ft per deg F per ft or Btu per hr per ft per deg F. Typical values of thermal conductivity in these units[1] are:

$$k_{air} = 0.014 \qquad k_{water} = 0.35$$
$$k_{shale} = 0.9 \qquad k_{ice} = 1.30$$
$$k_{granite} = 1.6 \qquad (k_{copper} = 225)$$

Since inorganic soil is composed of mineral particles, water and air, we would expect

[1] For conversion from the British system to the physical system of units; 1 Btu per hr per ft per deg F = 41.34×10^{-4} cal per sec per cm per deg C.

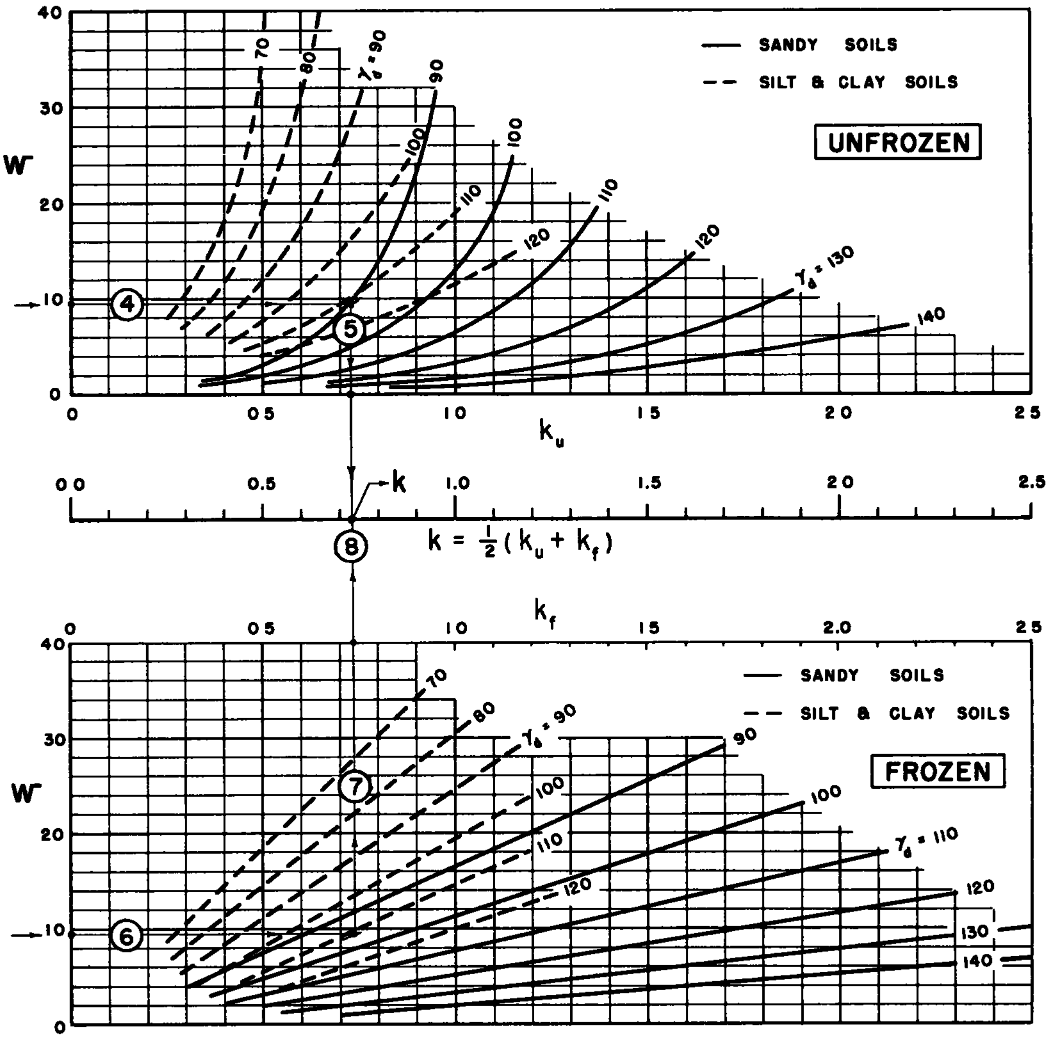

Figure 1. Thermal conductivity of soil after Kersten (1).

the thermal conductivity of soil to be about 1.0 Btu per hr per ft per deg F. Indeed, this would be a reasonable assumption for a preliminary investigation.

Kersten (1) conducted extensive thermal conductivity tests on inorganic cohesionless and cohesive soils. Results of these tests, conducted on both frozen and unfrozen soils, are summarized in Figure 1. Kersten states that "judicious use of the equations (essentially equivalent to the Figure 1 charts) with an understanding of their limitations will give conductivity values not more than 25 percent in error." For sandy soils having a relatively high silt and clay content, Kersten recommends the use of conductivity values intermediate between those given for sandy soils and silt and clay soils.

In the range of water contents (5-10 percent) and dry densities (125-135 lb per cu ft) commonly encountered in base-courses below pavements, thermal conductivity is very sensitive to moisture content and soil type. Scattered data indicate that the thermal conductivity of clean well-graded granular base-course materials lies about midway between values for sandy soils and silt and clay soils given in Figure 1.

Table 1 gives additional values of thermal conductivity which may be needed in depth of frost penetration computations.

Volumetric Heat C, expresses the change in thermal energy[2] in a unit volume of soil per unit change in temperature. Thus, the units of C are Btu per cu ft per deg F.[3] Volumetric heat is derived from the specific heat which expresses the change in thermal energy per unit weight per unit temperature change. Typical values of specific heat c in Btu per lb per deg F are:

$$c_{water} = 1.0 \qquad c_{rock} = 0.17 \text{ (soil minerals)}$$
$$c_{ice} = 0.5$$

at temperatures around the freezing point. Using these values of specific heat we may derive the following relationships for the volumetric heat, C, in Btu per cu ft per deg F for an inorganic soil:

For unfrozen soil:

$$C_u = \gamma_d\left(0.17 + \frac{w}{100}\right) \tag{1}$$

For frozen soil:

$$C_f = \gamma_d\left(0.17 + \frac{0.5w}{100}\right)$$

where:

γ_d = dry density (dry unit weight) of the soil, in pcf.

w = water content (moisture content) of the soil expressed, in percent of dry weight.

A typical value for the volumetric heat of soil would be 30 Btu per cu ft per deg F. Values of volumetric heat for other materials are given in Table 1.

Latent Heat L, expresses the change in thermal energy in a unit volume of soil when the soil moisture freezes or thaws at constant temperature. Thus, the units of L are Btu per cu ft. Latent heat depends only on the amount of water in a unit volume of soil. Since one pound of water gives off 143.4 Btu as it freezes, we may derive:

TABLE 1

THERMAL PROPERTIES OF COMMON MATERIALS[a]

At Temperatures Around the Freezing Point

Material	Thermal Conductivity, k, Btu/hr/ft/deg F[b]	Volumetric Heat, C, Btu/cu ft/ deg F[c]
Air	0 014	0 019
Water	0 35	62 4
Ice	1.30	28
Snow, loose	0 06	-
compact	0 20	-
Bituminous concrete	0 84	28
Portland-cement concrete	0 54[d]	30

[a] See Figure 1 for soil See References (2), and (3) for good tabulations of other materials
[b] 1 Btu per hr per ft per deg F = 41 34 x 10^{-4} cal per sec per cm per deg C
[c] 1 Btu per cu ft per deg F = 160 x 10^{-4} cal per cc per deg C
[d] Varies considerably with composition, moisture content, and density

$$L = 1.434\ w\ \gamma_d \tag{2}$$

A dense base-course below a pavement with a water content of 5 percent would have a latent heat of approximately 1000 Btu per cu ft.

Heat Storage

The relationships among the thermal properties defined above may be described by considering the following physical explanation of heat storage.

All physical objects can be considered to contain heat. A portion of this thermal energy (stored heat) is released when the object cools. If we consider a cubic foot of soil containing water within at least a portion of its voids, a graph representing the idealized thermal energy versus temperature characteristics of the soil is shown in Figure 2.

If the unfrozen soil is cooled, having started at a point represented by A, its thermal energy decreases by an amount C_u for each degree of temperature drop. When the temperature of the soil reaches the freezing point (point B) the soil moisture begins to freeze.

[2] Also referred to as heat content, internal energy, stored heat or enthalpy.
[3] 1 Btu per cu ft per deg F = 160 x 10^{-4} Cal per cu cm per deg C.

As the water freezes a thermal energy equal to L is released while the temperature of the soil remains nearly constant. After all water in the soil voids has frozen (point C) the temperature falls below freezing as heat is removed from the soil at the rate of C_f Btu per degree change in temperature.

In the absence of freezing or thawing the basic heat storage equation representing the change of thermal energy is simply:

$$u_1 - u_2 = C\,(v_1 - v_2) \qquad (3)$$

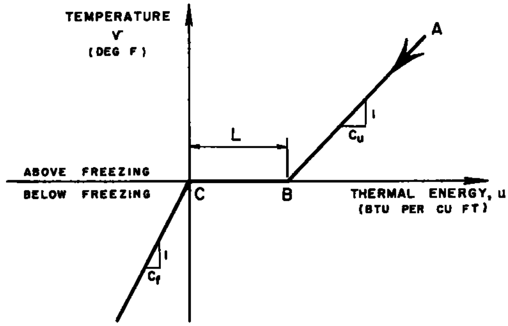

Figure 2. Idealized thermal energy diagram for a unit volume of soil.

where

u = thermal energy, in Btu per cu ft
v = temperature in deg F

The time rate at which the thermal energy changes is a function of the rate at which heat is transferred which in turn depends on the thermal conductivity of the soil.

Heat Transfer

There are three rather distinct methods by which heat may be transferred within an object or from one object to another. These methods are called conduction, convection and radiation. While convection and radiation play a dominant role in transferring heat between pavement surfaces and the air above, conduction plays a lone role in transferring heat within a soil mass unless water moves within the soil voids. For the time being we shall consider thermal conduction only.

Thermal Conduction. Atomically speaking, heat conduction is considered as a transfer of kinetic energy from the molecules of a warm part of a body to those in a cooler part. The rate at which heat is transferred by conduction is customarily given in the following form:

$$Q = k\,i\,A = k\,\frac{v_1 - v_2}{l}\,A \qquad (4)$$

where:

Q = rate of heat flow, in Btu per hr
i = thermal gradient in deg F per ft
= temperature difference ($v_1 - v_2$) divided by length l
A = area in sq ft

This relationship is known as the Fourier equation.

A more fundamental expression for Fourier's equation is:

$$q = -k\,\frac{\partial v}{\partial x} \qquad (5)$$

where q is the heat conducted per unit area per unit time in the x-direction. The minus sign follows from the second law of thermodynamics according to which heat flows from points of higher temperature to those at a lower temperature.

In this paper, thermal conduction will be considered in one direction only, namely vertical, where the positive x-direction is measured vertically downward from ground surface.

Thermal Continuity. In the absence of freezing or thawing the time rate at which the thermal energy of an element of soil changes plus the net rate of heat transfer into the element must equal zero. This follows from the conservation of thermal energy and may be expressed:

$$\frac{\partial u}{\partial t} + \frac{\partial q}{\partial x} = 0 \qquad (6)$$

The heat storage equation, shown initially in Equation 3, may be written more generally:

$$\frac{\partial u}{\partial t} = C \frac{\partial v}{\partial t} \tag{7}$$

Substitute Equation 7 in Equation 6 and introduce Equation 5 to obtain:

$$C \frac{\partial v}{\partial t} = k \frac{\partial^2 v}{\partial x^2}$$

or

$$\frac{\partial v}{\partial t} = a \frac{\partial^2 v}{\partial x^2} \tag{8}$$

where "a" is the diffusivity of the soil in sq ft per hr.

Equation 8 is known as the one-dimensional diffusion equation for heat.

At the interface between the frozen and unfrozen soil, $x = X$, the equation of continuity which must be satisfied is:

$$L \frac{dX}{dt} = q_u - q_f$$

where $(q_u - q_f)$ is the net rate of heat flow away from the interface and X is the depth of frost penetration in ft. More specifically:

$$L \frac{dX}{dt} = k_f \frac{\partial v_f}{\partial x} - k_u \frac{\partial v_u}{\partial x} \tag{9}$$

Equations 8 and 9 are the basic differential equations which must be solved for given initial conditions and boundary conditions to yield an expression for the depth of frost penetration. A solution to this problem is discussed later under a section on frost penetration formulas.

TABLE 2

PHYSICAL ANALOGIES IN CONDUCTION

Name	Fluid Flow Through Soil	Head Conduction	Current Conduction
1 Potential	Total head, h (ft)	Temperature, v (^{0}F)	Voltage, V (volts)
2 Storage	Fluid volume, W (cu ft per cu ft)	Thermal energy, u (Btu per cu ft)	Charge, Q (Coulombs)
3 Conductivity	Coefficient of permeability, k (ft per sec)	Thermal conductivity, k (Btu/°F/ft/hr)	Electrical conductivity, o [Coulombs per (sec) (meter) (volt)]
4 Flow	Rate of flow, Q (cu ft per sec)	Rate of flow, Q (Btu per hr)	Current, i (Coulombs per sec = amperes)
5 Negative Gradient Along x	Hydraulic gradient, i $i = -\frac{\partial h}{\partial x}$ (ft per ft)	Thermal gradient, i $i = -\frac{\partial v}{\partial x}$ (deg F per ft)	Electric Intensity, E $E = -\frac{\partial V}{\partial x}$ (volts per meter)
6 Conduction Along x	Darcy's Law $Q = -k_x \frac{\partial h}{\partial x} A$	Fourier's Law $Q = -k_x \frac{\partial v}{\partial x} A$	Ohm's Law $i = -o_x \frac{\partial V}{\partial x} A = \frac{V}{R}$
7 Capacitance	Coef of volume change, M (sq ft per cu ft) $M = \frac{dW}{dh} = \frac{\gamma_w a_v}{1+e} = \frac{k}{c_v}$ one dimensional	Volumetric heat, C (Btu/deg F/cu ft) $C = \frac{du}{dv}$	Capacitance, C (Coulombs per volt = farads) $C = \frac{dQ}{dV}$
8 Continuity (General Case)	$\frac{\partial W}{\partial t} + \nabla Q = O$	$\frac{\partial u}{\partial t} + \nabla Q = O$	$\frac{\partial Q}{\partial t} + \nabla i = O$
9 Continuity (Steady conduction)	$\nabla Q = O$	$\nabla Q = O$	$\nabla i = O$
(Laplace equation if material homogeneous and isotropic)	$\nabla^2 h = O$	$\nabla^2 v = O$	$\nabla^2 v = O$
10. Diffusion (One dimension)	$\frac{\partial h}{\partial t} = \frac{k}{M} \frac{\partial^2 h}{\partial x^2}$ ($\frac{k}{M} = c_v$, Coef of consolidation)	$\frac{\partial v}{\partial t} = \frac{k}{C} \frac{\partial^2 v}{\partial x^2}$ ($\frac{k}{C} = a$, Diffusivity)	$\frac{\partial V}{\partial t} = \frac{o}{C} \frac{\partial^2 V}{\partial x^2}$

Analogies. The equations which have been developed in preceding paragraphs are similar to those encountered in many other branches of physics. For example, soil engineers will recognize that Equation 4 is analogous to Darcy's Law which governs the flow of water through soil. Furthermore, Equation 8 is analogous to the equation representing one-dimensional consolidation of a fine-grained soil.

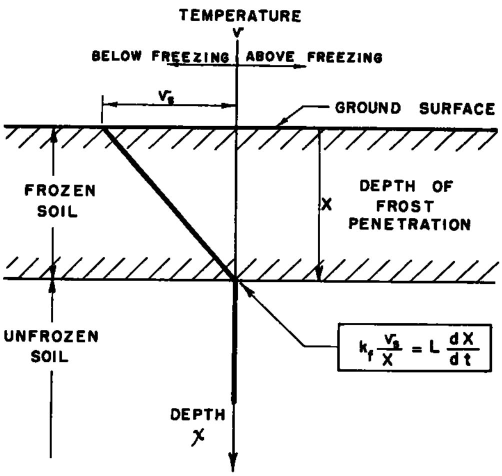

Figure 3. Thermal conditions assumed for Stefan Formula.

Table 2 has been prepared to show the formal correspondence between terms and equations found in three important branches of physics. Knowledge of these analogous relationships and those in magnetism, elastic equilibrium and others, give engineers and physicists a common language for discussion and for solving engineering problems. Often an explicit solution to an actual problem in one branch of physics will be available in analogous form in the literature of another branch.

