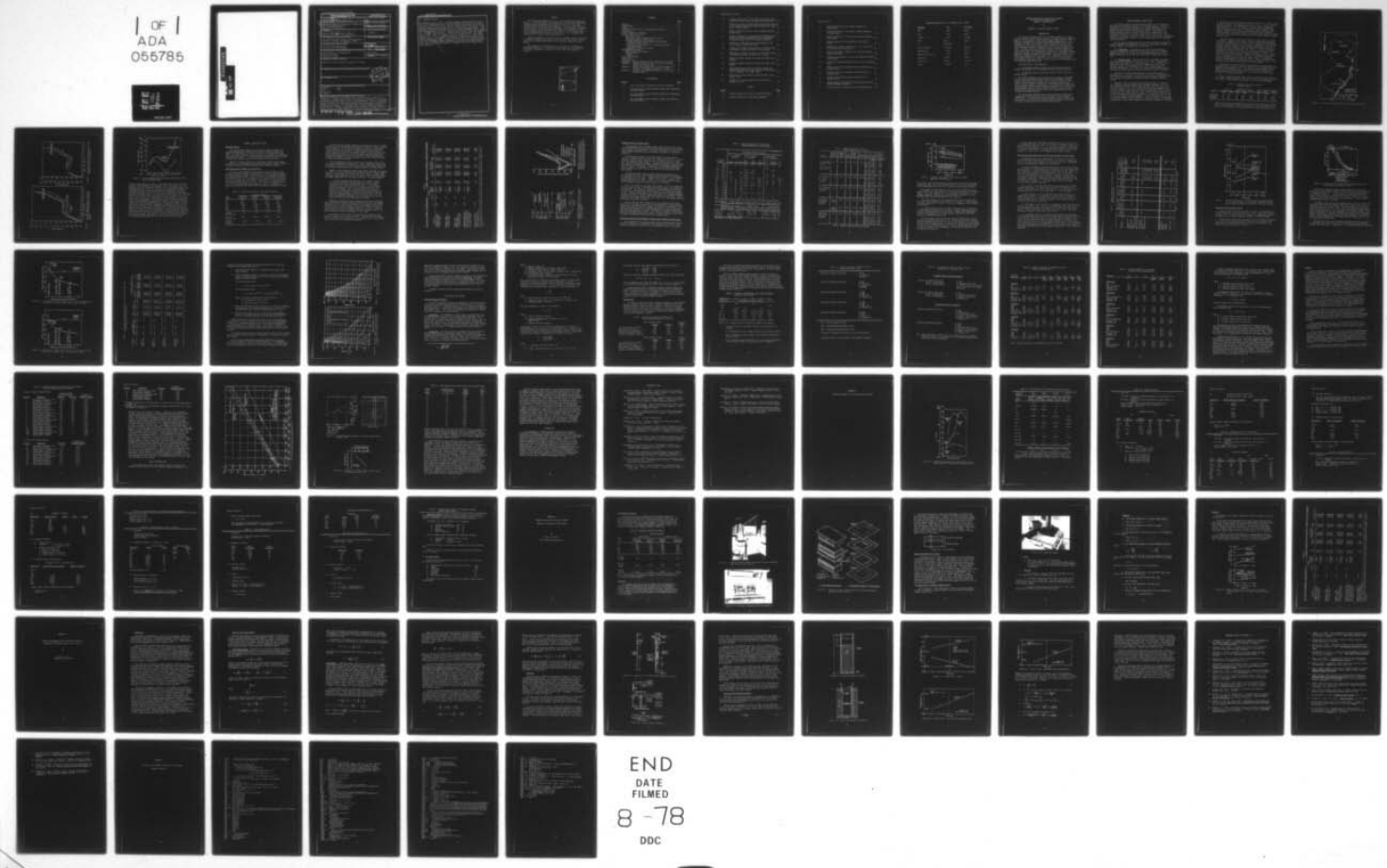
AD-A055 785 COLD REGIONS RESEARCH AND ENGINEERING LAB HANOVER N H F/G 13/2
IMPROVED DRAINAGE AND FROST ACTION CRITERIA FOR NEW JERSEY PAVE--ETC(U)
MAY 78 R L BERG, R W MCGAW

UNCLASSIFIED CRREL-SR-78-9 NL

| OF |
ADA
055785



END
DATE
FILMED
8-78
DDC

AD A055785

AD NO.

DDC FILE COPY

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: ~~Special Report~~ 14 CRREL-SR- 78-9
2. GOVT ACCESSION NO.
3. RECIPIENT'S CATALOG NUMBER

4. TITLE (and Subtitle): 6 IMPROVED DRAINAGE AND FROST ACTION CRITERIA FOR NEW JERSEY PAVEMENT DESIGN, Phase 2, Frost Action.

5. TYPE OF REPORT & PERIOD COVERED: 9 Special rept.

6. PERFORMING ORG. REPORT NUMBER

7. AUTHOR(s): 10 Richard L. Berg [redacted] Richard W. McGaw

8. CONTRACT OR GRANT NUMBER(s): NJ-7740

9. PERFORMING ORGANIZATION NAME AND ADDRESS
U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire 03755

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS

11. CONTROLLING OFFICE NAME AND ADDRESS
U.S. Dept. of Transportation
Federal Highway Administration

12. REPORT DATE: 11 May [redacted]78

13. NUMBER OF PAGES: 87 (12) 88p.

14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)
New Jersey Dept. of Transportation
1035 Parkway Avenue
Trenton, New Jersey

15. SECURITY CLASS. (of this report): Unclassified

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

DDC RECEIVED JUN 27 1978 F

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Freezing
Frost
Frost heave
Pavements
Roads
Soils

ABSTRACT

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

With the assistance of the Federal Highway Administration, the New Jersey Department of Transportation (NJDOT) is studying the feasibility of including open-graded drainage layers in their highway pavements. Before constructing actual pavements with open-graded drainage layers they chose to analytically examine the influence of the drainage layer on frost penetration beneath hypothetical pavements, CRREL conducted the analytical study under a contract with the NJDOT. Thermal conductivity values of several New Jersey soils, stabilized drainage layer materials, and pavement samples were measured using

CONT'

20. Abstract (cont'd)

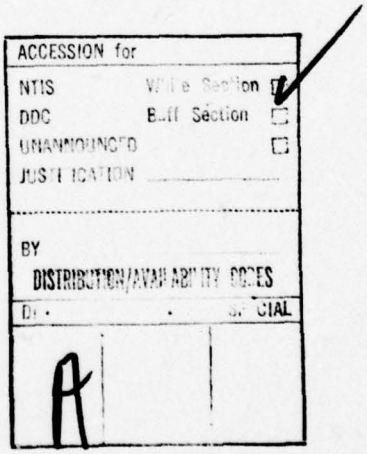
the Guarded Hot Plate method or the probe method. Measured values for the soils were somewhat less than predicted from Kersten's equations and ranged from 0.77 to 1.90 Btu/ft hr °F. Measured thermal conductivity values for the pavement samples were also somewhat lower than Kersten's observations. Frost penetration depths were computed using the modified Berggren equations. Mean air freezing indexes used in the computation ranged from 50 °F - days in Atlantic City to 480 °F - days in Newton. Design freezing indexes ranged from 250 °F - days to 900 °F - days for the same two sites. Maximum computed frost depths ranged from 0.8 to 2.1 ft beneath conventional pavements, i.e. those without drainage layers. For pavements incorporating an open-graded drainage layer, computed maximum frost depths ranged from 0.8 ft to 1.4 ft. It was concluded that frost penetration beneath a pavement including an open-graded drainage layer would be approximately equal to a pavement without the drainage layer at the same site.

PREFACE

This report was prepared by Dr. R.L. Berg, Research Civil Engineer, and Mr. R.W. McGaw, Research Civil Engineer, of the Northern Engineering Research Branch, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory. It is the final report on the CRREL study for the New Jersey Department of Transportation, Research Project NJ-7740, "Improved Drainage and Frost Action Criteria for New Jersey Pavement Design; Phase II, Frost Action." The New Jersey Department of Transportation received the funds from the U.S. Department of Transportation, Federal Highway Administration.

Special recognition is given to Mrs. D.J. VanPelt and SP4 C. Espiritu who reduced, tabulated and plotted the field data. Mrs. Van Pelt also conducted the thermal conductivity tests using the guarded hot plate method.

The contents of this report are not to be used for advertising or promotional purposes. The citation of trade names does not constitute an official endorsement or approval of the use of such commercial products.



CONTENTS

	Page
Preface	iii
Illustrations	iv
Tables	v
Conversion Factors for U.S. Customary and SI Units	vii
Introduction	1
Surface Transfer Coefficients	2
Thermal Conductivity Tests	7
Materials tested	7
Base course soils (guarded hot plate tests)	7
Pavement materials (probe tests)	11
Bituminous-stabilized open-graded drainage layer materials (probe tests)	11
Unstablilized open-graded Mix #1 (probe and guarded hot plate tests)	15
Comparison with Kersten's values	17
Calculated Frost Depths	23
Modified Berggren equation	23
Computations	25
Results	32
Other Considerations	34
Future Work	38
Literature Cited	39
Appendix A. Material Characteristics Furnished by the NJDOT	41
Appendix B. Thermal Conductivity Results on NJDOT Materials, Guarded Hot Plate Method	52
Appendix C. Theory and Equipment used on the Probe Method for Measuring the Thermal Conductivity of Soils	61
Appendix D. Listing of the Computer Program for the Modified Berggren Equation	76

ILLUSTRATIONS

Figure		Page
1.	State of New Jersey showing the test site locations.	4
2.	Air and pavement surface freezing indexes near Bordentown, 1975-1976 winter.	5
3.	Air and pavement surface freezing indexes near Bedminster, 1975-1976 winter.	5
4.	Air and pavement surface freezing indexes near Rockaway, 1975-1976 winter.	6

Illustrations (cont'd)

5. Thermal conductivity of base course and subbase course materials from New Jersey, guarded hot plate method. 10

6. Variation of ratio of thermal conductivity above freezing to the thermal conductivity below freezing, for sandy soils. Kersten (1949). 10

7. Thermal conductivity of New Jersey pavement materials, probe method. 14

8. Thermal conductivity of unstabilized Open-Graded Mix #1 by two test methods compared with the corresponding bituminous mixture and two of Kersten's materials. 17

9. Gradations of New Jersey base course materials used in the thermal conductivity tests. 18

10. Comparison of thermal conductivity of New Jersey soils with Kersten's values for sandy soils, unfrozen. 19

11. Comparison of thermal conductivity of New Jersey soils with Kersten's values for sandy soils, frozen. 19

12. Kersten's thermal conductivity values for sandy soils, unfrozen. 22

13. Kersten's thermal conductivity values for sandy soils, frozen. 22

14. Relationship between mean air freezing indexes and indexes determined for colder years for 30 consecutive years, Department of the Army (1966). 35

15. Statistically based design air freezing index values, Berube (1967). 36

16. Influence of freezing index on frost penetration beneath section 7. 36

TABLES

Table		Page
1.	Freezing indexes and n-factors, 1975-1976 winter.	3
2.	Physical properties of base course materials.	7

Tables (cont'd)

3.	Thermal conductivity test results, guarded hot plate method.	9
4.	Physical properties of New Jersey pavement materials for probe tests.	12
5.	Thermal conductivity of New Jersey pavement materials, probe method.	13
6.	Thermal conductivity of unstabilized Open-Graded Mix #1.	16
7.	Comparison of thermal conductivity of New Jersey soils (K) with Kersten's values (K_s).	20
8.	Initial and boundary conditions for modified Berggren equation solutions.	25
9.	Gradation specifications for base and subbase materials in New Jersey.	26
10.	Typical New Jersey pavement sections without a drainage layer.	27
11.	Tentative New Jersey pavement sections incorporating a drainage layer.	28
12.	Thermal properties for pavement sections without a drainage layer.	29
13.	Thermal properties for pavement sections with a drainage layer.	30
14.	Maximum seasonal frost penetration determined with the modified Berggren equation.	33
15.	Thaw depths from a statistically based design method.	37

CONVERSION FACTORS FOR U.S. CUSTOMARY AND SI UNITS

Multiply	By	To obtain
inch	0.025	meter
foot	0.305	meter
°F-days	0.556	°C-days
mile	1690.3	meter
°F	5/9 (F-32)	°C
lb/cu ft (pcf)	16.018	kg/cu m
Btu/ft hr °F	1.731	W/m K
Btu-in./sq ft hr °F	0.144	W/m K
Btu/cu ft	37,310	J/cu m
Btu/cu ft °F	67,158	J/cu m K
sq ft	0.093	sq m

IMPROVED DRAINAGE AND FROST ACTION CRITERIA FOR NEW JERSEY PAVEMENT DESIGN
PHASE 2: FROST ACTION

by

Richard L. Berg and Richard W. McGaw

INTRODUCTION

In early 1975 engineers from the New Jersey Department of Transportation (NJDOT) visited CRREL to discuss a program they were embarking upon with the assistance of the Federal Highway Administration. The NJDOT was interested in including open-graded drainage layers in their pavements to remove excess water. Before constructing actual field installations, they chose to determine, analytically, the effect of frost penetration beneath pavements when using an open-graded drainage layer. In early July 1975, the NJDOT issued a contract to CRREL to conduct this study, the principal goal of which is to determine analytically the impact of an open-graded drainage layer on the depth of frost penetration beneath pavements in New Jersey. The actual effect of a particular drainage layer will depend on its thickness and location in the pavement profile, the gradation of the soil and the stabilizing agent applied, the range of water contents likely to be encountered, and the resulting thermal properties.

The study had the following goals:

a. To determine whether the depth of frost penetration would change significantly if an open-graded drainage layer was used, and

b. To determine whether exceptional adverse effects should be guarded against.

The modified Berggren equation was used to estimate the maximum seasonal frost penetration depths beneath several pavement profiles. Previous experience has indicated that this equation predicts frost depths which are generally adequate for engineering applications. To provide a higher degree of reliability in the predictions, however, it was necessary to measure surface transfer coefficients for selected New Jersey pavements and to conduct thermal conductivity tests on representative base and subbase course materials, pavement materials and open-graded drainage materials.

Air and surface temperature data from the 1975-1976 winter were supplied by the NJDOT and were used to determine surface transfer coefficients. The NJDOT also furnished aggregate for the thermal conductivity tests on the base and subbase course materials and specimens for thermal conductivity tests on the paving materials.

SURFACE TRANSFER COEFFICIENTS

To calculate seasonal frost penetration depths, it is necessary to determine the temperature condition at the ground surface. Since air temperatures are generally available and surface temperatures are not, a correlation between these temperatures is required to establish the thermal boundary condition at the ground surface. Unfortunately no simple correlation exists between air and surface temperatures. The difference between air and surface temperatures at any specific time is influenced by latitude, cloud cover, time of year, time of day, atmospheric conditions, wind speed, exposure, surface characteristics and subsurface thermal properties.

Air and surface temperatures are converted to "freezing indexes" to compute seasonal frost penetration depths. To understand the concept of freezing indexes, two definitions are necessary:

(1) Degree-days. The degree-days for any one day equal the difference between the average daily temperature and 32°F. Freezing degree-days occur when the average daily temperature is below 32°F. Thawing degree-days occur when the average daily temperature is above 32°F.

(2) Freezing index. The freezing index is the number of degree-days between the highest and lowest points on a curve of cumulative degree-days versus time for a freezing season. It is a measure of the duration and magnitude of below-freezing temperatures for a given freezing season. Air temperatures for the air freezing index are measured approximately 4.5 ft above the ground while temperatures for the surface freezing index are measured immediately below the surface.

The ratio of surface freezing index to air freezing index is designated as the "n-factor" for freezing conditions. Determination of the n-factor for a specific location requires concurrent observations of air and surface temperatures throughout several complete freezing seasons.

Lunardini (1977) has conducted an extensive study of n-factors. For asphaltic concrete pavements under freezing conditions, he found that the range of values from 25 different sites was from 1.00 to 0.25 with an average of 0.57. For Portland cement concrete pavements subjected to freezing conditions, the range of values was 0.95 to 0.12 for 21 sites and the average value was 0.58. Moulton (1968) studied frost-related problems in West Virginia pavements. He reported values of 0.96 to 0.25 for asphaltic concrete pavements in the winter and 0.87 to 0.12 for Portland cement concrete pavements. It is the authors' opinion that a substantial amount of the variation in n-factors is due to differences in placement of the surface temperature probe and differences in proximity

of the surface and air temperature observation locations. Recent studies at CRREL by Eaton (1976) yielded n-factors of 0.57 to 0.43 for asphaltic concrete pavements. The Department of the Army (1966) recommended design values of 0.9 for both asphaltic concrete and Portland cement concrete pavements kept free of ice and snow. These values have been used for many years for computing maximum seasonal frost depths for pavement design. The larger values for n-factors would result in greater calculated frost penetration depths because the surface freezing indexes would be larger.

Since the range in values for n-factors is quite large, the CRREL investigators suggested that pavement surface temperatures be measured to obtain representative data for New Jersey. During the 1975-1976 and 1976-1977 freezing seasons, the New Jersey DOT measured air and surface freezing indexes on Portland cement concrete and asphaltic concrete pavements at three different locations (Figure 1). Site 1 was located on Interstate 295 near Bordentown, Site 2 was on Interstate 287 near Bedminster, and Site 3 was on Route 15 near Rockaway. Figures 2 - 4 contain air and surface freezing indexes from Sites 1, 2 and 3, respectively for the 1975-1976 winter. Data for the 1976-1977 winter are in Berg (in preparation). Both Portland cement concrete and asphaltic concrete pavement surface indexes are shown. Figure 2 illustrates the method, discussed above, for determining the air freezing index. Other freezing indexes are determined using the same procedure.

Three days of data are missing in mid-December at the Rockaway site and approximately 15 days of data are missing in late November and early December at the Bedminster site. The freezing season probably began during the time of missing data at both of these sites; however, n-factors determined from the partial freezing seasons are sufficiently accurate for the present studies.

Table 1 contains freezing index values, n-factors and the length of the freezing season measured at each of the three sites during the 1975-1976 winter. Bordentown was the warmest site, with an air freezing

Table 1. Freezing indexes and n-factors
1975-1976 winter

Site	Air		Asphaltic Concrete			Portland Cement Concrete		
	Season	Index	Season	Index	n-factor	Season	Index	n-factor
	Days	°F Days	Days	°F Days		Days	°F Days	
Bordentown	40	95	20	59	0.62	20	57	0.60
Bedminster	40	151	40	127	0.84	40	82	0.54
Rockaway	58	471	56	216	0.46	56	188	0.40

index of 95 °F-days, and Bedminster and Rockaway had air freezing indexes of 151 °F-days and 471 °F-days, respectively. At all three sites the n-factors for the asphaltic concrete surface were larger than those of the

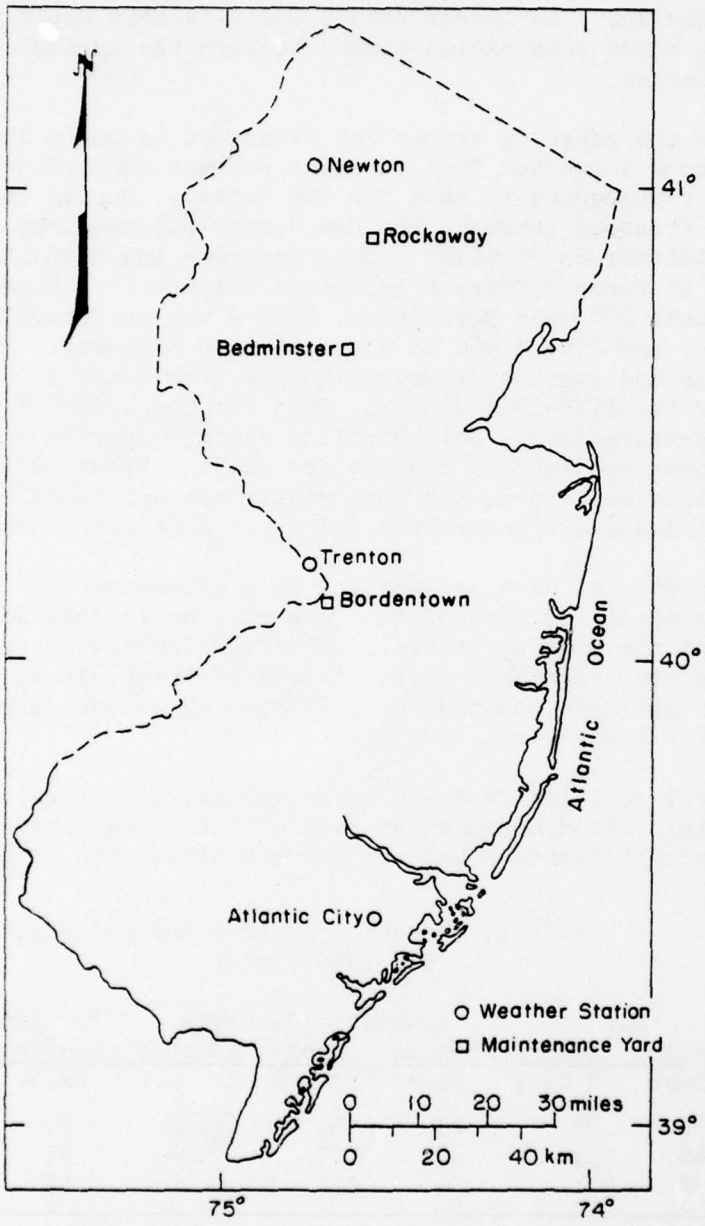


Figure 1. State of New Jersey showing the test site locations.

