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Performance of Insulated Pavements at Newton Field, Jackman, Maine

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Maureen A. Kestler
Richard L. Berg

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May 1992

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16. Abstract

In 1986, the runway at Newton Field, a small airport in Jackman, Maine, was reconstructed using a 2-in.-thick layer of extruded polystyrene insulation as part of the pavement structure. At the same time, a nearby town road was reconstructed using a conventional noninsulated pavement cross section for relatively heavy loads. Both pavements were monitored for frost penetration, frost heave, and seasonal changes in pavement strength. Since frost penetration beneath the insulation layer of the runway at Newton Field exceeded empirical estimates during winter 1986–1987, four additional test sections with varying combinations of insulation and subbase thicknesses were constructed adjacent to the airport's parking apron during summer 1987. Although the thermal performance of the insulated pavement test sections was comparable to design expectations for the following three years, evidence of discontinuities in the insulation layer in the Newton Field runway demonstrates the insulated pavement's susceptibility to variations in construction technique and site conditions. This report discusses pavement performance at each of the test sites over the observation periods 1986–1990 and 1987–1990.

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PREFACE

This report was prepared by Maureen A. Kestler and Dr. Richard L. Berg, Research Civil Engineers, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory.

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TABLES

Performance of Insulated Pavements at Newton Field, Jackman, Maine

MAUREEN A. KESTLER AND RICHARD L. BERG

PURPOSE/SCOPE

Three pavement test sections located in Jackman, Maine, have been monitored by CRREL over the past few winters: a portion of the insulated pavement on the Newton Field runway, insulated pavement test sections adjacent to Newton Field, and a conventional, noninsulated pavement at Nichols Road (Fig. 1a). To evaluate the effects of polystyrene insulation on pavement performance, each of the locations was monitored for frost penetration, frost heave, and variations in pavement strength.

This report discusses observations on the performance of the pavements over the duration of four winters: 1986–1987 through 1989–1990. More detailed results from the first (1986–1987) winter of observation at the Newton Field runway and greater detail concerning instrumentation, the construction sites, and the testing program are provided in Kestler and Berg (1991).

INTRODUCTION

The town of Jackman is located in northwestern Maine at an elevation of approximately 1175 ft above mean sea level (Fig. 1b). It has an average annual temperature of 38°F, and a design air freezing index of approximately 2570°F days.

Newton Field

The runway at Newton Field was reconstructed in 1986. The old runway was in extremely poor condition, as is shown in Figure 2. The longitudinal and transverse pavement surface was highly irregular and was disintegrating as a result of moisture entering the large number of cracks. Although located in the same vicinity as the old runway, the new 2900- × 60-ft runway is longer, wider, and at a slightly different orientation than the old runway.

According to U.S. Army Corps of Engineers and Maine Department of Environmental Protection criteria, the entire area is classified as a wetlands zone. Such near-surface water table conditions presented a challenge for both design and construction personnel. The soil profile for Newton Field is shown in Figure 3a.

Construction contracts for both insulated and noninsulated pavement were sent out for bid. The insulated alternative was selected when the bid was only 3.6% higher than the bid for the conventional pavement.

The project used 500,000 board feet of 2-in.-thick extruded polystyrene panels. The design was for total frost protection of the subgrade. The minimum compressive strength of the insulation was 40 psi, and the design load was a 30,000-lb single-wheel load.

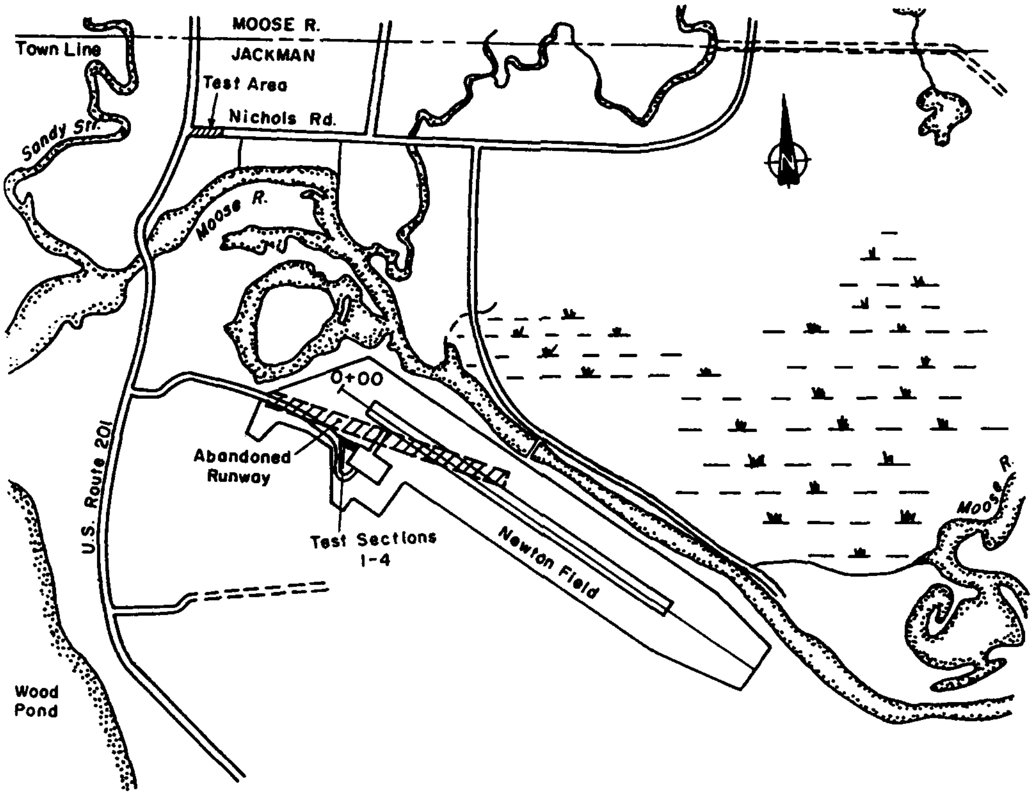
Figure 4a shows the typical cross section of the Newton Field runway. The pavement consists of $2^1/_2$ in. of asphalt concrete pavement, 12 in. of aggregate base course, a 2-in.-thick layer of extruded polystyrene insulation, and a sand leveling course of varying thickness (1 in. minimum), which was separated from the underlying wet silty subgrade by a geotextile.

Nichols Road

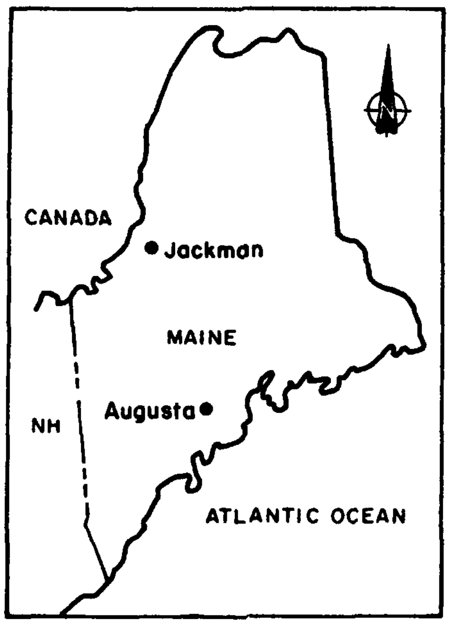
Also in 1986, the first 150 ft of Nichols Road was reconstructed to a cross section very similar to the noninsulated pavement alternative specified for the runway at Newton Field. Figure 4b shows a typical cross section of 3 in. of asphalt concrete pavement, 9 in. of gravel base, and 18 in. of sand subbase. The pavement structure is separated from the wet silty subgrade by a geotextile. Figure 3b shows the soil profile at Nichols Road.

Test sections 1–4

Since test results from the first winter of observation (1986–1987) showed substantial frost penetration beneath the insulation on the Newton Field runway, four test sections consisting of varying combinations of insulation and sand subbase thickness were constructed



a. Map of Jackman, Maine.



b. Location map.

Figure 1. Location of test site.

a. Looking northwest.

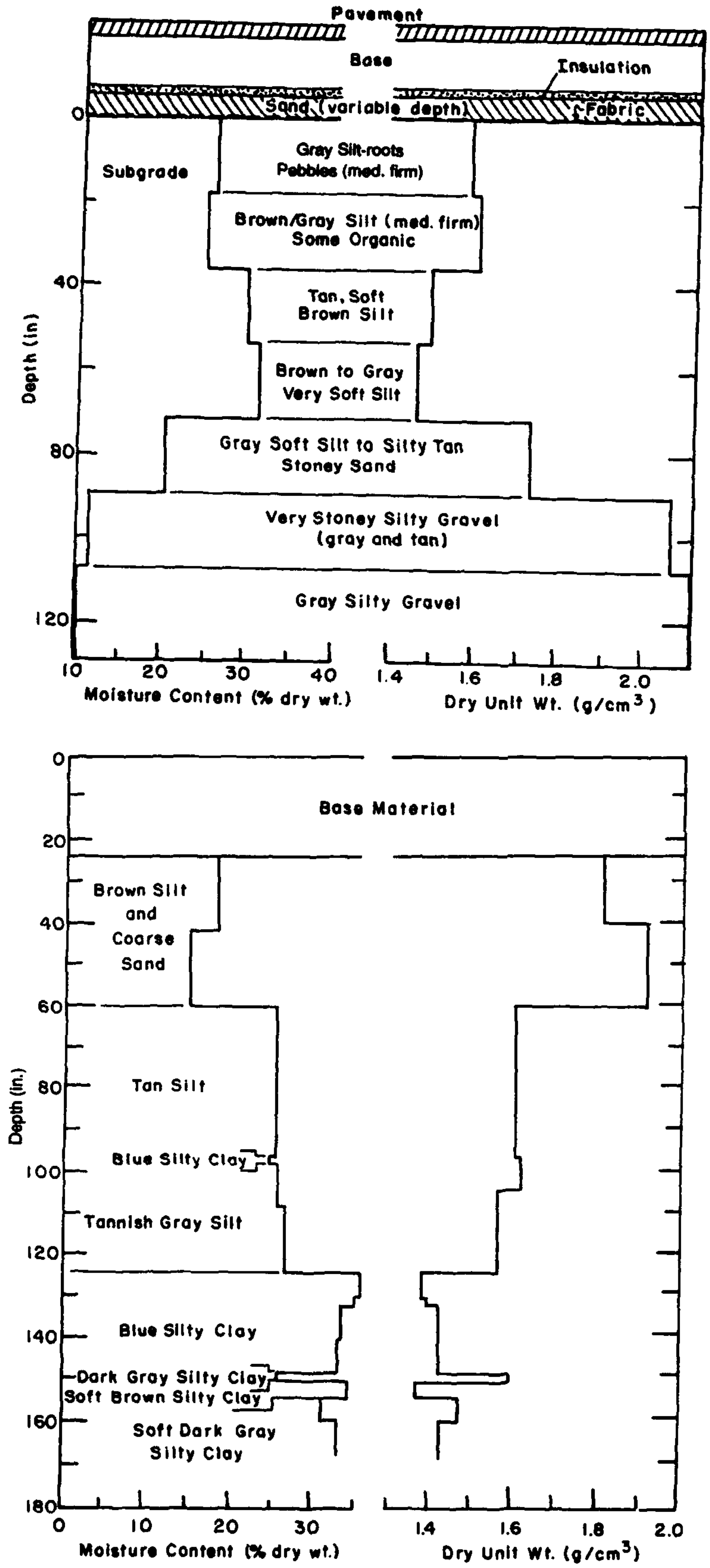
b. Boulder heave.