The author has found analogs extremely useful in classroom instruction as well as in the solution of actual thermal and fluid flow problems.

DEPTH OF FROST PENETRATION

We shall confine our thermal problem to predicting the maximum depth of frost penetration below a pavement surface maintained relatively free of snow. A solution to this problem can be achieved in a variety of ways dependent largely on the objectives of the investigation and use of the result. One objective may be to obtain a single solution for a given set of design soil and weather conditions. Another problem may involve studying the effects of certain variables on the depth of frost penetration. Furthermore, the method of solution will depend on the precision desired in the result and the reliability of the given data.

Methods of Solution

Depth of frost penetration predictions can be obtained by (1) rational and empirical formulas and charts; (2) prototype tests and laboratory experiments; (3) "hand" solutions (graphical and numerical) based on finite difference approximations to the differential equations; (4) "machine" computation, for example automatic computation on IBM equipment; and (5) analogs such as the hydraulic and electronic analog computers which have been built for the New England Division, Corps of Engineers.

By far the most common method of predicting frost penetration is by formulas and charts based on analytic solutions to the fundamental equations and modified by assumptions and observed data. This method will be considered in some detail in the following paragraphs. In particular, a rational formula, given the name modified Berggren formula, will be presented.

Frost Penetration Formulas[4]

Of the numerous formulas which have been developed for predicting the depth of frost penetration, only two will be presented here. They are the J. Stefan formula and the modified Berggren equation. We shall assume initially that the soil is homogeneous, in other words, that it is uniform with depth (non-stratified).

Stefan Formula. Thermal conditions assumed in the derivation of the Stefan Equation, the simplest of the frost penetration formulas, are shown in Figure 3. It is assumed

[4]Reference (2) represents the most thorough theoretical treatment of frost penetration which has been written in English. Reference (4) is also excellent.

that the latent heat of soil moisture is the only heat which must be removed when freezing the soil. Thus, the thermal energy which is stored in the form of volumetric heat and released as the soil temperatures drop to and below the freezing point is not considered. This assumption is equivalent to shifting the sloping lines in Figure 2 to vertical positions.

Under these assumptions the diffusion equations in the frozen and unfrozen soil, Equation 8, do not exist and Equation 9 reduces to:

$$L \frac{dX}{dt} = k_f \frac{v_s}{X} \tag{10}$$

where v_s is the difference between the ground surface temperature and freezing temperature of soil moisture, Figure 3, at any time. In physical language, Equation 10 states that the latent heat supplied by the soil moisture as it freezes a depth dX in time dt is equal to the rate at which heat is conducted to ground surface.

When Equation 10 is integrated:

$$X = \sqrt{\frac{2k_f \int v_s \, dt}{L}} \tag{11}$$

where $\int v_s \, dt$ in deg hr is known as surface freezing index, F. Usually F is expressed in deg days Fahrenheit in which case:

$$X = \sqrt{\frac{48 \; k_f \; F}{L}} \tag{12}$$

which is the common form used to present the Stefan formula.

Since the Stefan equation neglects the volumetric heat of the frozen and unfrozen soil, it will give depths of frost penetration which are always too large. The degree to which Equation 12 overpredicts depends obviously on the relative importance of volumetric heat to latent heat which in turn depends on climate at the site in question and water content of the soil. These factors can be discussed to better advantage after the modified Berggren formula is presented.

Modified Berggren Formula. A detailed derivation of a rational formula, initially presented by the author and Dr. Henry M. Paynter[5] in a report to the Corps of Engineers (5), is given in the Appendix. After this formula was developed, it was discovered to be essentially identical to a formula published by W. P. Berggren (6). Since the general form of the solution differed from that of Berggren, the author and Dr. Paynter have named their solution the modified Berggren formula.

This solution is based on the earlier work of Neumann, as reported in Carslaw and Jaeger (7), dealing with melting and freezing problems in still water. It is assumed, as shown in the Appendix that the soil is a semi-infinite mass with uniform properties and existing initially at a uniform temperature v_o deg. It is further assumed that the surface temperature is suddenly changed from its initial value v_o deg above freezing to a temperature v_s deg below freezing. Equations 8 and 9 govern the penetration of frost. These and other thermal conditions during frost penetration are summarized in Figure 4.

It is indeed fortunate that the results of this rather complex development can be written with very little sacrifice in accuracy in the following simple form which may be compared with the Stefan Equation 11:

$$X = \lambda \sqrt{\frac{2 \; k \; v_s \; t}{L}} \tag{13}$$

[5] Assistant Professor of Mechanical Engineering, M. I. T., and President, Pi-Square Engineering Co., Inc., Boston, Mass.

where

X = depth of frost penetration in ft
k = thermal conductivity of soil, in Btu per hr per ft per deg F generally taken as an average of values in the frozen and unfrozen state.
L = latent heat of soil, in Btu per cu ft
t = time in hr

and

λ = dimensionless correction coefficient which is given in Figure 5 as a function of two important dimensionless parameters, α and μ:

$$\text{Thermal ratio, } \alpha = \frac{v_o}{v_s} \tag{14}$$

$$\text{Fusion parameter, } \mu = \frac{C}{L} v_s$$

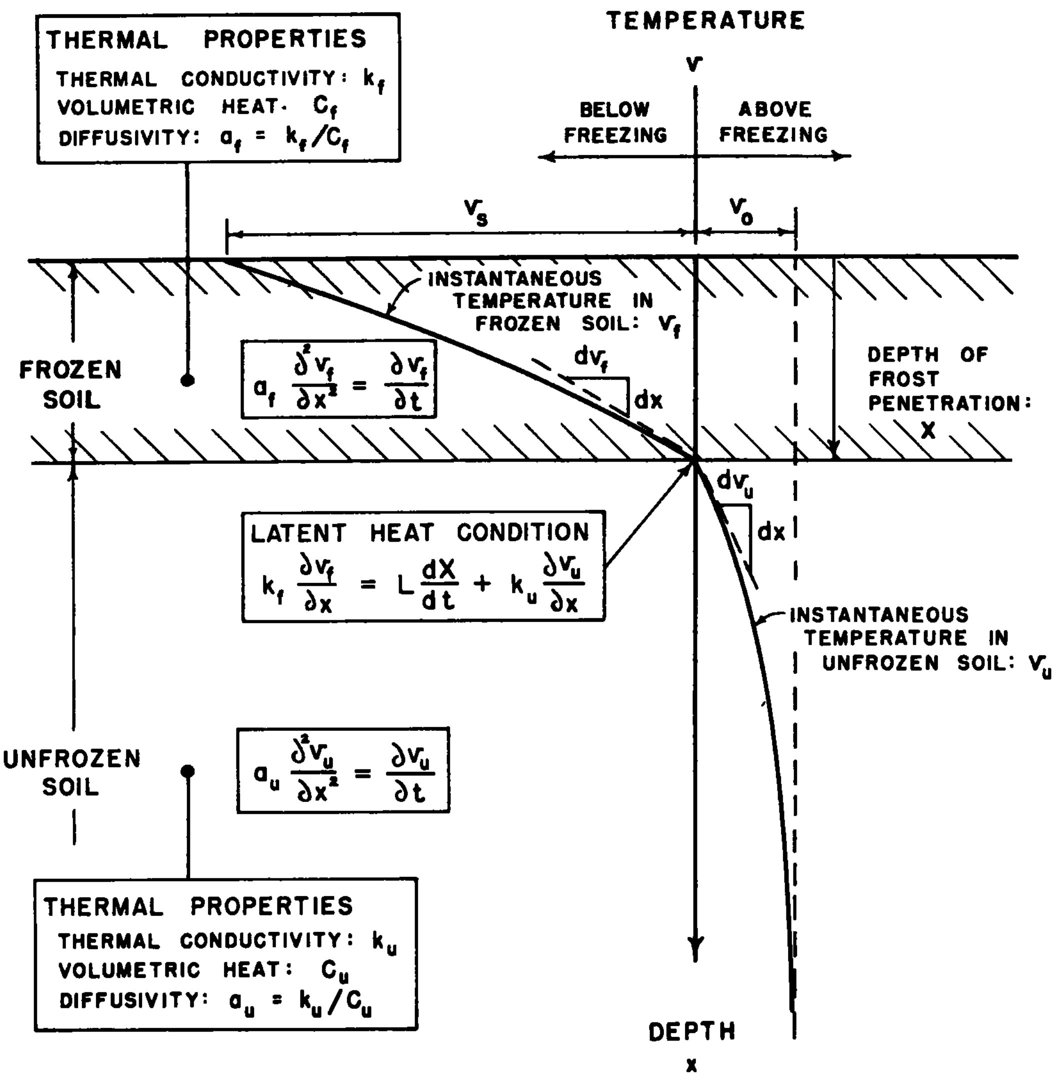

Figure 4. Thermal conditions assumed for modified Berggren formula.

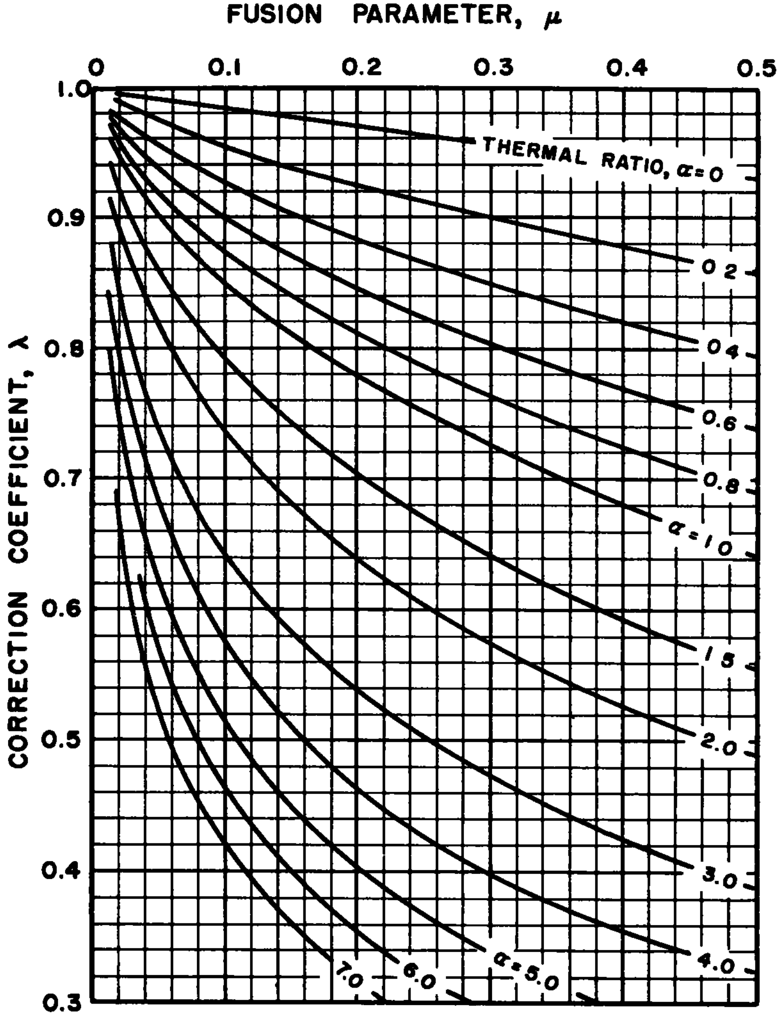

Figure 5. Correction coefficient in the modified Berggren formula.

The correction coefficient λ in equation 13 is essentially a term which corrects the Stefan formula for the effects of volumetric heat which it neglected. We noted that the Stefan equation always predicts too deep. Thus λ is always less than unity.

In the foregoing discussion of frost penetration formulas a number of assumptions were stated or implied in order to develop a rational mathematical approach for determining the maximum depth of frost penetration. While this approach is valuable as an aid to understanding many of the variables affecting the problem, we can no longer ignore complexities introduced by reality since we seek a useful solution to a real problem. The randomness of the weather and its effect on pavement temperatures and the usual heterogeneous nature of the soil are examples of important factors which must now be given consideration.

Effect of Surface Temperature Variations During the Freezing Period. In the development of the modified Berggren formula the surface temperature was assumed to change suddenly from v_0 degrees above freezing to v_s degrees below freezing where it remained constant. Thus, the relationship among the dimensionless parameters, Figure 5, is strictly applicable only to this case. In reality the pavement surface experiences daily as well as seasonal temperature fluctuations through local variations in air temperature, wind velocity, precipitation, solar radiation, etc. There can be little doubt that these

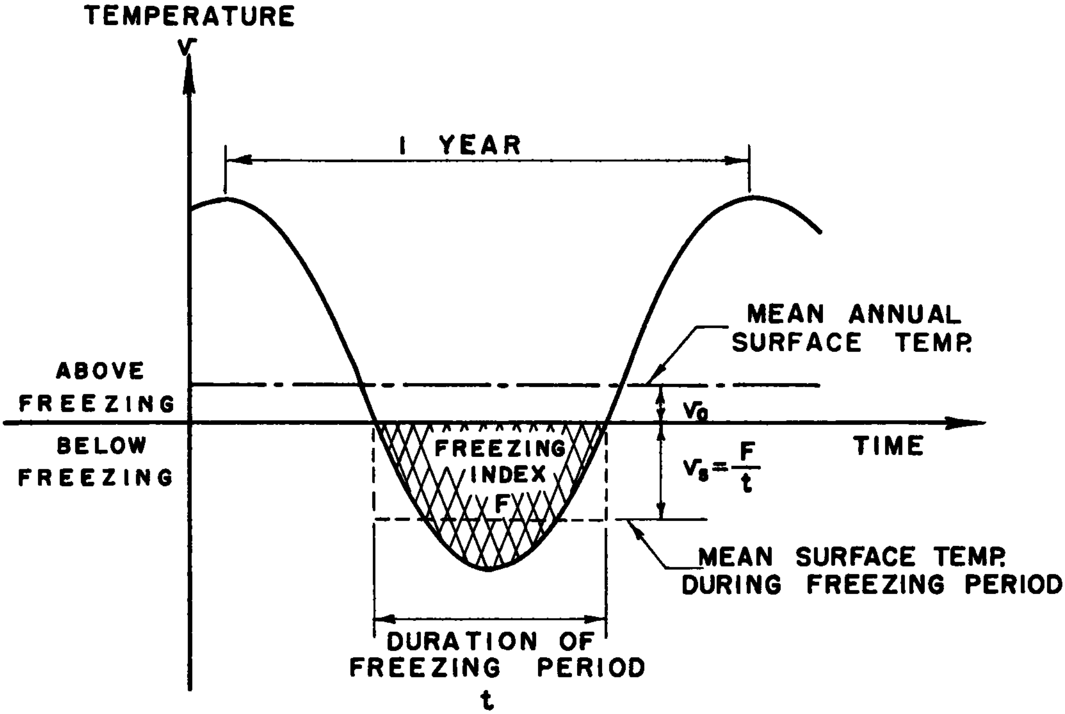

Figure 6. Sine curve for assumed annual variation in surface temperature.

variations have a marked effect on the rate at which freezing temperatures penetrate into the soil. However, the maximum depth of frost penetration is affected little if v_o and v_s are defined as shown in Figure 6.

In conjunction with studies for the Arctic Construction and Frost Effects Laboratory of the New England Division, Corps of Engineers, the author with Dr. Paynter obtained a series of frost penetration solutions assuming that surface temperature varies sinusoidally during the year, Figure 6. These computations, performed by the IBM Company on an automatic computer, gave values for the correction coefficient λ which varied less than 8 percent from those for the step change in surface temperature assumed in the modified Berggren formula. The IBM results and comparisons are given in Table 3. In view of this result and the results of statistical studies on observed depths of frost penetration, the following development from Equation 13 is indicated.

Assume that v_s, Figure 6, represents the time-average difference in temperature between ground surface and the freezing point during the freezing period. Therefore, the term v_s t in Equation 13 becomes the surface freezing index, F. If F is expressed in deg days Fahrenheit, the modified Berggren formula becomes:

$$X = \lambda \sqrt{\frac{48\,k\,F}{L}} \qquad (15)$$

which may be compared with the Stefan Equation 12. The correction coefficient λ may be obtained from Figure 5[6] as a function of α and μ which can now be expanded from equation (14) as follows:

$$\text{Thermal ratio } \alpha = \frac{v_o}{v_s} = \frac{v_o\,t}{F}$$
$$\text{Fusion parameter}, \mu = \frac{C}{L}\,v_s = \frac{CF}{Lt} \qquad (16)$$

[6] If desired, a slight correction for seasonal variation in surface temperature can be made by selecting a λ value differing from Figure 5 consistent with IBM results given in Table 3.

where v_0 is the deg F by which the mean annual temperature exceeds the freezing point of soil moisture, Figure 6.