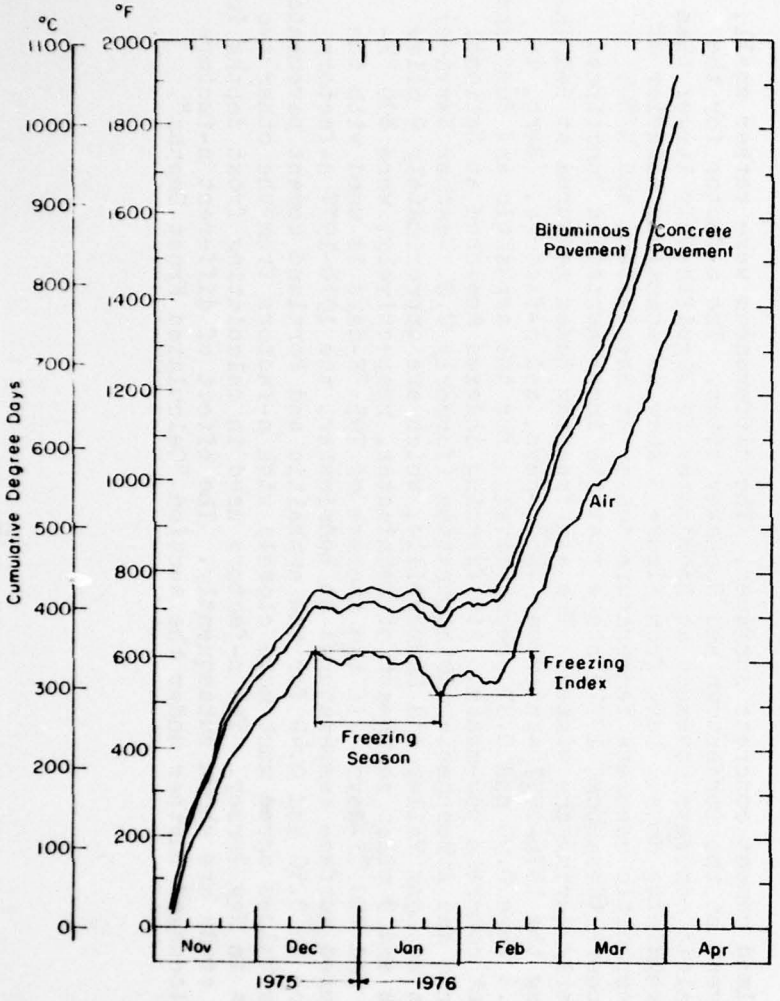


Figure 2. Air and pavement surface freezing indexes near Bordentown, 1975-1976 winter.

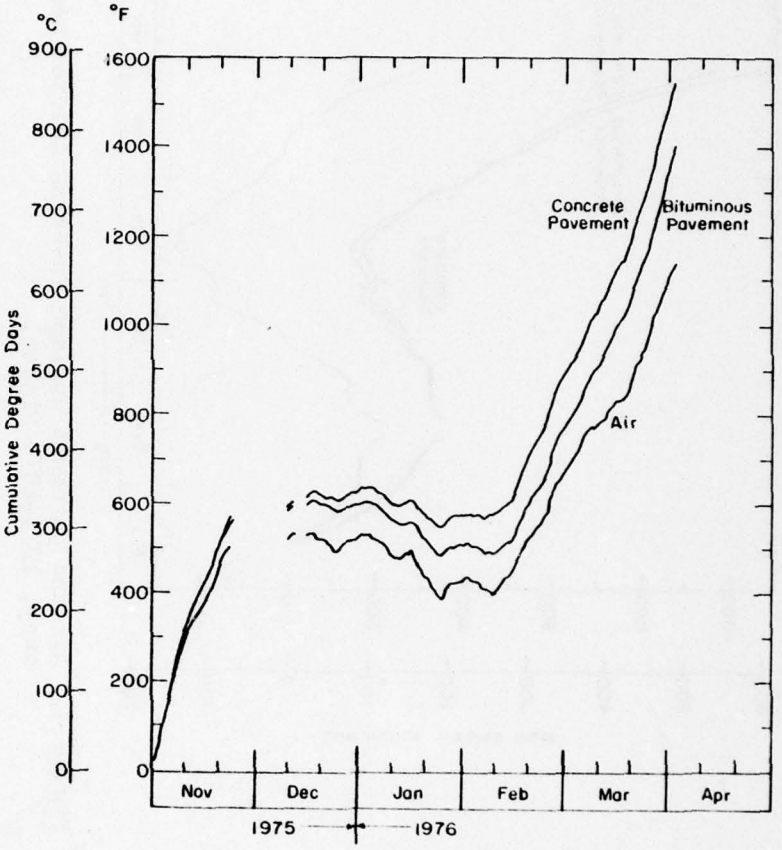


Figure 3. Air and surface freezing indexes near Bedminster, 1975-1976 winter.

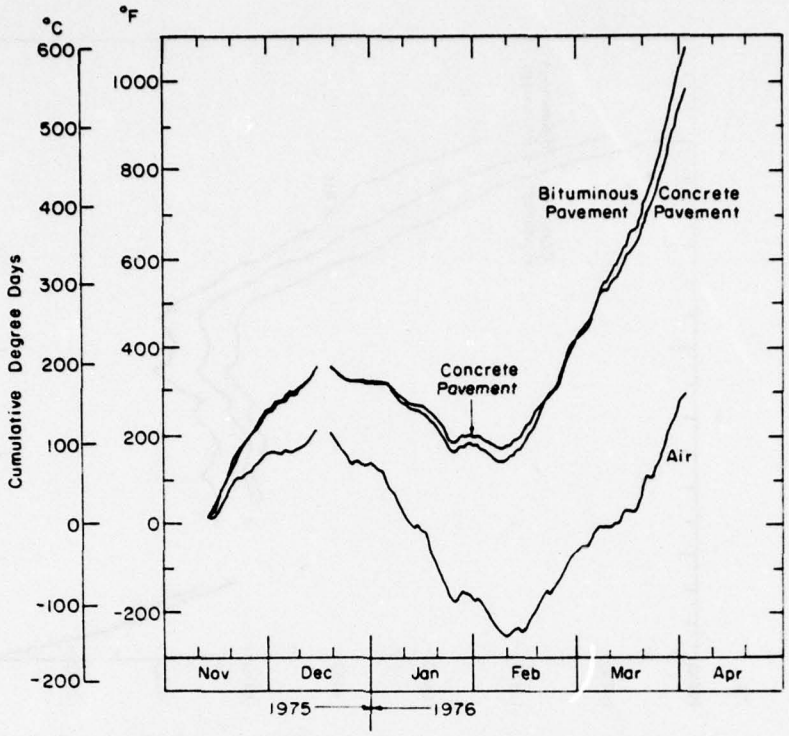


Figure 4. Air and pavement surface freezing indexes near Rockaway, 1975-1976 winter.

Portland cement concrete surfaces. The differences were rather small, however, at the Bordentown and Rockaway sites. The n-factor for the asphaltic concrete pavement at Bedminster is significantly larger than the remaining data. Data from Figure 3 show no anomalous behavior of the asphaltic concrete temperature sensor at Bedminster, and NJDOT engineers (Cosaboom, 1977) state that the instrumentation functioned properly during the winter. The air freezing index measured at Bedminster during the 1976-1977 winter was 446°F-days, and n-factors, (Berg, in prep.) were 0.90 and 0.87, respectively, for the asphaltic and Portland cement concrete pavements. Air freezing indexes measured at National Oceanic and Atmospheric Administration (formerly U.S. Weather Service) sites at Long Valley and Sommerville, which are approximately 9 miles north and 9 miles southeast of Bedminster, respectively, were 800 °F-days and 790 °F-days. If the average of 795 °F-days is used with the measured surface temperatures at Bedminster, the 1976-1977 n-factors reduce to 0.50 and 0.49 for the asphaltic and Portland cement pavements. These values agree much more closely with n-factors from the other two sites in New Jersey. The n-factors used in calculating frost depths in this study are shown subsequently. The effect of different n-factors is discussed further under the section "Calculated Frost Depths".

THERMAL CONDUCTIVITY TESTS

Materials Tested

The NJDOT shipped samples of typical base course, pavement, and candidate drainage-layer materials to CRREL for thermal conductivity tests. The base course soils were subjected to the Guarded Hot Plate (GHP) test, while the Probe test was used for the pavement and stabilized drainage-layer materials. One drainage-layer material was tested in the unstabilized condition, using both the GHP and the Probe tests.

Appendix A contains data sheets describing in detail the physical properties of all the materials as they were received at CRREL. Several of these properties are also discussed in the following sections.

Base Course Soils (Guarded Hot Plate Test)

Four base course materials were received from the NJDOT in bulk, and were compacted at CRREL to approximately 95% of the maximum density at optimum moisture content listed in Table 2. The maximum dry density and optimum moisture content values were obtained from data provided by the NJDOT (see Fig. A1). Each material was tested at two water contents in both the frozen and thawed states. Sample size for the Guarded Hot Plate apparatus was 20 in. x 20 in. x 3 in. thick. A complete description of the test equipment and procedure is provided in Appendix B. Physical properties of the base course materials are summarized in Table 2.

Table 2. Physical properties of base course materials.

Sieve Size	Actual % Passing			
	Ogdensburg Gneiss	Hamilton Lakes	Hamburg Shale	Pennington Traprock
4"	---	100	---	---
2"	100	---	100	100
3/4"	86.9	98.4	93.2	83.1
#4	48.0	74.4	69.1	55.1
#50	9.5	11.9	7.2	16.8
#200	3.8	1.4	2.8	8.9
Optimum Moisture (%)	8.1	7.6	9.5	11.1
Max. Dry Density (pcf)	135.6	123.6	127.8	135.9
Minimum Porosity	0.210	0.225	0.225	0.244

The two moisture contents chosen for the materials were (1) approximately 80% of the optimum moisture content, and (2) the "field capacity" of each material. The former was intended to simulate a rather dry field condition, and the latter a nearly saturated field condition. Initial plans were to conduct the higher moisture content tests at full water saturation; however, it was not possible to attain saturation because water tended to flow out of the mold. Field capacities were obtained by wetting the 3-inch thickness of soil with as much water as it would hold without draining.

The two temperatures chosen for each set of thermal conductivity tests were approximately 40°F and 22°F. These temperatures were chosen to provide a value in the frozen state and one in the thawed state, and to faciliate a comparison with the values reported by Kersten (1949).

Results of the Guarded Hot Plate tests (Table 3 and Fig. 5) indicate that three of the four unstabilized New Jersey soils have higher conductivities in the thawed state than in the frozen state. Kersten (1949) observed a similar behavior in soils he tested. Discussing differences in thermal conductivity at 25°F and at 40°F (Fig. 6) Kersten stated:

> "It was found that the difference in K values at these two points was chiefly dependent upon the moisture content. On relatively dry soils, for example those in the air-dry condition, very little difference was found. As the moisture content was increased, the K at 25 degrees became less than that above freezing. With a further increase in moisture content, the below-freezing value became progressively greater than that above freezing. The ratio of the frozen to unfrozen value also depended somewhat upon the density of the soil, the ratio in general being greater for a high density test than for a low density test at the same moisture content."

Probably the Ogdensburg Gneiss, Hamilton Lakes Sand and Hamburg Shale were tested at moisture contents where the ratio of the thermal conductivity in the frozen state to that in the thawed condition was less than unity, but the Pennington Traprock was tested at moisture contents where the ratio was greater than one. Tests at a series of moisture contents would be required to develop the relationships for these conditions but were unnecessary for this study.

The results of the GHP tests are compared with Kersten's mean values for sandy soils in a later section. They are also used, along with the results of the Probe tests, in the computation of frost penetration which will be discussed subsequently.

TABLE 3

NEW JERSEY DEPARTMENT OF TRANSPORTATION — THERMAL CONDUCTIVITY TEST RESULTS, GUARDED HOT PLATE METHOD

Material	Gs	Design OPT. MC %	Design 95% Max. Density pcf	Test Results MC %	Test Results Density pcf	Test Results Saturation %	Test Results Temp. °F	Test Results K' Btu/hr ft °F
OGDENSBURG GNEISS	2.75	8.1	128.8	6.7	129.4	56.8	23.8	1.055
(Sandy Gravel GW)				6.7	129.4	56.8	38.7	1.252
1 A Base Course			(Porosity	9.7	128.2	78.9	22.2	1.425
			0.249)	9.7	133.4*	93.3*	39.9	1.476
HAMILTON LAKES	2.66	7.6	117.4	6.3	117.3	40.6	21.8	1.025
(Gravelly Sand SW)				6.3	117.3	40.6	39.0	1.319
1 C Base Course			(Porosity	12.0	116.8	75.9	22.8	1.724
			0.293)	12.0	116.8	75.9	39.8	1.897
HAMBURG SHALE	2.75	9.5	121.4	8.3	118.8	51.6	22.5	0.766
(Well Graded -				8.3	120.9*	54.6*	40.6	0.853
Gravelly Sand Size)			(Porosity	14.4	116.5	83.7	22.6	1.065
1 C Subbase Course			0.293)	14.4	120.9*	94.7*	39.7	1.091
PENNINGTON TRAP ROCK	2.88	11.1	129.1	8.1	128.4	58.3	22.5	1.079
(Well Graded Mixture of				8.1	128.4	58.3	40.8	0.892
Sand and Gravel with			(Porosity	11.7	128.4	84.2	21.3	1.107
Less Than 10% Fines)			0.282)	11.7	128.4	84.2	40.2	0.993
5 A Subbase Course								
MOUNT HOPE GRANITE	2.68			4.5	121.2	31.8	24.9	0.362
(Unstabilized Open-				4.5	121.2	31.8	39.4	0.488
Graded Drainage Layer)								

*These samples densified upon thawing, resulting in a higher degree of saturation with no change in water content.

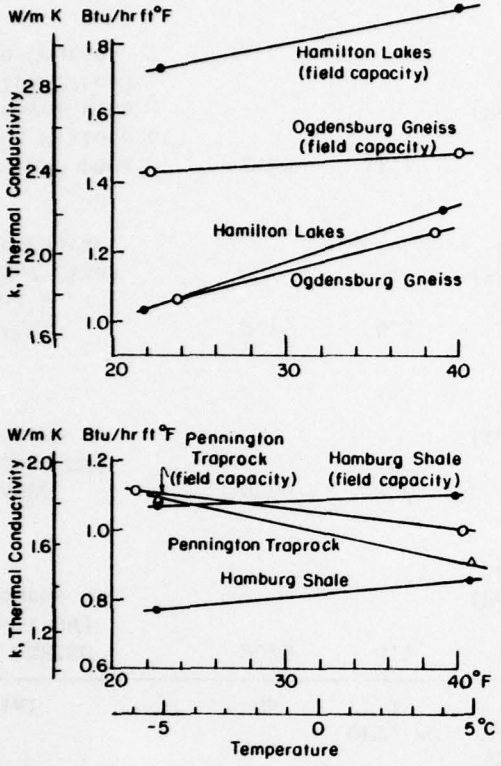


Figure 5. Thermal conductivity of base course materials from New Jersey, Guarded Hot Plate method.

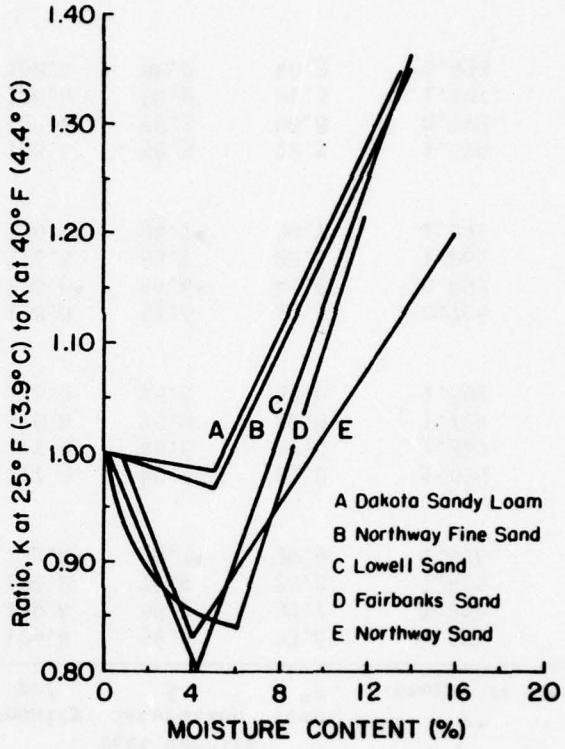


Figure 6. Variation of ratio of thermal conductivity above freezing to the thermal conductivity below freezing, for sandy soils (Kersten, 1949).

Pavement Materials (Probe Tests)

Two specimens each of Portland cement concrete and of three types of bituminous-stabilized pavement materials were received from the NJDOT and tested by the Probe method. A detailed description of the probe theory, sample preparation and experimental techniques is presented in Appendix C.

The bituminous mixtures included an asphaltic surface course (Mix #5), a binder course (Mix #2), and a base course (Mix #1). All the asphaltic concrete mixes contained about 5% bitumin (AC-20) by weight. All of the materials were tested in the as-received condition, (air-dry) both above and below freezing. Normally one sample of each material was used for the Probe tests. Properties of the pavement materials are given in Table 4. Properties of the stabilized open-graded drainage layer materials are also shown in the table but are discussed in the next section.

The Portland cement concrete samples had been prepared from a paving mixture on-site. The bituminous concrete samples were prepared in the NJDOT soils laboratory. Samples were cylinders 6 in. in diameter and approximately 8 in. long. A steel rod 0.070 in. in diameter was molded along the axis of each cylinder; it was removed prior to testing and the thermal conductivity probe inserted in its place.

Prior to testing, the specimens were brought to the test temperature with the probe in place. A pre-selected heating current was applied for a period of 10 or 15 minutes, during which the temperature of the probe was recorded. The current was then turned off and the specimen allowed to cool. Results obtained during both heating and cooling were averaged to obtain the value of thermal conductivity used in the pavement analysis; normally heating and cooling values were within 5% of each other. Table 5 presents the thermal conductivity test results for the pavement materials. These data are plotted in Figure 7.

Results from the tests indicated that the bituminous base course (Mix #1) and the binder course (Mix #2) had essentially the same thermal properties; therefore they were thereafter treated as one material. The results also indicate that the thermal conductivity of New Jersey pavement materials increases with lower temperatures, including temperatures below the freezing point of water. This trend is similar to results from Pennington traprock and may be the result of similar behavior except that the water is replaced by cement mortar or asphalt cement.

Bituminous-stabilized Open-graded Drainage Layer Materials (Probe Test)

Three gradations of bituminous-stabilized open-graded drainage-layer materials (crushed Mt. Hope granite) were prepared by the NJDOT. Mixes #1 and #2 contained 4% bitumin (AC-20) by weight, and Mix #3 contained

Table 4. Physical properties of New Jersey pavement materials for probe tests.

U.S. Sieve Size		Percentage Finer by Weight								
		P.C. concrete mixture			Bit. concrete mixtures			Open-graded bit. mixtures		
		Coarse	Fine	Comb'd.	#1 Base	#2 Binder	#5 Surface	#1	#2 (not tested)	#3
	mm	%	%	%	%	%	%	%	%	%
2"	50.8							100		
1-1/2	38.1				100	100			100	
1"	25.4	100		100	99.8	99.2		-	-	
3/4"	19.0	-		-	95.2	-		80	95	
1/2"	12.7	32		56	79.0	70.1	100	-	-	100
3/8"	9.5	20	100	49	70.7	-	98.6	50	65	70
#4	4.76	4	99	38	39.2	33.0	64.5	25	35	15
#8	2.38	1	92	34	28.8	16.9	50.5	-	-	5
#16	1.19	-	78	28	-	-	-	-	-	-
#30	0.595	-	49	18	-	-	-	-	-	-
#50	0.297	-	16	6	11.3	3.1	17.2	3	6	0
#100	0.149	-	4	1.4	-	-	-	-	-	-
#200	0.074	-	1	0.4	5.2	1.0	5.7	1	2	
Specific Gravity	Agg.	2.90	2.65	2.81	2.83	2.84	2.76	2.68	2.68	2.68
	Binder	P.C. = 3.15			AC-20 = 1.01			AC-20 = 1.01		
	Comb'd Solids*			2.87	2.60	2.60	2.54	2.55	2.55	2.55
Design Mixture (by wt)		P.C. = 18% Fine Agg. = 29% Coarse Agg. = 53%			Agg. = 95% AC-20 = 5%			Agg. = 96% AC-20 = 4%		97% 3%

*Calculated for design mixture.

Table 5. Thermal conductivity of New Jersey pavement materials, probe method.

Material	Cement/ Asphalt Content	Aggregate Voids (Total)	Air Voids	Aggre-gate Density	Overall Density	Test Temp.	Heat Input	Thermal Conductivity (Note 1)
	%	%	%	pcf	pcf	°F	Btu / ft-hr	Btu / hr-ft-°F
P.C. Concrete (Surface Course) Air-dry (Water/Cement) Ratio 6.0%	18.0	32.0	16.0	119.0	147.3	51.3	0.28	0.59
						50.7	0.29	0.62
						50.7	0.29	0.60
						50.7	0.18	0.57
						23.4	0.58	0.91
						23.4	0.58	0.87
						23.4	0.58	0.85
Bit. Concrete #1 (Base) #2 (Binder) Air-dry	5.0	18.2	6.0	144.8	152.3	50.5	0.18	0.55
						50.2	0.29	0.55
						23.9	0.29	0.66
						23.9	0.09	0.65
Bit. Concrete #5 (Surface) Air-dry	5.0	17.8	6.0	141.5	149.0	50.9	0.19	0.55
						50.5	0.19	0.55
						50.5	0.19	0.55
						23.5	0.22	0.74
						23.4	0.31	0.72
Open-graded Bit. Mix #1 Drainage) Air-dry	4.0 (Sample A)	28.2	20.4	120.0	124.9	37.8	0.30	0.69
						37.0	0.30	0.68
						24.4	0.29	0.73
						24.4	0.29	0.71
	4.0 (Sample B)	26.3	18.4	123.1	128.0	23.9	0.29	0.73
						23.9	0.43	0.75
Open-graded Bit. Mix #3 (Drainage) Air-dry	3.0	30.7	25.0	115.9	119.5	38.1	0.15	0.47
						23.4	0.10	0.52
						23.4	0.15	0.53
						23.4	0.15	0.45
						23.4	0.22	0.48
						23.4	0.22	0.47

NOTE 1: Average of heating and cooling values.