Figure 2. The Newton Field runway before 1986.

adjacent to the aircraft parking apron in July 1987. The test section site was wet, but was not as swampy as much of the runway site. Figure 5 shows a longitudinal section at the test sections. The pavement consists of $2^1/_2$ in. of bituminous concrete, 12 in. of aggregate base course, and 2- and 3-in.-thick layers of extruded polystyrene insulation in combination with 6 and 24 in. of sand subbase. Test section 1 most closely approximates the design used for the insulated runway. In contrast to single-thickness insulation panel placement in the runway, insulation panels in the test sections were placed in multiple layers with joints staggered as shown in Figure 6. The soil profile is the same as that of the Newton Field runway (Fig. 3a).

INSTRUMENTATION

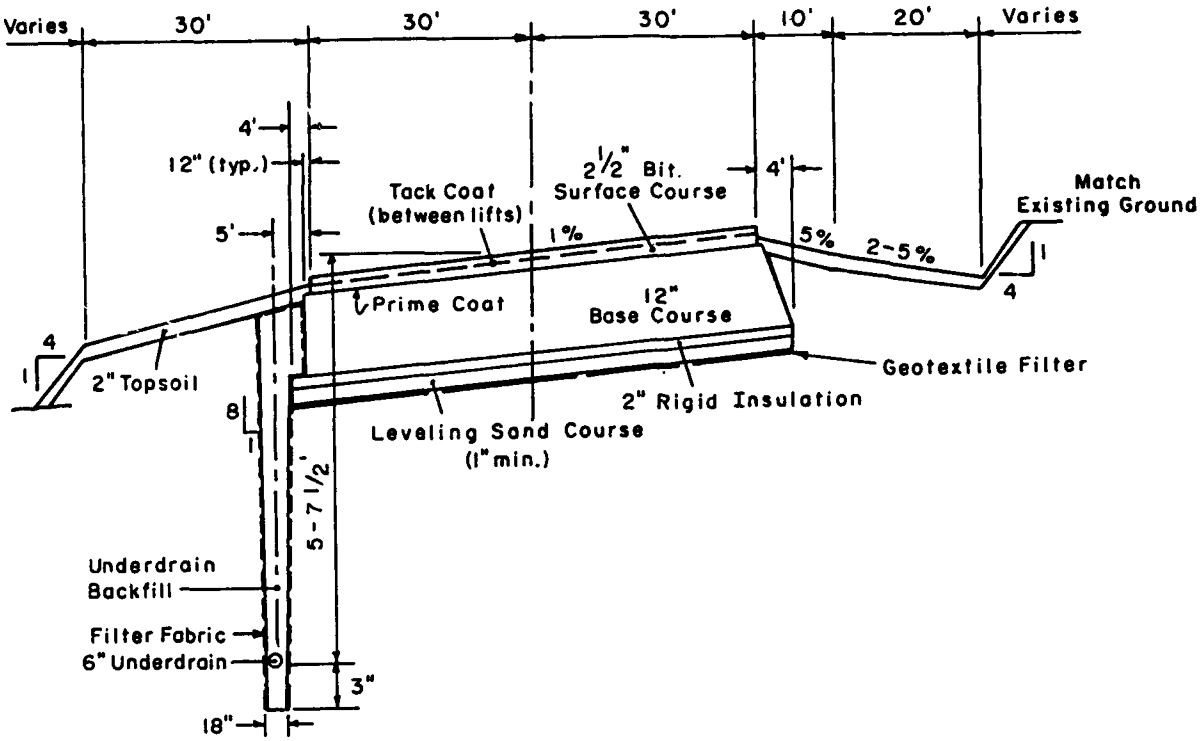
The instrumentation installed by CRREL during construction of each test site consisted of thermocouples to monitor subsurface temperatures and tensiometers to monitor soil pore water pressures. Instrumentation at the test sections also included thermistors to measure subsurface temperatures above, within, and beneath the insulation and electrical resistivity gages to indicate frozen/nonfrozen conditions beneath the insulation. The groundwater table was monitored via water wells at Newton Field and Nichols Road. Frost heave was measured by conducting periodic pavement surface elevation surveys with an engineer's level and rod.



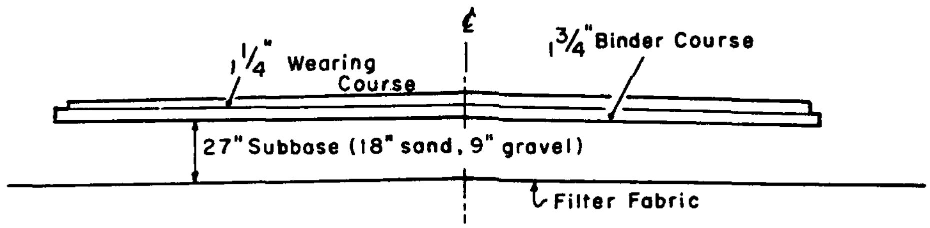
a. Newton Field.

b. Nichols Road.

Figure 3. Soil profiles.



a. The insulated runway pavement at Newton Field.



b. The noninsulated pavement at Nichols Road.

Figure 4. Typical cross sections of the pavements studied.

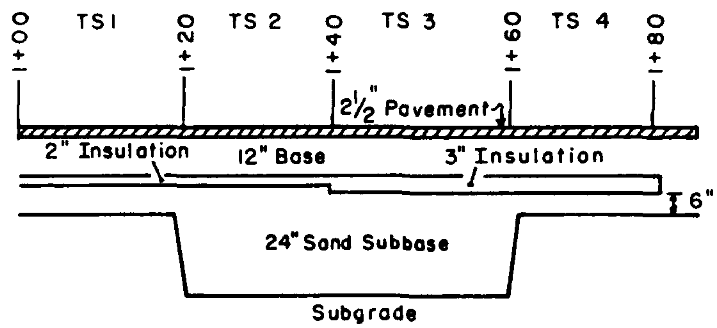


Figure 5. Longitudinal section of the insulated test sections.

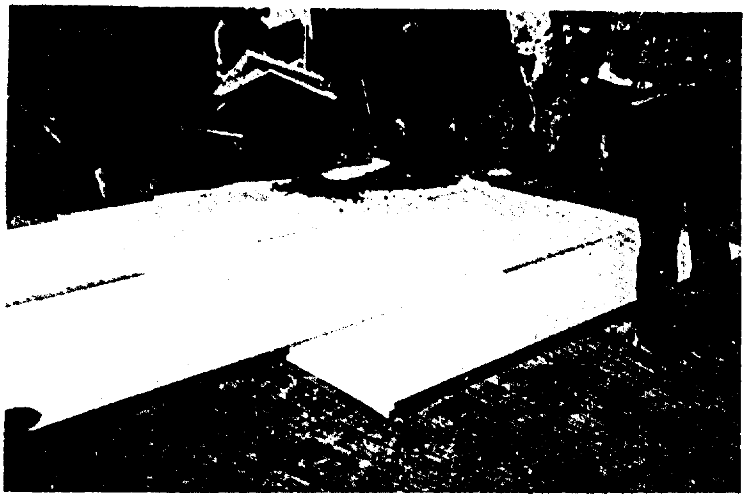


Figure 6. Placement of insulation panels at the test sections.

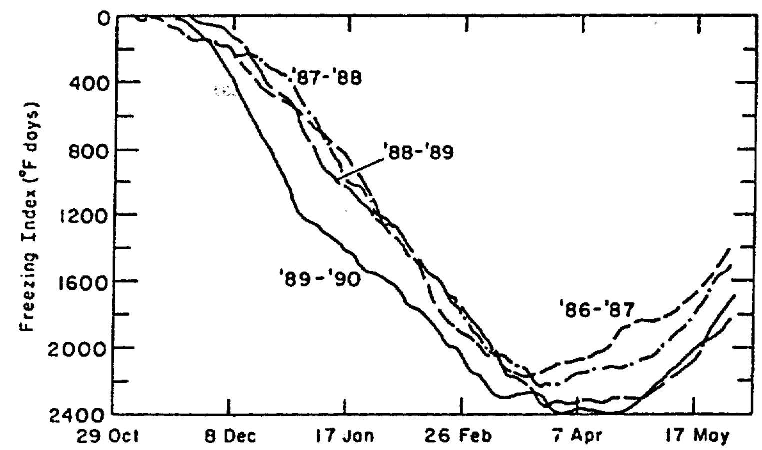


Figure 7. Air freezing index.