We are prepared now to demonstrate the degree to which the Stefan equation overpredicts by observing the magnitude of λ for the different climates and soil conditions. The importance of this demonstration as an aid to a fundamental understanding of frost penetration cannot be over-emphasized. Reference is made to Figure 5:

1. For soils having high water contents the volumetric heat C is small compared to latent heat L in which case the fusion parameter μ is small. Therefore, the correction coefficient λ approaches unity and the Stefan equation may be expected to give reasonable results.
2. For northern climates where the mean annual temperature approaches the freezing point of soil moisture, the thermal ratio α approaches zero and the correction coefficient is greater than 0. 9. Thus, the Stefan equation will yield depths of frost penetration not more than 10 percent greater than the actual.[7]

In general then, for northern climates represented by North Dakota, Canada and Alaska and for soils of high water content the correction coefficient may often be assumed equal to unity.[8] However, in more temperate climates represented, for example, by Kansas and Nebraska and for relatively dry or well drained soils, the correction coefficient may be as low as 0. 5 in which case the Stefan formula would predict twice the actual depth.

Relationship Between Pavement Surface Temperature and Air Temperature. The depth of frost penetration and indeed all subsurface temperatures are governed by variations in the ground surface temperature. Surface temperature has been used, therefore, in all developments which have been presented thus far. Unfortunately, pavement surface temperature variations are generally unknown while abundant data on air temperatures are usually available at or near a given site. Pavement temperature and air temperature are related but this relationship is one of the most complex and fundamentally important problems remaining to be solved in the frost penetration prediction.

The author has frequently used the following example as a vivid demonstration of the important air-pavement temperature relationship. At an air force base in Greenland in the early summer of 1953, thawing occurred to a depth of about 4 ft below a bituminous concrete pavement before the mean daily air temperature rose above 32 deg F. In other words, the pavement surface was enough warmer than the air to thaw 4 ft of base course before thawing would normally be expected to begin.

A fundamental investigation of factors affecting heat transfer at the air-ground interface has been the objective of recent research at MIT sponsored by the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers. A report based on the work of Scott (9), is in preparation as Reference 10. Current research is directed toward reducing Dr. Scott's theoretical studies to design criteria. In this paper, the author will describe briefly the important physical phenomena only.

TABLE 3

EFFECT OF SINUSOIDALLY VARYING SURFACE TEMPERATURE ON CORRECTION COEFFICIENT

Thermal Ratio, α	Fusion Parameter, μ	Correction Coefficient λ [a]	λ [b]
0	0 191	0 97	0 998
1 025	0 195	0 78	0 724
2 040	0 196	0 64	0 595
0	0 382	0 94	0 998
0 519	0 386	0 80	0 804

[a]Given in Figure 5, modified Berggren formula, for step change in surface temperature
[b]Determined from results of studies on IBM-701 computer for surface temperature varying sinusoidally

Transfer of heat between the pavement surface and air is effected through moisture evaporation and condenstion, snow and ice melt, and more important through direct and diffuse solar radiation, net longwave radiation between pavement and sky, and convection-conduction. The latter terms are shown graphically in Figure 7 as they could occur on a sunny day. The principal variables affecting the magnitude of these factors during a winter season may be summarized as follows:

Solar and longwave radiation:

1. Latitude (for sun's altitude) and elevation of the site,

[7] Indeed, the IBM results, Table 3, indicate a correction coefficient equal to unity for the α = 0 case. Thus, the Stefan equation should yield a nearly exact solution.
[8] Carlson and Kersten (8) report successful use of the Stefan formula in predicting the depth of freezing and thawing under pavements in Alaska.

2. Atmospheric vapor pressure (for solar absorption and scattering),
3. Cloud cover and type (for equivalent percent sunshine),
4. Atmospheric conditions (whether clear or industrial),
5. Type of surface (color and texture for absorbtivity to radiation).

Convection - conduction:
1. Wind velocity,
2. Type of surface (surface roughness),
3. Topography and vegetation in near vicinity.

Unfortunately, the magnitude of the radiation and convection-conduction heat transfer depends also on the unknown surface temperature itself. Therefore, the solution must be evolved through a successive approximation procedure.

Figure 8 has been prepared to represent idealized temperature curves during the winter season for various surfaces. The sine curves, all drawn about the same vertical axis, are presented for rough qualitative comparisons only. For example it is generally true that a pavement surface, even though maintained relatively free of snow, will exist at a temperature higher than the mean air temperature throughout the year. Furthermore, natural surfaces are warmer still in winter, resulting in part from insulating effects of snow, but during the summer are cooler than the surrounding air. An important factor affecting the latter is evaporation.

From the idealized curves in Figure 8 we can generate the following statements which may be important to the practicing engineer:

1. Estimates of frost penetration depths below a proposed pavement which are based

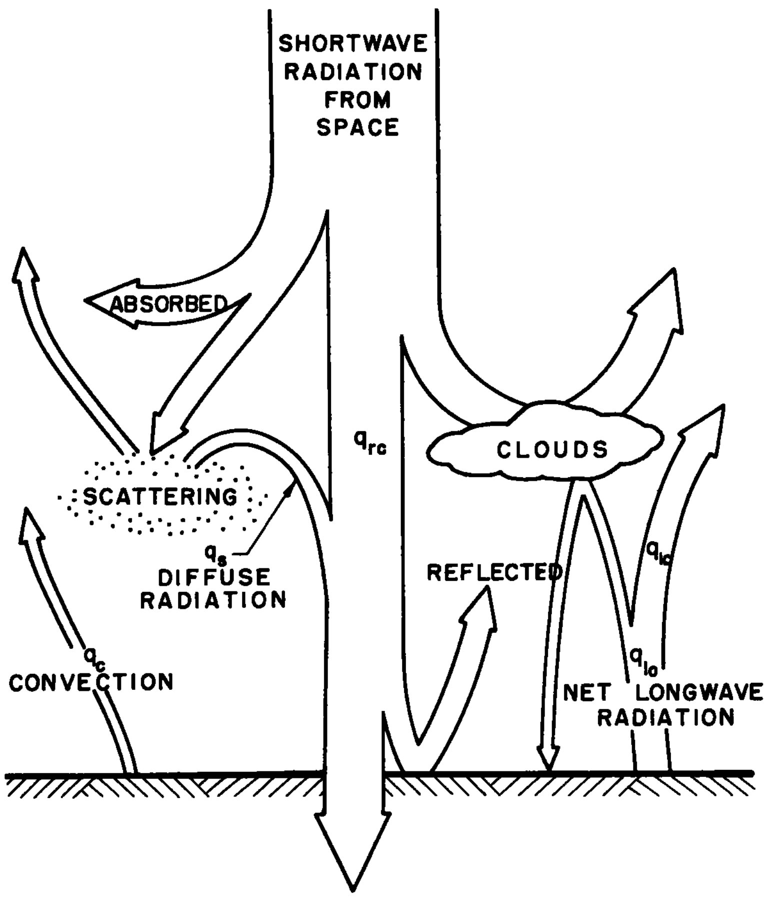

Figure 7. Heat transfer between ground surface and air on a sunny day.

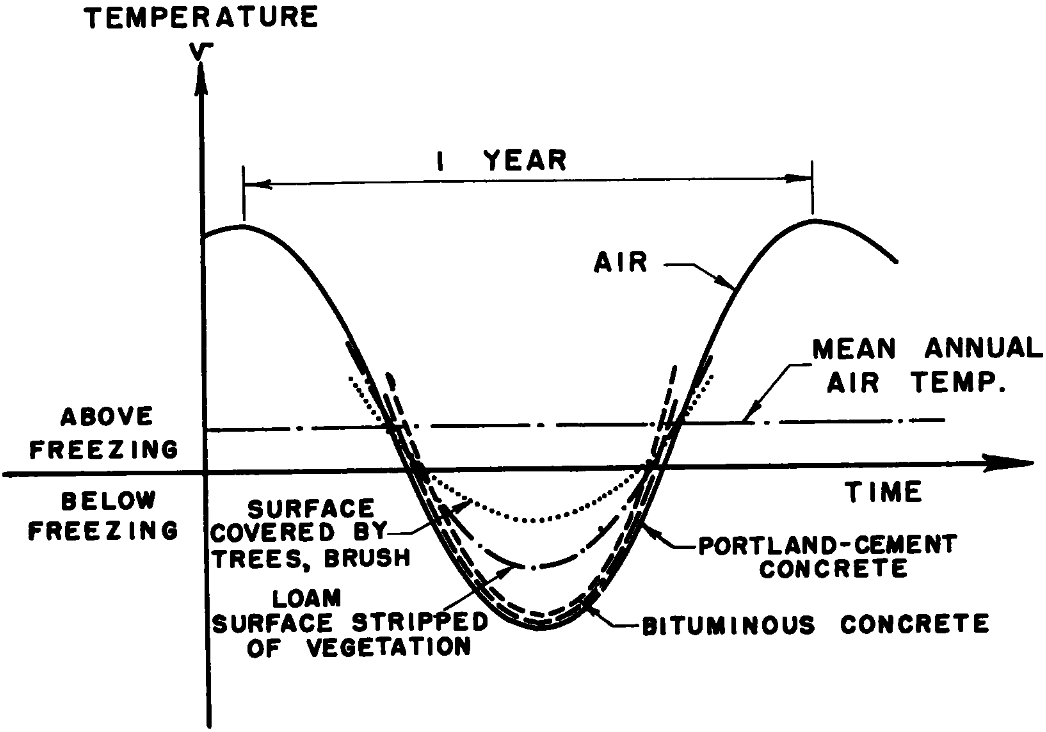

Figure 8. Idealized Temperature curves for various surfaces.

on records of observed frost depths in a given locality are likely to be unsafe for two reasons: (a) the pavement surface temperature will probably be colder than surrounding natural surfaces during the winter months; (b) if the pavement base-course is well drained it is likely to exist at a lower water content than natural soil in the surrounding area. Again, the frost depth will be greater below the pavement because, from Equation 15, we observe:

$$X \sim \frac{1}{\sqrt{L}}$$

where the latent heat L depends directly on the amount of moisture in the soil.[9]

2. In areas of permanently frozen ground (permafrost areas) major engineering problems arise because of changes in the subsurface thermal regime as a result of new surface conditions caused by construction. For example, it can be reasoned from Figure 8 that the seasonal depth of thawing will be greatly increased when an area is stripped of its vegetation and a pavement surface is constructed.

We have defined surface freezing index as the area under the temperature curve during the freezing period, Figure 6. It is clear that the pavement freezing index which we seek for the depth of frost penetration computation is less than the air freezing index which is generally known. The ratio of these indices has been called a surface correction factor n or C_f. It may be determined for a given pavement surface from direct observations of pavement and air temperatures (11). It is evident that the value of n which is applicable to one locality may be very different from that at another site even under identical pavement conditions. Indeed, a limiting value would be zero in a temperate climate with no pavement freezing index while the ratio approaches unity for colder climates.

[9] It is true that other factors in the equation vary in addition to F and L, notably the thermal conductivity k which is lower for the well-drained soil. However, this variation is more than offset by the latent heat.

A general rational approach to predicting pavement freezing index from air temperature data or from the air freezing index would be to base the computation on a surface temperature elevated a fixed amount relative to air temperature rather than on a fixed ratio of freezing indices. The author is frank to admit, however, that the more conservative approach is to base the prediction on air temperature. Indeed, in computations for the seasonal maximum depth of frost penetration below pavements this approach is recommended, at least until results of current studies are completed. Consistent with this assumption is that of basing v_0 on the mean annual air temperature rather than surface temperature.

Effect of Ice-Segregation During Freezing. It is well known that distinct layers or lenses of ice will form during freezing in frost-susceptible soils when certain moisture conditions are present. The thickness of individual ice layers is inversely proportional to the rate of frost penetration. However, the rate at which the ground surface heaves is essentially independent of the rate of frost penetration in a frost-susceptible soil. While a discussion of the ice-segregation phenomenon is outside the scope of this paper,

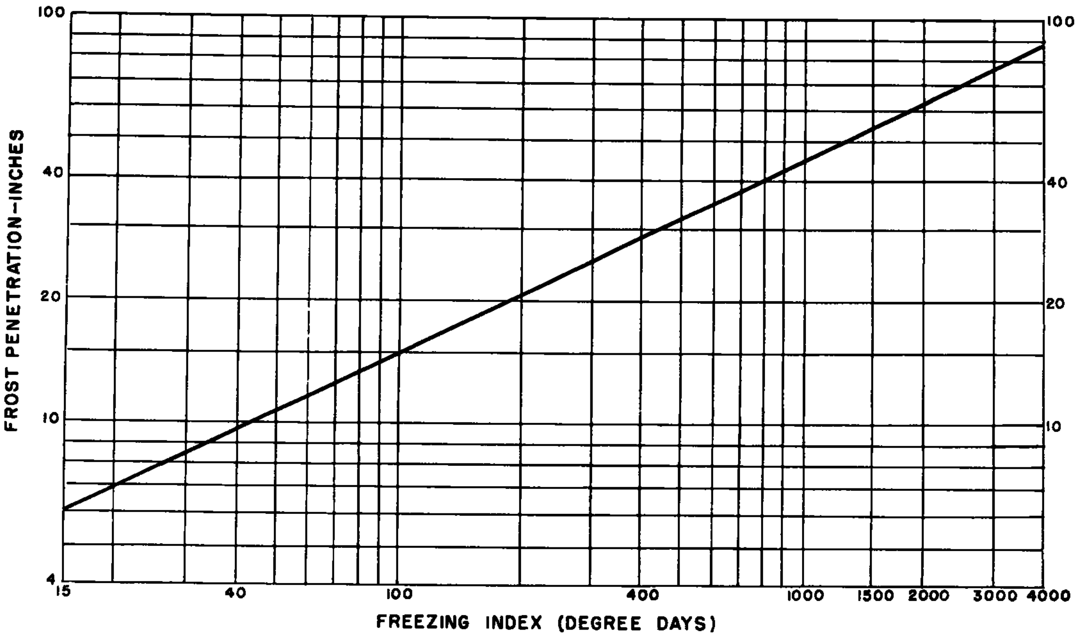

Figure 9. Relationship between frost penetration and freezing index in well-drained, granular, non-frost susceptible base course material. (Courtesy of the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers (13) and (14).

its effect on the maximum depth of frost penetration will be treated briefly.

In most highway and airfield pavement design today, the thickness of pavement and base is selected to prevent entirely or substantially freezing in a frost-susceptible subgrade. Therefore, even though frost penetration formulas have been proposed, (2) and (4), to account directly for ice segregation, the need for this refinement in practice is not common. The author believes that the following approximate treatment based on the modified Berggren formula should satisfy most requirements.

In the approximate treatment as well as in those formulas accounting for ice-segregation, the magnitude or rate of ice-lense formation must be known or assumed. Once the increase in moisture in the ice-segregated layer is assumed, its effect may be accounted for in the depth of frost penetration computation by altering thermal properties of the layer. The change in thermal conductivity and volumetric heat resulting from an increase in moisture content will be small compared to the latent heat effect. Therefore, in computations based on the modified Berggren or Stefan formulas, the depth of frost penetration below the frost heaved pavement surface may be computed using a latent heat L based on the equivalent moisture content and dry density of the ice-segregated zone.

As an example, assume that frost penetrates into a frost-susceptible subgrade having a water content of 18 percent and dry density of 113 pcf in the unfrozen state. If the subgrade is assumed to expand 25 percent in thickness from ice lense growth during freezing, the equivalent moisture content is about 30 percent and dry density 95 pcf. In this state the latent heat is about 4000 Btu per cu ft compared to 3000 in the initial state. Thus, in the computation for the depth of frost penetration, L is assumed to be equal to 4000 Btu per cu ft in the subgrade subject to ice-segregation.

It is evident that significant ice-segregation reduces the maximum depth of frost penetration by increasing the latent heat term.

Freezing Point of Soil Moisture. Water normally freezes at 32 F. However, it has been clearly demonstrated (12) that the freezing point of soil moisture in fine-grained soils is generally below 32 F. The author has avoided the use of 32 F for the freezing point in order to maintain flexibility in allowing for a depressed freezing point. Thus, in the frost penetration prediction, the terms F, t, v_o and v_s in Equations 15 and 16 and in Figure 6 may be determined on the basis of any assumed or known freezing point of soil moisture.

In the prediction of frost penetration through granular soils below highway and airfield pavements the freezing point should be taken at 32 F.