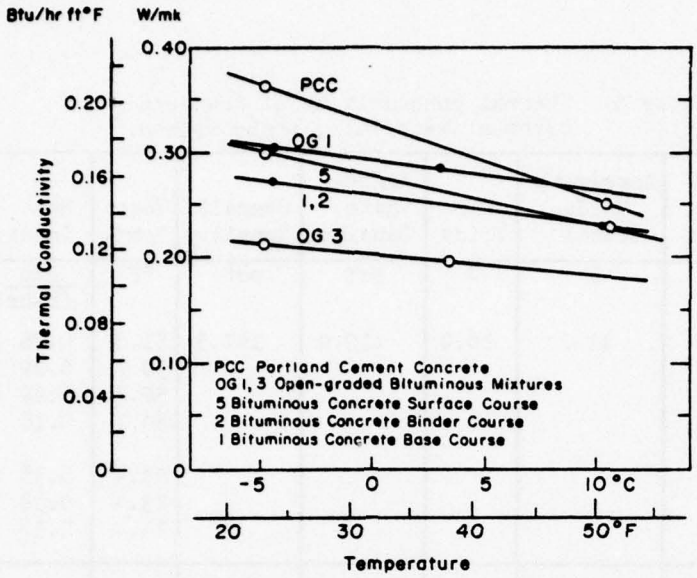


Figure 7. Thermal conductivity of New Jersey pavement materials, Probe method.

3% bitumen. Mix #2 was eventually eliminated from the testing program because the NJDOT felt that its density was too high and the resulting water permeability was too low for a suitable free draining material.

Two samples of open-graded Mix #1 (OG-1) and one sample of open-graded Mix #3 (OG-3) were tested, above and below freezing, by the Probe method. Tests were performed in the same manner as described for the pavement mixtures. Samples were 6-in.-diameter cylinders 8 in. long. All tests were in the as-received air-dry condition.

Physical properties of the bituminous stabilized open-graded mixtures are given in Table 4; test results are in Table 5. These results are shown graphically in Figure 7 to provide a comparison with the pavement mixtures.

It should be noted that in the above-freezing tests, the pavement materials were tested near 50°F to allow for higher temperatures caused by solar radiation absorbed at the surface. Because the open-graded mixtures would always be at a lower depth in the profile, they were tested at a temperature near 40°F.

The test results indicate that the thermal conductivity of OG-3 is about one-third lower than that of OG-1, both above and below freezing. Since the aggregate material is the same in both mixtures, the difference is evidently the combined effect of a lower dry density and lower asphalt content (115.9 pcf and 3% asphalt) for OG-3 as compared with OG-1 (121.5 pcf and 4% asphalt). These parameters, in turn, are controlled by the differences in gradation.

On the other hand, the thermal conductivity of OG-1 is similar to the pavement mixtures, which have much higher dry densities (142 to 145 pcf) and asphalt contents of 5%. In this case, the higher thermal conductivity of the quartzitic Mt. Hope aggregate evidently is sufficient to overcome the deficits in thermal conductivity associated with lower density, i.e. more voids, and a somewhat lower asphalt content.

Unstabilized Open-graded Mix #1 (Probe and Guarded Hot Plate Tests)

An aggregate having the same gradation as that used in the bituminous-stabilized open-graded Mix #1 (OG-1) was tested in a moist condition. Its use in this unstabilized condition would be as a drainage layer placed immediately beneath a rigid pavement.

To provide a comparison between test results from the Probe method and the Guarded Hot Plate method, the unstabilized OG-1 was tested using both techniques. A slight difference in gradation occurred because of the difference in the size of specimen used for each of the tests: for the Probe test specimen, material between the 2-in. and the 3/4-in. sizes was replaced with 3/4-in. material. The amount passing each of the smaller sieve sizes normally would have remained unchanged by this procedure; in actuality, the probe test sample was generally finer than the GHP sample (Appendix A). Apparently some breaking of corners occurred during previous handling.

This series of tests was performed both above and below freezing, by each test method. Dry density was 121.2 pcf (GHP) and 122.5 pcf (Probe); water content was the "field capacity" as defined above, which was approximately 4.5% on a dry weight basis for both specimens.

Results obtained for the unstabilized open-graded material are summarized in Table 6. For comparison, results found by Kersten for crushed granite having a density of 120 pcf and a moisture content of 4%, and a sandy soil at a moisture content of 4.5%, are also listed. These data are plotted in Figure 8.

The GPH values at 22°F and 40°F are between 70% and 75% of the Probe values. The reason is due partially to the somewhat finer gradation of the Probe sample, and also to the skin effect in the GHP sample, which tends to reduce the effective heat flow across the sample. The slight difference in density should have little effect.

In a comparison between the stabilized and unstabilized drainage layer material (both tested by the Probe method and at the same density), the frozen OG-1 which was stabilized had a 50% greater thermal conductivity than the frozen OG-1 which was unstabilized. In contrast, the unfrozen mixtures had such similar thermal conductivities that two other test values were obtained (at approximately 50°F and 65°F). The similarity persisted throughout the unfrozen zone; thus, water and asphalt seem to have nearly the same effect on the thermal conductivity of this open-graded aggregate.

Table 6. Thermal Conductivity of Unstabilized Open-Graded Mix #1.

Material	Water Content	Aggregate Voids (Total)	Air Voids	Aggregate Density	Overall Density	Test Temp.	Heat Input	Thermal Conductivity (Note 1)
	%	%	%	pcf	pcf	°F	Btu / ft-hr	Btu / hr-ft-°F
GUARDED HOT PLATE:								
Open-graded Mix#1 (Unstabilized)	4.5	27.5	18.8	121.1	126.5	39.4	-	0.44
						24.7	-	0.36
PROBE TEST:								
Open-graded Mix #1 (Unstabilized)	4.5	26.4	17.5	123.1	128.6	64.4	0.42	0.54
						51.3	0.31	0.60
						41.4	0.58	0.65
						41.3	0.58	0.66
						40.7	0.30	0.65
						39.9	0.58	0.65
						25.0	0.58	0.47
						23.5	0.30	0.51
						22.3	0.30	0.47
KERSTEN'S TESTS:								
Crushed Granite	4.0	28.0	20.3	120.0	124.8	40.0	-	0.83
						25.0	-	0.81
Sandy Soils	4.5	28.7	20.8	120.0	125.4	40.0	-	1.11
						25.0	-	0.90

NOTE 1: Average of heating and cooling values.

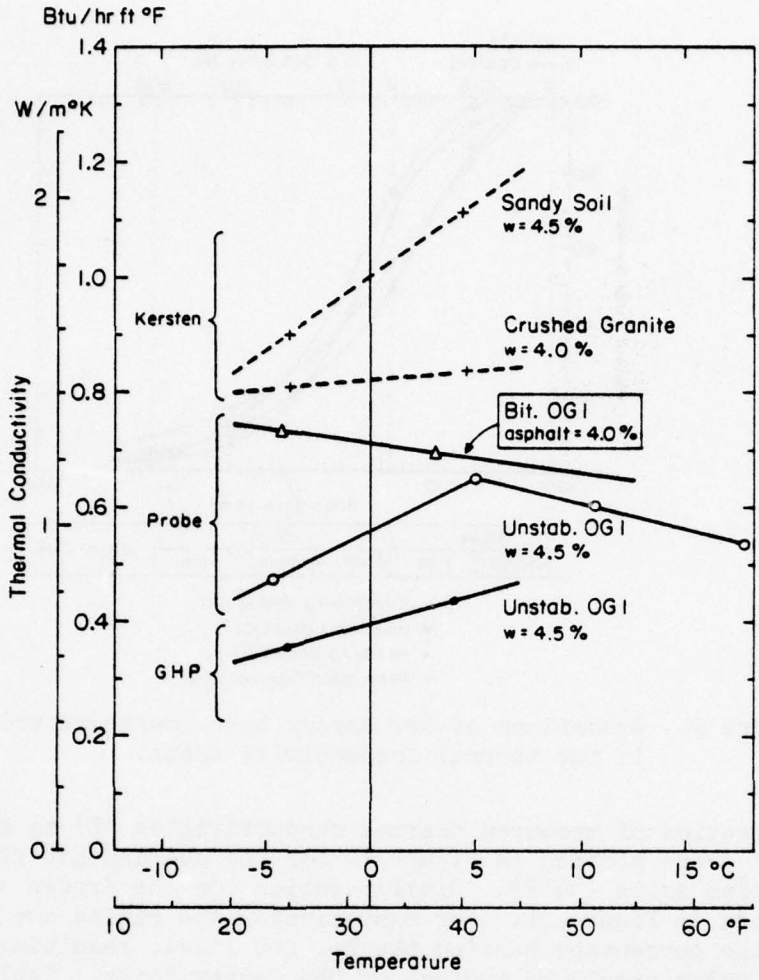


Figure 8. Thermal conductivity of unstabilized open-graded Mixture #1 (w = water content) by two methods, compared with the corresponding bituminous mixture and two Kersten materials.

Comparison with Kersten's Values

Kersten (1949) has given equations and graphs for estimating the thermal conductivity of two main groups of soils: sandy soils, and silt and clay soils. He states that the accuracy is approximately ±25% for an unknown soil of similar gradation, density, and water content.

The New Jersey base course aggregates tend to be sandy in character, and are therefore compared here with Kersten's predicted values for sandy soils. Gradation specifications are given in Appendix A, and gradation curves are plotted in Figure 9.

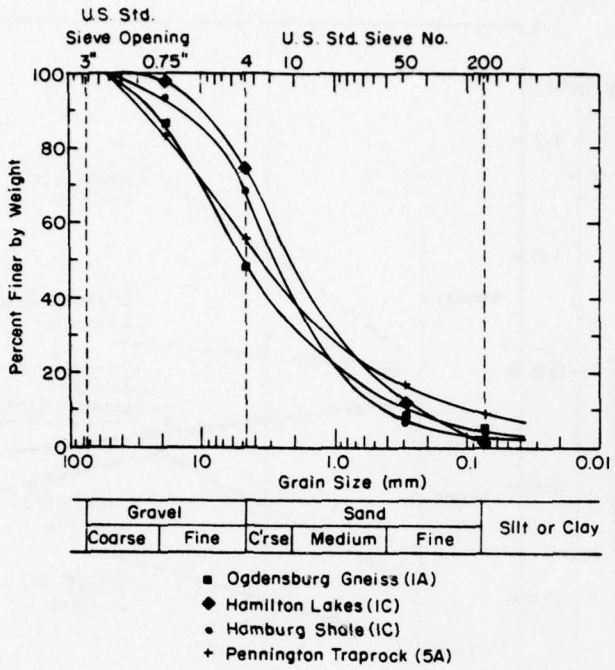


Figure 9. Gradations of New Jersey base course materials used in the thermal conductivity tests.

The ratios of measured thermal conductivities (K) to Kersten's values (K_s) are plotted in Figure 10 for the guarded hot plate tests on the unfrozen soils (40°F). Similar ratios for the frozen soils (22°F) are plotted in Figure 11. For convenience, the ratios are plotted against the percentage passing the No. 200 sieve, resulting in a relationship which should be typical of New Jersey soils. Table 7 gives supporting data for the two graphs.

The Hamilton Lakes sand (a well-graded sand) is the only one of the New Jersey soils which had thermal conductivity values, both frozen and unfrozen, similar to Kersten's data. The other three soils have conductivities ranging from 50% to 75% of the corresponding Kersten values. These results are nearly as one would expect, inasmuch as Kersten states that his relations are for soil aggregates high in quartz content. The Hamilton Lakes Sand is quartzitic, whereas the Hamburg Shale, Ogdensburg Gneiss, and Pennington Traprock have low quartz contents. As Kersten showed, a quartz aggregate has the highest thermal conductivity; a traprock aggregate has the lowest.

The four New Jersey soils considered in these tests were chosen to be representative of the majority of New Jersey soils used as base and subbase course materials. Consequently, it is suggested that the following

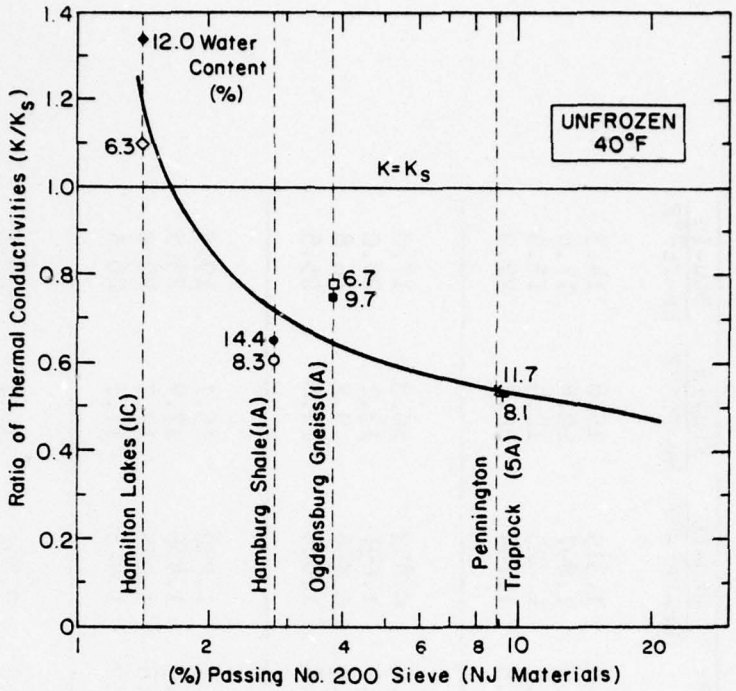


Figure 10. Comparison of thermal conductivity (K) of New Jersey soils with Kersten's values (K_s) for sandy soils, unfrozen.

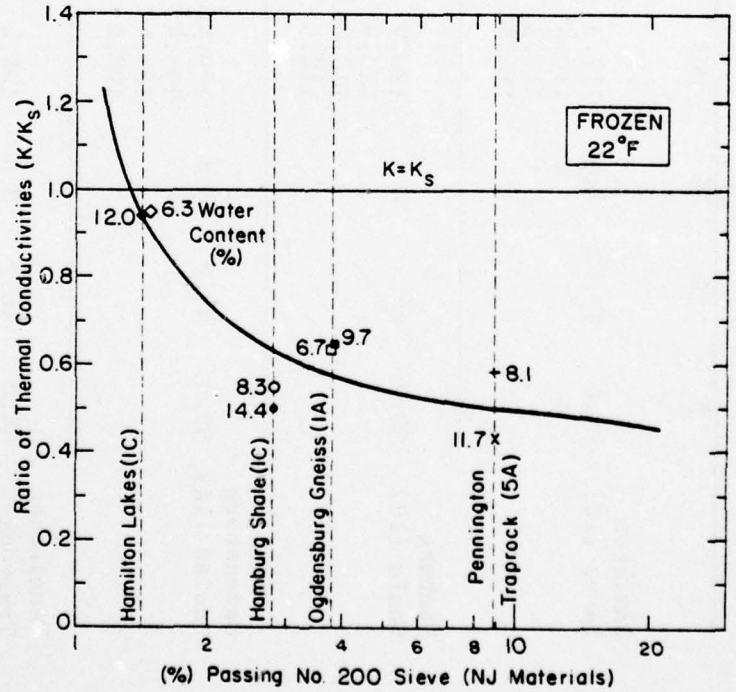


Figure 11. Comparison of thermal conductivity (K) of New Jersey soils with Kersten's values (K_s) for sandy soils, frozen.

Table 7. Comparison of thermal conductivity of New Jersey soils (K) (K) with Kersten's values (K_s).

New Jersey Soil	Gradation			Dry Density	Water Content	Temp.	Thermal Conductivity		
	Gravel	Sand	-#200				Measured	(K)	(K_s)
	%	%	%	pcf	%	°F	Btu-ft / hr-ft^2-°F	Btu-in / hr-ft^2°F	Btu-in / hr-ft^2°F
Hamilton Lakes (1C)	25.6	73.0	1.4	117.3	6.3	39.0	1.319	15.8	14.3
				116.8	12.0	39.8	1.897	22.8	17.0
				117.3	6.3	31.8	1.025	12.3	13.0
				116.8	12.0	22.8	1.724	20.7	22.0
Hamburg Shale (1C)	30.9	66.3	2.8	120.9	8.3	40.6	0.853	10.3	17.0
				120.9	14.4	39.7	1.091	13.1	20.0
				118.8	8.3	22.5	0.766	9.2	16.8
				116.5	14.4	22.6	1.065	12.8	25.5
Ogdensburg Gneiss (1A)	52.0	44.2	3.8	129.4	6.7	38.7	1.252	15.1	19.5
				133.4	9.7	39.9	1.476	17.7	23.5
				129.4	6.7	23.8	1.055	12.7	20.0
				128.2	9.7	22.2	1.425	17.1	26.5
Pennington Traprock (5A)	44.0	46.2	8.9	128.4	8.1	40.8	0.892	10.7	20.1
				128.4	11.7	40.2	0.993	11.9	22.0
				128.4	8.1	22.5	1.079	13.0	22.5
				128.4	11.7	21.3	1.107	13.3	31.0

procedure be used to estimate the thermal conductivity of other New Jersey base and subbase course soils:

a. For the new soil, measure or estimate the dry density and water content.

b. Using the equations below, or Figures 12 and 13, find Kersten's value of thermal conductivity (K_s) for a soil having the same density and water content.

For unfrozen sandy soils:

$$K_s = (0.7 \log w + 0.4)\, 10^{0.01\gamma}$$

For frozen sandy soils:

$$K_s = 0.076\ (10)^{0.013\gamma} + 0.032\ (10)^{0.0146\gamma}\ (w)$$

where K_s = thermal conductivity, Btu-in/ft^{2}hr °F
γ = dry density of soil, lb/ft^3
w = moisture content, % dry wt.

c. Enter the appropriate curve in Figure 10 or Figure 11 (unfrozen or frozen) with the percentage passing the No. 200 sieve for the New Jersey soil; find the corresponding ratio (K/K_s).

d. Multiply the Kersten value (K_s) by the ratio determined in step (c); the resulting value should be within approximately ±10% of the thermal conductivity of the New Jersey soil.

Kersten measured the thermal conductivity of only one bituminous paving mixture. It had a dry density of 138 pcf and an asphalt content of about 6%. The measured values for the New Jersey paving materials are approximately 70% of Kersten's value. This value is reasonable, considering that the New Jersey materials have asphalt contents of about 5%, and the aggregates have lower thermal conductivities, but higher dry densities.

The bituminous-stabilized open-graded mixtures have no direct counterpart in Kersten's series of tests. Consequently, no comparison of these can be made.

The New Jersey unstabilized open-graded materials in the moist condition are similar to Kersten's crushed granite. In a comparison with crushed granite (at 120 pcf and 4% water content), the unstabilized OG-1 had a conductivity only 60% of that found by Kersten at 25°F, and

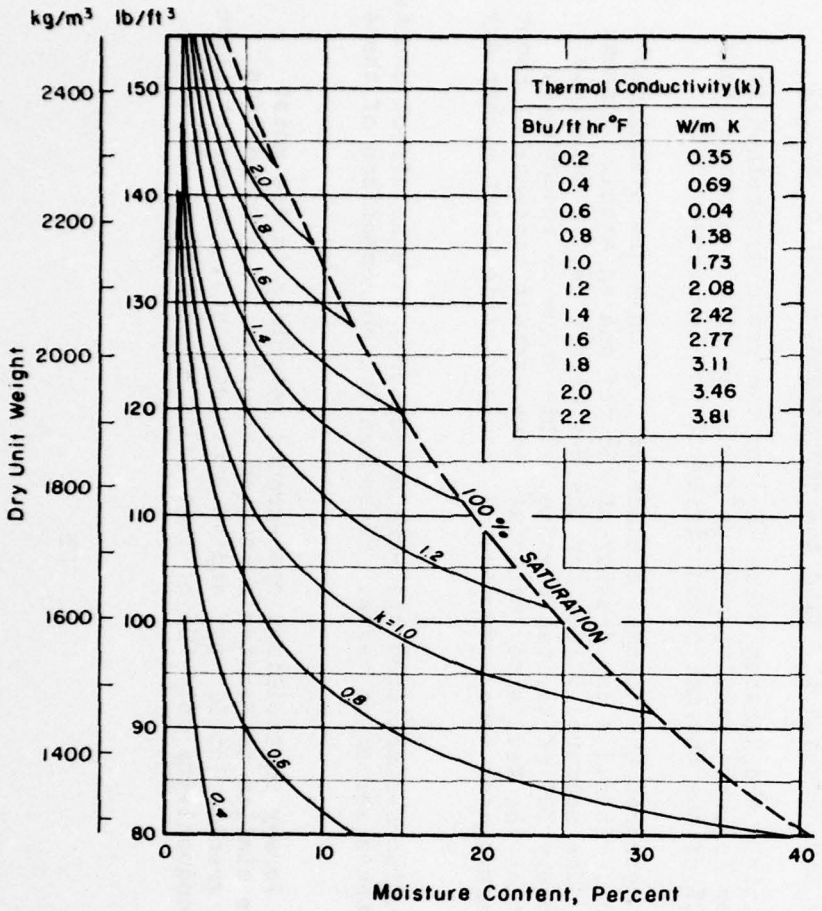


Figure 12. Kersten's thermal conductivity values for sandy soils, unfrozen (in Btu/ft hr °F).