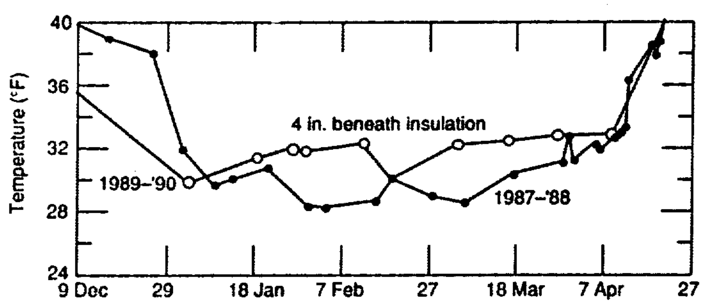


Figure 8. Temperatures at similar depths in 1987–1988 and 1989–1990.

A concrete benchmark was constructed by the Maine Department of Transportation adjacent to the Newton Field test sections; a spike in the concrete base of an existing flagpole served as a benchmark at Nichols Road. Pavement stiffness was measured nondestructively with a falling weight deflectometer (FWD).

Most tests have been conducted throughout each winter and spring since construction; however, no FWD tests were conducted during winter/spring 1989–1990.

FREEZING INDEX

The air freezing index for each winter was calculated from temperatures recorded at Central Maine Power (CMP), located approximately one half mile southwest of Newton Field. The winter of 1986–1987 was an average winter with an air freezing index of 2185°F days. Although each of the following winters was sequentially colder, the design freezing index (average of the three coldest winters in the past 30 years) of 2570°F days was never attained at Jackman during the four winters of observation.

The air freezing index is expressed as a function of time in Figure 7. In contrast to the similarly sloping curves corresponding to the first three winters, the steeper slope of the 1989–1990 freezing index indicates

December was a comparatively cold month. According to CMP's records, not once during December 1989 did the air temperature rise above freezing. By 1 January 1990, the air freezing index was approximately twice that determined for any of the three preceding winters; however, as is also shown in the figure, the rate of accumulation in the freezing index for January 1990 was less than during earlier winters. Although the final freezing index for 1989–1990 exceeded that of the three previous winters, a combination of the distribution over time of the colder temperatures and the presence of heat at depth resulted in frost penetration depths generally equal to or less than those of the first three winters.

The warming at depth, throughout periods of subfreezing air temperatures, is exemplified by Figure 8. During the month of December 1989 (air freezing index for winter 1989–1990 = 2405°F days), the temperatures 4 in. beneath the insulation were considerably colder than during the month of December 1987 (air freezing index for winter 1987–1988 = 2236°F days). Once the rate of accumulation in air freezing index decreased, the temperatures shown for the two winters were quite similar.

CRACKS

The asphalt concrete was placed in two lifts. The first was placed immediately following construction in 1986, and the second was placed in 1987. Two major transverse cracks developed in the first lift during the first year: one at station 8+00, and the second at station 30+00. Each of the two cracks reflected through the final layer.

The progression of crack development following placement of the final $1^1/_4$-in. asphalt concrete lift in 1987 is shown in Figure 9. Although the cracks had been sealed in September 1989 with a rubberized crack sealer and polyfiber, nearly all cracks have since reopened. In all instances, the failure occurred due to a loss of adhesion between the asphalt pavement and the sealing agent. The crack shown in Figure 10a was unsuccessfully sealed with silicone shortly before being resealed with the rubberized crack sealer.

Generally, the cracks have manifested themselves in two varieties: small longitudinal cracks typically occurring in localized groups, and individual long, wide, transverse cracks. Elev-

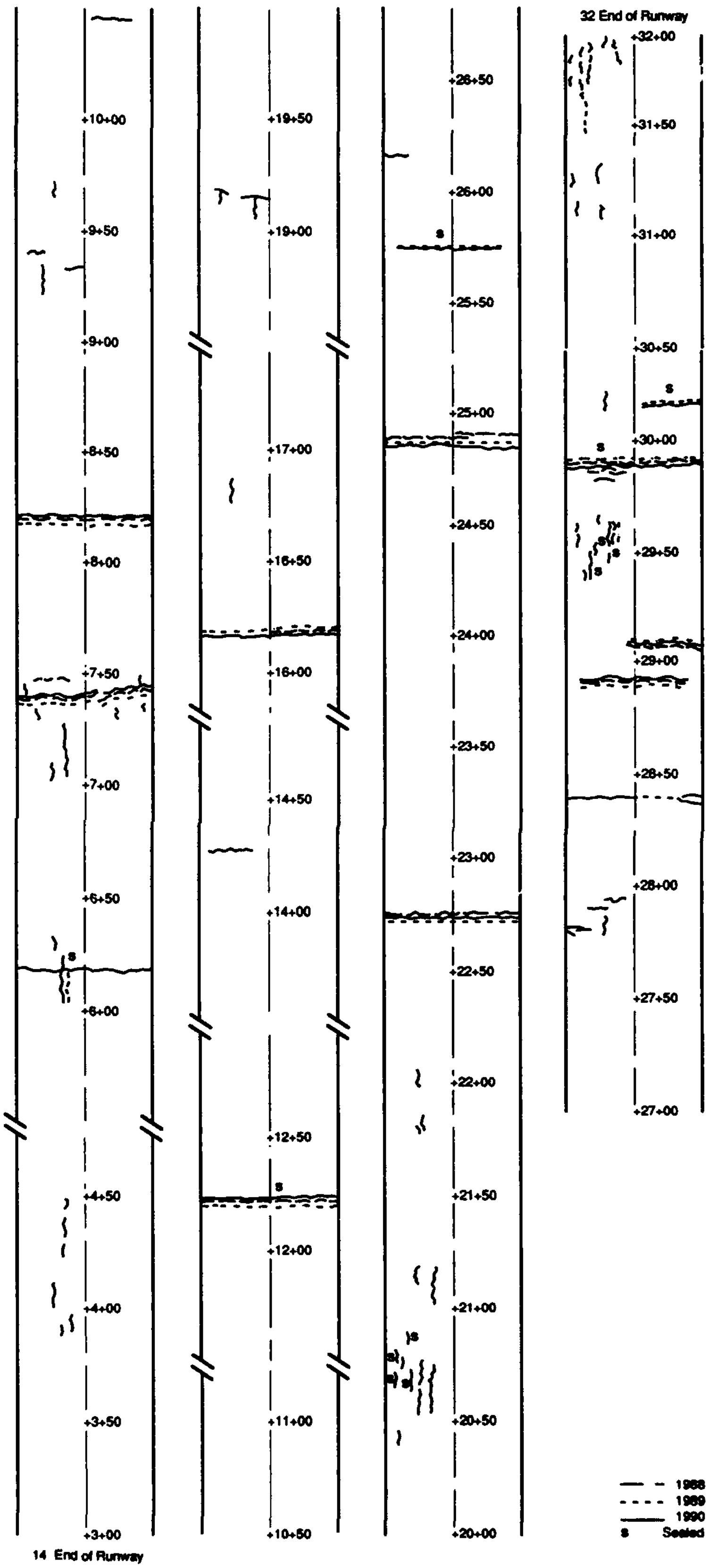


Figure 9. Newton Field pavement crack map.

a. Crack at station 8+22 initially sealed with silicone, then with a rubberized crack sealer.

b. Crack at station 16+15.

Figure 10. Cracks in the Newton Field runway.

c. Settlement of pavement at station 29+95

Figure 10 (cont'd).

en transverse cracks extend across, or nearly across, the entire runway, and two transverse cracks extend across the taxiway. Crack density is particularly high at the southeast (32) end of the runway; elsewhere, cracks are fairly evenly distributed.

The asphalt concrete is beginning to exhibit signs of secondary cracking. One possibility is that the base course has eroded from beneath some of the larger cracks; however, no evidence of removed material has been observed at the edges of the runway. Figure 10b illustrates a crack at least 9 in. deep at station 16+15, and Figure 10c shows the results of the secondary cracking and ensuing settlement of approximately 3 in. at station 29+95. While Figures 10b and c represent the most extreme of the crack-related failures exhibited by the four-year-old pavement, progressive deterioration is inevitable unless the cracks are successfully sealed.

No cracks have been observed on the smaller pavement sections (i.e., the 300- × 125-ft apron or the 80- × 20-ft test sections) or on Nichols Road.

Table 1. Maximum frost heave at each of the test sites, 1986–1990.

Test sites	*Approximate maximum vertical displacement (in.)*
Insulated pavements:	
Test sections	1
Newton Field, surface elevation grid	
10 × 25 ft grid	
25 points per 100 ft section of grid	
Station 4+00—Station 5+00	2.5
Station 5+00—Station 6+00	1.5
Newton Field, centerline—every 100 ft	
Station 3+00—Station 32+00	1
Station 4+00—Station 6+00	3
Station 8+00—Station 9+00	2
Station 30+00—Station 32+00	3
Noninsulated pavement:	
Nichols Road	
Station 0+00—Station 1+50 (new)	1
Station 1+50—Station 2+00 (transition)	2
Station 2+00—Station 3+00 (old)	3.5

* Baseline elevations from summer 1987.