Deviations From the One-Dimensional Assumption. The frost penetration prediction has been formulated as a one-dimensional problem. This assumption implies that ground surface temperature at a given time is everywhere uniform over the surface area and that no variation in soil conditions or subsurface temperature exists in a horizontal direction.

The one-dimensional assumption is more nearly satisfied in frost penetration below pavements than below most natural surfaces. This is especially true when the depth of frost penetration is small compared to the pavement width. Indeed, we could not question the assumption when considering frost penetration below airfield pavements.

Frost penetration below the edge of typical highway pavements is generally less than that below the center due largely to the insulating effects of snow on the shoulders. Thus, the one-dimensional assumption would be conservative, yielding depths slightly greater than the actual. One should not assume, however, that the frost problem is less serious below the edges. Ice-segregation, surface heave, and eventual subgrade weakening is frequently more pronounced at the pavement edge.

Preliminary Estimates of Frost Penetration. It is frequently necessary to obtain a quick estimate of the depth of frost penetration below a pavement surface before the data required in a comprehensive analysis are available. Further, we shall see that an analysis presented under the following heading for stratified soil requires in itself a first estimate of frost penetration depth. Thus, it seems appropriate to introduce the following results.

An empirical curve (Figure 9)[10] gives an approximate relationship between depth of frost penetration and air freezing index in non-frost susceptible soil beneath airfield pavements. Thus, the chart must be considered to apply only to pavement surfaces kept relatively free of snow and underlain by well-drained granular base-course materials. Figure 9 was determined from frost penetration data gathered during the winters 1944-45 and 1945-46 at airfields having both bituminous concrete and Portland-cement concrete pavements (14). The observed depths deviated a maximum of about 50 percent from the value given in Figure 9 while half the observations were within plus or minus 10 percent.

A preliminary estimate of the frost penetration depth below a proposed pavement surface may be obtained from Figure 9 if the air freezing index is known. The latter may be determined from weather bureau data (daily mean air temperatures) for the locality

[10] Figure 9 as well as Figures 10 and 11 are reproduced through the courtesy of the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers. They are published in References 13 and 14.

or estimated from a freezing index map such as that shown in Figure 10 which gives mean values.[11]

Frost Penetration Through Non-Uniform (Stratified) Soil. All practical analytic solutions for the depth of frost penetration are derived for uniform soil. The modified Berggren equation is no exception. Therefore, for the non-uniform or multilayer system which occurs typically below highway and airfield pavements, approximate computation techniques based on the mathematical solutions must be used. Since no exact solution for the multilayer case exists, there is no possibility of verifying, in a simple way, the reliability of the approximate procedure itself. It is true that a numerical approach or an analogue computer will give an "exact" solution. Some progress has been made in this direction (5), (9), and (15).

In computing the maximum depth of frost penetration below a typical highway or airfield pavement the following semi-empirical adaptation of the modified Berggren formula is suggested. We shall assume that the pavement profile is given as are values of water content and dry density for each layer. The layers making up the profile will be numbered consecutively downward starting with the Portland-cement concrete or bituminous concrete layer as No. 1.

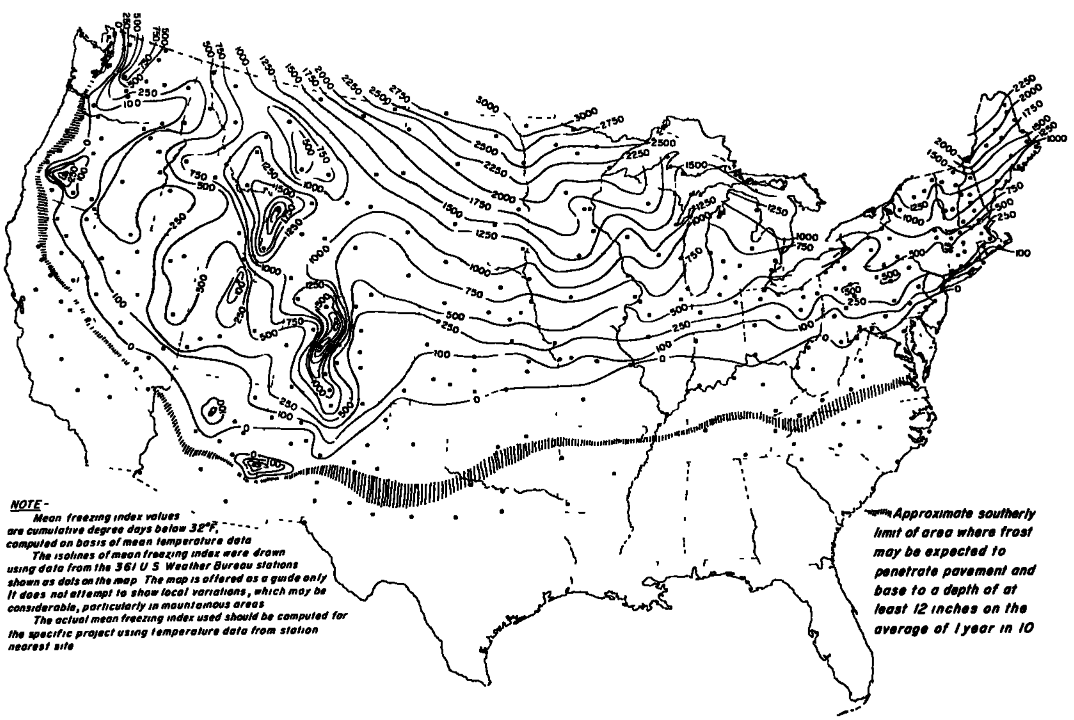

Figure 10. Distribution of mean freezing index values in continental United States. (Courtesy of the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers (13) and (14).

[11] Linell (13) has recommended the use in design of a one-year-in-ten freezing index rather than the mean, for example, the average of the three coldest years in a 30 year cycle. The one-year-in-ten index which differs little from the one-year-in-five, is approximately equal to the mean index plus 500 deg days. Linell points out that the freezing index to be used in design involves an economic balance between cumulative loss from frost damage over the life of the pavement and cost of protective measures to prevent frost damage.

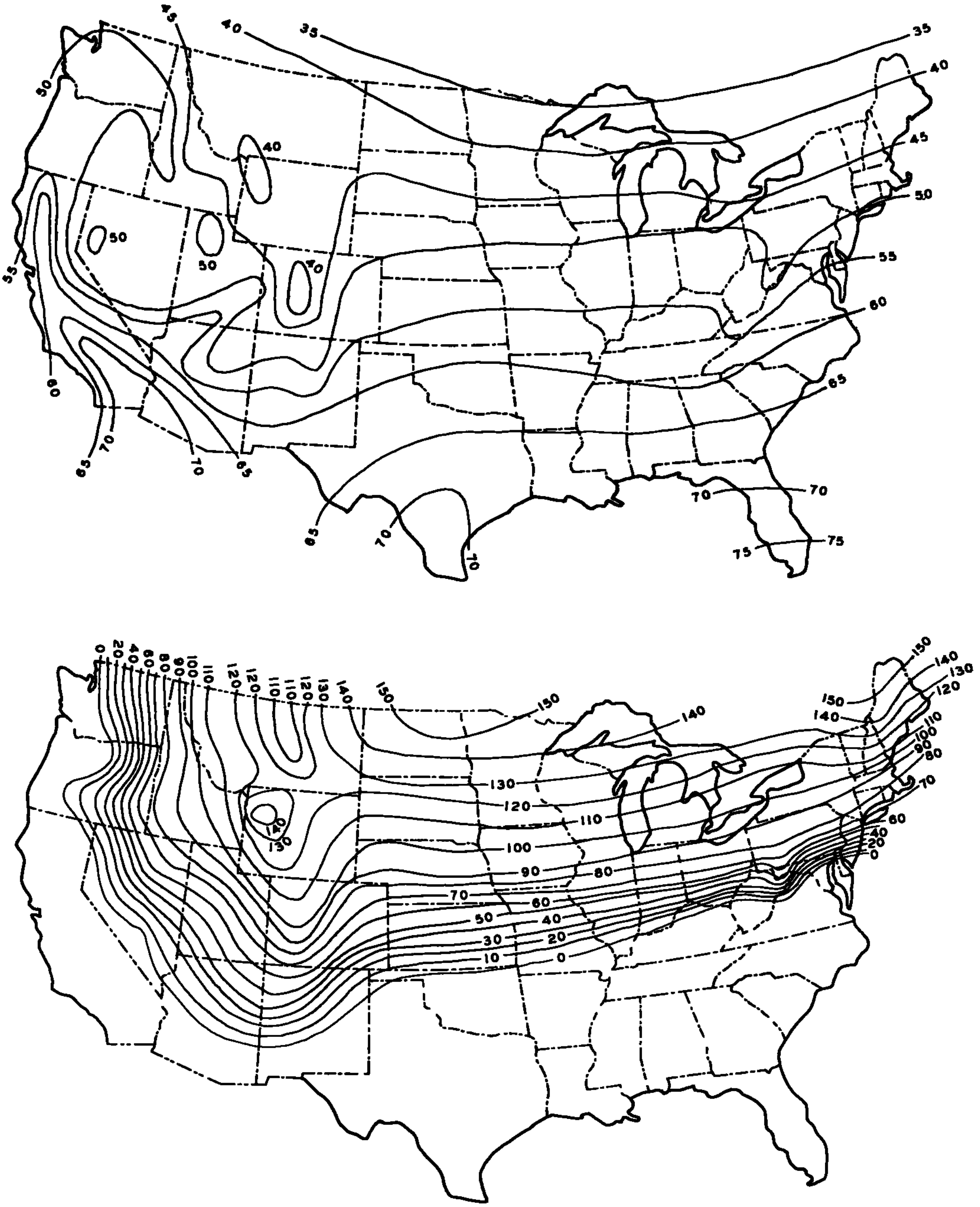

Figure 11. Mean annual air temperature, in degrees F (upper); and duration of normal freezing index, in days (lower). (Courtesy of the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers (13) and (14).

Step 1. Determine the pavement freezing index F, which may be conservatively assumed to be equal to the air freezing index.

Step 2. Determine the duration of the freezing period t, and the mean annual air temperature from weather bureau data for the locality or from maps such as those shown in Figure 11.

Then:

$$v_o = \text{mean annual temperature minus } 32$$

Step 3. From the given w and γ_d for each layer, determine the thermal properties k, C and L for each stratum within the estimated depth of frost penetration:

$$k = \frac{k_f + k_u}{2} \quad \text{from Figure 1 and Table 1}$$

$$C = \frac{C_f + C_u}{2} \quad \text{from Equation 1}$$

$$L = 1.434\ w\ \gamma_d$$

Step 4. Compute an effective $\frac{L}{k}$ from the following equation [12]:

$$\left(\frac{L}{k}\right)_{eff.} = \frac{2}{X^2}\left[\frac{d_1}{k_1}\left(\frac{L_1 d_1}{2} + L_2 d_2 + \ldots . L_n d_n\right) + \frac{d_2}{k_2}\left(\frac{L_2 d_2}{2} + L_3 d_3 + \ldots . L_n d_n\right) + \ldots\ldots + \frac{d_n}{k_n}\left(\frac{L_n d_n}{2}\right)\right]$$

where X is the estimated depth of frost penetration in feet and d is the thickness of a layer within the depth. Thus:

$$X = d_1 + d_2 + \ldots . d_n$$

Step 5. Compute weighted values of C and L within the estimated depth of frost penetration from:

$$C_{wt} = \frac{C_1 d_1 + C_2 d_2 + \ldots . C_n d_n}{X}$$

$$L_{wt} = \frac{L_1 d_1 + L_2 d_2 + \ldots . L_n d_n}{X}$$

Step 6. Compute the effective values of α and μ from Equation 16:

$$\alpha = \frac{v_o t}{F} \qquad \mu = \frac{C_{wt} F}{L_{wt} t}$$

Step 7. Determine the correction coefficient λ from Figure 5.
Step 8. Compute the depth of frost penetration from Equation 15:

$$X = \lambda \sqrt{\frac{48 F}{\left(\frac{L}{k}\right)_{eff}}}$$

If the computed depth differs appreciably from the assumed depth, steps 4 through 8 may be repeated. Seldom are more than two cycles necessary.

EXAMPLE

As an example of the foregoing procedure, we select the following pavement profile with given thermal properties:

[12] This equation may be derived from developments presented in Reference 5, pages 43 and 44. It is determined by considering the partial freezing indices required to freeze each layer of soil.

Given

Layer	k Btu per hr per ft per deg F	C Btu per cu ft per deg F	L Btu per cu ft
3 in. bit.concrete	0.8	28	0
6 in. base-course	1.00	23	850
21.5 in. subbase	1.30	25	1200
subgrade	1.70	27	2900

Mean annual temperature: 37 F (v_0 = 5 F)
Surface freezing index F: 1568 deg days F
Duration of freezing period t: 157.5 days

Solution

Results from Steps 1, 2 and 3 are given.
Step 4.
The estimated depth of frost penetration X, is 5.7 ft
Thus:

$$\left(\frac{L}{k}\right)_{eff} = \frac{2}{5.7^2}\left[\frac{0.25}{0.8}\left(0 + \overset{425}{(850)\ (0.5)} + \overset{2150}{(1200)\ (1.79)} + \overset{9160}{(2900)\ (3.16)}\right) + \frac{0.5}{1.00}\left(212 + 2150 + 9160\right) + \frac{1.79}{1.30}\left(1075 + 9160\right) + \frac{3.16}{1.70}\left(4580\right)\right]$$

$$\left(\frac{L}{k}\right)_{eff} = 1970$$

Step 5.

$$C_{wt} = \frac{(28)(0.25) + (23)(0.5) + (25)(1.79) + (27)(3.16)}{5.7} = 26.0$$

$$L_{wt} = \frac{0 + (850)(0.5) + (1200)(1.79) + (2900) + (3.16)}{5.7} = 2060$$

Step 6.

$$\alpha = \frac{v_0 t}{F} = \frac{(5)(157.5)}{1568} = 0.503$$

$$\mu = \frac{C_{wt} F}{L_{wt} t} \quad \frac{(26.0)(1568)}{(2060)(157.5)} = 0.126$$

Step 7.

From Figure 5, λ = 0.90

Step 8.

$$X = \lambda \sqrt{\frac{48 F}{\left(\frac{L}{k}\right)_{eff}}} = 0.90 \sqrt{\frac{(48)(1568)}{1970}}$$

$$X = 5.6 \text{ ft}$$

Nordal (15) obtained solutions to this problem with a hydraulic analogue computer assuming in one case a step change in surface temperature and in another case a surface temperature varying sinusoidally to accumulate a freezing index of 1568 deg days F in 157.5 days. Depth of frost penetration from these solutions are 5.7 ft and 5.8 ft respectively, which compare favorably with the semi-empirical computational procedure based on the modified Berggren formula. Other solutions obtained by Nordal also agree favorably with computed depths.

We now have then a computational procedure known to give reliable predictions for the depth of frost penetration if reliable data are given. Thus, it would appear that our ability to predict the actual depth of frost penetration below a given pavement depends primarily on the reliability of thermal properties and surface temperature used in the computation. The need for continued research on these factors and in other areas related to frost-action in soil are very ably presented by Johnson and Lovell (16).

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Appendix

DERIVATION OF A RATIONAL FORMULA FOR THE PREDICTION OF FROST PENETRATION

By Harl P. Aldrich, Jr. and
Henry M. Paynter

BASIC ASSUMPTIONS

A solution is sought to the thermal problem of a semi-infinite soil mass of uniform properties and at a uniform initial temperature subjected to changes of temperature at the surface, which are assumed uniform over the surface extent to yield a one-dimensional problem.

Figure 4 illustrates the nomenclature and significant variables in this situation. Further assumptions are best listed in the form of specific conditions as follows:

Condition I - At the Ground Surface

It is assumed that the surface temperature is suddenly changed from an initial temperature v_o above freezing to a temperature v_s below freezing. This temperature value is then maintained constant and uniform over the entire surface.

Condition II - In the Frozen Soil

It is assumed that the diffusion equation:

$$a_f \frac{\partial^2 v_f}{\partial x^2} = \frac{\partial v_f}{\partial t}$$

with $a_f = k_f / C_f$ measuring the diffusivity of the frozen soil is satisfied throughout the frozen soil mass, subject to the surface temperature condition (I) and the latent heat condition (III) at its boundaries.

Condition III - At the Moving Frost Interface

It is assumed that at the interface between the frozen soil (above) and the unfrozen soil (below) the temperature remains constant at the freezing point of the soil moisture. It is further assumed that the heat flow upward just above the interface in the frozen soil is equal to the sum of the heat flow just below the interface in the unfrozen soil plus the heat flow due to the removal of the latent heat of fusion of the soil moisture as it freezes.