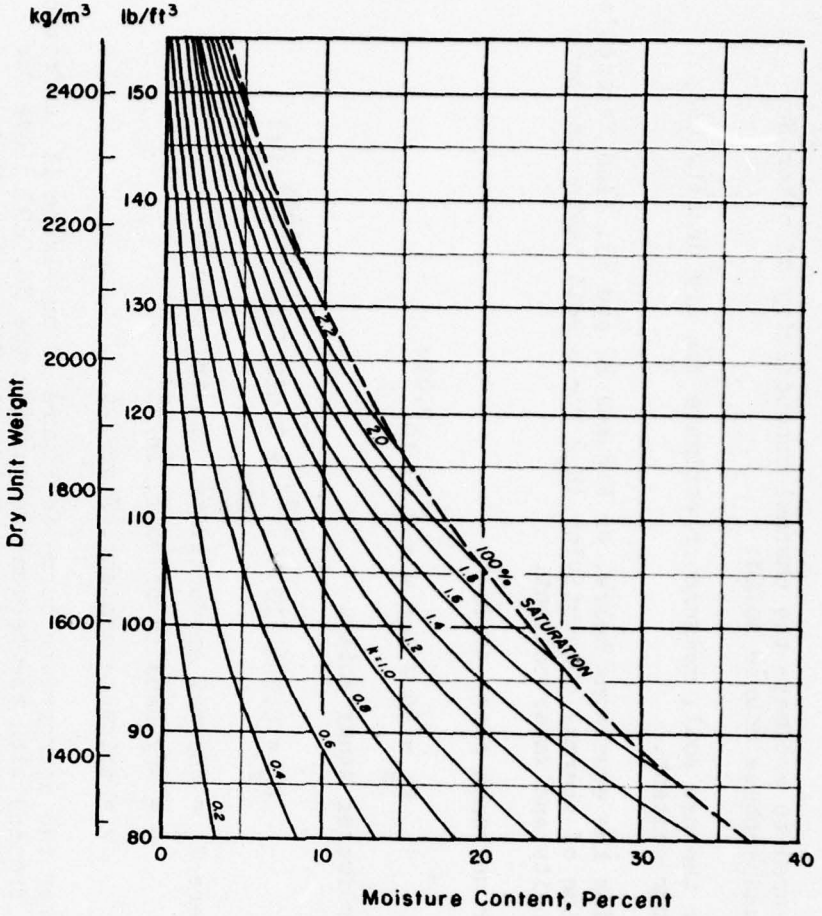


Figure 13. Kersten's thermal conducitivity values for sandy soils, frozen (in Btu/ft hr °F).

about 80% of Kersten's value at 40°F. These figures are based on the Probe test results, inasmuch as the Probe samples and Kersten's samples were both cylindrical, and heat flow was radial. The Guarded Hot Plate test results on the unstabilized OG-1 were 45% and 52%, respectively, of Kersten's values for crushed granite at 25°F and 40°F.

A comparison between thermal conductivity values of the unstabilized open-graded mixtures and those for Kersten's sandy soils illustrates that the equations and graphs given by Kersten should not be used to estimate the thermal conductivity of an open-graded mixture. The Probe test results were only 50% of Kersten's recommended values, both above and below freezing; the Guarded Hot Plate results were only 40% of Kersten's values.

A summary of results from the open-graded mixtures was given in Figure 8.

CALCULATED FROST DEPTHS

Modified Berggren Equation

The modified Berggren equation was developed by Aldrich and Paynter (1953) under a contract with the Arctic Construction and Frost Effects Laboratory (ACFEL). A complete description of the necessary assumptions and simplifications made during its development are contained in Aldrich and Paynter (1953). A few of the more important assumptions and limitations of the equation are discussed below.

Assumptions. The mathematical model assumes one-dimensional heat flow with the entire soil mass at its mean annual temperature prior to the initiation of freezing. When the freezing season starts, it is assumed that the surface temperature changes as a step function from the mean annual temperature to the mean temperature during the freezing season and remains at this new temperature throughout the entire freezing season. The effect of latent heat is considered as a heat sink at the moving frost line, and it is assumed that the soil water freezes at 32.0°F.

Limitations. The model on which the modified Berggren equation is based further assumes an isothermal system at the beginning of the freezing season and therefore cannot normally be used to calculate thaw depths during the frost melting period. Aldrich and Paynter (1953) state, "Attempts to calculate depth-time curves based on partial freezing indices are likely to encounter substantial errors."

The modified Berggren equation for homogeneous soils is:

$$X = \lambda\sqrt{\frac{48\ K\ nF}{L}}$$

where

X = depth of freeze, ft
K = thermal conductivity of the soil, Btu/ft hr°F
L = volumetric latent heat of fusion, Btu/cu ft
n = conversion factor for air index to surface index, dimensionless
F = air-freezing index, °F-days
λ = a coefficient which takes into consideration the effect of temperature changes in the soil mass

The λ coefficient is a function of the freezing index, the mean annual temperature of the site, and the thermal properties of the soil. Freezing of low-moisture-content soils in the lower latitudes is greatly influenced by this coefficient. It is determined by two factors: the thermal ratio, α, and the fusion parameter, μ. The thermal ratio is:

$$\alpha = \frac{v_o}{v_s}$$

where

v_o = mean annual site temperature minus 32°F (MAT -32)
v_s = surface freezing index, nF, divided by the length of the freezing season, t, or nF/t.

The fusion parameter is equal to the expression

$$\mu = v_s \times \frac{C}{L}$$

where

v_s = same as previously defined
C = average volumetric heat capacity of the soil, Btu/cu ft°F
L = same as previously defined

A multilayer solution to the modified Berggren equation is used for nonhomogeneous, i.e. layered, soils, by determining that portion of the surface freezing index required to penetrate each layer. The sum of the thicknesses of all the frozen layers is the depth of frost. The partial freezing index required to penetrate the top layer is given by

$$F_1 = \frac{L_1 d_1}{24\lambda_1^2}\left(\frac{R_1}{2}\right)$$

where

d_1 = thickness of the soil layer, ft
$R_1 = \frac{d_1}{K_1}$ = thermal resistance of the layer, sq ft hr °F/Btu

The partial freezing index required to penetrate the nth layer is

$$F_n = \frac{L_n d_n}{24\lambda_n^2}\left(\Sigma R + \frac{R_n}{2}\right)$$

where ΣR is the total thermal resistance above the nth layer and equals

$$\Sigma R = R_1 + R_2 + R_3 \ldots + R_{n-1}$$

When the summation of the partial indexes, $F_1 + F_2 + F_3 \ldots + F_n$, is equal to the surface freezing index, the frost depth has been determined.

Aitken and Berg (1968) presented a computer program solving the modified Berggren equation for multiple-layered soil and pavement profiles. A revised version of that computer program was used in this study; this program is listed in Appendix D. Aitken and Berg discuss, in greater detail, the various parameters that are used in the computer program to solve the modified Berggren equation.

Computations

To establish initial and upper boundary conditions for the modified Berggren equation solutions, monthly and annual weather records from weather stations in New Jersey were surveyed. Three sites with long term records were chosen for the calculations. Trenton and Atlantic City were chosen for southern sites and Newton was chosen as representative of a northern site. Data used in the modified Berggren equation solutions are summarized in Table 8.

Table 8. Initial and boundary conditions for modified Berggren equation solutions.

	Mean Freezing Season		
	Newton	Trenton	Atlantic City
Air Freezing Index, °F-days	480	130	50
Length of Season, days	95	40	10
Mean Annual Air Temperature, °F	49.6	53.8	54.4
N-Factor (Portland cement)	0.44	0.44	0.44
N-Factor (Asphaltic concrete)	0.50	0.50	0.50

	Design Freezing Season		
	Newton	Trenton	Atlantic City
Air Freezing Index, °F-days	900	320	250
Length of Season, days	110	83	82
Mean Annual Air Temperature, °F	49.6	53.8	54.4
N-Factor (Portland cement)	0.44	0.44	0.44
N-Factor (Asphaltic concrete)	0.50	0.50	0.50

Two sets of freezing season data were used in the solutions, one an average or mean freezing season and the other a design freezing season. The design freezing season is chosen as the average of the three coldest winters in the 1946-1976 period.

Table 9 contains gradation specifications for base and subbase materials used in New Jersey highway pavements, Table 10 contains thicknesses of various layers in typical New Jersey pavement sections without a drainage layer, and Table 11 contains proposed thicknesses of various layers for New Jersey pavement sections incorporating either stabilized or unstabilized drainage layers. Both of these sets of information were furnished by engineers from the New Jersey Department of Transportation. Thermal properties for pavement sections without drainage layers are given on Table 12 and thermal properties for pavement cross sections incorporating drainage layers on Table 13.

Table 9. Gradation specifications for base and subbase materials in New Jersey

Designation	1A	1B	1C	2B	5A
Sieve Size	Percentage by Weight Passing				
4"	100		100		
2-1/2"					100
2"	70-100	100		100	
3/4"	50-95	65-100	60-100	70-100	50-90
No. 4	30-60	40-75	30-100	30-80	25-60
No. 50	5-25	5-30	5-35	10-35	5-25
No. 200	0-7	0-7	0-5	5-12	3-12

NOTES: 1A is usually used as a base course beneath PCC pavement.

1B may be used as a base and/or subbase course beneath a PCC pavement.

1C is generally used as a subbase beneath 1A or 5A base courses.

2B is a gravel base course commonly used beneath shoulders and secondary roads.

5A is a quarry-processed material and is used beneath bituminous stabilized base course, in bituminous concrete pavements.

Table 10. Typical New Jersey pavement sections without a drainage layer.

Section #1 Southern New Jersey:	9" PCC 12" Base (1B) Subgrade*
Section #3 Northern New Jersey:	9" PCC 6" Base (1A) 6" Subbase (1C) Subgrade*
Section #5 Southern New Jersey:	3" FABC 6" BSBC 5" Base (2B) 5" Subbase (1B)
Section #6 Southern New Jersey	3" FABC 6" BSBC 6" Base (5A) 11" Subbase (1B)
Section #7 Northern New Jersey	3" FABC 6" BSBC 6" Base (5A) 12" Subbase (1C)

PCC - Portland Cement Concrete

FABC - Fine Aggregate Bituminous Course

BSBC - Bituminous Stabilized Base Course

*Values for 1C material were used in the thermal analysis.

See Tables 6 and 9 for properties of the granular materials.

Table 11. Tentative New Jersey pavement sections incorporating a drainage layer.

Portland Cement Concrete Pavements

Section #2 Southern New Jersey 2A OGL is stabilized 2B OGL is unstabilized	9" PCC 6" drainage layer (OGL) 6" Subbase (1B or 2B) with the top 2" lime fly ash or cement stabilized.
Section #4 Northern New Jersey 4A OGL is stabilized 4B OGL is unstabilized	9" PCC 6" Drainage layer (OGL) 6" Subbase (1A) with 2" Stabilization

Bituminous Concrete Pavements

Section #8 Southern New Jersey	3" FABC 6" BSBC course 5" Drainage layer (OGL) 2" Subbase (1B) Stabilized w/cement or lime-fly ash 3" Subbase (1B)
Section #9 Northern New Jersey	3" FABC 6" BSBC 6" Drainage layer (OGL) 2" Subbase (1C) stabilized w/cement or lime-fly ash 10" Subbase (1C)

OGL - Open graded layer. This layer may be stabilized with an asphalt cement in either type of pavement or used unstabilized under Portland cement concrete pavements.

Table 12. Thermal properties for pavement sections without a drainage layer

Material Property	γ_d	w	d	k_{dry}	k_{wet}	C_{dry}	C_{wet}	L_{dry}	L_{wet}
	lb/ft^3	%	ft	$\frac{Btu}{fthr°F}$	$\frac{Btu}{fthr°F}$	$\frac{Btu}{ft^3°F}$	$\frac{Btu}{ft^3°F}$	$\frac{Btu}{ft^3}$	$\frac{Btu}{ft^3}$
SECTION 1									
PCC	140	0	0.75	0.57	-	30.0	-	0	-
Base (1B)	129	6.7(9.7)	1 .00	1.15	1.45	28.41	31.31	1245	1802
Subgrade (1C)	118	14.4	5.00	-	1.08	-	32.80	-	2447
SECTION 3									
PCC	140	0	0.75	0.57	-	30.0	-	-	-
Base (1A)	129	6.7(9.7)	0.50	1.15	1.45	28.41	31.31	1245	1082
Subbase (1C)	117	6.3(12.0)	0.50	1.17	1.81	25.42	30.42	1061	2022
Subgrade (1C)	118	14.4	10.00	-	1.08	-	32.80	-	2447
SECTION 5									
FABC	138	0	0.25	0.54	-	28.0	-	0	-
BSBC	138	0	0.50	0.53	-	28.0	-	0	-
Base (2B)	129	6.7(9.7)	0.42	1.15	1.45	28.41	31.31	1245	1802
Subbase (1B)	129	6.7(9.7)	0.42	1.15	1.45	28.41	30.42	1245	1802
Subgrade (1C)	118	14.4	5.00	-	1.08	-	32.80	-	2447
SECTION 6									
FABC	138	0	0.25	0.54	-	28.0	-	0	-
BSBC	138	0	0.50	0.53	-	28.0	-	0	-
Base (5A)	128	8.1(11.7)	0.50	0.99	1.05	29.54	37.54	1493	2157
Subbase (1B)	129	6.7(9.7)	0.92	1.15	1.45	28.41	31.31	1245	1802
Subgrade (1C)	118	14.4	15.00	-	1.08	-	32.80	-	2447
SECTION 7									
FABC	138	0	0.25	0.54	-	28.0	-	0	-
BSBC	138	0	0.50	0.53	-	28.0	-	0	-
Base (5A)	128	8.1(11.7)	0.50	0.99	1.05	29.54	32.99	1493	2157
Subbase (1C)	117	6.3(12.0)	1.00	1.17	1.81	25.42	30.42	1061	2022

Note: Moisture contents in parentheses are the "wet" values.

Table 13. Thermal properties for pavement sections with a drainage layer

Material	γ_d	w	d	k	C	L
	lb/ft^3	%	ft	$\frac{Btu}{fthr^oF}$	$\frac{Btu}{ft^3{}^oF}$	$\frac{Btu}{ft^3}$
SECTION 2A						
PCC	140	0	0.75	0.57	30.0	0
OGL (stabilized)	120	7.0	0.5	0.51	26.6	1204
Subbase (1B)	129	6.7	0.5	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.0	1.08	32.8	2447
SECTION 2B						
PCC	140	0	0.75	0.57	30.0	0
OGL (unstabilized)	121	4.5	0.50	0.40	25.9	784
Subbase (1B)	129	6.7	0.50	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.00	1.08	32.8	2447
SECTION 4A						
PCC	140	0	0.75	0.57	30.0	0
OGL (stabilized)	120	7.0	0.5	0.51	26.6	1204
Subbase (1A)	129	6.7	0.5	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.0	1.08	32.8	2447
SECTION 4B						
PCC	140	0	0.75	0.57	30.0	0
OGL (unstabilized)	121	4.5	0.50	0.40	25.9	784
Subbase (1A)	129	6.7	0.50	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.00	1.08	32.8	2447
SECTION 8						
FABC	138	0	0.25	0.54	28.0	0
BSBC	138	0	0.50	0.54	28.0	0
OGL (stabilized)	120	7.0	0.42	0.51	26.6	1204
Subbase (1B)	129	6.7	0.42	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.0	1.08	32.8	2447
SECTION 9						
FABC	138	0	0.25	0.54	28.0	0
BSBC	138	0	0.50	0.54	28.0	0
OGL (stabilized)	120	7.0	0.50	0.51	26.6	1204
Subgrade (1C)	118	14.4	5.0	1.08	32.8	2447

Values of thermal properties shown in the tables are average values for frozen and unfrozen conditions. The volumetric heat capacity, which is the quantity of heat required to change the temperature of a unit volume by one degree, is determined from

$$C = \gamma_d [c + 0.75 (w/100)]$$

where

C = volumetric heat capacity, Btu/cu ft°F
γ_d = dry unit weight of soil, lb/cu ft
c = specific heat of dry soil, Btu/lb °F
w = moisture content of soil, % dry weight.

The volumetric latent heat of fusion is the quantity of heat required to freeze the water in a unit volume of soil without a change in temperature. It is determined from

$$L = 144 \gamma_d (w/100)$$

where the symbols are as defined above.

The thermal conductivity values used in most of the computations were determined from:

$$K_a = (K_u + K_f)/2$$

where

K_a = average thermal conductivity, Btu/ft hr°F
K_u = thermal conductivity at about 40°F
K_f = thermal conductivity at about 22°F

The values of K_u and K_f were generally those obtained from the laboratory measurements made during this study. Thermal conductivity values of the Portland cement and asphaltic concrete specimen furnished to CRREL by the NJDOT were lower than the values normally used. Therefore several frost depth calculations were made using the larger values. The outcome of the computations is discussed in the section "Results."

Information from Tables 8, 10, 11, 12 and 13 was used in the computer program discussed previously to calculate maximum frost penetration depths for various conditions. For each pavement profile shown in Table 10 solutions were run using the mean freezing index and the thermal properties in the dry state to compute a maximum frost penetration depth for the mean year and again in the wet state to compute a minimum frost penetration depth for the mean year. Two similar sets of calculations were made for each of the design freezing seasons. Therefore, four frost penetration depths were computed for each of the pavement sections listed in Table 10. Calculations were made using only the "dry" soil properties for profiles shown in Table 11.

Results

Table 14 contains the results of the computations. The first part of the table contains data from pavement profiles without drainage layers and the second part contains data for pavement profiles incorporating drainage layers. Frost penetration in Section 1 progresses only slightly beneath the Portland cement concrete pavement. In Section 3 the frost penetrates slightly into the base material, and in Section 5, the frost barely penetrates the pavement and the bituminous stabilized base course. In Section 6 the same phenomenon occurs and in Section 7 frost penetrates into the base course beneath the asphalt pavement and the bituminous stabilized base course. Thus, for pavement profiles without drainage layers the calculated maximum frost depth in the southern part of the state barely penetrates the pavement section during either a mean or a design season or with wet or dry subsoils. In the northern part of New Jersey, frost may penetrate into the subbase material.

The last six calculations in the first part of Table 14 were made to illustrate the influence of higher thermal conductivities of the pavement layer. For the southern sites, increasing the thermal conductivity of the pavement did not significantly alter the computed frost depths. At the northern site, the computed frost depth increased by about 25% when the thermal conductivities of the pavement layers were increased to the values shown in the footnote to Table 14.

Data in the lower part of Table 14 indicate that frost penetration into pavement profiles with open-graded layers is equal to or slightly less than beneath similar pavement profiles without drainage layers. The difference is due to the lower thermal conductivity of the open-graded material. Whether the open-graded material is stabilized or unstabilized has little or no influence on the depth of frost penetration into the pavement.

These computations suggest that frost penetration beneath a New Jersey highway containing a drainage layer will be equal to or slightly less than the frost penetration beneath a "typical" highway currently constructed there.

The n-factors shown on Table 8 were used to compute the maximum frost depths shown on Table 14. Had the average n-factor for Portland cement and asphaltic concrete pavements shown on Table 1 been used in the calculations, maximum frost depths on Table 14 would increase by 8% for Portland cement concrete pavements and 13% for asphaltic concrete pavements. Thus the maximum difference in the calculated values would be an increase of 0.3 ft for Section 7** in Table 14.

Frost penetration depths calculated in this report have used a design freezing index which was determined by the average of the three coldest

Table 14. Maximum seasonal frost penetration determined with the modified Berggren equation.

Profiles without drainage layers:

Section	Conditions	Maximum Seasonal Frost Penetration Dry Soil	Wet Soil	Thickness of pavement section
1	Mean Season (Atlantic City)	0.8 ft	0.8 ft	1.75 ft
1	Mean Season (Trenton)	0.8	0.8	1.75
1	Design Season (Atlantic City)	0.8	0.8	1.75
1	Design Season (Trenton)	0.8	0.8	1.75
3	Mean Season (Newton)	1.0	0.9	1.75
3	Design Season (Newton)	1.6	1.6	1.75
5	Mean Season (Atlantic City)	0.8	0.8	1.58
5	Mean Season (Trenton)	0.8	0.8	1.58
5	Design Season (Atlantic City)	0.8	0.8	1.58
5	Design Season (Trenton)	0.8	0.8	1.58
6	Mean Season (Atlantic City)	0.8	0.8	2.17
6	Mean Season (Trenton)	0.8	0.8	2.17
6	Design Season (Atlantic City)	0.8	0.8	2.17
6	Design Season (Trenton)	0.8	0.8	2.17
7	Mean Season (Newton)	1.0	1.0	2.25
7	Design Season (Newton)	1.7	1.5	2.25
1*	Design Season (Atlantic City)	0.8	-	1.75
1*	Design Season (Trenton)	0.8	-	1.75
3*	Design Season (Newton)	2.0	-	1.58
5**	Design Season (Atlantic City)	0.8	-	1.58
5**	Design Season (Trenton)	0.8	-	1.58
7**	Design Season (Newton)	2.1	-	2.25

Profiles with drainage layers:

Section	Conditions	Dry Soil	Thickness of pavement section
2A	Mean Season (Atlantic City)	0.8 ft	1.75 ft
2A	Mean Season (Trenton)	0.8	1.75
2A	Design Season (Atlantic City)	0.8	1.75
2A	Design Season (Trenton)	0.8	1.75
2B	Mean Season (Atlantic City)	0.8	1.75
2B	Mean Season (Trenton)	0.8	1.75
2B	Design Season (Atlantic City)	0.8	1.75
2B	Design Season (Trenton)	0.8	1.75
4A	Mean Season (Newton)	1.0	1.75
4A	Design Season (Newton)	1.4	1.75
4B	Mean Season (Newton)	0.9	1.75
4B	Design Season (Newton)	1.3	1.75

Table 14 (cont'd)

Section	Conditions	Dry Soil	Thickness pavement section
8	Mean Season (Atlantic City)	0.8	1.58
8	Mean Season (Trenton)	0.8	1.58
8	Design Season (Atlantic City)	0.8	1.58
8	Design Season (Trenton)	0.8	1.58
9	Mean Season (Newton)	1.0	1.42
9	Design Season (Newton)	1.4	1.42

* Thermal conductivity of PCC increased to 1.0 Btu/ft hr °F, other data same as in Table 12.

** Thermal conductivities of FABC and BSBC increased to 0.86 Btu/ft hr °F, other data same as in Table 12.

winters in the last 30 years of record. Figure 14, from Department of the Army (1966), illustrates the relationship between mean air freezing indexes and indexes determined for colder years for 30 consecutive years. Another method of developing design air freezing index values is to use the relationships developed by Berube (1967), Figure 15. His method is statistically based and the designer can choose the design life and the level of probability that a computed freezing index will be exceeded during the design period. The computed freezing index can then be used to estimate the frost depth. An example of the relationships that can be obtained from this procedure is shown in Figure 16. Conditions similar to those from Newton where the mean freezing index equals 480°F-days (Table 8), with a pavement cross section similar to Section 7 were used in the computations for the figure. A design engineer may wish to study the effect of differing design life on the maximum frost penetration depth beneath the pavement surface. He may also wish to evaluate the effect on frost penetration of different probabilities of a design index being exceeded. Results from a sample study are shown in Table 15. These indicate that for a given design life, the frost depth increases as the probability that the design freezing index will be exceeded decreases. Note that the probability the 900 °F-day design index previously used in the calculations will be exceeded is 85% for a 30-year design life but only 50% for a 10-year design life. The frost depth varied by more than 40% (2.4-1.7/1.7) depending upon the design life and the probability that the design freezing index will be exceeded at least once during this design life. Using Figure 15 the design engineer can determine a design frost depth based on the design life and importance of the road, i.e. the more important the road, the lower the allowed probability that the design freezing index can be exceeded.