FROST HEAVE

As was noted earlier, frost heave was measured periodically with an engineer's rod and level. Table 1 shows the maximum vertical displacements, which were fairly similar throughout the four winters of observation. The maximum frost heave at test sections 1–4 and the noninsulated pavement at Nichols Road was approximately 1 in. While the maximum frost heave along most of the runway was only slightly greater than that observed at the test sections, substantial frost heave occurred in three localized areas: station 4+00 to station 6+00 exhibited approximately 3 in. of frost heave; stations 8+00 to 9+00, approximately 2 in.; and stations 30+00 to

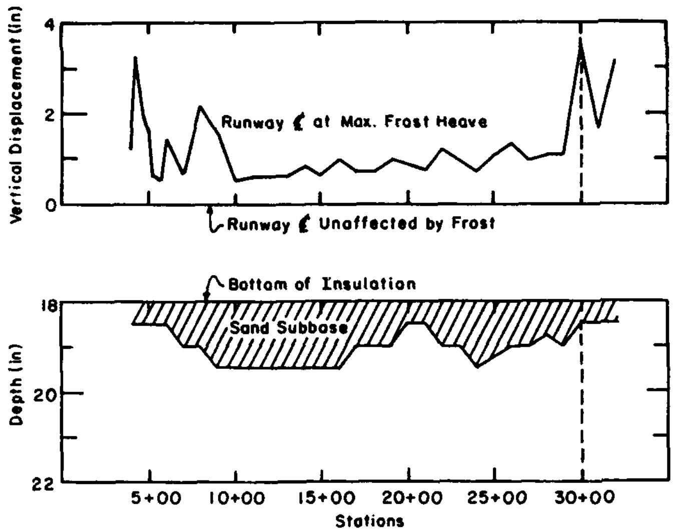


Figure 11. Profile of frost-heaved centerline and corresponding depth of sand subbase, March 1990.

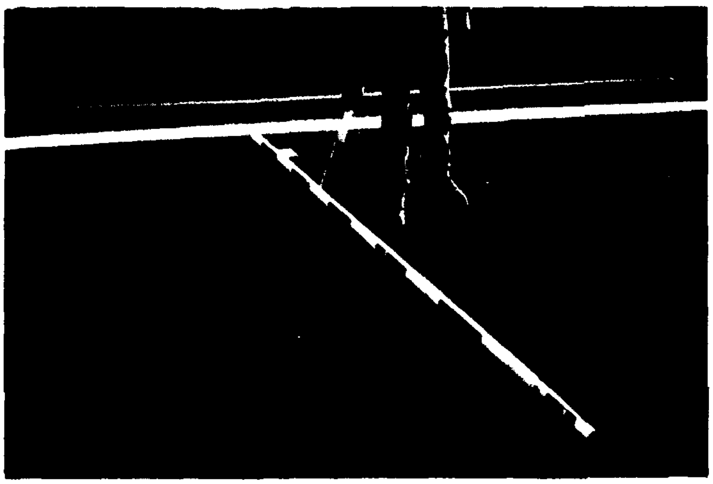


Figure 12. Differential frost heave near station 8+00. The "valley" is about 6 in. below the "ridges."

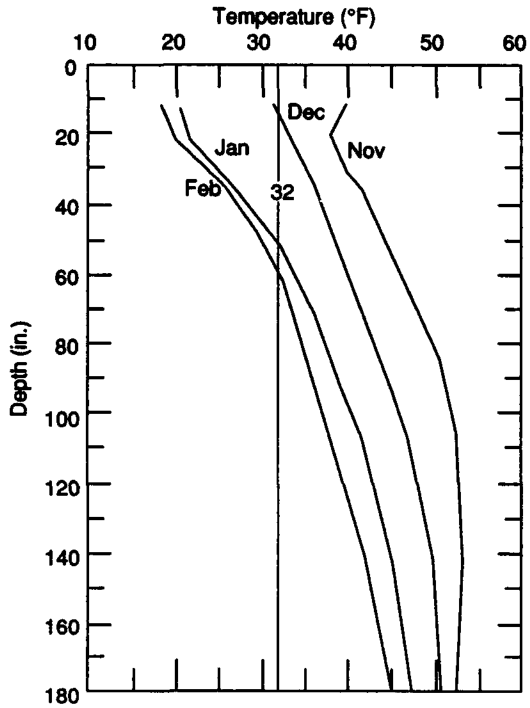
32+00, approximately 3 in. Figure 11 shows both the March 1990 centerline profile and the depth of sand subbase along the length of the runway. The areas that experienced appreciable frost heave generally correspond to the two ends of the runway that were excavated to a lesser depth. In addition, both ends of the runway were particularly spongy during construction.

Localized differential heave was exhibited each spring in the vicinity of station 8+00 and between stations 29+00 and 32+00. Figure 12 depicts the irregular pavement surface near station 8+00 in early April 1988; in this area the differential movement was about 6 in. A variety of methods have been employed to investigate the causes of the localized differential frost heaving; the methods and observations are discussed later in this report.

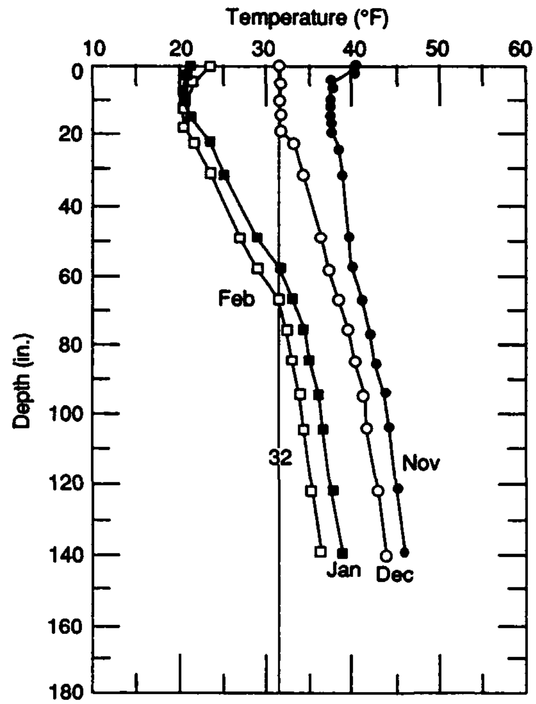
SUBSURFACE TEMPERATURES

Subsurface temperatures were recorded at the test sites with thermocouples and thermistors. Figures 13a and b show the changes in temperature with increasing depth beneath the pavement surface during winter 1987–1988 for the noninsulated pavements at Nichols Road and the unpaved road adjacent to the test sections, respectively. For any given winter month shown, temperatures gradually increase with increasing depth beneath the pavement surface. Figures 13c and d demonstrate the effectiveness of the insulating layer for the insulated pavements at the test sections and the runway.

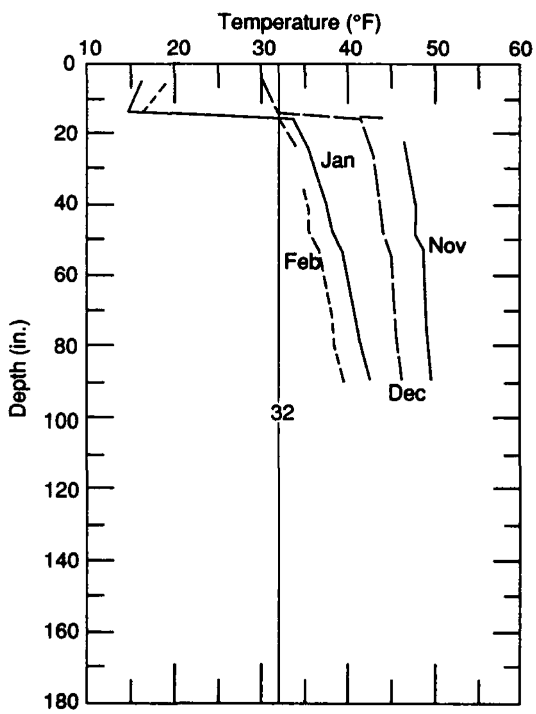
A distinct temperature discontinuity occurs immediately beneath the insulation on the runway (Fig. 13d). Two vertical thermocouple assemblies, one located both in and above the insulation and the second located entirely below the insulation, are separated horizontally by approximately 5 ft. It is believed that the temperature discontinuity between the thermocouple assemblies is caused either by damage to the insulation or by the horizontal separation of the insulation panels. The problem probably occurred during construction. Evidence of this problem was encountered during construction near stations 4+50 and 8+00. At both locations, trucks, bulldozers, and other construction traffic caused a large subgrade "flow" that in turn raised the insulation. Under



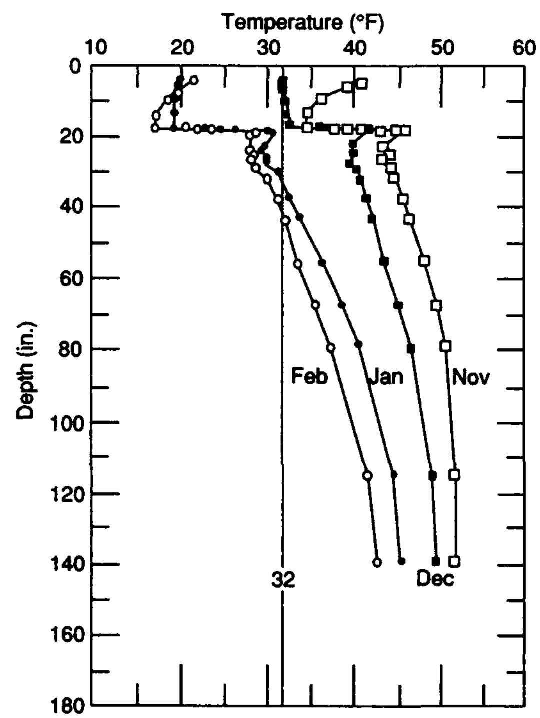
a. Nichols Road, winter 1987–1988.



b. Subsurface temperatures at unpaved road adjacent to test sections, winter 1987–1988.



c. Subsurface temperatures at test sections, winter 1987–1988.



d. Subsurface temperatures at Newton Field, station 4+50, winter 1987–1988.

Figure 13. Subsurface temperatures at the study sites.