Condition IV - In the Unfrozen Soil

It is assumed that the diffusion equation:

$$a_u \frac{\partial^2 v_u}{\partial x^2} = \frac{\partial v_u}{\partial t}$$

with $a_u = k_u / C_u$ measuring the diffusivity of the unfrozen soil is satisfied throughout the unfrozen region, subject to the latent heat condition (III) at the frost interface (the upper boundary of the unfrozen soil) and to the lower boundary condition that, at all times, the temperature approaches the initial temperature for sufficiently great depths.

GENERAL SOLUTION

The solution for the temperature in the frozen soil v_f which satisfied conditions I and II may be written:

$$v_f = -v_s + A \operatorname{erf}\left(\frac{x}{2\sqrt{a_f t}}\right) \qquad \text{(A-1)}$$

while the solution for v_u satisfying condition IV may be written:

$$v_u = v_o + B\left[1 - \operatorname{erf}\left(\frac{x}{2\sqrt{a_u t}}\right)\right] \tag{A-2}$$

At the frost interface where x = X, in order to satisfy condition III it is necessary that:

$$v_f = v_u = 0 \tag{A-3}$$

from which:

$$A \operatorname{erf}\left(\frac{X}{2\sqrt{a_f t}}\right) = v_s \tag{A-4}$$

$$B\left[1 - \operatorname{erf}\left(\frac{X}{2\sqrt{a_u t}}\right)\right] = - v_o \tag{A-5}$$

Since these last conditions must be satisfied for all values of time,

$$\frac{X}{\sqrt{t}} = \text{constant, } \gamma \tag{A-6}$$

or

$$X = \gamma \sqrt{t} \tag{A-7}$$

Thus, the constant γ depends upon v_s, v_o, and the thermal coefficients. The manner of this dependence may be found by considering the latent heat requirement of condition III. This thermal continuity condition relates the temperature gradients each side of the interface to the rate of movement of the interface in the form:

$$L\frac{dX}{dt} = k_f \frac{\partial v_f}{\partial x} = k_u \frac{\partial v_u}{\partial x} \tag{A-8}$$

which becomes, upon substituting Equations A-4, A-5, and A-7, performing appropriate differentiation, and simplifying:

$$\frac{v_s k_f}{\sqrt{\pi a_f}} \cdot \frac{e^{\frac{\gamma^2}{4a_f}}}{\operatorname{erf}\left(\frac{\gamma}{2\sqrt{a_f}}\right)} - \frac{v_o k_u}{\sqrt{\pi a_u}} \cdot \frac{e^{-\frac{\gamma^2}{4a_u}}}{\left[1 - \operatorname{erf}\left(\frac{\gamma}{2\sqrt{a_u}}\right)\right]} = \frac{L}{2}\gamma \tag{A-9}$$

Furthermore, by making the substitutions:

$$\alpha \equiv \frac{v_o C_u}{v_s C_f} \tag{A-10}$$

$$\delta \equiv \sqrt{\frac{a_f}{a_u}} \tag{A-11}$$

$$\mu \equiv \frac{v_s C_f}{L} \tag{A-12}$$

$$Z = \frac{\gamma}{2\sqrt{a_f}} \tag{A-13}$$

it is possible to rewrite Equation A-9 in a simplified non-dimensional form; namely,

$$\mu\left[\frac{e^{-Z^2}}{\operatorname{erf} Z} - \frac{\alpha}{\delta}\,\frac{e^{-\delta^2 Z^2}}{(1 - \operatorname{erf} \delta Z)}\right] = \sqrt{\pi}\, Z \tag{A-14}$$

If all the terms in Z are then carried to the right-hand side, one has a direct relation between μ , α , δ , and Z in the form:

$$\mu = \frac{\sqrt{\pi}\; Z}{\left[\frac{e^{-Z^2}}{\operatorname{erf} Z} - \frac{\alpha}{\delta} \frac{e^{-\delta^2 Z^2}}{(1 - \operatorname{erf} \delta Z)} \right]} \tag{A-15}$$

This last may then be inverted to obtain a transcendental relation for Z in terms of the parameters α , δ , and μ, which may be solved graphically to give, in symbolic form:

$$Z = \int (\alpha , \delta , \mu) \tag{A-16}$$

From Equation A-13, one may obtain

$$\gamma = 2\sqrt{a_f}\; Z \tag{A-17}$$

and combining with Equation A-7 for the frost penetration depth:

$$X = \gamma \sqrt{t} = 2\, Z \sqrt{a_f\, t} \tag{A-18}$$

It is found convenient to rearrange this last expression so that the radical is expressed in terms of μ in the form:

$$X = \frac{2\; Z}{\sqrt{2\,\mu}} \sqrt{2\,\mu}\, \sqrt{a_f\, t} \tag{A-19}$$

By defining the new dimensionless correction coefficient λ as :

$$\lambda \equiv \sqrt{\frac{2\; Z^2}{\mu}} \tag{A-20}$$

and by substituting under the radical in Equation A-19 for the physical variables, one finally obtains for X, the general expression:

$$X \quad \lambda \sqrt{\frac{2\, k_f\, v_s\, t}{L}} \tag{A-21}$$

where the correction coefficient λ is a function of the three dimensionless parameters α , μ , δ .

PHYSICAL SIGNIFICANCE OF PARAMETERS

(a) Thermal Ratio (α): The thermal ratio α , defined as:

$$\alpha = \frac{v_o\, C_u}{v_s\, C_f} \tag{A-22}$$

measures the ratio of heat stored initially in the unfrozen soil to the heat loss in the frozen soil. If it may be assumed, as with many existing formulas, that the difference between the volumetric heats C_u and C_f is not usually significant, the thermal ratio α may be written:

$$\alpha = \frac{v_o}{v_s} \tag{A-23}$$

which is the ratio of the initial ground temperature above the freezing point to the as sumed constant surface temperature below freezing during the freezing period.

(b) Diffusivity Ratio (δ): The (root) diffusivity ratio δ , defined as:

$$\delta = \sqrt{\frac{a_f}{a_u}} \tag{A-24}$$

measures the relative values of diffusivity $a = k/C$ in the frozen and unfrozen soils. It is clear that for most soils of low moisture content δ is approximately unity. Tabulated herewith are typical values of δ for representative soil types and moisture contents.

TYPICAL VALUES OF THE DIFFUSIVITY RATIO[1]

$$\delta = \sqrt{\frac{k_f C_u}{k_u C_f}}$$

Assumed Dry Density = 110 pcf

Moisture Content	Values of δ	
	Silt or Clay	Sand
%		
0	1.00	1.00
5	1 07	0.92
10	1.15	1.14
15	1.22	1.31
20	1.30	1.50

The effect of variations in δ may be found directly from the λ, α, μ graph, Figure 5, by making use of an equivalent α -value given by the expression:

$$\alpha_\delta = \frac{\alpha}{\delta}\left[\frac{1 - \operatorname{erf} \lambda \sqrt{\frac{\mu}{2}}}{1 - \operatorname{erf} \delta \lambda \sqrt{\frac{\mu}{2}}}\right] e^{-(\delta^2 - 1)\frac{\lambda^2 \mu}{2}} \tag{A-25}$$

which can be approximated by the empirical equation:

$$\alpha_\delta \approx \frac{\alpha}{\delta}\left[1 + \frac{\delta - 1}{2\sqrt{1 + \alpha}}\right] \tag{A-26}$$

A comparison of exact values of λ determined from Equation A-15 and A-20 with estimated values using Figure 5 and Equation A-26 shows only negligible differences for all practical values of δ.

The following table indicates the relationship between α, δ and γ_δ as given by Equation A-26.

EQUIVALENT THERMAL RATIO (α_δ)

$$\text{Values of } \alpha_\delta = \frac{\alpha}{\delta}\left[1 + \frac{\delta - 1}{2\sqrt{1 + \alpha}}\right]$$

Values of	Values of δ						
α	0.9	1.0	1.1	1.2	1.3	1.4	1.5
0	0	0	0	0	0	0	0
1	1.07	1.00	0.94	0.89	0.85	0.82	0.79
2	2.16	2.00	1.87	1.76	1.67	1.59	1.53
3	3.25	3.00	2.80	2.62	2.48	2.36	2.25
4	4.35	4.00	3.72	3.47	3.28	3.11	2.97

[1] Source of data for k and C: "Final Report, Laboratory Research for the Determination of the Thermal Properties of Soils", Corps of Engineers, St. Paul District, June 1949.

Since it has been demonstrated that a 1 percent error in water content produces only about a 1.5 percent change in the value of δ, the above tabulation in conjunction with Figure 5 would indicate less than a 1 percent change in the computed depth of penetration due to this variation. Moreover, typical soils have values of δ of the order of 1.15, which would indicate an effective value of α_δ which is roughly 15 percent smaller than that computed assuming $\delta = 1.0$, yet this effect can produce a variation in λ and therefore a change in predicted depth of less than 5 percent for typical thermal conditions.

Since inclusion of a term involving δ would increase the complexity of representation of the formula in graphical form, and would further increase the tendency to predict depths of penetration which are too deep, it seems reasonable to base at least preliminary calculations on the assumption that $\delta = 1.0$.

Consistent, then, with the previous assumption that $C=C_u=C_f$ and $\delta = \sqrt{\left(\frac{k_f}{k_u}\right)\left(\frac{C_u}{C_f}\right)} = 1.0$, there would follow then the necessary condition that $k = k_f = k_u$. In practical application, this would suggest the use of average values of C and k.

(c) Fusion Parameter (μ): The fusion parameter μ, defined as:

$$\mu = \frac{v_s \, C_f}{L} \qquad \text{(A-27)}$$

measures the heat removed in the frozen soil (below the freezing point) compared to the latent heat of the soil moisture.

When $\mu = 0$, the only significant soil properties affecting the depth of frost penetration are the latent heat L, and the thermal conductivity, k_f; on the other hand, as μ becomes large, the stored heat in the soil volume becomes proportionately, more significant.

(d) Correction Coefficient (λ): From Equation A-21, it can be seen that the correction coefficient λ is a correction to the calculated depth of frost penetration accounting for latent heat only.

The value of λ may be found from Figure 5 which shows the relationship between α, μ and λ for the case of $\delta = 1$. Data for this figure were obtained by assuming values of z and solving Equation A-15 for μ. Once μ was obtained λ followed directly from Equation A-20.

The Soil Freezing Experiment

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The purpose of this article is to present in as convenient and simple a form as possible a discussion of fundamental concepts and theories as a basis for frost action research, particularly with reference to the phenomenon of the upward flow of moisture in soil in the soil freezing experiment. A pilot soil freezing experiment relative to suction measurement is described and its results presented.

The improvised apparatus used in the freezing experiment demonstrates vividly where and how the ground-water is transferred and convinces one that upon freezing of the soil sample the ground-water is sucked up vertically into the soil.

Subpressure or suction values, as obtained analytically by means of the hydrodynamic and thermodynamic theories, are compared with those obtained experimentally.

Although much research work is still to be done along these lines, it is hoped, however, that this presentation will partially fill the existing gap in the present literature in respect to methodological organization and help to range this subject into a discipline supported on a theoretical basis.

● THE paving of the relatively narrow and yet expensive highway and railway ribbon, as well as that of airports, has a comparatively shallow foundation as contrasted with larger and heavier structures, the foundations of which are usually set well below the frost penetration depth line. This fact, as well as the fact that they are fully exposed to the weather makes their pavements especially vulnerable to climatic influences. Earthwork soils in particular are subjected to periodic temperature and moisture variations. For example, frost action may cause differential heaves on roads, but variation in soil moisture content as a consequence of frost action may affect the strength of the soil, particularly during a thawing period after frost has left the ground. This causes loss of bearing capacity of soil and so-called "spring breakups" on roads built on and of improper soil. The resulting affect of such conditions usually is damage to roads and their pavements. Thus it can be inferred that frost action imposes difficulties in design, construction, exploitation, and maintenance of highways. It also impairs traffic safety. In addition, repairs of roads damaged by frost usually cost huge sums.

It becomes necessary, therefore, in highway and airport engineering, to estimate, among other factors, the frost penetration depth in soil. By a correct estimate of this penetration, it is possible to provide either proper insulation or adequate drainage courses underneath the pavement to take care of the thawing waters, and to determine the necessary amount by which the ground-water table should be lowered. If the ground-water table is sufficiently low, upon freezing the suction height of the upward-sucked soil moisture from the ground-water will not be intercepted by the depth of the maximum frost penetration which is known to prevail in the region under consideration. Hence, we conclude that data on frost penetration and upward suction of soil moisture, which contributes to the thickening and to the downward progress of the frozen layer of soil along with soil properties, are useful in arriving at adequate design of highways and and runways.

A sketch illustrating the concept of unidimensional upward flow of soil moisture toward the frost boundary (or ice lenses) upon freezing is shown in Figure 1 (5).

Unfortunately the interrelationship of the various factors involved in damaging roads is very complex. A variation in any one of the factors influences to a greater or lesser extent the others, the properties of the soil, as well as the whole thermal system "soil-moisture-temperature." Soil, because of the variation in properties effectuated by moisture variations in it, is a difficult engineering material to deal with and to study. It is nonhomogenous, possessing many properties. Such studies become particularly difficult when soil is subjected to temperature potentials, temperature cycles, and moisture transfer.

Because in the freezing process the soil properties vary with the variation in its moisture content which, in turn, is associated with temperature variations, the study

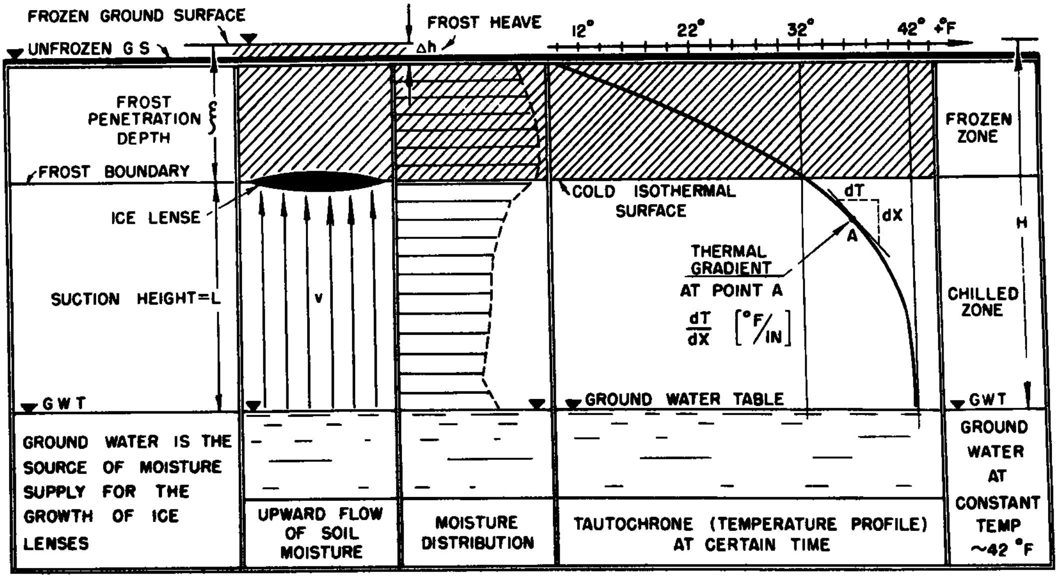

Figure 1. Sketch illustrating the concept of unidimensional upward flow of soil moisture toward the frost boundary (or ice lenses) upon freezing. Open system.

of frost penetration and moisture migration upon freezing and thawing in soil is simultaneously a geotechnical and heat transfer problem, a problem of considerable difficulty. This explains why soil studies under thermal conditions have in general never been highly popular or attractive. One of the reasons is that the process of heat transfer takes place slowly. Also, there are no convenient, simple, and handy laboratory means available for experimentation in order to confine the flow of heat and the migrating moisture along some well defined path. Consequently, little progress was made in arriving at more effective methods for evaluating the thermal system "soil-moisture-temperature." Researchers have avoided such studies simply because they are cumbersome, involved and unattractive.

However, because of the economic and national importance of the performance of highways under freeze-thaw conditions and in order to understand better the complex freezing process in soil where temperature differences, heat transfer and moisture migration are involved, it is necessary to familiarize and refresh ones thinking with some basic concepts pertaining to this subject. It is believed that familiarity with these concepts will be a great aid in methodological organization of the subject as well as in supporting a discipline to be built on basic knowledge already available.