OTHER CONSIDERATIONS

The coldest winter, which will normally cause the deepest frost penetration, may not cause the most extensive damage to pavements. For

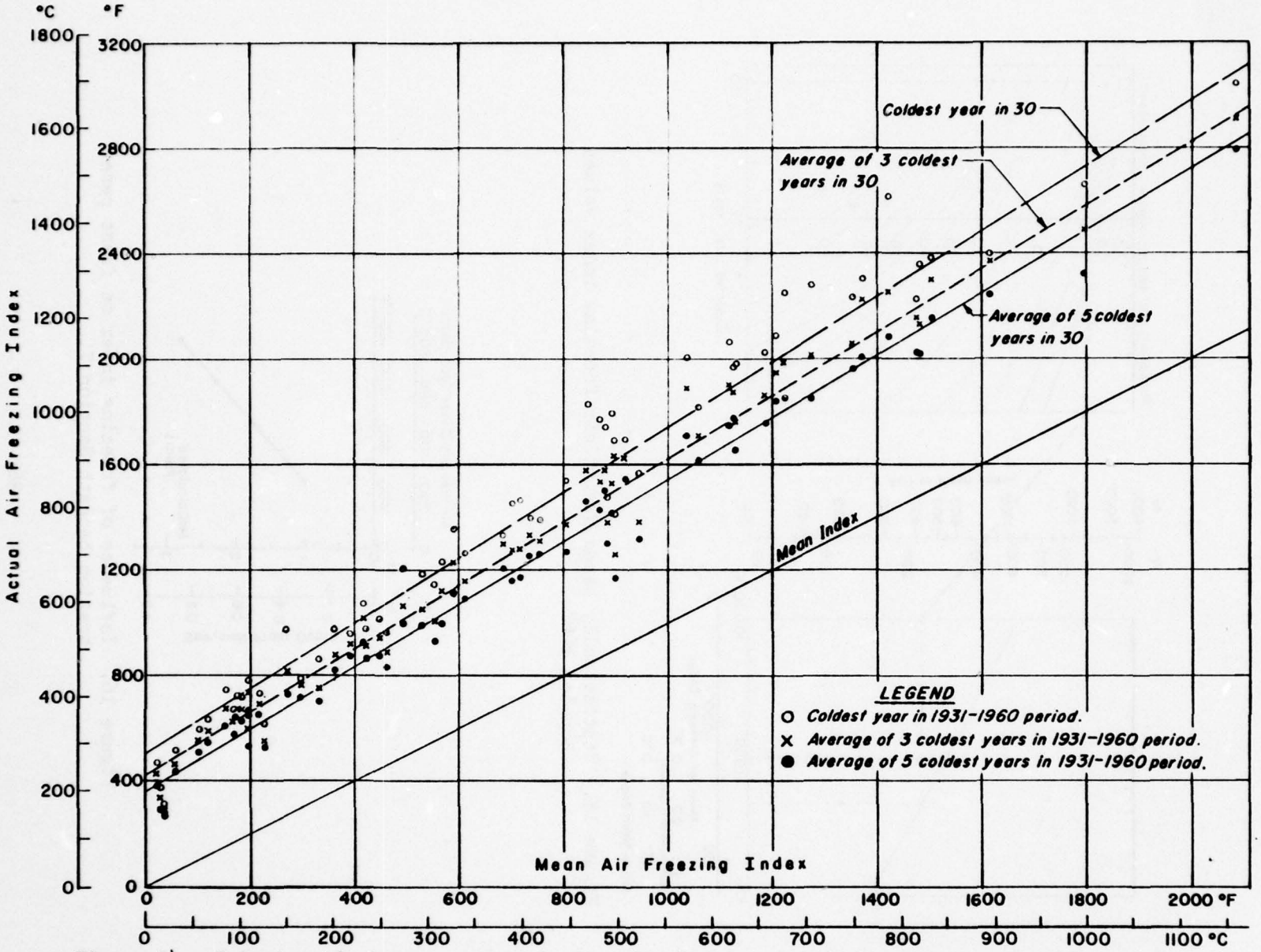


Figure 14. Relationship between mean air freezing index and freezing indexes of colder years in New England during 1931-1960 period.

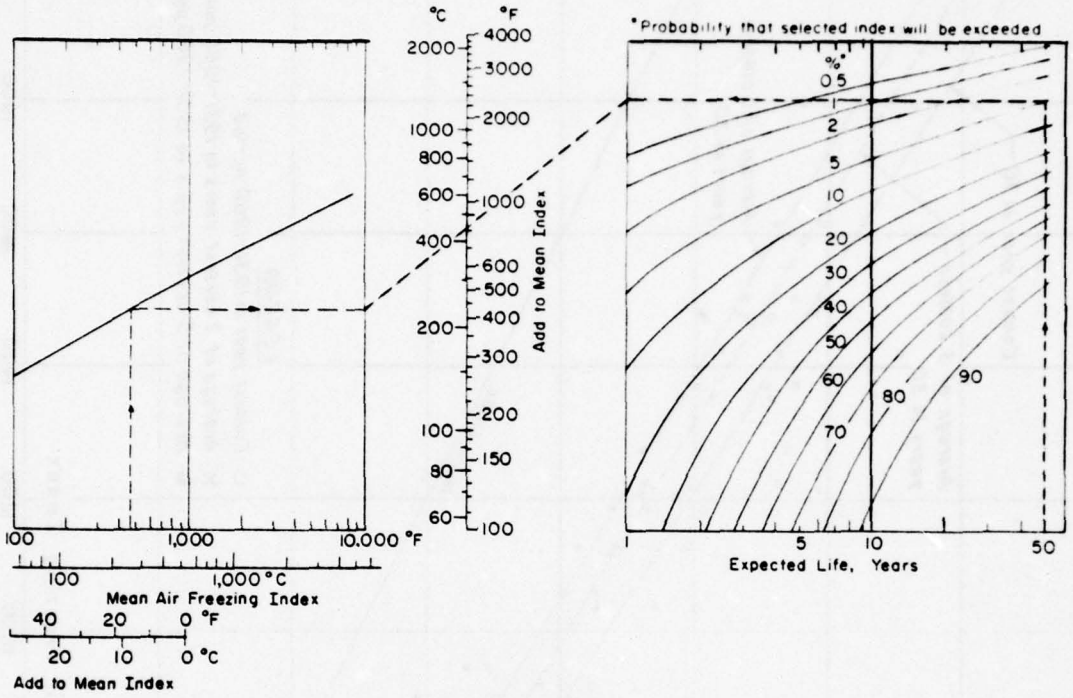


Figure 15. Statistically based design air freezing index values (Berube, 1967).

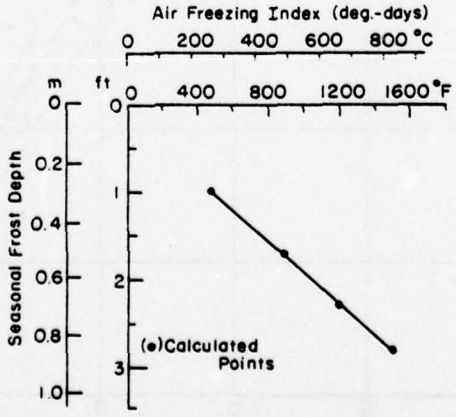


Figure 16. Influence of freezing index on frost penetration beneath Section 7.

Table 15. Thaw depths from a statistically based design method.

Design Life Years	Probability of Being Exceeded %	Design Index °F-days	Frost Depth ft
10	5	1120	2.1
10	10	1080	2.0
10	20	1020	1.9
10	50	900	1.7
30	5	1260	2.4
30	10	1180	2.2
30	20	1130	2.1
30	50	1020	1.9
30	85	900	1.7
50	5	1280	2.4
50	10	1230	2.3
50	20	1170	2.2
50	50	1070	2.0

example, Cosaboom (1977) noted that the 1975-1976 winter, although relatively mild, caused several freeze-thaw cycles in the upper pavement layers, resulting in extensive damage to New Jersey pavements. In contrast the 1976-1977 winter was extremely cold with very few freeze-thaw cycles in the pavement layers. Relatively little pavement damage was experienced during the 1976-1977 winter.

Since quantitative data on moisture contents, precipitation and temperatures within the pavement systems are not available, it is impossible to state with certainty the exact cause of this difference in behavior. From the qualitative data which are available, and from results of field and laboratory tests conducted by others, for example Jessberger and Carbee (1970) and Fredrickson (1963), on the reduction of strength of soils upon thawing, the major problem is caused by excessive water in the previously frozen material. Frequent freeze-thaw cycles may permit more opportunity for moisture to enter the subsurface materials through cracks in the pavement. If vertical and lateral drainage do not permit sufficient water removal between successive freeze-thaw cycles, expansion of excess water upon freezing can damage pavements at cracks and joints, and extensive weakening at the cracks and joints may cause additional breakage upon thawing. Incorporation of a drainage layer beneath the pavement should reduce the severity of this problem. The drainage layer must be part of a reliable subsurface drainage system wherein all components function satisfactorily under frozen and unfrozen conditions. Cedegren et al. (1972), the Department of Transportation (1972) and Cedegren (1974) provide guidelines for designing and installing drainage layers for pavement systems.

Since the thermal conductivities of the open-graded drainage layers are generally somewhat lower than the thermal conductivities of the base and subbase course materials, a question has been raised as to whether this will cause the pavement surface over an open-graded drainage layer to become icy before a pavement without the drainage layer does (Cosaboom, 1977). This problem has occurred when insulation layers are incorporated in pavement cross-sections and is similar to the behavior of bridge decks which sometimes become ice-covered when the adjacent roadway is not. The difference in thermal conductivity between the open-graded layer (0.54 Btu/ft hr °F) and the base course and subbase course materials (about 1.1 Btu/ft hr °F) is considerably less than the difference between a thermal insulating layer (0.02 Btu/ft hr °F) and the base course and subbase course materials. Therefore we do not anticipate any significant difference in surface conditions between pavements with and without an open-graded drainage layer. Other environmental conditions, i.e. shaded or open, south side or north side of a hill, etc., will probably cause more differences in surface conditions than the use of an open-graded drainage layer. We suggest, however, that when segments of highways containing open-graded drainage layers are constructed in New Jersey, careful observations be made to determine whether differential icing conditions occur over these pavements.

FUTURE WORK

At the suggestion of CRREL engineers, New Jersey Department of Transportation engineers and technicians installed frost tubes for monitoring frost penetration depths and access tubes for use in monitoring subsurface moisture contents with a nuclear probe. During the 1976-1977 winter, the New Jersey Department of Transportation measured frost penetration depths and moisture contents at several sites. Air and surface temperatures will again be measured at selected sites in New Jersey and n-factors and freezing indexes will be calculated from the temperature data. The New Jersey Department of Transportation has furnished these data to CRREL and we are comparing measured frost penetration depths with depths calculated using the modified Berggren equation. These data will be used to evaluate our confidence in the calculated frost penetration depths and to validate our tentative conclusion that the use of drainage layers should have no detrimental effects on frost action in New Jersey pavement systems.

LITERATURE CITED

Aitken, G.W. and R.L. Berg (1968). Digital Solution of the Modified Berggren Equation to Calculate Depths of Freeze or Thaw in Multi-layered Systems. CRREL Special Report 122.

Aldrich, H.P. and H.M. Paynter (1953). Analytical Studies of Freezing and Thawing of Soils, First Interim Report. Arctic Construction and Frost Effects Laboratory (ACFEL) Technical Report 42.

Berg, R.L. (In preparation). Improved Drainage and Frost Action Criteria for New Jersey Pavement Design, Phase II (Data Analysis). Final contract report by CRREL for NJDOT.

Berube, P.H. (1967). Standard Deviations of Air Freezing and Thawing Indexes at Selected Stations in North America. CRREL Technical Note, unpublished.

Cosaboom, B. (1977). Personal Communication.

Cedegren, H.R. (1974). Drainage of Highway and Airfield Pavements. John Wiley & Sons, New York. 258 pp.

Cedegren, H.R., K.H. O'Brien and J.A. Arman (1972). Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Sections. Report No. FHWA-RD-72-30. Department of Transportation, Federal Highway Administration, 400 7th St. S.W., Washington, DC 20590. 24 pp.

Department of the Army (1966). Arctic and Subarctic Construction - Calculation Methods for Determination of Depths of Freeze and Thaw in Soil. TM 5-852-6 (Department of the Air Force Manual AFM 88-19, Chapter 6).

Department of Transportation (1972). Implementation Package for a Drainage Blanket in Highway Pavement Systems. Federal Highway Administration, Washington, DC. 25 pp.

Eaton, R.A. (1975). Comparison of the Performance of All-Bituminous Concrete and Reduced Subgrade Strength Highway Pavement Test Sections under Freezing Conditions. CRREL Technical Report 270.

Eaton, R.A. (1976). CRREL Full-Depth Test Sections, Performance During First Three Winters. In TRB Record 572, pp. 85-96. Transportation Research Board, Washington, DC.

Fredrickson, F.C. (1963). Frost Considerations in Highway Pavement Design: West - Central United States. Highway Research Record No. 33, pp. 35-60

Jessberger, H.L. and D.L. Carbee (1970). Influence of frost action on the bearing capactiy of soils. Highway Research Record No. 304, pp. 14-26.

Kersten, M.S. (1949). Laboratory Research for the Determination of the Thermal Properties of Soils. Final Report. ACFEL Technical Report 23.

Lunardini, V. (1977). Geothermal Aspects of n-Factor and Calculation Methods. Internal Report. NRC Contract 032-579, Ottawa, Canada.

Moulton, L.K. (1968). A Study of the Relationship Between Air Temperatures and Depth of Frost Penetration as Related to Pavement Performance of West Virginia Highways. Final Report, Research Project No. 7, Eng. Exp. Station, W. Virginia University.

APPENDIX A

Material Characteristics Furnished by the NJDOT

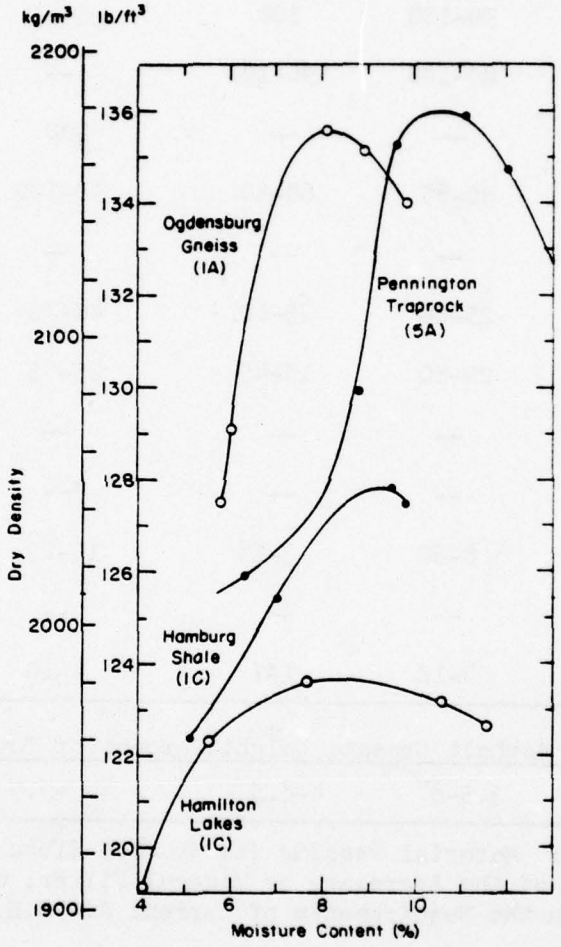


Figure A-1. Compaction curves for New Jersey soils, using ASTM standard D 698-Method D (Standard Proctor).

Table A1. Specifications for Bituminous Concrete Mixtures

Gradation Mix No.	Base 1	Binder 2	MA-BC 4	FA-BC 5
Sieve Size	Grading of total aggregate (coarse plus fine, plus filler if required). Amounts finer than each laboratory sieve (square opening), weight percent.			
2"	100	--	--	--
1-1/2"	90-100	100	--	--
1"	80-100	90-100	--	--
3/4"	--	--	100	
1/2"	50-85	60-80	90-100	100
3/8"	--	--	--	80-100
No. 4	25-60	25-60	40-70	55-75
No. 4	20-50	15-45	25-55	30-60
No. 16	--	--	--	--
No. 30	--	--	--	--
No. 50	8-30	3-18	10-25	10-30
No. 100	--	--	--	--
No. 200	4-12	1-7	4-10	4-10
	Asphalt Cement, Weight Percent of Total Mixtures			
	3.5-8	4-8.5	4.5-9.5	5-10

NOTE: Material Passing The No. 200 Sieve May Consist of Fine Particles of the Aggregate or Mineral Filler, or Both. In Accordance with the Requirements of Current A.A.S.H.O. Designation T 90.

Table A2. Bituminous Mix #1.

I. Materials:

#57 Stone - Houdaille Construction Materials, Bound Brook, N.J. (Diabase)
#8 Stone - Houdaille Construction Materials, Bound Brook, N.J. (Diabase)
Sand - Houdaille Construction Materials, Kenville, N.J.
Mineral Filler - Limestone Products, LimeCrest, N.J.
Asphalt Cement - ARCO AC-20

II. Aggregate Gradation

	#57		#8		Sand	
Sieve Size	Specs. (% passing)	Actual (% passing)	Specs.	Actual	Specs.	Actual
2"	100	100	100	100		100
1-1/2"	100		100	100		100
1"	95-100	99.7	100	100		100
3/4"		88.0	100	100		100
1/2"	25-60	47.5	100	100		100
3/8"		29.3	85-100	96.8		100
#4	0-10	7.0	10-30	23.1		98.1
#8	0-5	2.2	0-10	3.1		88.1
#50				2.1		22.7
#200						1.5

III. Cylinder Properties

A. Voids - 6 (in each)
B. Dimensions - 1) 6" (Diam.) x 8.30"
2) 6" (Diam.) x 8.42"
C. Weights of materials used (each)

1) #57----------3440 gms.
2) #8 ----------2150 gms.
3) Sand ----------2150 gms.
4) Mineral Filler---414 gms.
5) Asphalt Cement --453 gms.

Table A2 (cont'd)

IV. Cylinders Design Specifications

Bituminous Mix #1 - Base Course

Sieve Size	Specifications (% passing)	Actual (% passing)
2"	100	100
1-1/2"	90-100	100
1"	80-100	99.8
3/4"		95.2
1/2"	50-85	79.0
3/8"		70.7
#4	25-60	39.2
#8	20-50	28.8
#50	8-30	11.3
#200	4-12	5.2

Asphalt Cement, Weight Percentage of Total Mixtures:

Spec. ----- 3.5-8.0
Actual ---- 5.0

Table A3. Bituminous Mix #2 - Binder Course

I. Materials

#57 Stone - Houdaille Construction Materials, Bound Brook, N.J. (Diabase)
8 Stone - Houdaille Construction Materials, Bound Brook, N.J. (Diabase)
Sand - Houdaille Construction Materials, Kenville, N.J.
Asphalt Cement - Arco AC-20

II. Aggregate Gradation

	#57		#8		Sand	
Sieve Size	Specs. (% passing)	Actual (% passing)	Specs. (% passing)	Actual (% passing)	Specs.	Actual
2"	100	100	100	100		100
1-1/2"	100	100	100	100		100
1"	95-100	97.7	100	100		100
3/4"			100	100		100
1/2"	25-60	14.5	100	100		100
3/8"			85-100	98.3		100
#4	0-10	0.8	10-30	35.6		99.1
#8	0-5	0.7	0-10	6.7		88.1
#50				1.2		16.5
#200				1.2		1.9

Table A3 (cont'd)

III. Physical Properties

Once the pre-measured material (aggregate, sand, and asphalt cement) has been combined and compacted into a 20" x 20" x 3" form, one should have a representative #2 mixture with 6% voids

Weights of materials used

1) #57.....20,577.4 gms
2) #8.....13,718,3 gms
3) Sand.....11,431.9 gms
4) Asphalt Cement.... 2,406.7 gms

IV. Bituminous Mix #2 (Binder Course)

Sieve Size	Specs. (% passing)	Actual (% passing)
2"		
1-1/2"	100	100
1"	90-100	99.2
3/4"		
1/2"	60-80	70.1
3/8"		
#4	25-60	33.0
#8	15-45	16.9
#50	3-13	3.1
#200	1-7	1.0

Asphalt Cement, Weight Percentage of Total Mixture:

Specs.....4.0 - 8.5%
Actual.....5.0%

Table A4. Bituminous Mix #5.

I. Materials:

#8 Stone - Houdaille Construction Materials, Bound Brook, N.J. (Diabase)
Sand - Houdaille Construction Materials, Kenville, N.J.
Mineral Filler - Limestone Products, Limecrest, N.J.
Asphalt Cement - ARCO AC-20

Table A4 (cont'd)

II. Aggregate Gradation

Sieve Size	Specs (% pass)	Actual	Specs.	Actual
2"	100			
1-1/2"	100			
1"	100			
3/4"	100			
1/2"	100	100		100
3/8"	85-100	96.8		100
#4	10-30	23.1		98.1
#8	0-10	3.1		88.1
#50		2.1		22.7
#200				1.5

III. Cylinder Properties

A. Voids 6% (each)
B. Dimensions 1) 6" (diam.) x 8.26"
2) 6" (diam.) x 8.40"
C. Weights of materials used
1) #8----------3764 gms
2) Sand -------4207 gms
3) Mineral Filler -----421 gms
4) Asphalt Cement -----443 gms

IV. Cylinder's Design Specifications

Bituminous Mix #5 - Surface Course

Sieve Size	Specifications (% passing)	Actual (% passing)
2"		
1-1/2"		
1"		
3/4"		
1/2"	100	100
3/8"	80-100	98.6
#4	55-75	64.5
#8	30-60	50.5
#50	10-30	17.2
#200	4-10	5.7

Asphalt Cement, Weight Percentage of Total Mixtures:

Specs ---- 5-10
Actual---- 5

Table A5. Specific Gravity of Bituminous Cylinder Materials

#57 ---------- 2.91
#8 Stone------- 2.93
Sand ---------- 2.63
Mineral Filler------- 2.75
Asphalt Cement ------ 1.01

Table A6. Portland Cement Concrete Cylinders

I. Materials:

Portland Concrete Cement
Concrete Sand (Fine) Aggregate
Coarse Aggregate

II. Aggregate Gradations (% passing by weight)

	Fine Aggregate		Coarse Aggregate	
Sieve Size	Specs.	Actual	Specs.	Actual
1-1/2"			100	100
1"			95-100	100
1/2"			25-60	32
3/8"	100	100		
#4	95-100	99	0-10	4
#8	80-100	92	0-5	1
#16	50-85	78		
#30	25-60	49		
#50	10-30	16		
#100	2-10	4		
#200	0-3	1		
Fineness Modulus	2.3-3.1	2.62		

III. Specific Gravity of Materials

Coarse Aggregate ----- 2.90
Fine Aggregate ------- 2.65
PC ------------------- 3.15

IV. Maximum w/c Ratio --- 6.0*

*This is the maximum ratio allowed by specification. The actual ratio used in the cylinders is not available

Table A6 (cont'd)

V. Class B Concrete Weight Proportions

1 - 1.65 - 2.96*

*For one pound of Portland Cement, one must have 1.65 pounds of Fine Aggregate and 2.96 pounds of Coarse Aggregate.