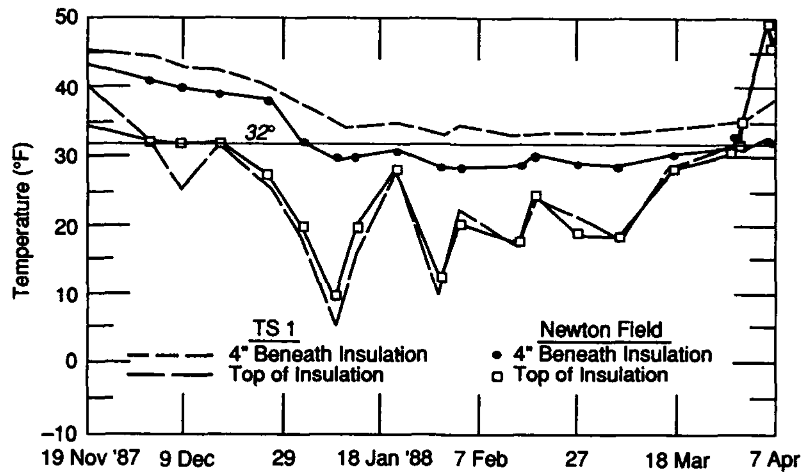


Figure 14. Comparison of temperatures above and beneath the insulation at Newton Field and in test section 1.

direction of the resident engineer, the insulation was removed, the subgrade was removed to the desired depth, and the geotextile, insulation, and base course were all replaced.

The difference in insulating ability of the insulation at the two pavement sites is also seen in Figure 14. Although the temperatures at the top of the insulation at each site are comparable, 4 in. beneath the insulation the temperatures differ substantially and fairly consistently over time. For winter 1987–1988, the 32°F isotherm never penetrated to this depth at the test sections, but at a comparable depth at the runway, the temperature remained below 32°F for nearly three months. This performance was the same for all three winters during which subsurface temperatures were obtained at both sites.

FROST PENETRATION

The progression of frost penetration with time is shown in Figure 15. For the 1987–1988 winter, the insulated pavement at the runway (station 4+50) allowed approximately 2 ft of frost penetration beneath the insulation. Although appreciably less than at noninsulated Nichols Road, the actual frost depth far exceeded all estimates. The unexpected frost penetration beneath the insulation on the runway during winter 1986–1987 constituted the primary reason for installation of the test sections in 1987. In contrast to the limited effectiveness of the insulation at the runway, the insulation at test section 1 prevented frost from penetrating into the subgrade. While Figure 15 is based upon 1987–1988 data, the curves are representative of each year; frost never

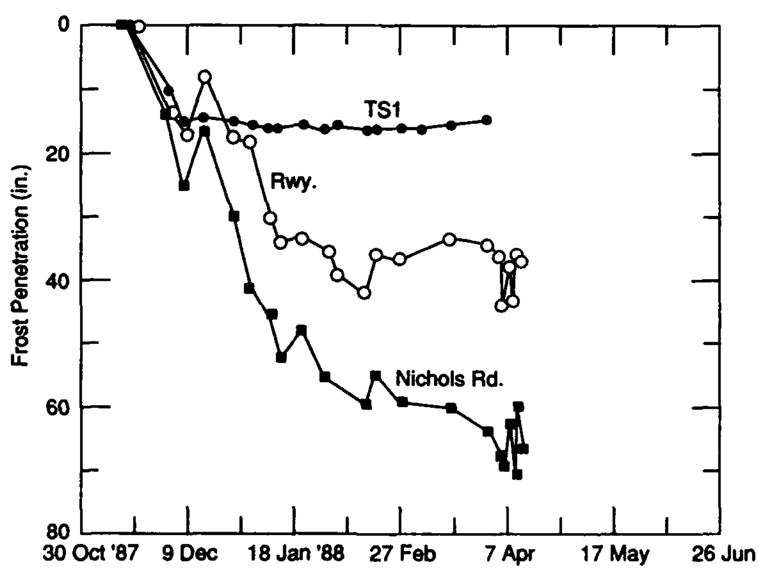


Figure 15. Progression of frost penetration with time, winter 1987–1988.

Table 2. Maximum frost depth (ft) at each of the test sites, 1986–1990.

Frost penetration test sites	*Depth (ft)* 1986-87	1987–88	1988–89	1989–90
Air freezing index (°F days) *	2185	2136	2344	2405
Nichols Road	5	5.5	5.5	5
Newton Field runway	3.5	3.5	4	3
Test section 1	—	1.2	1.6†	1.6†
Unpaved road (adjacent to test sections)	—	5	5	4.5

* Design air freezing index = 2570°F days.
† 5 in. beneath bottom of insulation.

penetrated more than 5 in. beneath the insulation at test section 1. When frost did penetrate the insulation, it lasted in the subbase less than 7 weeks during winter 1987–1988, and less than 3 weeks during winter 1989–1990. Maximum frost depths at each of the test sites for each year of observation are summarized in Table 2.

CONDITION OF THE INSULATION

On 27 July 1987, a 10- × 31-ft section of pavement near station 30+00 was removed due to excessive localized settlement (Fig. 16). Figure 17 shows the overlapping and damaged insulation that was removed and replaced. The damaged insulation at this particular location had been attributed by the consulting/design firm to settlement of the backfill above the 10-in. PVC sewer line that crosses the runway at a depth of approximately 8 to 9 ft beneath finished grade. It is probable, however, that similar insulation damage or separation of adjacent panels caused the temperature anomaly at station 4+50. Similar problems also are probably responsible for the substantial differential frost heave at both ends of the runway. These areas were undoubtedly more unstable during construction than areas with a thicker layer of granular material beneath the insulation. Since the subgrade possessed very low strength during construction, movement of the insulation panels due to construction traffic probably caused the individual boards to move, resulting in gaps in some locations and overlap in others.

During the years following the removal of the pavement section at station 30+00, a variety of nondestructive methods have been employed in an attempt to confirm the suspicion that damaged and/or separated insulation panels are not limited to station 30+00. The methods, discussed in subsequent paragraphs, have included hand-excavating the base course alongside the

Figure 16. 1987 removal of pavement at station 30+00.

Figure 17. Overlapping and damaged insulation.

asphalt concrete pavement and the use of both infrared photography and ground-penetrating radar.

Trench excavation

In April 1988, a 2- × 5-ft trench was hand-excavated immediately adjacent to the north edge of the runway at station 4+25. The insulation proved to be intact, with only a $^1/_4$-in. gap between panels. A similar 2- × 8-ft trench at station 7+75 on the south edge of the runway again yielded intact insulation; however, a $2^1/_2$-in. gap was observed between insulation panels (Fig. 18). A third trench was started at station 4+75 on the south edge of the runway, but was abandoned when insulation was not encountered at the excavated depth of 18 in.

Prior to both the pavement removal at station 30+00 and hand-excavating alongside the runway, in March 1987, 25 smaller holes were manually dug at random locations alongside the runway to determine the depth to insulation. Although the plans specify 12 in. of aggregate base course atop the insulating layer, actual depths at which insulation was encountered varied from 6 in. at station 8+17 to 16 in. at station 7+00. Measured depths to insulation and respective locations are listed in Table 3.

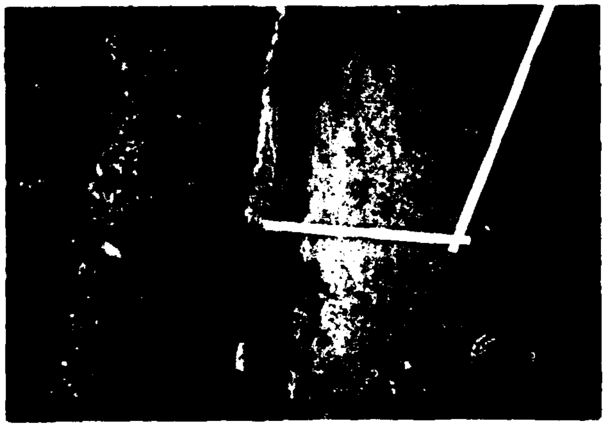
Infrared photography

Figure 19 illustrates the variation in pavement surface temperatures in the immediate vicinity of a crack. In an attempt to record similar surface manifestations of the temperature variations caused by underlying gaps between panels or by damaged insulation, an infrared camera was used by CRREL personnel during the spring of 1988 to photograph the pavement surface (Fig. 20). The equipment records the infrared image on video tape, and the corresponding surface temperature of the object is indicated on the screen. Due to equipment problems, the

Table 3. Insulation depths (in.) measured at edges of runway.

Station	*North edge of runway*	*South edge of runway*
4+50	—	15
7+00	16	—
7+50	9	—
8+00	7	—
8+17	6	10.5
8+22	7	11
8+27	7.5	12.5
8+50	7.5	12.5
9+00	7	13.5
9+50	12	—
10+00	9	10.5
15+00	13	10.5
20+00	13.5	14
25+00	13	15.5
27+00	—	12
30+00	9	—

a. Trench excavation.



b. Gap between insulation panels

Figure 18. Station 7+75, south edge of Newton Field runway.

infrared imaging feature was not used; only surface temperatures were recorded.

Pavement surface grids were established near stations 4+50 and 8+00, and surface temperatures were recorded. Although the edge of the insulation was readily apparent, insulation discontinuities were not identified. The runway exhibited considerably greater variation in surface temperatures than did the test sections; however, neither the increased surface temperature range nor the distribution of temperatures could provide conclusive evidence that gaps were present or that the insulation was damaged. Although the temperature variation could be indicative of gaps between panels or damaged insulation, it could also be caused by nonuniform thickness, densities, and/or water contents of any of the (upper)

Figure 19. Variation in pavement surface temperature due to the presence of a crack.

Figure 20. Infrared equipment.

pavement layers; subtle differences in color of the pavement surface; differential frost heave; or changes in thickness of cloud cover or air temperature during the time required to move from one grid point to the next.

Infrared tests were conducted under both daytime and nighttime conditions. Daytime conditions included both total sunlight and total overcast. Although absolute temperatures differed, relative temperature ranges at a specific site were similar.

Table 4 shows the pavement temperatures recorded near station 8+00 with the infrared equipment on 20 April 1988.

Table 4. Pavement surface temperatures (°F) near station 8+00 recorded with infrared equipment.