GENERAL CONCEPTS

In the study of the complex frost problems associated with soil moisture migration, it is necessary to establish, as in any other field of physics and research, a so-called working system. The latter aids one to avoid some difficulties that may arise in research work and helps to develop the subject. A working system can be considered as a useful means by which to organize and orient the subject and the set of ideas, essential principles, and facts within a certain frame of general concepts. The following is a short description of some of the general concepts.

System. A system is generally understood to be the quantity of matter under consideration. More specifically, a system is a separated region of space or a finite part of matter set apart from its surroundings. Attention is focused on the system; in the system, changes in the state of matter and transfer of energy and/or mass can be studied. When energy transfer between the system and its surroundings takes place only under the influence of temperature difference, the transferred energy is called heat. As known from physics, heat possesses only one measurable property, temperature.

In the soil freezing experiment an effort was made to simulate field conditions as nearly as possible. For example, a vertically supported cylindrical soil sample (imagined to be a part of the soil in the ground) with a simulated ground-water table distant H below the ground surface is frozen from the top downward (Figure 1). This means that in the soil sample, upon freezing, changes in temperature, moisture content, phase (water is converted to ice), and volume (frost heaves) take place. The changes in the existing factors are usually effectuated by a temperature potential as a driving force in energy (heat) and mass (water) transfer. All these changes taking place in a physical body of matter under consideration are to be included in the region of study. Hence, a region where transfer of heat and soil moisture can be studied is defined as a system. In this sense, the concept of a system in thermal soil mechanics can be compared with the free body diagram as it is utilized in analyzing problems in technical mechanics, for example, statics (7), or it is analogous to the concept of "system" as used in studies of thermodynamics (6). Because of its nature, the particular system in the soil freezing experiment and its prototype in the field can be termed "soil-moisture-temperature."

The concept of a system is illustrated in Figure 2. This is an open system, that is, one where water can enter from below from the ground-water freely, and, after thawing of the frozen soil, leave it again. Also, energy (heat) can cross the lower and upper boundaries of the system. Thus exchange of energy and mass is possible with the surroundings.

Surroundings. The region outside the thermal system is called the surroundings. In Figure 2, the system is shown surrounded laterally by an insulating substance (impregnated cardboard tube and vermiculite) through which, it is assumed, no heat or moisture flow. However, internal energy in the form of heat can be transferred to the surroundings vertically across the horizontal unfrozen and frozen boundaries of the system. The prerequisite for the transmission of heat, in turn, is the temperature difference or potential between the system and its surroundings. A potential is popularly called a "driving force" causing changes in the state of a system. Hence, proper temperatures, as one of the several possible potentials, are one of the necessary factors in the freezing process.

Process. In a broad sense, a process is any event in nature in which a redistribution or transformation of energy occurs (1). Whenever a system undergoes a physical change of any kind from one state to another, this change is termed a physical process. In thermodynamics, a distinction is made between two kinds of processes, namely, reversible and irreversible ones.

According to Weber (10) a reversible process is defined as one where, at any stage, a differential decrease in the driving force causes the process to proceed in the opposite direction. After a reversible change, both the system and surroundings may attain their

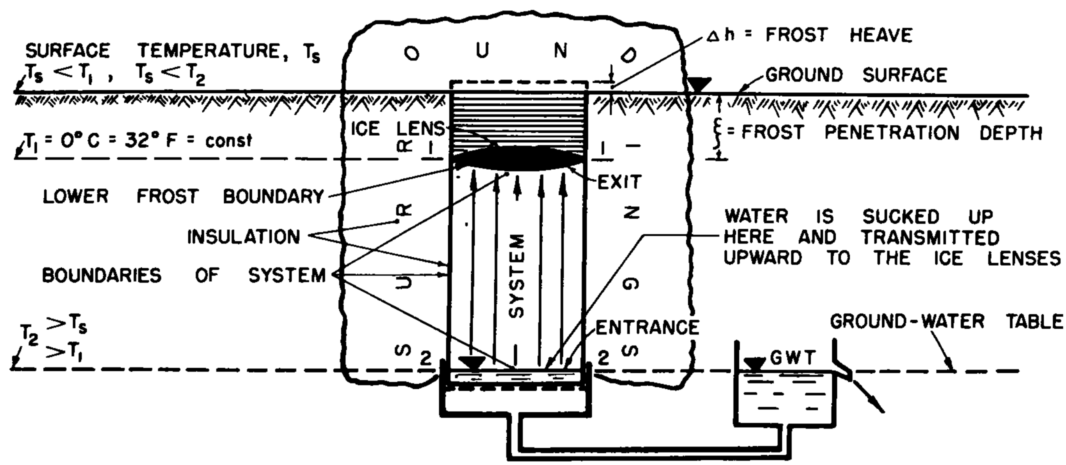

Figure 2. System, boundaries and surroundings.

original condition. An irreversible process is defined as one in which the system and surroundings can never be completely restored.

Aside from the concepts of system and surroundings, it is advantageous to attach to the system certain simple and idealized properties. For example, it might be assumed that the matter of the system is a uniform and homogeneous soil through the moisture films of which a perfect and easily mobile soil moisture flows, and through the soil of which, upon freezing the top surface of the soil sample, heat and moisture are transferred upward from a warmer region (ground-water table) to a colder one (lower boundary of the freezing ice lenses). The freezing lenses are connected with the ground-water table by a communicating system of soil-moisture films (Figure 3), and moisture is moved upwards against gravity (3).

Hence, the processes of moisture and heat flow within this thermal system are of the nature of soil mechanics, hydrodynamics, heat transfer and thermodynamics. From this discussion one realizes the complexity of the freezing process associated with heat and moisture transfer in a porous medium such as soil. Here heat is transferred by the soil particles in contact as well as by the moisture flowing upward through moisture films toward the freezing ice lenses. Very little is known as to what is the proportion of heat transfer through the soil particles as compared with that through the soil moisture. In addition, other factors must be taken into account, such as variation in water affinity to soil and changes in the viscosity of water effected by temperature variations, the amount of specific surfaces of soil, the various types and sizes of the constituent parts of the soil particles, as well as the soil void ratio. All these and other factors are to be recognized as constituting some of the difficulties in confining the flow of heat and moisture in a porous medium and thermal system like soil along or within some well-defined paths or channels. Therefore, certain assumptions are necessary to simplify studies.

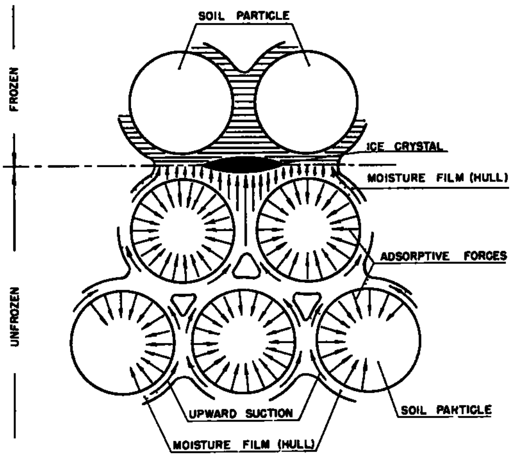

Figure 3. Sketch illustrating the concept of the upward flow of soil moisture toward an ice crystal.

Assumptions. From the previous discussion one gathers that the system in the soil-freezing experiment is by no means a simple one. It is a complex, multiple-component system where heat transfer is associated with the transfer of soil moisture in a porous medium. Therefore, in order to study physical processes and to obtain a general insight into them, it is necessary, as in almost every other branch of knowledge, to simplify the actual process considerably. Factors which are of minor importance are usually eliminated, and attention is focused on only the major phenomenon in the particular process. For these and other reasons, as well as for constructing a soil freezing apparatus, certain assumptions are to be made at the outset for agreement on certain facts.

In making assumptions, attention is focused on the interior of the system.

1. It is assumed that in the system under consideration there is a ground-water source present at a certain distance below the ground or pavement surface.

2. The freezing ice lenses are connected freely with the ground-water table, by means of moisture films which are adsorbed to the soil particles.

3. The temperature conditions in soil are such that the moisture films are uninterrupted.

4. Upon the application of a freezing temperature gradient, a suction within the moisture films is inaugurated, causing an upward flow of soil moisture in the liquid phase toward the forming ice lenses.

5. For reasons of simplicity, a laminar, unidimensional upward flow of soil moisture and heat transfer is assumed. The transference of mass and energy takes place

upward unidimensionally and parallel to the longitudinal, vertical axis of the cylindrical system. Because of the large areal extent of the soil in the field, it is further assumed that no lateral transfer of heat and moisture takes place. In the laboratory, the upward direction of the flow, it is believed, can be achieved by isolating the soil system from its surroundings with insulation.

6. The system under consideration is an open system, that is, one such as is found in the field where additional moisture can enter and be sucked up vertically into an imaginary vertical soil cylinder as a result of freezing.

7. The film moisture and the soil particles are the medium between the ground-water table (entrance) at a temperature above freezing and the forming ice lenses (exit) at freezing temperature, and serves as a means of free communication for the transfer of energy.

8. No "eddies" or turbulence in the process of transfer of soil moisture and heat through the system take place.

9. The vapor movement in the soil is negligible, and therefore ignored.

10. The moisture transfer between the soil and the air is negligible.

11. The temperature underneath the ice lenses is regulated by the supply of cold from above, and the temperature of the upward flow of soil moisture from below. However, for simplicity, it is assumed that moisture in soil freezes at a theoretical freezing point, namely, 0 deg C = 32 deg F. This is done because the true freezing in soil cannot satisfactorily be determined. Factors which may affect the freezing point of soil moisture, such as salinity, are ignored.

These assumptions for the approximation of the problem oversimplify the actual process considerably. However, they have the advantage of being explicit, and can be justified as a means of approach to a better understanding of the processes partaking in the soil freezing experiment.

THEORETICAL BASIS

In studies of processes associated with freezing of soil, both kinds of approaches can be employed, namely, theoretical and experimental.

In theoretical studies, the hydrodynamic (also known as the suction force theory) and the thermodynamic methods usually are considered. Although different in nature, both methods of study, however, have one common feature. This is the fact that it is possible by means of them to calculate the magnitude of the subpressure or suction necessary to cause, upon freezing, the upward flow of soil moisture from ground-water to the growing ice lenses. It is simply a question as to how these suction values compare with each other.

Hydrodynamic Considerations. The suction force theory based on hydrodynamic considerations (8, 4) gives the following expression for the calculation of subpressure:

$$P_s = \frac{(\gamma_w).(\Delta h).(H - \xi)}{(1.09).(k_s).(t)}, \qquad (1)$$

where

P_s = subpressure or suction,
γ_w = unit weight of water,
Δh = allowable amount of frost heave relative to a certain riding length,
H = position or distance of the ground-water table below ground or road surface,
ξ = frost penetration depth,
$H - \xi = L$ = suction length,
1.09 = coefficient to take care of the expansion of water by an amount of 9 percent upon freezing,
k_s = suction coefficient of upward-sucked soil moisture, and
t = duration of freezing period in the field or freezing experiment.

By suction is understood the maximum possible subpressure, P_s, to which the pore moisture of the soil, upon freezing, is subjected in order to cause an upward flow towards the ice lenses.

In this theory the type of experimental system is a hydraulic one. The potential inaugurating the freezing process is temperature. A hydraulic gradient is present, causing a current flow of soil moisture, the current density of which is a volume of moisture flowing through a unit area in a unit of time and having for its conductivity a volume of moisture flowing through a unit area in a unit of time under a unit pressure gradient.

Thermodynamic Considerations. The study of a simplified thermal system "soil-moisture-temperature" can be based also on thermodynamic considerations. In the discipline of thermodynamics, there are two ways by which a thermal system may interact with its surroundings; namely, they are by the performance of work, and by heat transfer. As shown by Dr. Winterkorn, the difference in the amount of heat of a certain volume of water is equal to the amount of (heat) energy transferred in the upward suction process from a region of higher temperature (ground-water) to that of lower temperature (forming ice lenses). According to the second law of thermodynamics, the free energy or maximum theoretical work available in an ideal, mechanically reversible process, where the heat is being exchanged at two constant temperature levels only, can be calculated as follows (9, 11):

$$W_{max} = P_{max} \cdot V = Q \cdot \frac{T_2 - T_1}{T_2}, \tag{2}$$

where

$P_{max} \cdot V$ = work of the steady flow process (or displacement energy),
P_{max} = maximum absolute pressure difference,
V = specific volume,
W_{max} = maximum work available or free energy that can be obtained from conversion of heat,
Q = total amount of heat transferred from a temperature level of T_2 (ground-water temperature) to a temperature level of T_1 (temperature of freezing ice lenses).

This amount of heat consists of latent heat of fusion of water, Q_L, and the difference in the amount of heat of the transferred matter at the two boundary temperatures without change in phase, Q_P. Because Q_L is relatively large as compared with Q_P, the latter is omitted from the derivation of the subpressure or suction function.

Assuming that V = 1 cm; that the unit weight of water is $\gamma_W = 1$ g/cm^3; that 1 cm^3 of water and 1 cm^3 of ice weigh approximately lg; that Q_L = 80 calories, and that 1 calorie = 42,700 g-cm = (4.27) · (10^4) g-cm, the maximum theoretical subpressure, P_{max}, or the difference in pressure due to the work function between the ground-water table and the forming ice lense (for finite difference in temperature between the temperature levels T_2 and T_1 or constant temperature gradients with depth) is

$$P_{max} = \frac{Q_L}{V}\left(1 - \frac{T_1}{T_2}\right), \tag{3}$$

or

$$P_{max} = 3.42 \times 10^6 \left(1 - \frac{T_1}{T_2}\right) \left(\frac{g}{cm^2}\right) \tag{4}$$

The term

$$Q_L \frac{T_1}{T_2}$$

is the amount of heat unavailable for producing work.

When the temperature gradient varies as the depth coordinate in soil increases, then for the calculation of subpressure for a differential process the following differential equation is in order

$$dW = Q_L \frac{dT}{T}, \tag{5}$$

or

$$P_{max} = \frac{W_{max}}{V} = 3.42 \times 10^6 \ \ln\left(\frac{T_1}{T_2}\right) \left(\frac{g}{cm^2}\right) \tag{6}$$

In the thermodynamic theory in the suction process the acting potential is temperature. The conductivity is the thermal conductivity. The temperatures operated within this theory are in the absolute thermodynamic temperature system, in Kelvin degrees. The second law of thermodynamics applies to every case of practical importance where heat is converted to work.

Both theories, the hydrodynamic as well as the thermodynamic, presuppose that at the entrance of the system all properties which fix the state of the fluid maintain fixed values, that is, they do not vary with respect to time.

This presupposition concerning the ground-water in soil or the simulated "ground-water" in the experimental device can be approved, particularly relative to temperatures. Ground-water temperature measurements by the author through several winters showed that the temperatures vary from 6 deg C to 10 deg C, the average of which, 8 deg C, for a whole freezing season can be considered as constant. Hence, the viscosity of water can practically be considered as constant also. Thus during the laboratory soil freezing experiment it is possible to maintain the state, at the entrance of the system, at constant or fixed values.

Furthermore, in these theories, the conditions should be such that at the exit (ice lens) from the system fluid properties and velocity do not vary. When moisture molecules have reached the forming ice lens and freeze, the velocity of the upward flow of moisture is zero. Thus, the velocity condition can be considered as satisfied. However, the condition that fluid properties should not vary is not satisfied because just at the exit of the system (ice lens) water upon freezing is converted into ice – in other words, a change in phase takes place accompanied by changes in properties. However, assuming the simplest conditions, in other words, the properties just before converting the water into ice, this condition also can practically be considered as satisfied. It is to be noted, however, that the exit properties, because of the complex porous system, may be and usually are quite different from the entrance properties. Between the two points of reference, the properties might even be unsteady. The latter point becomes particularly obvious when we observe the work-energy Equation 2. This equation merely fixes the entrance and exit temperatures. It says nothing about what happens and how the process takes place within the system between its entrance and exit. This fact can be considered a disadvantage. On the other hand, it also can be considered an advantage, as it gives the final effect of the system, masking out processes between entrance and exit.

Theory, in general, also requires that the flow of soil moisture at the exit must be equal to the flow at the entrance (condition of continuity of flow). This requirement is satisfactorily fulfilled in the hydrodynamic or suction force theory (3) and indirectly in the thermodynamic theory. The latter, however, does not consider the resistance to flow of soil moisture through the system. It treats the process with a 100 percent efficiency and gives maximum possible suction values for every type of material and length of duration, which does not correspond to what can be observed in nature.

In the hydrodynamic theory the resistance to flow of moisture is reflected in the suction coefficient, k_s. It does not, however, consider temperatures directly. Their effect is masked out. Indirectly they are reflected in the suction process itself, as well as in the suction coefficient, and the amount of frost heave.