Table A7. Open-graded Mix #1

I. Materials

Crushed Stone - Mount Hope Materials (Granite)
Asphalt Cement - AC-20

II. Aggregate Gradation

Sieve Size	Specs. (% passing)	Actual (% passing)
2"	100	100
3/4"	70-95	80
3/8"	40-65	50
#4	20-35	25
#50	0-6	3
#200	0-2	1

III. Specific Gravity

Aggregate - 2.68
Asphalt Cement - 1.01

IV. Density

Approximately 131 pcf

V. Cylinder Sizes

Sample A 6" (diam.) x approximately 8"
Sample B 6" (diam.) x approximately 8"

VI. Asphalt Content

4% by weight

Unstabilized Open-graded Mix #1.

Sieve Size	% Passing Specs.	Probe Test	Guarded Hot Plate
2"	100	100	100
3/4"	70-95	97	73
3/8"	40-65	68	43
#4	20-35	35	20
#50	0-6	5	3
#200	0-2	3	0

Table A8. Open-graded Mix #3.

I. Materials

Crushed Stone - Mount Hope Materials (Granite)
Asphalt Cement - AC-20

II. Aggregate Gradation

Sieve Size	% Passing
1/2"	100%
3/8"	70%
#4	15%
#8	5%
#50	0%

III. Specific Gravity

Aggregate - 2.68
Asphalt Concrete - 1.01

IV. Density

Approximately 118 PCF

V. Cylinder Sizes

A. 6" (diam.) x approximately 8"
B. 6" (diam.) x approximatley 8"

VI. Asphalt Content

3% by weight

Table A9. Mineralogical Content of Pennington Traprock and Mt. Hope Granite.

A. Pennington Traprock from the Pound Brook Quarry is quite uniform in mineralogy and texture. Mineral percentage varies a little as do the effects of weathering (upper level) and hydrothermal alteration.

Essentially the rock is composed of the following:

1.	Labradorite feldspar	40%	±
2.	Pyroxene (mainly augite)	40%	±
3.	Glass	5-10	±
4.	Magnetite	0-5	±
5.	Trace accessories	Tr	±

There is usually some alteration which forms the following:

1. Labradorite - epidote-zoisite, sericite
2. Pyroxene - chlorite
3. Glass - chlorite, albite

Fresh rock is gray. Altered rock is greenish gray to dull foliage green.

Texture is ophitic, feldspar crystals contained within pyroxene, typical of basalt.

B. Mt. Hope Granite

Alaskite and/or Hornblende Granite from Mt. Hope is also a fairly uniform rock composed of:

1.	Microperthite (microcline with exsolved albite)	45%
2.	Quartz	25%
3.	Oligoclase	20%
4.	Hornblende	0-10
5.	Pyroxene	
6.	Magnetite	0-5
7.	Misc. Trace accessories	Tr

The granitic rocks are quite uniform, equigranular, medium grained and gneissoid.

APPENDIX B

Thermal Conductivity Results on NJDOT Materials, Guarded Hot Plate Method

By

Doris J. VanPelt

Civil Engineering Technician

Description of Testing

This testing consisted of determining the thermal conductivity using the guarded hot plate method under four different conditions. The test conditions were as follows: Less than optimum moisture content at 95 percent maximum density (a) above freezing, (b) below freezing; and field capacity* at 95 percent maximum density, (c) above freezing, and (d) below freezing. The physical properties (supplied by NJDOT) of the thermal conductivity test samples are shown in Table B1.

Table B1. Physical Properties of Samples

Sieve Size	Actual % Passing			
	Base Course		Subbase Course	
	Ogdensburg Gneiss	Hamilton Lakes	Hamburg Shale	Pennington Traprock
4"	---	100	---	---
2"	100	---	100	100
3/4"	86.9	98.4	93.2	83.1
#4	48.0	74.4	69.1	55.1
#50	9.5	11.9	7.2	16.8
#200	3.8	1.4	2.8	8.9
Optimum Moisture (%)	8.1	7.6	9.5	11.1
Max. Dry Density (pcf)	135.6	123.6	127.8	135.9
Minimum Porosity	0.210	0.225	0.225	0.244

*Field capacity here is the amount of water a 3-inch thick specimen will hold without further drainage.

Apparatus

The apparatus used in testing the samples was a guarded hot plate testing machine, conforming to ASTM designation C177-71, ASTM 1973. Figures B1 and B2 show the physical apparatus and the test stack installed, ready to be insulated and closed up for test. Figure B3 shows a cross section of the stack assembly and the thermocouple locations in the testing plate surfaces, when assembled for the simultaneous testing of two specimens.

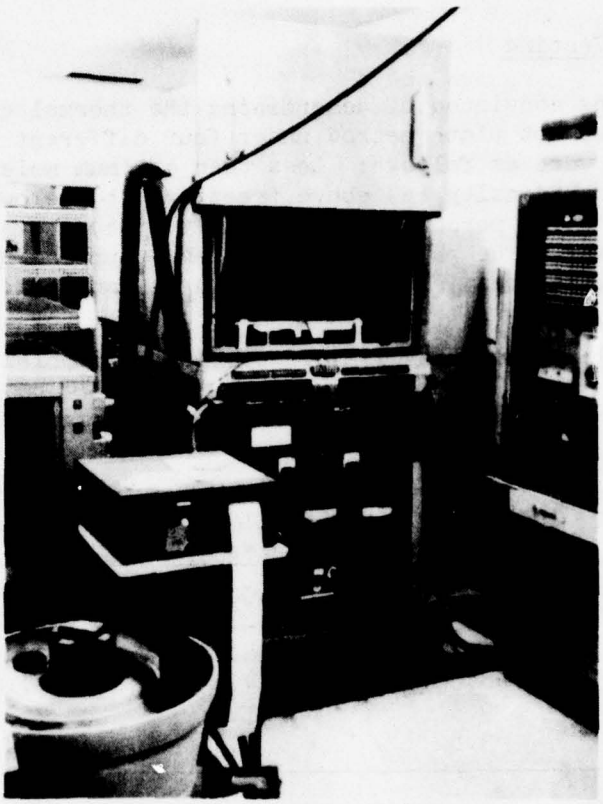


Figure B-1. Guarded Hot Plate thermal conductivity testing apparatus. (with testing stack removed).

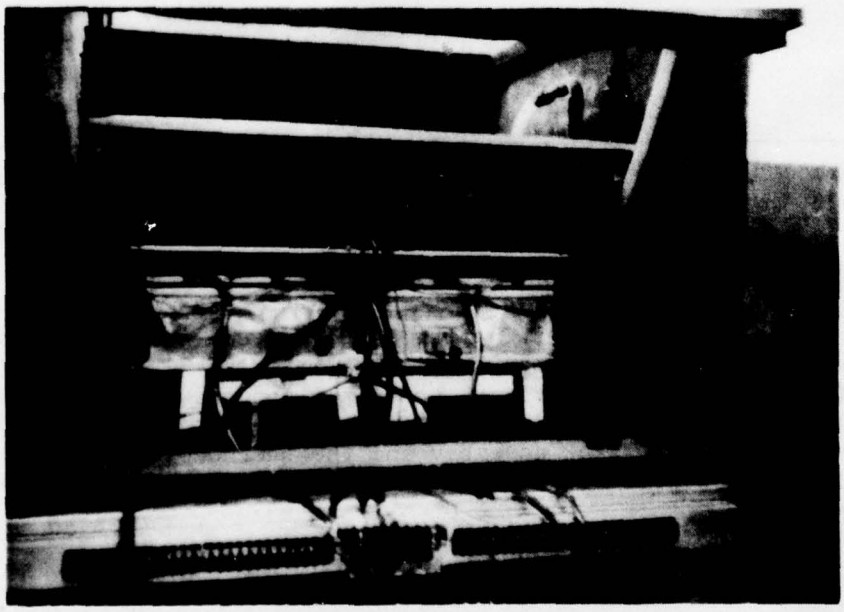
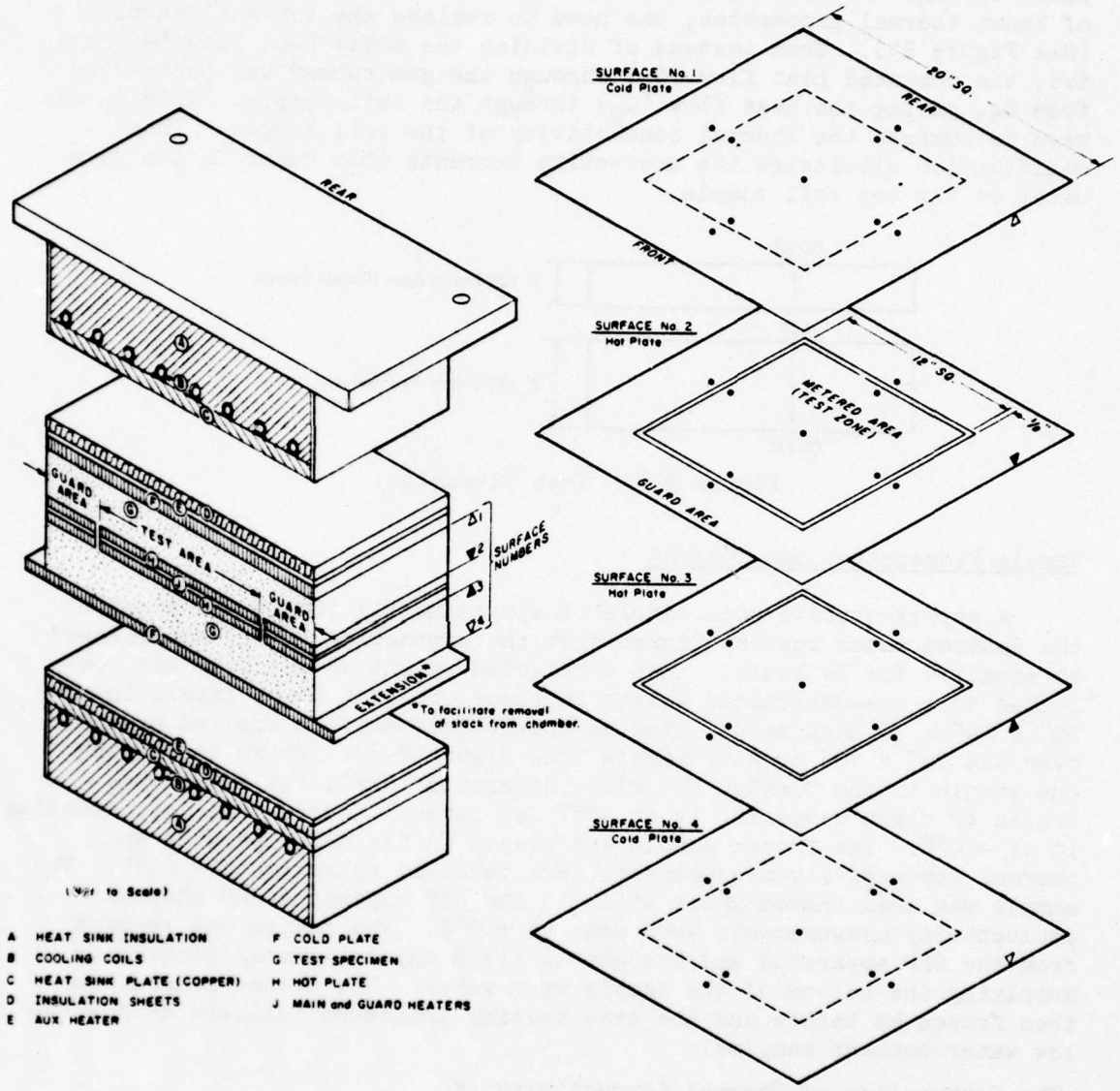


Figure B-2. Test stack, installed, prior to insulating.



a. *Stack assembly cross section.* **b. *Thermocouple locations (•) in plate surfaces.***

Figure B-3. Guarded Hot Plate thermal conductivity testing apparatus (ASTM Designation C177-71).

The thermal conductivity tests were performed in the guarded hot plate apparatus (Figure B1) in accordance with ASTM procedures, except for the following modifications. Instead of using two identical samples and evenly splitting the total heat flow, (Q_t), from the center guarded plate through the sample, a standard gum rubber sample one-inch thick, of known thermal properties, was used to replace the top soil sample (See Figure B4). Then instead of dividing the total heat flow (Q_t) by two, the computed heat flow (Q_1) through the gum rubber was subtracted from Q_t, giving the heat flow (Q_2) through the soil sample. This Q_2 was used to compute the thermal conductivity of the soil sample. This modification eliminates the convective currents that occur in the pore water of the top soil sample.

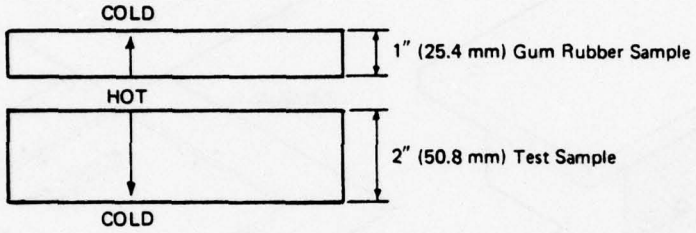


Figure B-4. Test Situation.

Sample Preparation and Testing

A representative soil sample of approximately 100 lb was mixed at the desired water content (taken from the compaction curve) and allowed to equalize for 24 hours. Then a selected weight of wet soil was compacted to a pre-determined volume in three one-inch thick layers in a 20" x 20" x 3" deep mold, using an appropriate uniform applied pressure over the 20" x 20" area of sample (see Figure B-5). Prior to placing the sample in the Guarded Hot Plate Apparatus (GHP), the sample was frozen by first tempering it at 40°F for several hours, then fast freezing it at -10°F. The frozen sample was placed in the GHP apparatus and thermal conductivity measurements were taken at approximately 23°F. The sample was then thawed while still in the GHP apparatus and thermal conductivity measurements were made at +40°F. The sample was removed from the GHP apparatus and brought to field capacity water content by supplying the bottom of the sample with water. The wetted sample was then frozen as before and the same testing procedure followed as for the low water content samples.

The Computation of Thermal Conductivity, K

The following is a short description of what is actually measured and what is computed, using the guarded hot plate method of measuring thermal conductivity.

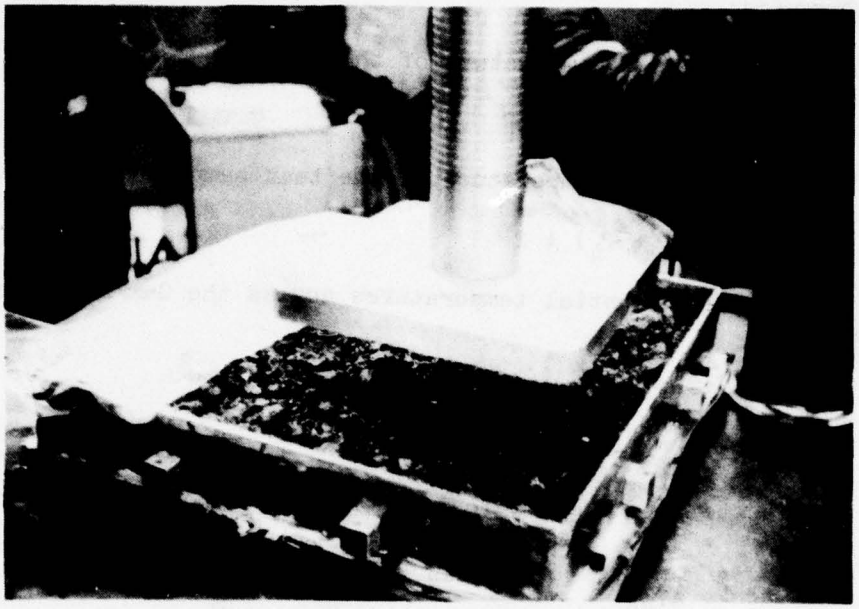


Figure B-5. Compacting Sample.

1. Known:

   a. The test area (A) is 1 ft^2 (Figure B3).
   b. The thermal conductivity of the gum rubber sample (K_{GR}) in Btu/hr ft^2 °F/in). This calibration versus temperature was supplied with the gum rubber from the National Bureau of Standards.

2. Measured:

a. The thickness in inches of both the gum rubber (X_{GR}) and the test sample (X_S), prior to starting the test.

b. The average temperature (°F) of each of the four testing plates, T_1, T_2, T_3, and T_4 (Figure B3). The five center test area thermocouples on each of the four test plates are read separately and then averaged together.

c. The main heater voltage (E) and current (I). This is the guarded heater within the center of the test stack.

3. Computed:

a. The average temperature of the gum rubber sample:

$$T_{GR} = (T_1 + T_2) \div 2$$

b. The average temperature of the test sample:

$$T_S = (T_3 + T_4) \div 2$$

c. The differential temperatures across the Gum Rubber and the test sample:

$$\Delta T_{GR} = T_2 - T_1$$

$$\Delta T_S = T_3 - T_4$$

d. The temperature gradient of the Gum Rubber and the test sample:

$$i_{GR} = \frac{\Delta T_{GR}}{X_{GR}} \qquad i_S = \frac{\Delta T_S}{X_S}$$

e. The total heat flow (Q_t) in Btu/hr through both the gum rubber and the test sample. The mean heater power (watts) is computed by:

$$P = EI$$

then using a constant conversion of 3.415 Btu/hr/watt

$$Q_t = (P)(3.415)$$

f. The Thermal Conductivity of the gum rubber (k_{GR}) taken from the NBS calibration curve at T_{GR}.

g. The heat flow through the gum rubber (Q_{GR}):

$$Q_{GR} = k_{GR} i_{GR} A$$

h. The heat flow through the test sample (Q_s)

$$Q_s = Q_t - Q_{GR}$$

i. Finally the Thermal Conductivity of the test sample (k_s)

$$k_s = Q_s / i_s A \quad \text{in Btu/hrft}^2\text{°F/in.}$$

Results

The results of the thermal conductivity tests are shown in Table B2 and Figure B6.

In all cases the wetted samples had higher thermal conductivities. Also, three of the soils developed lower conductivities when frozen than when thawed. These results are consistent with previous experience, which has shown that thermal conductivity rises generally with water content. Samples having very few fines will have lower conductivities when frozen because there is little unfrozen water to wet the mineral grains and thereby enhance the heat transfer from ice to soil particles.

In contrast, the Pennington Traprock has considerably more fines than the other three soils, and predictably a greater amount of unfrozen water at subfreezing temperatures. For this soil the frozen condition has a higher thermal conductivity than does the unfrozen condition.

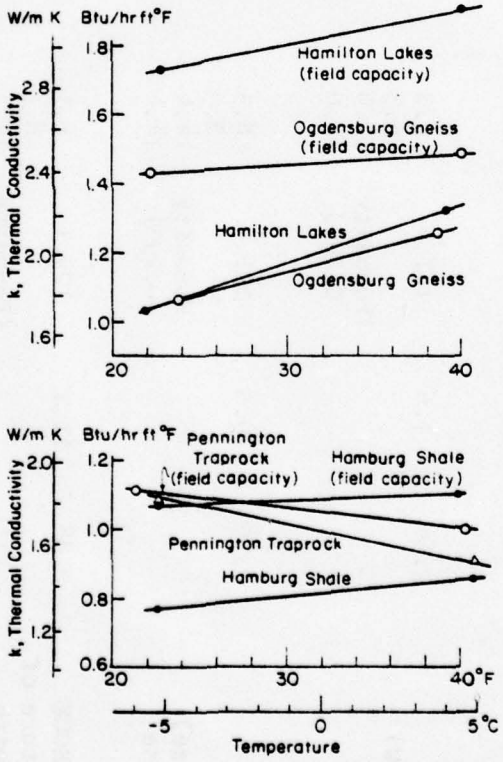


Figure B-6. Thermal conductivity of base course and subbase course materials from New Jersey, guarded hot plate method.

TABLE B-2

NEW JERSEY DEPARTMENT OF TRANSPORTATION — THERMAL CONDUCTIVITY TEST RESULTS, GUARDED HOT PLATE METHOD

Material	Gs	Design OPT. MC %	Design 95% Max. Density pcf	Test Results MC %	Test Results Density pcf	Test Results Saturation %	Test Results Temp. °F	Test Results K' Btu/hr ft °F
OGDENSBURG GNEISS	2.75	8.1	128.8	6.7	129.4	56.8	23.8	1.055
(Sandy Gravel GW)				6.7	129.4	56.8	38.7	1.252
1 A Base Course			(Porosity	9.7	128.2	78.9	22.2	1.425
			0.249)	9.7	133.4*	93.3*	39.9	1.476
HAMILTON LAKES	2.66	7.6	117.4	6.3	117.3	40.6	21.8	1.025
(Gravelly Sand SW)				6.3	117.3	40.6	39.0	1.319
1 C Base Course			(Porosity	12.0	116.8	75.9	22.8	1.724
			0.293)	12.0	116.8	75.9	39.8	1.897
HAMBURG SHALE	2.75	9.5	121.4	8.3	118.8	51.6	22.5	0.766
(Well Graded -				8.3	120.9*	54.6*	40.6	0.853
Gravelly Sand Size)			(Porosity	14.4	116.5	83.7	22.6	1.065
1 C Subbase Course			0.293)	14.4	120.9*	94.7*	39.7	1.091
PENNINGTON TRAP ROCK	2.88	11.1	129.1	8.1	128.4	58.3	22.5	1.079
(Well Graded Mixture of				8.1	128.4	58.3	40.8	0.892
Sand and Gravel with			(Porosity	11.7	128.4	84.2	21.3	1.107
Less Than 10% Fines)			0.282)	11.7	128.4	84.2	40.2	0.993
5 A Subbase Course								
MOUNT HOPE GRANITE	2.68			4.5	121.2	31.8	24.9	0.362
(Unstabilized Open-				4.5	121.2	31.8	39.4	0.488
Graded Drainage Layer)								

*These samples densified upon thawing, resulting in a higher degree of saturation with no change in water content.