Distance from centerline (ft)	Temperature (°F) Station 8+00 8:00 am	Station 7+90 8:10 am	Station 7+90* 8:15 am	Station 7+80 8:20 am	Station 7+70 8:25 am
27.5 R	40.0	40.5	40.5	41.0	42.2
25.0 R	40.5	41.6	41.4	42.2	42.3
22.5 R	40.5	42.1	42.0	42.2	43.0
20.0 R	41.0	41.1	41.4	42.5	42.9
17.5 R	40.8	41.7	41.7	43.0	43.0
15.0 R	39.8	41.4	41.7	42.2	43.0
12.5 R	39.9	41.0	41.0	41.0	42.0
10.0 R	40.9	41.5	—	41.6	—
7.5 R	40.5	40.0	—	43.3	—
5.0 R	40.4	40.6	—	42.5	—
2.5 R	39.3	40.0	—	42.0	—
0.0	39.6	42.2	—	42.5	—
2.5 L	42.0	42.0	—	42.3	—
5.0 L	41.2	41.5	—	42.0	—
7.5 L	41.0	41.5	—	41.7	—
10.0 L	40.9	41.5	—	—	—

* Check for repeatability.

Ground-penetrating radar

Ground-penetrating radar investigations were conducted at Newton Field by CRREL personnel on 25–26 March 1987 and 15–16 June 1988 (Delaney 1988). The primary objective of the 1987 study was to locate the depth of frost penetration beneath the insulation. Although the frost depth could not be determined from the radar survey, depths to the insulation panels appeared to vary from approximately 5 to 24 in. beneath the pavement surface. Results from the 1987 investigation are discussed in further detail by Martinson (1989) and by Kestler and Berg (1991).

The two objectives of the 1988 survey were to map iregularities in the insulation beneath the runway surface and to locate buried vertical PVC drain pipes associated with a separate study at Jackman conducted by Allen (1991). Only the insulation-related investigations will be discussed in this report.

As noted earlier, the two ends of the runway had experienced appreciable total and differential frost heave, and the thermocouple assemblies at station 4+50 had indicated both a temperature discontinuity and substantial frost penetration beneath the insulation. Consequently, ground-penetrating radar surveys were conducted, primarily at both ends of the runway. Pavement surface grids were established at stations 4+50 and 30+75. A less detailed surface grid was set up at station 30+00, where the section of pavement had been removed and replaced in July 1987. Additional radar surveys were conducted along the runway centerline and at 10 runway cross sections. Nine of the runway cross sections were at the southeast end of the runway. Radar survey stations are listed in Table 5.

Figure 21 shows the instrumentation, which includ-

Table 5. Stations at which ground-penetrating radar surveys were conducted, June 1988.

Centerline	*Cross sections*	*Grid sections*	*Repaired runway*	*Pavement test section*
Sta 3+00 to	Sta 16+00	Sta 4+50	Sta 30+00	Section 1
Sta 32+00	Sta 29+05	Sta 30+75		Section 2
	Sta 29+95			Section 3
	Sta 30+50			Section 4
	Sta 30+75			
	Sta 31+00			
	Sta 31+25			
	Sta 31+50			
	Sta 31+75			
	Sta 32+00			

ed an impulse radar control unit (XADAR) coupled to a 900-MHz antenna. The antenna, which was mounted on plywood, was towed or manually pulled across the pavement surface, and data was recorded on magnetic tape.

Part of the signal emitted by the radar equipment is reflected back to the receiving antenna when an abrupt change in water content (caused by a change in dielectric constant) is detected. Travel times of radar emissions can be correlated to depths if the electrical properties of the subsurface are known. In the event the dielectric constant is not known, it can be back-calculated (Martinson 1989), provided the depth to a particular feature is known:

$$e = (tc/2d)^2$$

where e = dielectric constant
t = "round trip" time of radar pulse
c = the speed of light in a vacuum (1 ft/ns)
d = depth from the surface to the known feature.

Assuming the gravel base course between the insulation and the asphalt concrete is of relatively uniform water content, approximate depths from the pavement surface to the bottom of the insulation can be determined from the graphic radar records, which resemble subsurface profiles (Fig. 22). Typically, the uppermost set of dark bands represents the antenna direct coupling; the next series of bands represents the interface between the insulation panels and the subbase (i.e., the bottom of the insulation); and the third, less distinct set is simply a multiple of the above insulation/subbase interface bands.

The transducer was pulled manually along the centerline of the test sections; the corresponding radar record is shown in Figure 22a. The unequal depths of the insulation at the parking apron and test sections are indicated by the discontinuity in the second set of dark bands.

In contrast to the relatively uniform depth of the insulation panels at the test sections, Figures 22b and c show the irregular base course thickness and corresponding nonuniform depth of the insulation panels along the centerline at the two ends of the runway. For the insulation panels at shallow depths, the vertical stresses imposed by the design single-wheel load of 30 psi exceed the panels' 40 psi compressive strength. It is possible that this could lead to crushing of the insulation and ultimately lower the efficiency of the insulating layer. In Figure 22d, the individual 2-ft wide panels can be identified and are indicated on the figure.

NONDESTRUCTIVE TESTING

Pavement stiffness was measured nondestructively with the CRREL falling weight deflectometer (FWD) at each of the test sites and along U.S. Route 201 north of Jackman during winter–spring of 1986–1987, 1987–1988, and 1988–1989.

Briefly, the FWD operates as follows: An impulse load is applied to the pavement surface through a 12-in.-diameter circular plate, and seven sensors, spaced at desired distances from the center of the load, measure vel-

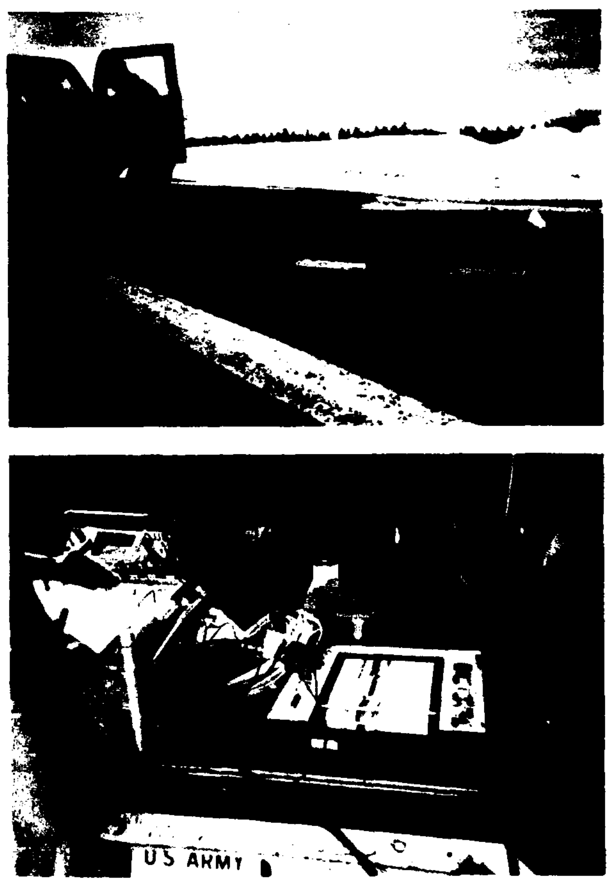
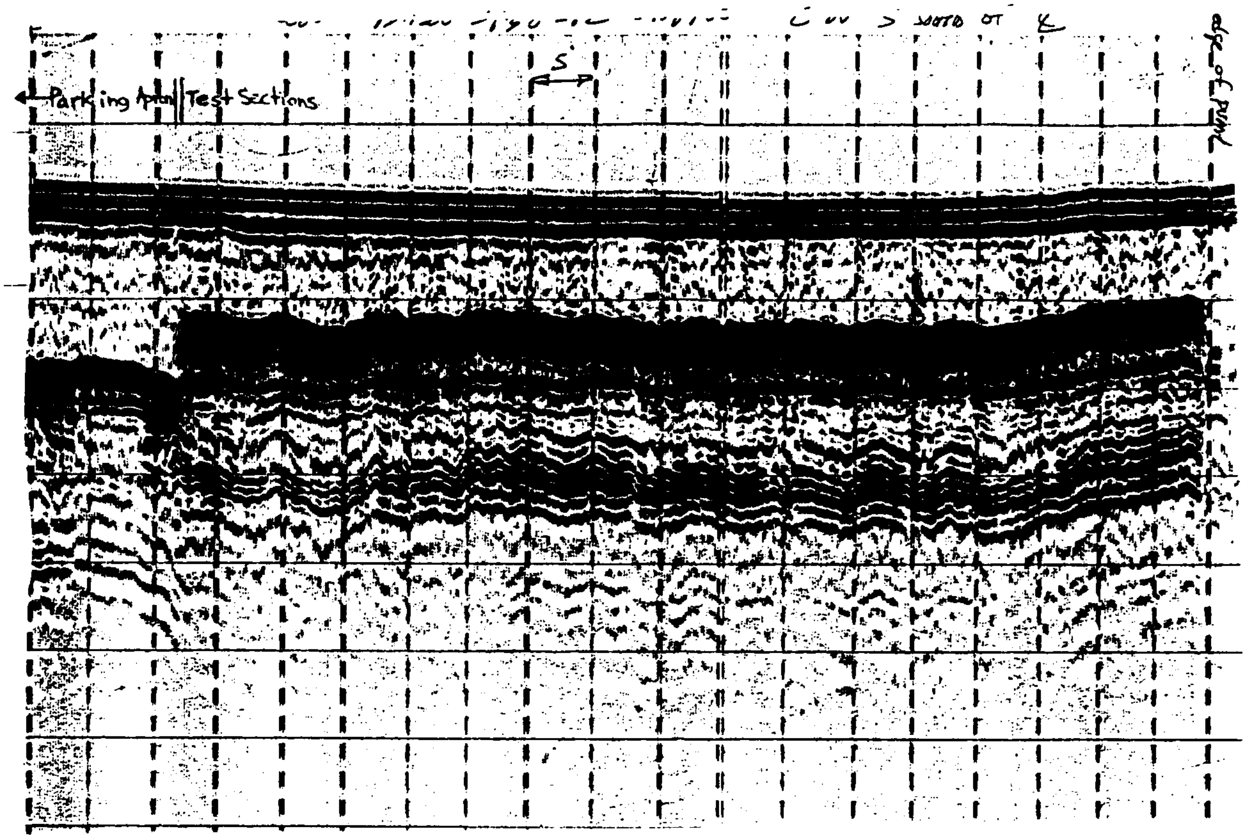
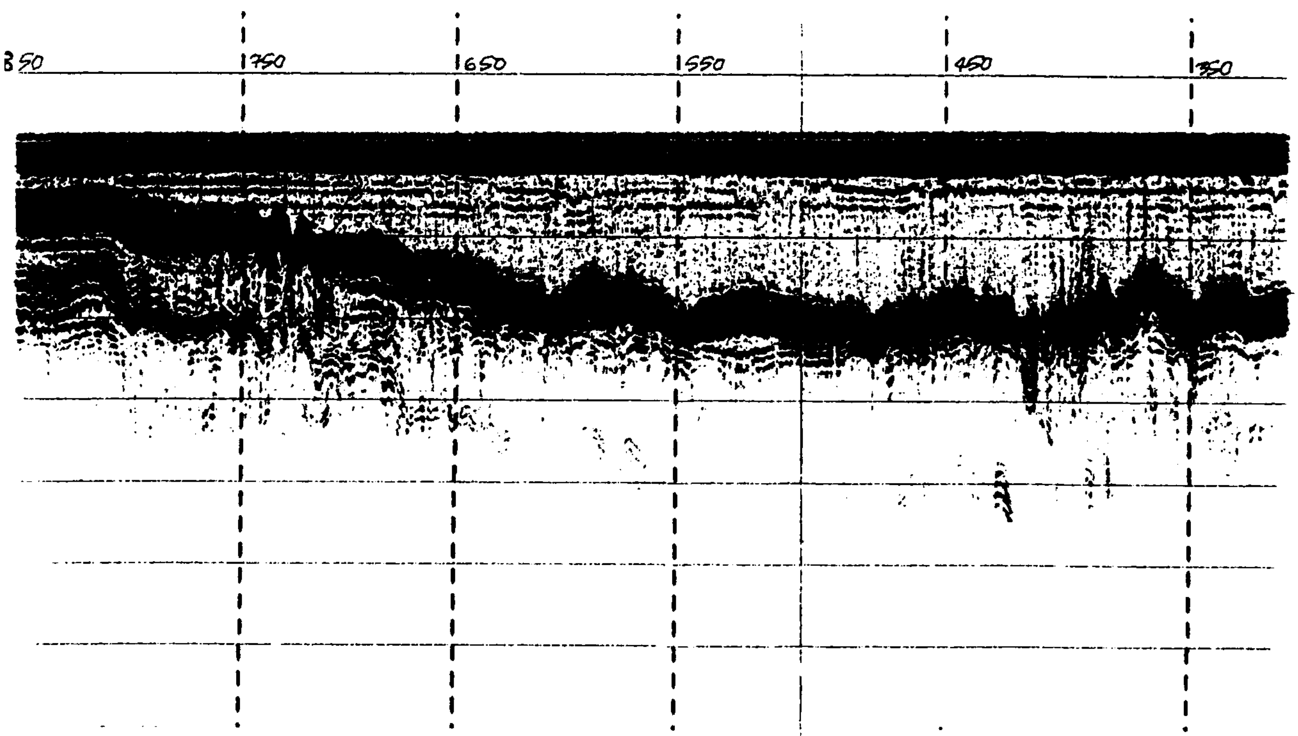