Because the thermal system "soil-moisture-temperature" in nature does not work with 100 percent efficiency, it is a practical necessity to ascertain the subpressure or suction values in soil upon freezing experimentally.

VALUE OF EXPERIMENTS

Studies of a complex system like that of "soil-moisture-temperature" can, according to the author's belief, be readily and most effectively studied experimentally in the laboratory on a small scale. The purpose of a small scale soil freezing experiment is (1) to gain a better understanding and knowledge of the freezing phenomenon, its process and its resulting effect; (2) to try, through observation, to explain the nature of frost penetration into soil with its associated heat and moisture transmission, with particular reference to the measurements of suction values of soil moisture; (3) to establish the

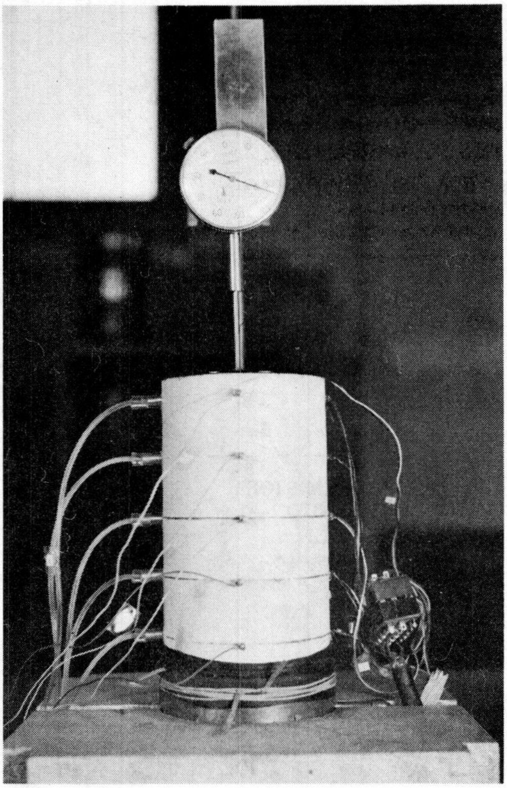

Figure 4. Photograph of the improvised soil freezing apparatus.

frost penetration problem in soil on a somewhat satisfactory scientific basis, and (4) to obtain qualitative experimental data for checking the theories and derivations of physical laws pertaining to this subject, as well as for calculating frost penetration depths in highway soils.

Because almost all physical laws are derived or obtained from experiment, so it is hoped that small scale soil freezing experiments might lead to a formulation of some physical relationships in this discipline.

Small scale laboratory research has the following advantages over full or natural scale field studies: (1) easy handling of experimental apparatus; (2) less expense than incurred in field studies; (3) independence from the mercy of the climate; (4) simulation and reproduction of freezing processes in soil in the frost research laboratory at any time, of any duration and variation; (5) simplification and reduction of a complex system into a less complex one; (6) elimination of extraneous factors and concentration on the elements of interest under investigation; (7) close observation of the processes; and (8) making of exact measurements, thus obtaining reliable data for evaluation and correlation.

After the small scale research is over, the findings can be checked in the field.

THE SUCTION PROCESS OF THE SOIL MOISTURE

During freezing the suction process of soil moisture takes place at a slow rate, and does not attain a state of equilibrium. Although the suction process is slow, a considerable amount of soil moisture can be transmitted during a relatively long period of time. However, it is the slow process of flow which often is overlooked and forgotten. This is the main factor where the danger of damage to roads and runways lies. The suction process continues until all of the soil moisture is consumed, or until the constantly freezing soil layer has grown in thickness and reached the ground-water table, or when the freezing process is checked by an increase in temperature, that is, the moisture in the soil redistributes and a new state of suction equilibrium is again attained. Upon the application of a new freezing thermal gradient, the state of equilibrium is interrupted, and upward flow of soil moisture towards the cold isothermal surface starts again.

When the soil moisture reaches the cold isothermal surface the film water is converted to ice, and gives up heat. The definite amount of heat which is released in the freezing process without change in temperature is called the latent heat of fusion of water. The magnitude of such a heat is 80 calories for one gram of water.

The attachment of the water molecules to the ice crystals induces suction in the soil moisture films. Hence, the lower, cold isothermal surface of the frozen layer (exit of the system) can be assumed to be an acceptor of the prevailing subpressure in the moisture films. Therefore, the subpressures or suction in soil freezing experiments are to be measured at the downwards progressing, ice-forming isothermal boundary.

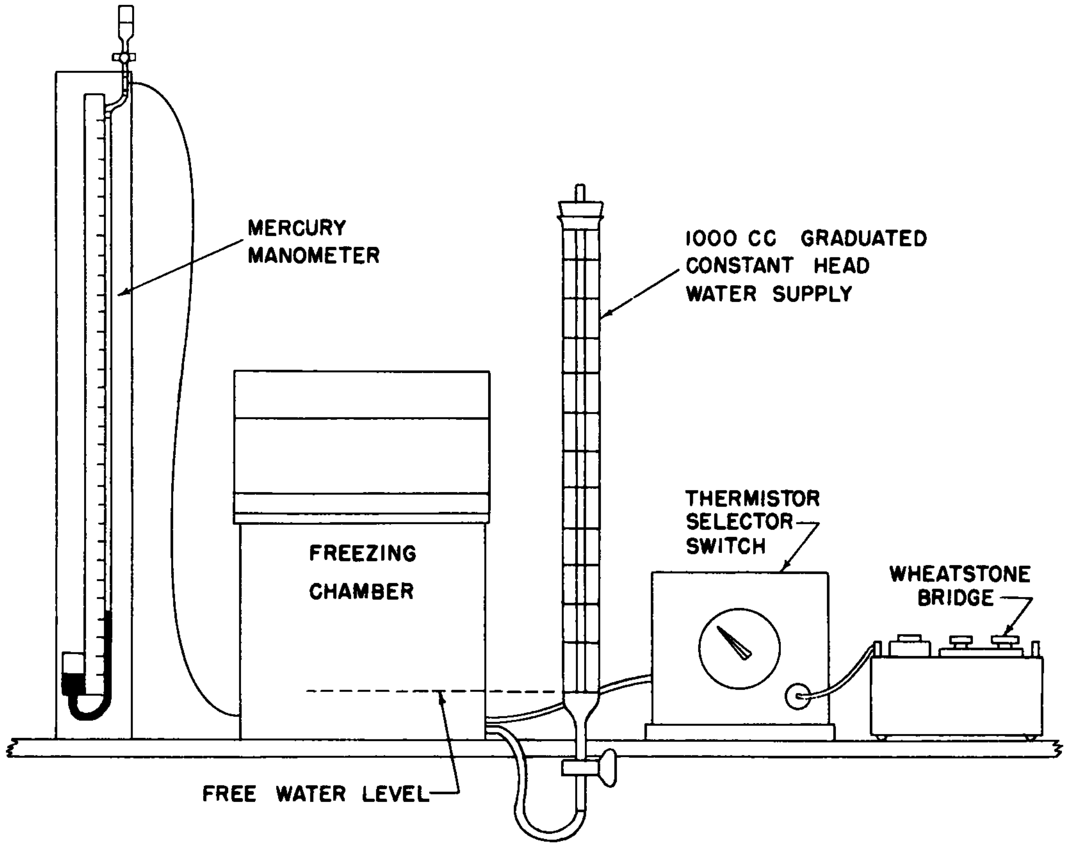

Figure 5. Apparatus used for freezing test.

THE SOIL FREEZING EXPERIMENT

General Notes. The processes in a soil freezing experiment correspond very nearly to those occurring in nature. Under the action of freezing temperatures the soil and soil moisture freeze. The frost penetrates the soil from the ground surface downward. Also, volume changes in the soil sample in the form of frost heaves are produced in the freezing process.

First, water freezes in the larger voids of the soil. The nuclei of ice crystals so formed start to grow, sucking up soil moisture from the surrounding soil and from ground-water or proximity of other water sources until the soil underneath the cold isothermal boundary of ice lenses freezes. This frozen layer of soil underneath the so-called ice line interrupts the supply of moisture to the ice lenses. The same process then starts deeper, forming in silty soils parallel layers of segregated ice layers.

Objectives. The main objectives of the improvised soil freezing experiment here described are to demonstrate the upward sucked soil moisture from the "ground-water table" towards the freezing ice lenses upon freezing the soil sample from the top downwards, that is, upon the application of a thermal potential, and to report on subpressure or suction measurements in the freezing soil.

No attempt was made in the experiment to simulate any particular climatic freezing condition. The main purpose was to see whether suction can be measured. It is hoped that after perfection of the freezing apparatus, there will be provided enough observation and test data to establish a method or criterion index for the evaluation of frost-susceptible soils as well as for the calculation of frost penetration depth.

Apparatus. The improvised soil freezing apparatus was constructed mainly for the demonstration of the upward motion of soil moisture from "ground-water" upon freezing as well as for the purpose of trying to measure suction in soil upon freezing. Figure 4 is a photograph of the equipment. Figure 5 illustrates the complete apparatus in line drawing. The apparatus consists of a freezing chamber (Figure 6). The soil specimen to be frozen is contained in an open-ended 4⅛ in. inside diameter by 8½ in. long waterproofed cardboard tube, the inside of which was greased with technical vaseline. The base of the soil sample is inserted in a perforated brass receptable or cup which is in communication with a constant level water supply, adjusted to give a free water level, specifically, ground-water table in the soil sample.

The soil specimen, when positioned for freezing, is enclosed within an insulated box made from Celotex. During a freezing experiment the space between the soil specimen and the sides of the box is filled with vermiculite insulating material.

Freezing is done by means of dry ice contained in a sidewise insulated can. There is a space provided between the top of the soil specimen and the ice can to permit heaving of the frozen soil. An Ames dial indicator indicates the amount of heaving. The temperatures within the soil specimen during freezing are measured in terms of electrical resistance by means of six helically spaced thermistors. A thermistor unit is illus-

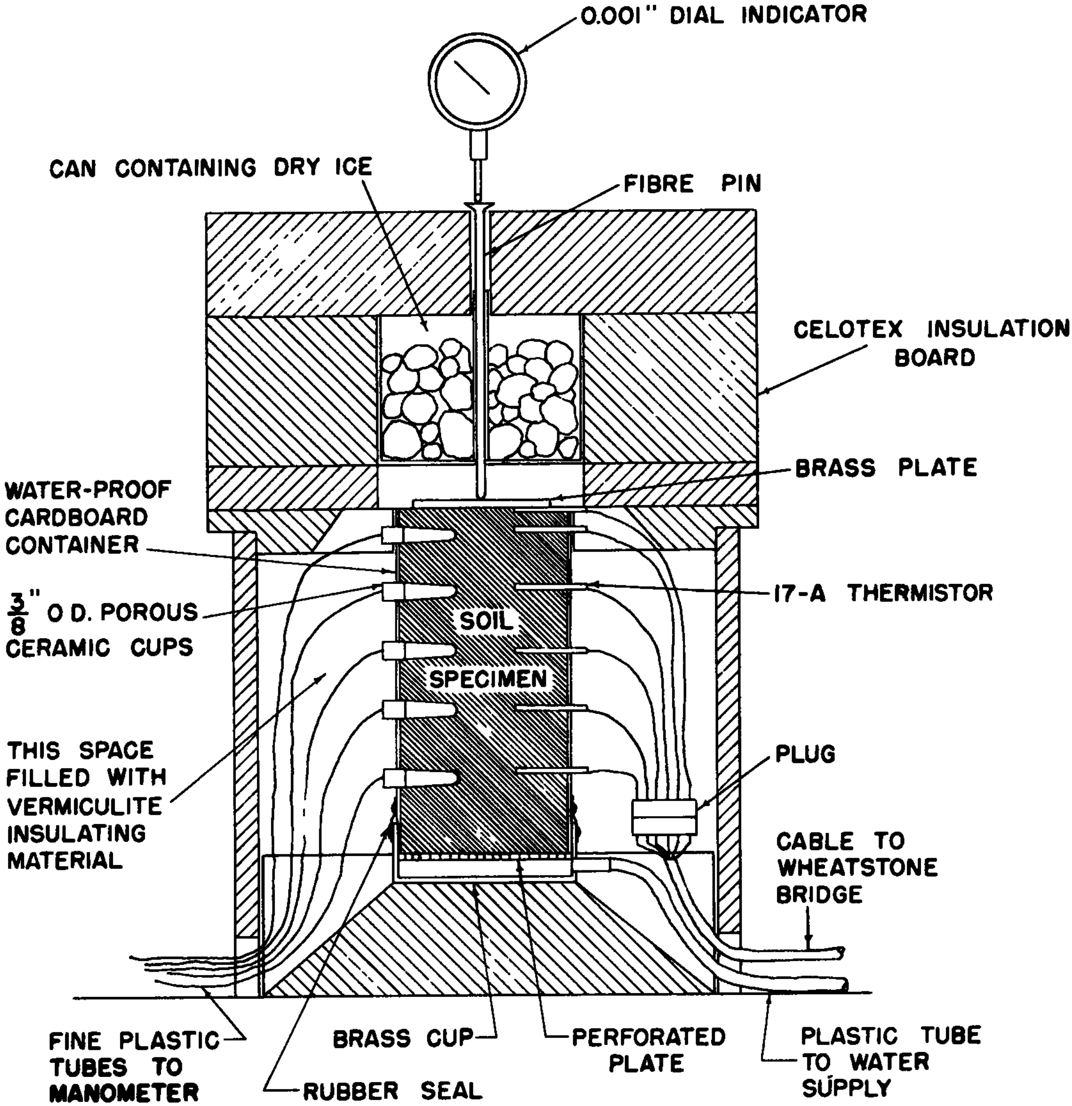

Figure 6. Freezing chamber.

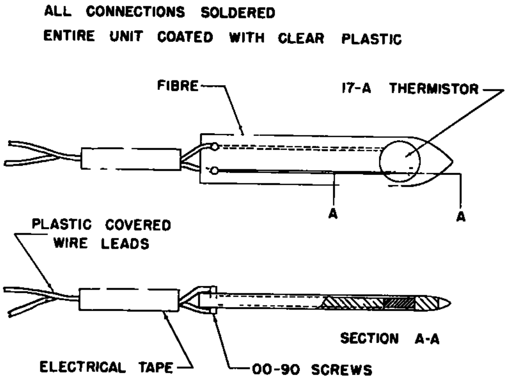

Figure 7. Thermistor unit.

trated in Figure 7. The thermistors were spaced from surface down, at 0, ½, 2, 3½, 5 and 6½ in. depths, respectively.

Subpressures were measured by a mercury manometer connected to the system by means of a suction cup at a depth of 3½ in. from the top of the specimen. The latter provides a connecting link or continuity between the moisture films in the soil specimen and the suction-measuring manometer. Any number of suction cups, as the system permits, can be inserted to learn the variation in subpressure along the height of the soil specimen. The mercury-water manometer was justified because the freezing and suction processes are relatively slow. It permitted accurate subpressure measurements. It was constructed with as small a core as possible in order to minimize the amount of water in the manometer that was consumed as an additional moisture supply for the growth of the ice lenses in the freezing soil specimen.

Soil. The soil used in this freezing experiment was a Dunellen soil, the grain size distribution of which is shown in Figure 8. The soil is a silty glacial outwash material. Its consistency limits are liquid limit, 16 percent, and non-plastic. The maximum dry density and optimum moisture content of this soil, determined according to the standard Proctor compaction method, are 120 pcf and 12 percent, respectively. The soil at a moisture content of 12 percent was compacted into a cardboard tube in five layers, applying ten blows per layer of a 5.5-lb compaction rammer falling 12 in. The soil specimen prepared in this way had a dry density of 120.9 pcf.