APPENDIX C

Theory and Equipment Used in the Probe Method for Measuring the Thermal Conductivity of Soils.

by

Richard W. McGaw
Research Civil Engineer

Background

The thermal conductivity of a soil is one of the major factors influencing the rate at which freezing temperatures penetrate into ground under natural conditions. Of particular interest is the conductivity in the frozen zone adjacent to the location of freezing. Temperatures in this zone will generally be in the vicinity of 0°C.

Kersten (1949) reported thermal conductivities of various soils for temperatures of 25°F and 40°F. He used a steady-state method which required a temperature differential of approximately 10°F. While representing a considerable improvement over the 40°F differential formerly required by the guarded hot-plate apparatus, a 10 degree differential still results in errors caused by the migration of moisture or vapor during the many hours required to reach a steady condition. Latent heat effects at temperatures just below freezing cause further difficulties with fine-grained soils, in which the amount of frozen water may change appreciably with an increase in temperature.

Over the past two decades a transient method of heating has been developed that reduces the intensity of these effects by requiring temperature differentials of no more than several degrees. This is the probe, or modified line heat-source, method. Basically, the method utilizes a constant rate of heat production along a line source to produce an expanding cylindrical temperature field in the adjoining material. The rate of increase of temperature at any point in the material depends in general on its distance from the source and the diffusivity of the material. However, after a sufficient interval of time the temperature rise varies only with the conductivity of the material, and in a simple logarithmic manner; for points near the heat source, corresponding to a thin probe, this period can be reduced to 15 minutes or less for most materials. Migration of internal fluids is thereby greatly minimized.

Because of the inherent simplicity of the method, the relatively small temperature differentials, and the short heating periods required, the probe method is well suited to the measurement of the thermal conductivity of soils and other materials near freezing temperatures. Lachenbruch (1957) reported an investigation on frozen soils in place, in which the probe method was used. Also, a series of laboratory probe tests on frozen soils was performed under contract for CRREL (Wechsler, 1965). Since that time, CRREL has been engaged in a continuing laboratory program with the following objectives: (1) to develop a simple, reliable apparatus and technique for performing such tests, (2) to determine any special difficulties which may be encountered at temperatures close to freezing, and (3) to compare measured conductivities of selected soils with previously reported values. A general objective of the program has been to seek a physical explanation for the observed thermal conductivities. Several reports are available describing results of this program (McGaw 1967, 1968, 1969, 1971, 1974).

Theory of the Probe Method

The earliest mention of a line heat-source method of measuring conductivity has been attributed to Schleiermacher (1888). In this century it was suggested by Stalhane and Pyk (1931) as a means of measuring the thermal conductivity of a liquid. The first practical use of the method is credited to Van der Held and Van Drunen (1949) who used a heated wire, also in connection with liquids. Both the heated-wire method and the probe method are based on the simple line heat-source theory.

Line-Source Theory. Beginning from an initial isothermal condition, and with a constant heating rate (q) per unit length of source, the simple line heat-source theory results in the following well-known expression for the temperature rise at a distance r from the line source during the heating time t:

$$T_r = \frac{-q}{4\pi K} \text{ Ei } \left(\frac{-r^2}{4 \alpha t}\right), \tag{1}$$

where α is the thermal diffusivity of the external medium, and K is its thermal conductivity (Carslaw and Jaeger, 1959). The exponential integral (Ei) is tabulated by Jahnke et al (1960), but is also commonly approximated by the series expansion,

$$-\text{Ei } \left(\frac{-r^2}{4\alpha t}\right) = \ln \frac{4\alpha t}{2} - \gamma - \left(\frac{r^2}{4\alpha t}\right) + 1/4 \left(\frac{r^2}{4\alpha t}\right)^2 + \ldots, \tag{2}$$

where $\gamma = 0.5772$. When t is sufficiently large (or r is sufficiently small) that the value of

$$\frac{\alpha t}{r^2} >> 1/4 ,$$

then

$$\left(\frac{r^2}{4\alpha t}\right) << 1 , \tag{3}$$

and all but the first two terms of the expansion are considered to be negligible. Hence, equation (1) becomes

$$T_r = \frac{q}{4\pi K} \left(\ln \frac{4\alpha t}{r^2} - \gamma\right) . \tag{4a}$$

or

$$t_r = \frac{q}{4\pi K} \ln t + \left(\ln \frac{4\alpha}{r^2} - \gamma\right) , \tag{4b}$$

With α and r unchanging, the expression in parentheses is a constant. Consequently, in the simple theory the temperature rise plotted against the logarithm of time will be a straight line when the criterion given in equation (3) holds.

In practice, the temperature rise (ΔT) between any two large successive times (t_1 and t_2) is utilized, in which case equation (4b) becomes

$$\Delta T_r = T_2 - T_1 = \frac{q}{4\pi K} \ln \frac{t_2}{t_1} \quad , \tag{5}$$

the value of the conductivity being given by the basic line-source equation:

$$K = \frac{q}{4\pi \Delta T_r} \ln \frac{t_2}{t_1} \quad . \tag{6}$$

Probe Theory. Hooper and Lepper (1950) were the first to use a probe, or portable heating element, for measuring the conductivity of soils in their natural condition. They were followed shortly after by de Vries (1952), Lentz (1952), Hooper and Chang (1953), and others. These investigators found that definite precautions must be taken in the design and operation of a probe if it is to approximate a simple line source. The dimensions of the probe, its thermal properties, and the contact resistance between the probe and the test material all must be taken into account, either directly or indirectly. Consequently, beginning with D'Eustachio and Schreiner (1952), an extensive series of analyses of the possible errors in the probe method was carried on by Van der Held et al (1953), Blackwell (1954), Vos (1955), Buettner (1955), Jaeger (1956), Blackwell (1956), Jaeger (1958), de Vries and Peck (1958), and others. A summary of these studies is given by Wechsler (1965).

Typical solutions for the temperature rise at the inner radius of a hollow metal probe, heated along its outer radius b, have been given by Blackwell (1954). If the probe has a heat capacity M_1C_1 per unit of length and a contact resistance $\frac{1}{H}$ per unit of area between its outer surface and the external medium, the large-time solution is:

$$T = \frac{q}{4\pi K} \left[\ln 4\tau - \gamma + \frac{2K}{bH} + \frac{1}{2\tau} \right.$$

$$\left. \left[\ln 4\tau - \gamma + 1 - \frac{2}{\beta} \left(\ln 4\tau - \gamma + \frac{2K}{bH} \right) \right] + O\left(\frac{1}{\tau^2}\right) \right] , \tag{7}$$

where $\tau = \frac{\alpha t}{b^2}$ and $\beta = \frac{2\pi b^2 c}{M_1 C_1}$, ρc being the volumetric heat capacity of the external medium.

Jaeger (1956) found an identical solution for a solid cylindrical probe under the same conditions. Consequently, whether a probe with an internal heater is considered to be hollow or solid makes no change in the large-time solution. Moreover, a related solution by Blackwell (1954), in which he takes into account the finite conductivity of the probe material, is also identical to equation (7) except for the addition of one other term:

$$\frac{1}{2\tau} \left[- \frac{2\alpha}{b^2} (\Delta_1 + \Delta_2) \right] ; \qquad (8)$$

here, Δ_1 and Δ_2 are functions of the inner and outer radii, $\underline{a}$ and $\underline{b}$. According to Blackwell, this term is usually insignificant in magnitude. It therefore appears that the finite conductivity of a probe also produces no significant change in the large-time solution, so long as the probe material is a good conductor in relation to the external medium.

An axial flow of heat, brought about by temperature gradients along the probe or by conduction along the electrical or thermocouple leads, may introduce serious distortions from equation (7) by reducing the effective heat input by an unknown (and perhaps varying) amount. Lentz (1952) and Hooper and Chang (1953) took note of conduction along the leads, while Blackwell (1956) analyzed the axial flow error in the probe itself. Blackwell found that temperature gradients along a carefully constructed probe of good conducting material should be negligible if the length of the probe is at least 25 times the diameter. In practice, length-to-diameter ratios of 100 or more are usually employed, so that axial heat losses are generally assumed to be negligible. On the other hand, loss of heat from the system through the leads is a distinct possibility whenever the ends of the probe are at a higher temperature than the surroundings. This latter factor appears to have been overlooked in the more recent literature.

Assuming that adequate measures have been taken to control axial heat losses, equation (7) may be accepted as the basic probe equation, in which contact resistance and the thermal properties of the probe have been taken into account. As in the line heat-source analysis, incorporating the criterion of equation (3) will eliminate terms proportional to powers of $(\frac{1}{\tau})$. That is, for times sufficiently large that $\tau >> 1/4$, the temperature rise at the center of the probe is given by

$$T = \frac{q}{4\pi K} \left[\ln \frac{4\alpha t}{b^2} - \gamma + \frac{2K}{bH} \right], \qquad (9a)$$

or

$$T = \frac{q}{4\pi K} \left[\ln t + \left(\ln \frac{4\alpha}{b^2} - \gamma + \frac{2K}{bH} \right] \right. . \qquad (9b)$$

Since $\underline{b}$ and $\underline{\gamma}$ are constants, the expression in parentheses in equation (9b) is a constant when α, K, and $(\frac{1}{H})$ are unchanging during a test period. In this case the temperature rise between successive large times reduces to equation (5), the value of the conductivity being given by equation (6) as before. Thus, in a well-designed test the basic line-source equations also hold for the probe method.

Equation (9b) may also be written in the following form, which is sometimes convenient when measuring the conductivity of a frozen soil near the freezing point:

$$T = \frac{q}{4\pi K}\left[\ln t + \left(\ln \frac{4}{b^2} - \gamma\right) + \ln \frac{K}{\rho c} + \frac{2K}{bH}\right] . \qquad (9c)$$

Here, the term in parentheses is an absolute constant, while the last two terms may each be variable. There is some danger that a progressive change in either term during a test will give an erroneous slope to the graph of $\underline{T}$ versus $\underline{\ln t}$ while not noticeably affecting the linearity. Consequently, special care must always be used in the interpretation of test data for frozen soils.

Apparatus

The probe used at CRREL was constructed by Arthur D. Little, Inc. of Cambridge, Massachusetts. It is similar in construction to that used by D'Eustachio and Schreiner (1952), although somewhat smaller in diameter and approximately twice as long. A stainless steel sheath of 0.065 in. outside diameter contains a heating coil of constantan wire (66.0 ohms/in.) within which is placed a chromel/constantan thermocouple. The thermocouple lies on the axis of the probe and at its mid-length. A molded plastic block at one end supports the thermocouple and power leads. The free length of the probe below the block is 8.00 in., resulting in a length/diameter ration of 125. A schematic of the probe construction is shown in Figure C1.

The probe circuitry consists of a heater circuit and a thermocouple circuit. The heater circuit (Figure C2a) contains the probe coil of 544 ohms, a switch, a DC milliameter sensitive to 0.001 ma, and a regulated DC voltage power supply. The heat input to the coil is read as I^2R.

The probe thermocouple circuit (Figure C2b) contains the chromel/constantan probe junction, a reference junction placed in an ice bath, a bias voltage (accuracy 0.1%), and indicating amplifier, and a millivolt recorder. At any time the probe junction temperature can be read directly as the algebraic sum of the bias voltage ($\pm$) and the scale voltage (+) as shown either on the amplifier indicator or the recorder.

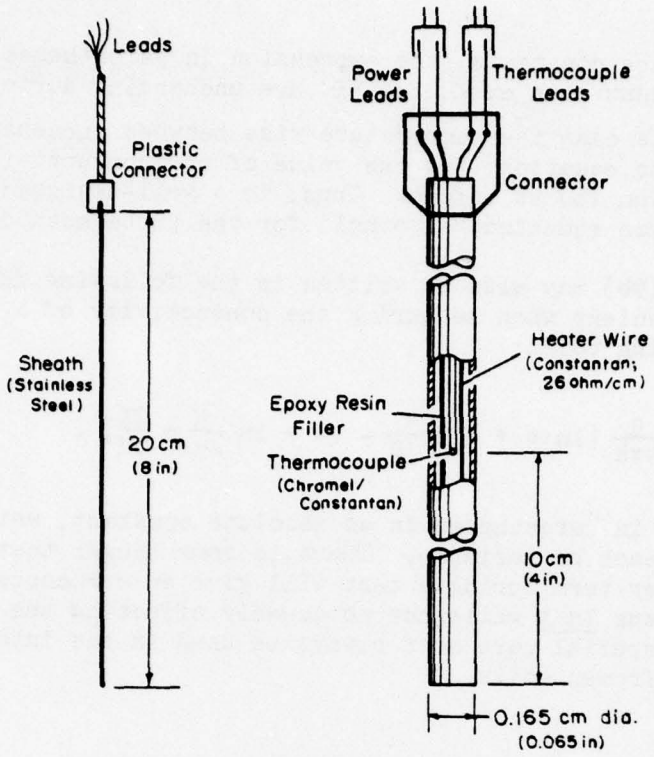


Figure C-1. Schematic of the CRREL thermal conductivity probe.

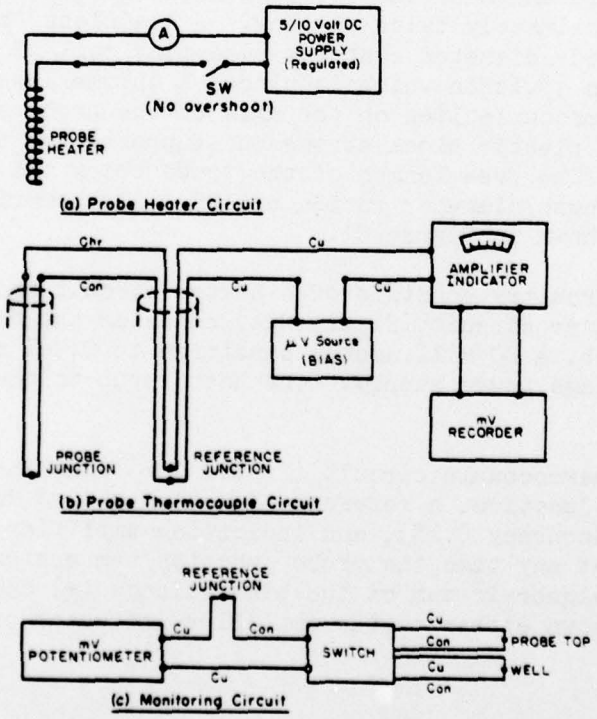


Figure C-2. Probe circuits (Schematic).

The recorder is operated as zero-left with +20 microvolts full scale (0.5F). Prior to a test the bias voltage is adjusted to maintain the trace on the recorder scale from approximately 10 seconds of heating. In most cases linearity of the trace will have been established before 60 seconds of heating.

Standard test specimens are three in. (76 cm) in diameter and approximately 8-1/2 in. (63 cm) long. They are molded in an aluminum cylinder 12 in. (30 cm) long having a wall thickness of 1/4 in. (0.63 cm) and a rigid insulating bottom (Figure C3). This container fits snugly into an aluminum cooling well bathed by circulating pre-cooled ethylene glycol (Figure C4). A sealed air gap surrounding the circulating chamber provides insulation from the room temperature, which is maintained at +15°C (59°F). One-inch thick foam insulation is placed beneath and above the cooling well assembly. Insulation is also applied to the top of the specimen container after it has been placed in the well and the probe inserted.

Temperature regulation of the ethylene glycol refrigerant is accomplished in an external cooling bath, in normal operation. In the case of the New Jersey specimens, which were 6 in. in diameter because of the large aggregate, a somewhat modified procedure was devised. In place of the standard container, a waterproof plastic cylinder was fitted snugly over the specimen, enclosing the sides and the bottom of the specimen. This assembly was then suspended directly in the ethylene glycol refrigerant, thereby cooling the specimen to the regulated bath temperature.

After insertion of the probe, a 1-in. thick disc of styrofoam insulation was fitted over the specimen to protect it from the room air, which is maintained at 40°F. Probe heating was then accomplished in the normal manner, and thermal conductivity was calculated from the data as described in the next section.

Evaluation/Calibration Procedure

Evaluation of the probe apparatus is accomplished at a temperature of +5.0°C using three materials whose conductivities are well known. These materials are Ottawa Standard Sand (20-30 mesh), both dry and saturated, and distilled water.

Typical results obtained for water, dry sand, and saturated sand at +5°C are given in Figures C5, C6, and C7. From the value of the measured slope (ΔT vs ln t), thermal conductivity is calculated from the following simple relation:

$$K = \frac{C\ I^2}{\Delta T_1}\ , \qquad (10)$$

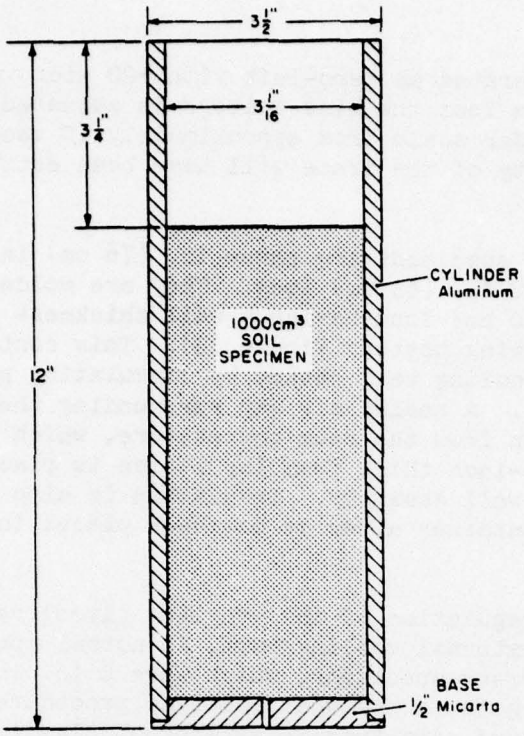


Figure C-3. Specimen Container (standard).

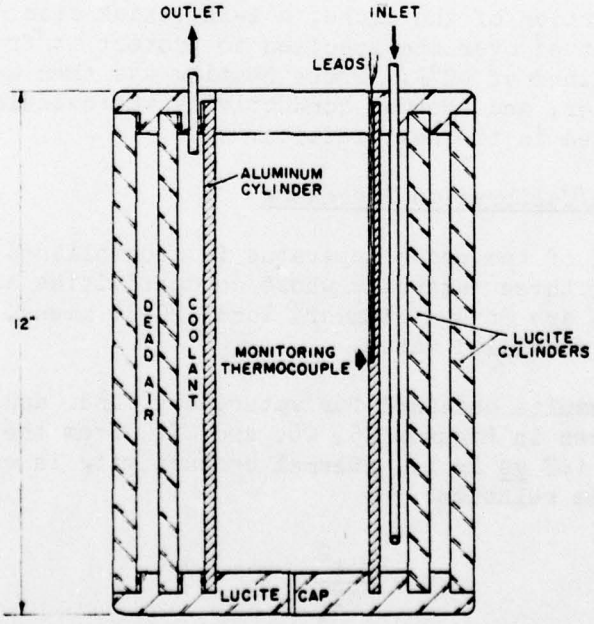


Figure C-4. Cooling Well Assembly (standard).

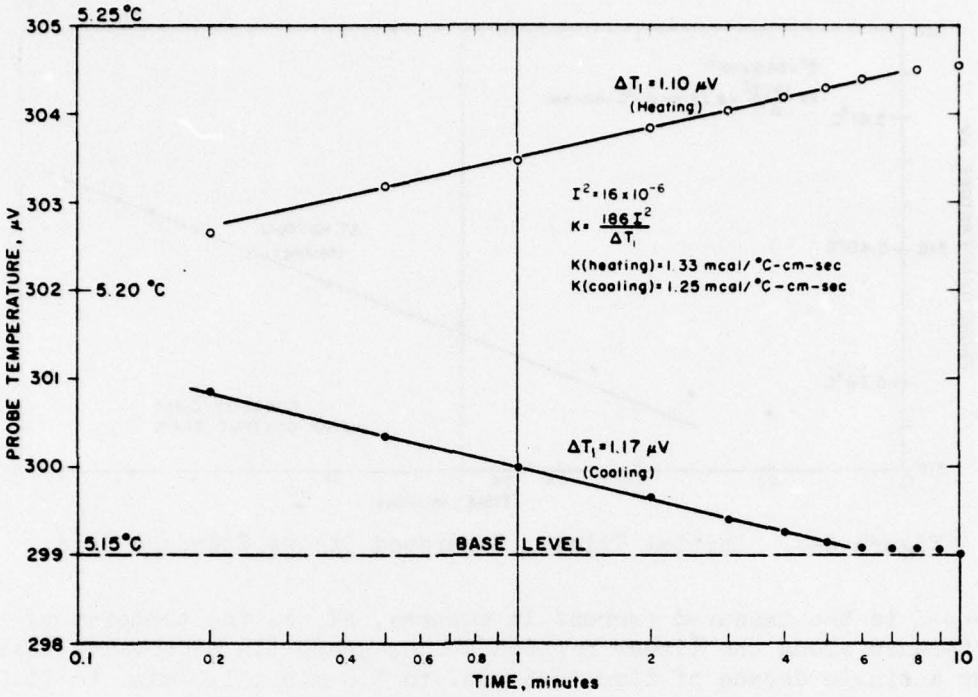


Figure C-5. Initial Trial: Water.