Figure 21. Ground-penetrating radar equipment.

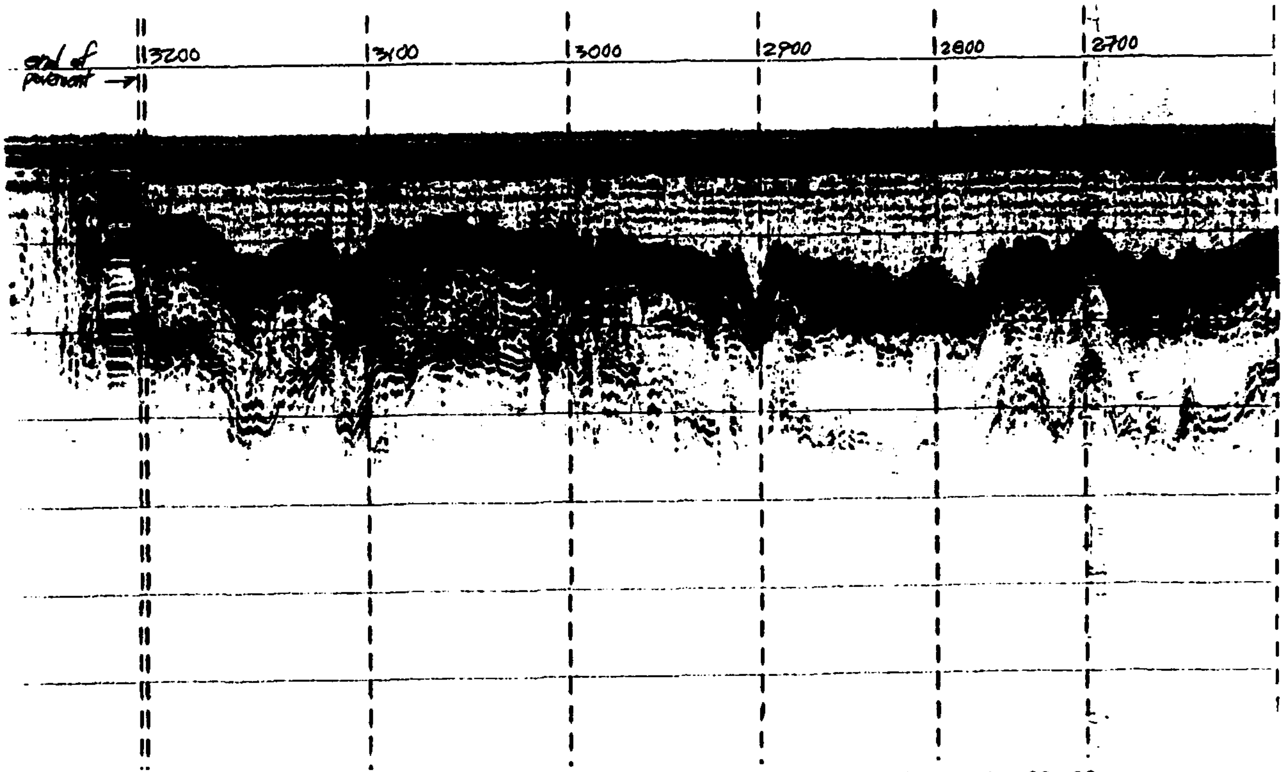


a. Centerline profile at test sections.

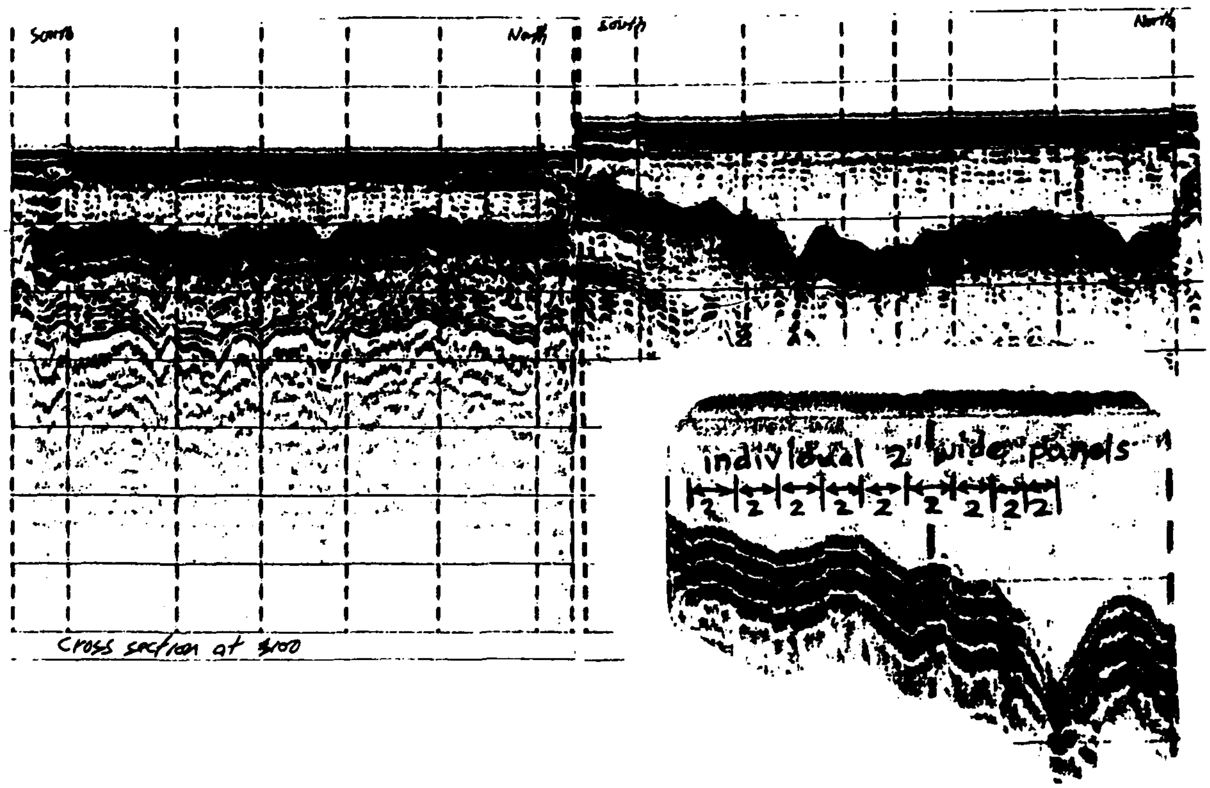


b. Centerline profile at west end of Newton Field, station 3+50 to station 8+50.