Freezing Test. The soil specimen, before freezing, was allowed to absorb water from the ground-water supply. This was done partly to simulate field conditions and partly to establish the moisture films within the specimen. Forty-four cm^3 of water

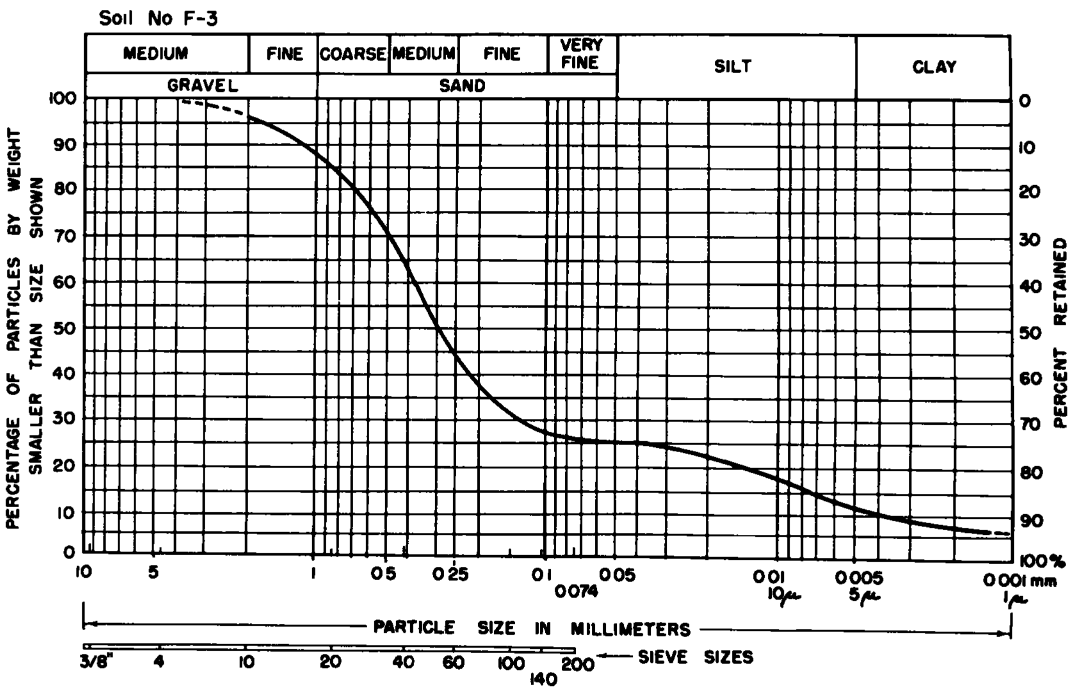

Figure 8. Grain size distribution curve.

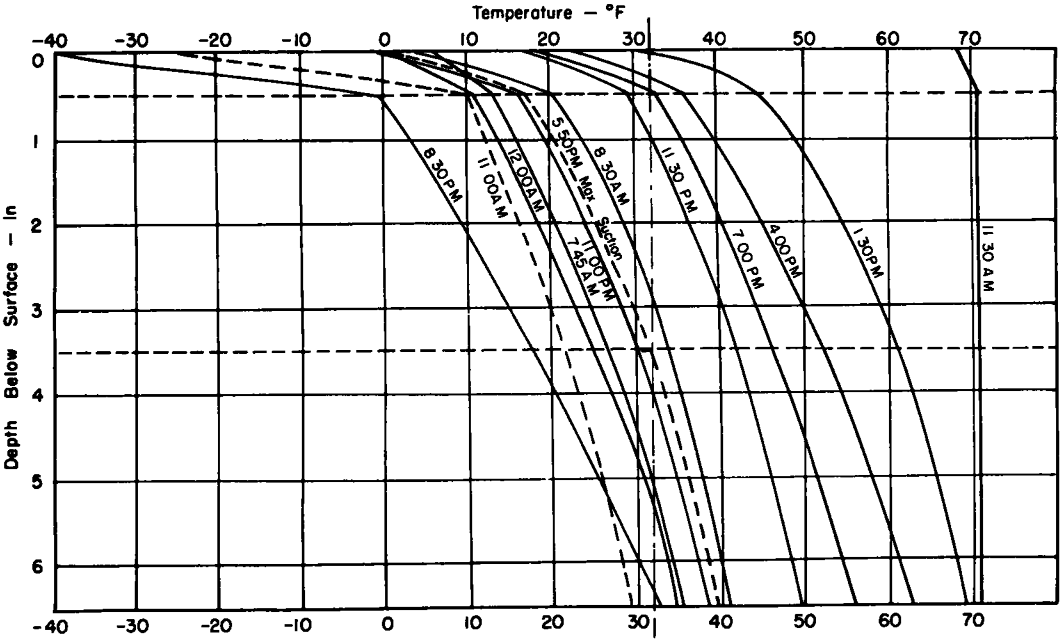

Figure 9. Temperature - depth curves (Tautochrones).

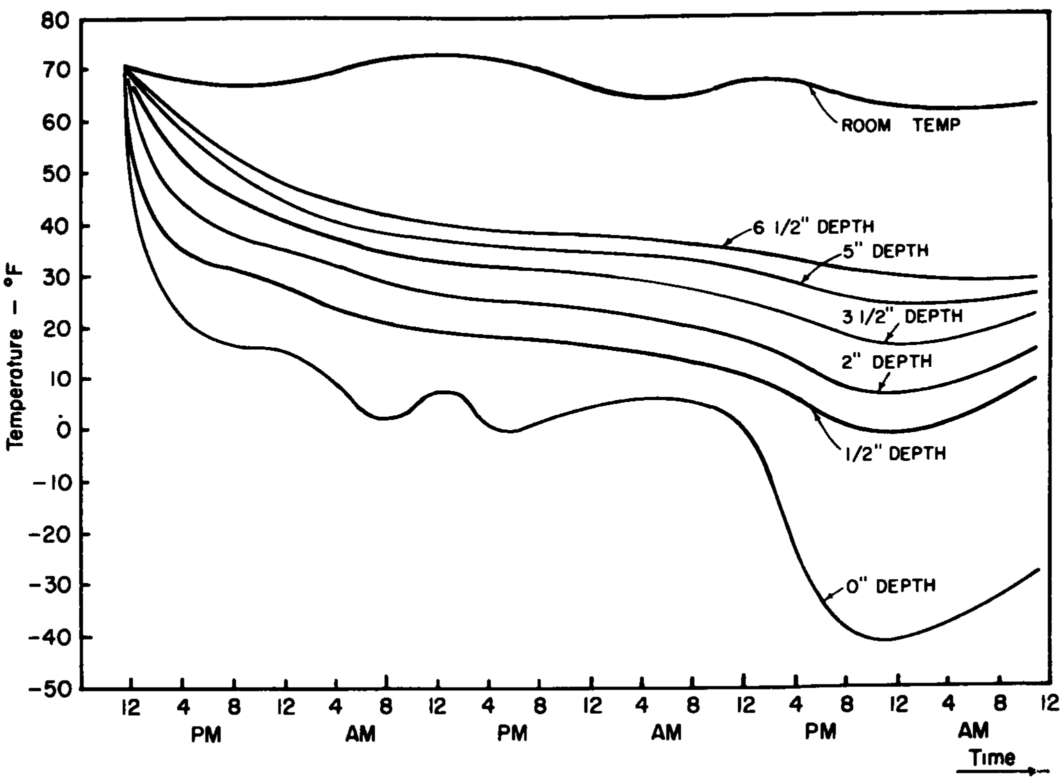

Figure 10. Time - temperature curves.

were absorbed during a 40-hour period, increasing the moisture content in the specimen to approximately 13.5 percent. The manometer indicated a slight moisture tension which reduced as absorption progressed.

The freezing chamber was then assembled, the dry ice can filled and observations started. Freezing was started at 11:30 a.m., April 22, 1954. The last readings were taken at 11:20 a.m., April 25, 1954. Soil temperatures, subpressures, volumes of upward-sucked ground-water and amounts of frost heaves were measured during the freezing process. Room temperatures were also noted. These data were recorded at 15-minute intervals for one and a half hours. This time interval was then extended to two hours.

Results. The results of a certain soil freezing experiment (Test No. 4), undertaken with illustrative purposes in mind, are shown here in a graphical form. Figure 9 represents some of the temperature-depth curves or tautochrones of the soil, the shapes of which agree with theoretical ones. They show the depth of frost penetration and indicate, indirectly, the rate at which the theoretical freezing temperature of 0 deg C = 32 deg F penetrated the soil specimen. The curved tautochrones indicate that after the frost had penetrated approximately half-way down (3½ in.) into the soil specimen, the tautochrone in the unfrozen part of the soil can practically be assumed to be straight lines, and thus the temperature gradient constant.

It can be noted from this graph that the soil specimen was subjected to an intense cold. At the start of the freezing experiment, 11:30 a.m., the temperature of the soil specimen was still at room temperature (note vertical, constant tautochrone). The colder the surface temperatures, the deeper the frost penetration (intersection of tautochrones with the 32 deg F-line). Because of the thermal properties of the soil (for example, heat capacity, latent heat of fusion and conductivity), the frost penetration into the soil suffered a time lag and retardation. The deeper the soil specimen was frozen, the more the tautochrones moved to the colder side on the graphs.

The dashed tautochrone marked at 5:50 p.m. corresponds to temperature conditions

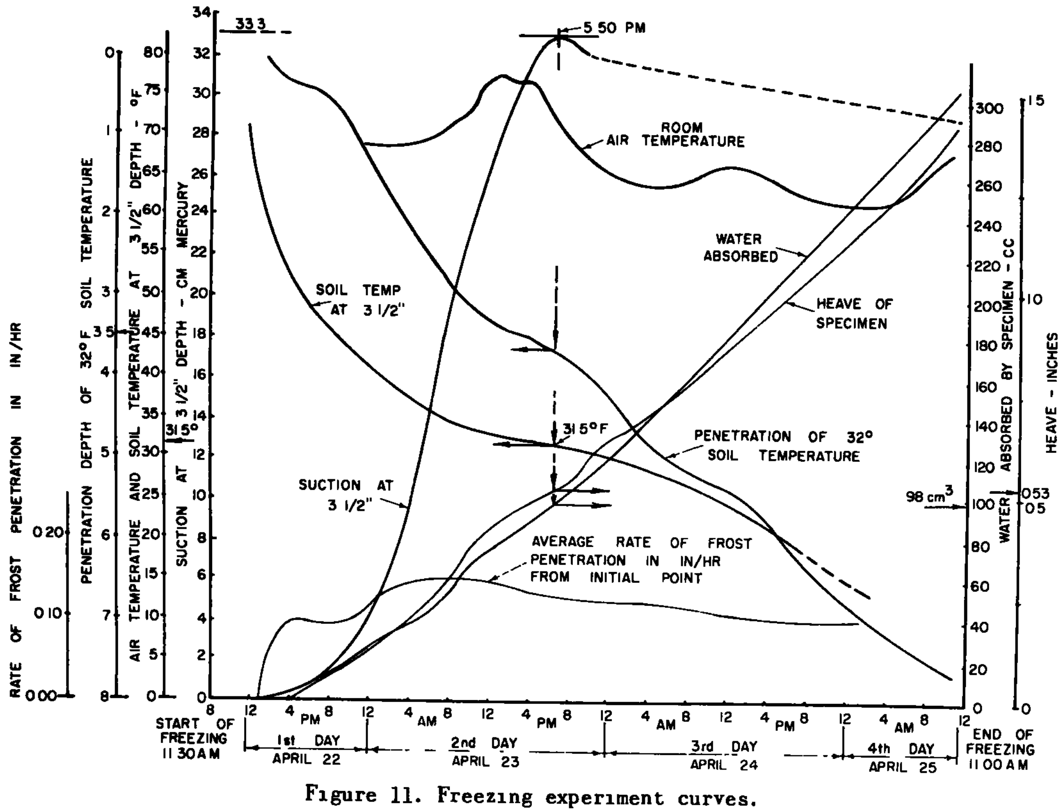

Figure 11. Freezing experiment curves.

when frost penetrated the soil specimen 3½ in. deep, reaching the suction point, and where and when the maximum subpressure or suction for that point was measured.

The soil specimen was then frozen all the way down to the ground-water table 6½ in. below the top of the specimen. The dashed line tautochrone marked 11: 00 a. m. indicates the last tautochrone in the freezing process. Note that the lower branch of this tautochrone approaches assymptotically the 32 deg F-line.

Figure 10 shows the time-soil temperature curves at the positions of the thermistors. The fluctuation of the soil surface temperature is a result of fluctuating room air temperature and the removal of the top of the chamber for inspection and replenishing of dry ice. This figure illustrates well the influence of the surrounding temperatures on the freezing process within the thermal system, and indicates the need, in soil freezing studies, for a controllable constant temperature room in order to keep, as nearly as possible, the ground-water and the suction manometers at constant temperatures. Such a room is desirable likewise for housing the actual soil freezing chambers, and to provide automatically controllable soil freezing equipment for freezing soil specimens either at constant temperatures or under cyclic variations.

In Figure 11 are shown the following relationships with time as observed in this experiment: (1) air room temperature as a function of time; (2) penetration depth in soil of the 32 deg F temperature; (3) rate of frost penetration; (4) water absorbed from ground-water by upward suction into soil upon freezing; (5) soil temperature at 3½ in. depth during the whole freezing process; (6) subpressure or suction in soil upon freezing at 3½ in. depth; and (7) frost heave.

The adverse influence of the room air temperature on the freezing process of the soil specimen is clearly reflected.

The rate of penetration of frost into the soil specimen decreases with time. This can be explained by the fact that the heat conduction of the growing ice lenses, or frozen layers, is considerably less than that of water. Besides, heat is transferred upward from a warmer region to a colder one, releasing heat to the surface and thus retarding the rate of frost penetration. Also, the rate of frost penetration in the soil decreased at each peak of the air temperature.

Frost penetration started at 1: 00 p. m., heaving at 4: 00 p. m. on the same day. By the time frost penetrated the soil 3½ in. (at 5: 50 p. m. next day), the amount of water "sucked up" from the burette was 98 cm^3, and frost heave was measured at 0. 53 in. The measured head of the subpressure or suction at the 3½ in. depth reached a maximum of 33. 3 cm mercury = 14. 85 ft of water or 0. 462 t/ft^2 vacuum when the soil temperature at that suction measuring point was 31. 5 deg F, indicating that this soil medium froze at a temperature of less than 32 deg F.

After the frost passed below the suction point, its manometer was no longer operative. After the whole specimen was frozen through, the manometer indicated a slight drop in subpressure. At the end of the experiment the manometer showed 29. 2 cm mercury (see the dashed part of the suction curve in Figure 11).

COMPARISON OF THEORETICAL AND MEASURED SUCTION VALUES

1. The suction value, P_S, calculated for the described experimental conditions by means of the hydrodynamic Equation 1 with k_S = 0. 0000028 ft/min and for t = 1820 min, is P_{SN} = 0 083 t/ft^2.

2. The thermodynamic theory, assuming a linear temperature gradient (chord method), by Equation 2 or 3 gives an average maximum theoretical suction value of P_{STc} = 0. 523 t/ft^2.

For a variable course of temperature gradient, the maximum theoretical suction value (by Equation 6)is approximately P_{STV} = 0. 529 t/ft^2.

3. The suction value measured in the experiment was P_{Se} = 0. 462 t/ft^2.

A comparison of these values shows that the hydrodynamic suction value is about 15. 9 percent of that obtained from the thermodynamic expression. Of course, much of the suction value, P_{SH}, depends upon the accurate determination of the suction coefficient, k_S. Besides, more experiments are needed before any conclusion may be drawn.

The measured suction value, $P_{Se} = 0.462$ t/ft^2 is about 5.6 times greater than that calculated by the hydrodynamic formula, that is, $P_{SH} = 0.083$ t/ft^2. Assuming that the thermodynamic suction value is of 100 percent efficiency, the efficiency of the experimental system under consideration is

$$\eta = \frac{P_{Se}}{P_{STc}} = \frac{0.462}{0.523} = 0.88$$

CONCLUSIONS

1. There exists a theoretical basis for experimental frost action research.
2. It is possible to range the soil freezing experiment into a hydrothermodynamic system.
3. The system soil-moisture-temperature is a very complex one. However, upon simplification of the system, it is possible to obtain theoretically as well as experimentally a satisfactory insight into the processes which take place within this system, particularly with reference to soil moisture transfer upon freezing and its associated suction force.
4. Preliminary tests indicate that it is possible to measure the magnitude of suction in soils upon freezing.
5. The improvised apparatus used in the soil freezing experiment simulates fairly closely the system as it is found in nature. It demonstrates vividly the upward transfer of soil moisture from the ground-water table towards the freezing ice lenses, and shows how frost penetrates the ground.
6. The adverse influence of fluctuating temperatures of the surroundings of the system on the freezing process and on the temperature of the ground water indicates the necessity of a constant temperature room and a controllable freezing equipment for a better and more accurate control of tests to permit simulation of varying climatic conditions.
7. Although it is generally recognized that the second law of thermodynamics applies to every case of practical importance where heat is converted to work, a comparison of the calculated and measured suction values in this experiment indicated that much research is still to be done in order to learn the relationship between theoretical and experimental suction values.
8. Suction studies should be done on various types of soil, since each type of soil at its maximum dry density and temperature conditions at the same freezing conditions has a different coefficient of permeability and rate and amount of heaving. The position of the ground-water table influences the suction considerably. The number of experiments should be large enough to permit drawing some definite conclusions.

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