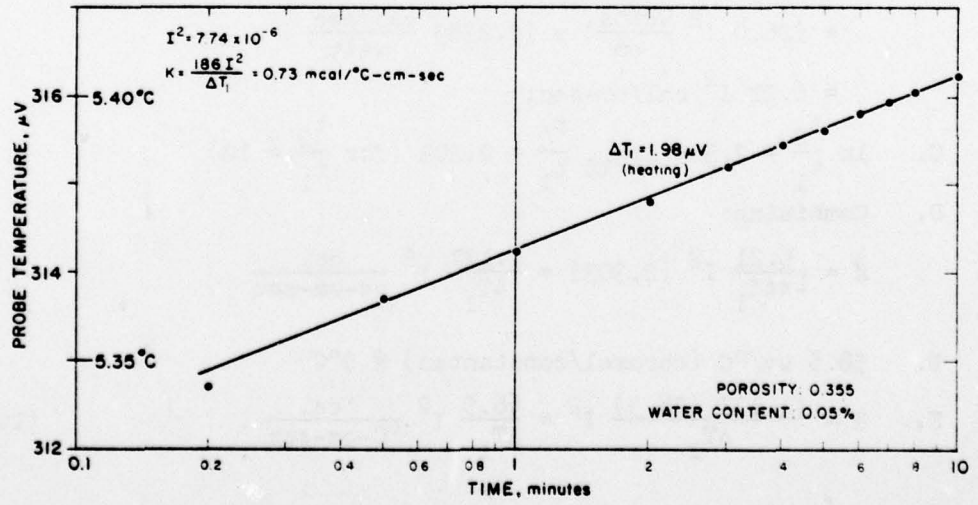


Figure C-6. Initial Trial: Air-Dry Ottawa Standard Sand.

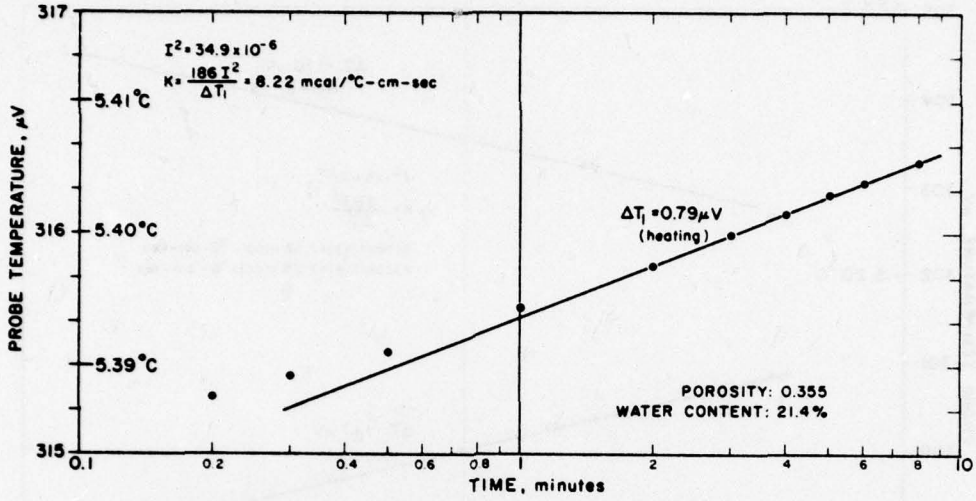


Figure C-7. Initial Trial: Saturated Ottawa Standard Sand.

where I is the measured current in amperes, ΔT_1 is the temperature difference along the linear portion of the graph (in microvolts) taken over a single decade of time (0.5 min. to 5.0 min.; 1.0 min. to 10.0 min., etc.), and C is a constant. Equation (10) is derived from the probe constants and equation (6) as follows:

A. $$K = \frac{q}{4\pi\Delta T} \ln \frac{t_2}{t_1} \quad ; \qquad (6)$$

B. $q = I^2R$, per unit length of probe, where R = 26.0 ohms/cm.

$$= (26.0\ I^2\ \frac{\text{Watts}}{\text{cm}}) \times (0.2389\ \frac{\text{cal/sec}}{\text{watt}})$$

$$= 6.21\ I^2\ \text{cal/cm-sec};$$

C. $$\ln \frac{t_2}{t_1} = 2.303 \log_{10} \frac{t_2}{t_1} = 2.303 \quad (\text{for } \frac{t_2}{t_1} = 10)$$

D. Combining:

$$K = \frac{6.21}{4\pi\Delta T_1} I^2\ (2.303) = \frac{1.137}{\Delta T_1} I^2\ \frac{\text{cal}}{\mu\text{v-cm-sec}}$$

E. 58.5 μv/°C (chromel/constantan) @ 0°C

F. $$K = \frac{(1.137)(58.5)}{\Delta T_1} I^2 = \frac{66.5}{\Delta T_1} I^2\ \frac{\text{cal}}{^\circ\text{C-cm-sec}} . \qquad (10)$$

The tests for water typically reach good linearity prior to 30 seconds of heating. With the lowest level of heat input (1.5 mw/in.) convective disturbances do not set in until after 5 minutes of heating; with higher heat inputs, convection begins after about 2 minutes. Both heating and cooling curves give similar results. In three tests the following conductivity values were obtained: 1.33, 1.37, 1.34 (avg. 1.347, heating); 1.25, 1.37, 1.34 (avg. 1.320, cooling). The value at +5°C is reported to be 1.34 mcal/°C-cm-sec. (Dorsey, 1940, Table 130 (BN)).

There is no problem from convection in the dry sand. Linearity is established prior to 12 seconds on heating and 30 seconds on cooling. In five trials the following conductivity values were obtained: 0.71, 0.72, 0.77, 0.73, 0.72 (avg. 0.730, heating); 0.75, 0.69, 0.67, 0.72, 0.72 (avg. 0.710, cooling). Kersten (1949) found 0.77 mcal/°C-cm-sec for the same density at 40°F by a steady-state method; de Vries and Peck (1958) found 0.73 at +17.°C by a transient method using a heated wire in a glass capillary.

The tests using water and dry sand show that adequate precision in the computation of conductivity is obtained when the slope of the linear portion (ΔT_1), over one decade of time is at least 1 μv. In the course of the saturated sand trials it was found that from four to five times the heat input was required to obtain a value of ΔT_1 equal to 1 μv. Woodside and Messmer (1961) found the conductivity of saturated 20-30 Ottawa sand to be about 8.0 mcal/°C-cm-sec, or 10 times that of dry sand. Trials with saturated Ottawa sand give a value of 8.22 mcal/°C-cm-sec.

REFERENCES CITED IN APPENDIX C

1. Blackwell, J.H. (1954). A transient-flow method for determination of thermal constants of insulating materials in bulk, Park I - Theory, Jour. Applied Physics, Vol. 25, No. 2, 137-144.

2. Blackwell, J.H. (1956). The axial-flow error in the thermal-conductivity probe, Canadian Jour. of Physics, Vol. 34, 412-417.

3. Buettner, K. (1955). Evaluation of soil heat conductivity with cylindrical test bodies, Trans. Amer. Geophys. Union, Vol. 36, 831-837.

4. Carslaw, H.S., and J.C. Jaeger (1959). Conduction of Heat in Solids (2nd ed.), Clarendon Press, Oxford.

5. D'Eustachio, D., and R.E. Schreiner (1952). A study of a transient method for measuring thermal conductivity, Trans. Amer. Soc. of Heating and Ventilating Engrs., Vol. 58, 331.

6. deVries, D.A. (1952). A nonstationary method of determining thermal conductivity of soil in situ, Soil Science, Vol. 73, No. 2, 83-89.

7. deVries, D.A. (1963). Thermal properties of soils: Chap. 7 of physics of Plant Environment, W.R. van Wijk (ed.), North-Holland Publ. Co., Amsterdam.

8. deVries, D.A. and A.J. Peck (1958). On the cylindrical probe method of measuring thermal conductivity, with special reference to soils, Australian Jour. of Phys., Vol. 11, 255-271; 409-423.

9. Dorsey, N.E. (1940). Properties of Ordinary Water Substance, Reinhold Publ. Corp., New York.

10. Hooper, F.C., and F.R. Lepper (1950). Transient heat flow apparatus for the determination of thermal conductivities, Heating, Piping, and Air Conditioning, Vol. 22, No. 8, 129-134.

11. Hooper, F.C. and S.C. Chang (1953). Development of the thermal conductivity probe, Trans. Amer. Soc. of Heating and Ventilating Engrs. Vol. 59, 463-472.

12. Jaeger, J.C. (1956). Conduction of heat in an infinite region bounded internally by a circular cylinder of a perfect conductor, Australian Jour. of Physics, Vol. 9, 167-179.

13. Jaeger, J.C. (1958). The measurement of thermal conductivity and diffusivity with cylindrical probes, Trans. Amer. Geophys. Union, Vol. 39, No. 4, 708-710.

14. Jahnke, Emde, and Losch (1960). Tables of Higher Functions, McGraw-Hill Book Co., New York.

15. Kersten, M.S. (1949). Laboratory research for the determination of the thermal properties of soils, U.S. Engineer Division, St. Paul, Minn.

16. Lachenbruch, A.H. (1957). A probe for the measurement of the thermal conductivity of frozen soils in place, Trans. Amer. Geophys. Union, Vol. 38, No. 5, 691-697.

17. Lentz, C.P. (1952). A transient heat flow method of determining thermal conductivity, Canadian Jour. of Tech., Vol. 30, 153-166.

18. McGaw, Richard, A comparison of heat conduction equations for granular materials, USACRREL Tech. Note, March 1967.

19. McGaw, Richard, Thermal Conductivity of compacted sand/ice mixtures Highway Resarch Record No. 215, Highway Research Board, National Research Council, Jan. 1968.

20. McGaw, Richard, Heat conduction in saturated granular materials, Highway Research Board Special Report 103 (Effects of Temperature and Heat on the Engineering Behavior of Soils) National Research Council, Washington, D.C., Jan. 1969.

21. McGaw, Richard, The use of the cooling curve in the probe conductivity test, XIth International Conference on Thermal Conductivity, Albuquerque, N.M., Sept. 1971.

22. McGaw, Richard, Thermal conductivity of organic sediments from two Wisconsin lakes, USACRREL Special Report 129, Nov. 1974.

23. Schleiermacher, A. (1888), Wiedemann Annalen Physik, Vol. 3.

24. Stalhane, B., and S. Pyk (1931). Teknisk Tidskift, Vol. 61, 389-93.

25. Van der Held, E.F.M., and F.G. Van Drunen (1949). A method of measuring the thermal conductivity of liquids, Physica, Vol. 15, No. 10, 865-81.

26. Van der Held, E.F.M., J. Hardebol, and J. Kalshoven (1953). On the measurement of the thermal conductivity of liquids by a non-stationary method, Physica, Vol. 19, 208-216.

27. Vos, B.H. (1955). Measurements of thermal conductivity be a non-steady-state method, Applied Scientific Research, Sect. A, Vol. 5, 425-438.

28. Wechsler, A.E. (1965). Development of thermal conductivity probes for soils and insulations, Arthur D. Little, Inc., Cambridge, Mass.

29. Woodside, W. (1958). Probe for thermal conductivity measurement of dry and moist materials, Heating, Piping, and Air-Conditioning, Vol. 30, 163-170.

30. Woodside, W., and J.H. Messmer (1961). Thermal conductivity of porous media, Part I, Jour. of Applied Physics, Vol. 32, No. 9, 1688-1699.

APPENDIX D

Listing of the Computer Program for the Modified Berggren Equation.

```
100 *      MODIFIED BERGGREN SOLUTION FOR MULTILAYERED SYSTEMS  (GWA VERSION)
110 *      G. AITKEN/R. HERG, MARCH,1966
120 *
130 *
140 *      LATEST UPDATE 4/9/68, GWA
160 *      THIS VERSION PROVIDES FOR
170 *         1. ALPHA HEADER AND SUMMARY DATA
180 *         2. TYPE IN OF BASIC DATA
190 *         3. KICK OUT FOR DEPTH CHANGE LESS THAN .03 FT
200 *
210 *                              OPERATING INSTRUCTIONS
220 *
250 *      A.  SET FLAG1=1. FOR FROST DEPTH SOLUTION, FLAG1=0.
260 *      FOR THAW DEPTH SOLUTION.
270 *      B.  SET FLAG2=1. FOR FILE INPUT ; FLAG2=0. FOR TERMINAL
280 *       INPUT.
290       REAL KIND
300       CHARACTER*8 NAME
310 101    PRINT,"INPUT FLAG1=1. IF LOOKING FOR FROST DEPTH"
320       INPUT,FLAG1
330       PRINT,"ARE YOU READING FROM A FILE- TYPE '1.' IF YES"
340       READ(0,1) FLAG2
350       IF (FLAG2.EQ.1.) GO TO 111
360       GO TO 112
370 111   PRINT,"INPUT AN 8-LETTER FILENAME"
380       READ(0,1) NAME
385       OPENFILE 1,NAME
390 112   A1=.0705230784
400       A2=.0422820123
410       A3=.0092705272
420       A4=.0001520143
430       A5=.0002755672
440       A6=.0000430638
441 109   IF (FLAG2.EQ.1.) GO TO 104
442       PRINT 108
443 108   FORMAT(1H ,58HTYPE DEGREE-DAYS (DEGREE),N-FACTOR (EN),LENGTH OF FREEZING,/
444      &52H OR THAWING SEASON (T),MEAN ANNUAL TEMPERATURE (ATM))
445       READ(0,1) DEGREE,EN,T,ATM
446       GO TO 117
447  104  READ(1,1) DEGREE,EN,T,ATM
448 1     FORMAT(V)
449 117   CONTINUE
670       DEPTH1=0.
680       DEPTH=0.
690       SUMD=0.
700       SUMFD=0.
710       SUMCD=0.
720       SUMIND=0.
730       RES=0.
740       SUMRES=0.
750       NN=10
760       N=0
770       SI=0.
850 105      DEGREE=ABS(DEGREE)
870       V0=ABS(ATM-32.)
880       VS=EN*DEGREE/T
890       DEGREE=DEGREE*EN
900       ALP=V0/VS
```

```
910 2       FLAG=-100.
913         FLAGA=-100.
914         IF (FLAG2.EQ.1.) GO TO 113
915         PRINT,"INPUT KIND (A REAL NUMBER - LESS THAN 2. MEANS THE SOIL"
916         PRINT," IS COARSE-GRAINED, GREATER THAN OR EQUAL TO 2. BUT"
917         PRINT," LESS THAN 10. MEANS THE SOIL IS FINE-GRAINED, AND"
918         PRINT," GREATER THAN OR EQUAL TO 10. MEANS YOU WILL HAVE TO"
919         PRINT," SPECIFY ADDITIONAL THERMAL PARAMETERS),GAMMA (DENSITY),"
920         PRINT," WC (WATER CONTENT, PERCENT ON A GRAVIMETRIC BASIS),"
921         PRINT," AND D (LAYER THICKNESS, IN FEET)"
927         GO TO 114
928 113     READ(1,1) KIND,GAMMA,WC,D
929         GO TO 115
930 114     READ(0,1) KIND,GAMMA,WC,D
931 115     CONTINUE
932         IF(KIND-10.)4,5,5
940 4       IF (KIND-2.) 6,7,7
945 *       COURSE-GRAINED
950 6        THERMT=(.7*ALOG10(WC)+.4)*(10.**(.01*GAMMA))
960         THERMF=.076*(10.**(.013*GAMMA))+.032*(10.**(.0146*GAMMA))*WC
970         THERM=(THERMT+THERMF)/24.
980         GO TO 20
985 *       FINE-GRAINED
990 7        THERMT=(.9*ALOG10(WC)-.2)*(10.**(.01*GAMMA))
1000         THERMF=.01*(10.**(.022*GAMMA))+.085*(10.**(.008*GAMMA))*WC
1010         THERM=(THERMF+THERMT)/24.
1020 20        C=GAMMA*(.17+(.75*WC/100.))
1030         FUSION=144.*GAMMA*(WC/100.)
1040         GO TO 21
1050 5       IF (FLAG2.EQ.1.) GO TO 220
1060 217     PRINT 208
1070 208     FORMAT('INPUT C, THERM, FUSION')
1071 *     C - HEAT CAPACITY
1072 *     THERM - THERMAL CONDUCTIVITY
1073 *     FUSION - HEAT OF FUSION
1080 209     READ(0,9) C,THERM,FUSION
1090 9       FORMAT(V)
1100         GO TO 21
1110 220     READ(1,9) C,THERM,FUSION
1120 21        A=THERM/C
1130         SUMD=SUMD+D
1140         DEPTH=SUMD
1150 10        SUMFD=SUMFD+(FUSION*D)
1160         BARL=SUMFD/SUMD
1170         SUMCD=SUMCD+(C*D)
1180         BARC=SUMCD/SUMD
1190         IF(BARL)37,37,36
1200 36        U=VS*(BARC/BARL)
1210         GARB=1.77245/U
1220         R=.15
1230 30        ERF=1.-1./((1.+A1*R+A2*R**2+A3*R**3+A4*R**4+A5*R**5
1240        &   +A6*R**6)**16)
1250         EXPF=EXP(-(R**2))
1260         ERFC=1.-ERF
1270         CHECK=EXPF/ERF-((ALP)*EXPF/ERFC)
1280         CHECKA=R*GARB
1290         IF(FLAG) 1002,1002,1003
1300 1002    FLAG=100.
```

```
      IF (CHECK-CHECKA)1004,1000,1000
1004  FLAGA=100.
      GO TO 1001
1003    IF(FLAGA)1000,1000,1001
1000    IF(CHECK-CHECKA)35,35,34
1001    IF(CHECK-CHECKA)1007,35,35
1007    R=R-.001
      GO TO 30
34      R=R+.001
      GO TO 30
35      F=((2.*(R**2))/J)**.5
      GO TO 38
37      F=0.
      U=0.
38      RES=ABS(D)/THERM
      TOTALR=SUMRES+RES/2.
      IF (BARL) 40,40,39
39     AYERI=(FUSION*(ABS(D))/(24.*(F**2)))*TOTALR
       GO TO 41
40     AYERI=0.
41     SI=SI+AYERI
      N=N+1
      PRINT
      IF (N.EQ.1) PRINT 200
200      FORMAT(1H ,25X,53HMULTILAYER SOLUTION OF THE MODIFIED
     & BERGGREN EQUATION    )
      IF (N.EQ.1) PRINT 106
106      FORMAT(1H ,11HALPHA INPUT)
      IF (N.EQ.1) PRINT 510
510      FORMAT(1H ,//)
      IF (N.EQ.1) PRINT 201
201      FORMAT(1H ,11X,3HDRY,4X,5HWATER,4X,5HLAYER,4X,4HHEAT,4X,7HTHERMAL,
     &2X,6HLAYER,7H FUSION,7H LAMBDA,2X,5HLAYER,3X,4HCUM.,5X,5HLAYER,4X
     &,4HCUM.,/,10X,7HDENSITY,3H CONTENT,3X,4HSIZE,3X,8HCAPACITY,3X,5HCO
     &ND.,4X,4HHEAT,3X,4HPAR.,11X,4HRES.,3X,4HRES.,5X,5HINDEX,3X,5HINDEX
     &,/,10X,7H(GAMMA),2X,4H(WC),6X,3H(D),5X,3H(C),5X,7H(THERM),9H (FUSI
     &ON),2X,3H(U),3X,3H(F),4X,5H(RES),9H (SUMRES),2X,7H(AYERI),3X,4H(SI
     &),/)
      PRINT 11,N,GAMMA,WC,D,C,THERM,FUSION,U,F,RES,SUMRES,AYERI,SI
11       FORMAT(3X,I3,2X,2F8.2,2X,F7.3,2X,F5.2,2X,F8.4,2X,F5.0,2X,F5.2,
     &  2X,F4.2,2X,F6.2,2X,F6.2,2X,F7.0,2X,F7.0)
      IF(NNN)15,12,12
12       IF(DEGREE-SI)13,100,14
14       IF((DEGREE-SI)-10.)100,100,501
501       SUMRES=SUMRES+RES
      GO TO 2
13       CON=SUMD-D
      DEPTHO=DEPTH-D
      SIN1=SI-AYERI
      NNN=-10
15       IF(DEGREE-SI)17,100,16
16       IF(DEGREE-(SI+10.))100,100,18
18       SUMFD=SUMFD-(FUSION*D)
      SUMCD=SUMCD-(C*D)
      D=D*((DEGREE-SIN1)/AYERI)
      IF((ABS(SUMD-(CON+D)))-.03)100,100,102
102      SUMD=CON+D
      SI=SIN1
```

```
1700       GO TO 10
1710  17     IF((DEGREE+10.)-SI)18,100,100
1720  100 DEPTH=DEPTHO+D
1730       PRINT 701,DEGREE
1740  701  FORMAT(1H ,////,5X,8HINDEX = ,F7.0,1X,13HDEGREE DAYS F)
1750       PRINT 702,F
1760  702  FORMAT(1H ,4X,11HN FACTOR = ,F5.2)
1770       IF (FLAG1.NE.1.) GO TO 33
1780       PRINT 703,DEPTH
1790       GO TO 747
1800  33   PRINT 203,DEPTH
1810  203  FORMAT(1H ,4X,46HDEPTH OF THAW PENETRATION FOR THIS SOLUTION
1820      & = ,F4.1,1X,5HFEET.)
1830  703  FORMAT(1H ,4X,47HDEPTH OF FROST PENETRATION FOR THIS SOLUTION
1840      & = ,F4.1,1X,5HFEET.)
1850   747 PRINT 704,T
1860  704  FORMAT(1H ,4X,19HLENGTH OF SEASON = ,F5.0,1X,5HDAYS.)
1870       PRINT 705,ATM
1880  705  FORMAT(1H ,4X,19HMEAN ANNUAL TEMP = ,F5.1,1HF/)
1890  500  PRINT 204
1900  204  FORMAT('SOLUTION COMPLETE.  TYPE "100000." IF YOU DO NOT WANT',/
1910      &' TO CONTINUE AND "99999." IF YOU DO.')
1915  1999 IF (FLAG2.EQ.1.) READ(1,1) TAG
1920        IF (FLAG2.NE.1.) READ(0,1) TAG
1930       IF (TAG.EQ.100000.) GO TO 2001
1932       IF (TAG.EQ.99999.) GO TO 109
1935       GO TO 500
1939   2001 CONTINUE
1940       END
```