Figure 22. Ground-penetrating radar profiles.



c. Centerline profile at east end of Newton Field, station 26+00 to station 32+00.



d. Cross sections at east end of Newton Field, stations 29+05 and 31+00.

Figure 22 (cont'd). Ground-penetrating radar profiles.

ocity. Maximum vertical deflections at each sensor are determined by integrating the velocity–time history. The cross sectional area (through the center of the plate) bounded by the undeformed pavement surface and the vertically displaced pavement surface is termed the deflection basin area (Fig. 23).

With the exception of the thickness of the insulating layer, the cross sections for test sections 2 and 3 are identical. Consequently, differences in the shape and area of the deflection basins in Figure 24 are attributed primarily to the additional 1-in. thickness of insulation in test section 3.

The deflection basin areas of test sections 2 and 3 through winter/spring 1988–1989 are shown in Figure 25. In 1989, the pavement and base course in the test sections remained frozen through mid-March. This is reflected by the small deflection basin areas in the figure. The areas increased in late March and remained relatively constant through early May while the groundwater table immediately adjacent to the test sections remained high. High pavement temperatures also caused larger areas.

Figure 26 contains similar data for station 4+50 on the Newton Field runway. The figure expresses area, impulse stiffness modulus (ISM), and resilient modulus in terms of ratios to initial (15 Oct 1987) reference

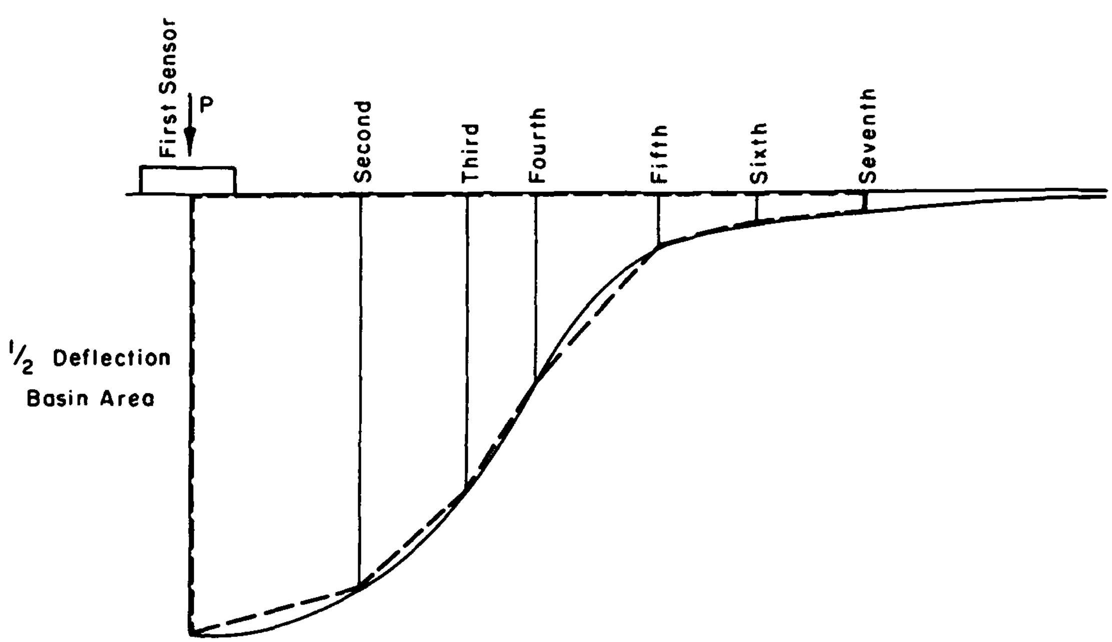


Figure 23. Applied load and resulting deflection basin.

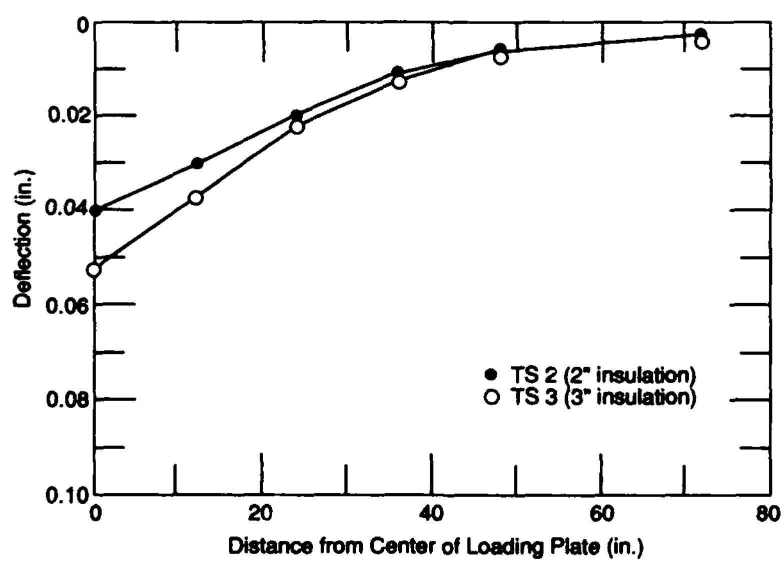


Figure 24. Typical deflection basins, test sections 2 and 3, spring 1989.

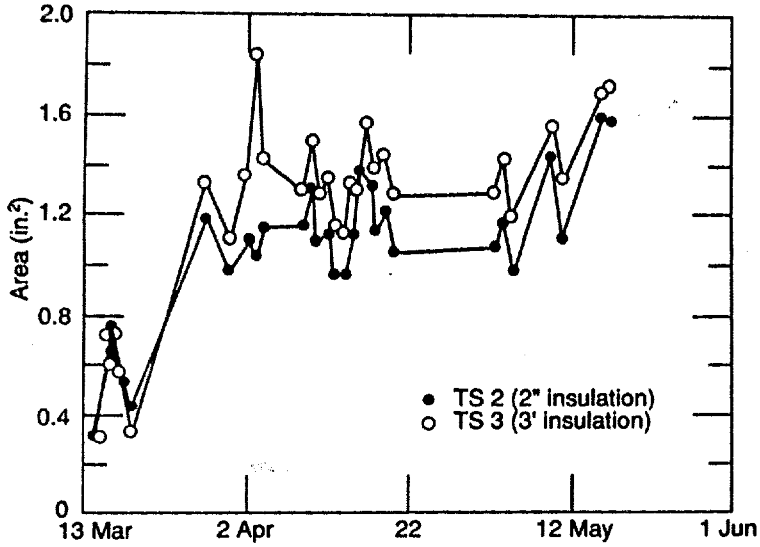
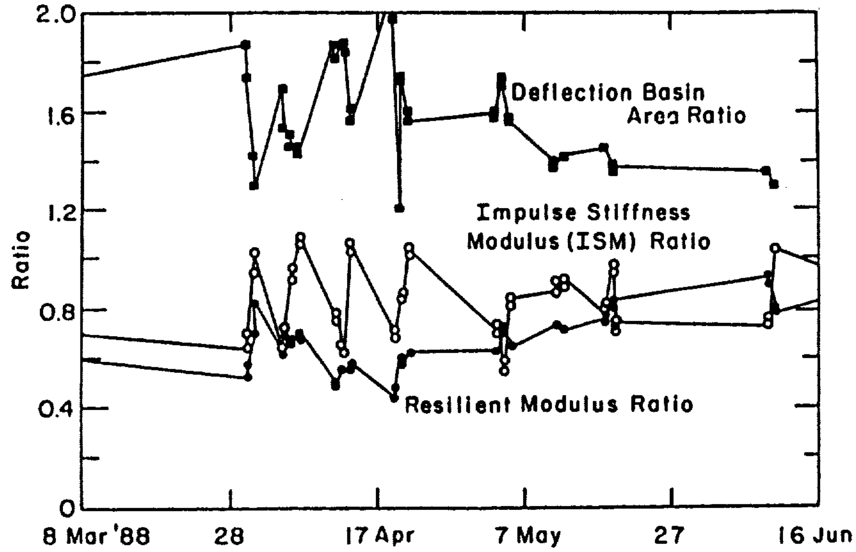
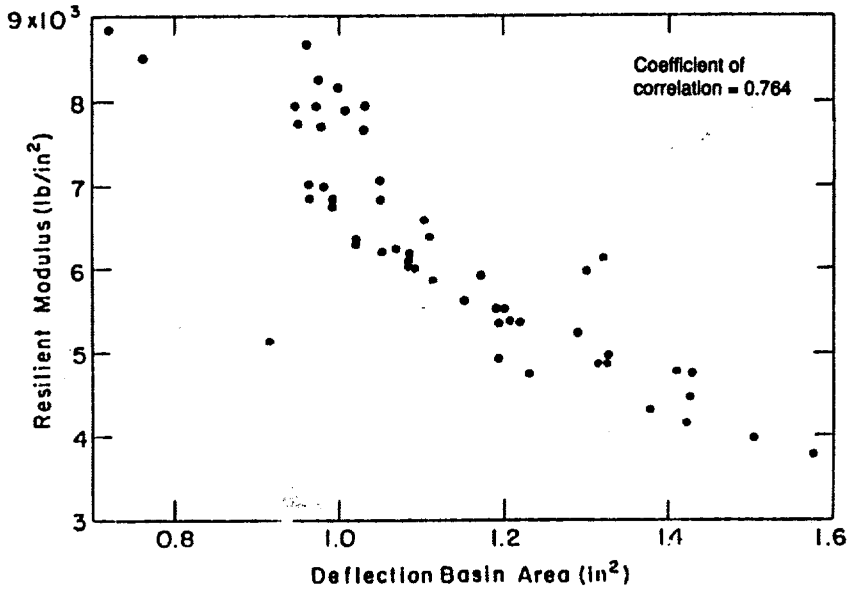


Figure 25. Deflection basin area vs time, test sections 2 and 3, spring 1989.



a. Deflection basin area ratio, resilient modulus ratio, and ISM ratio.



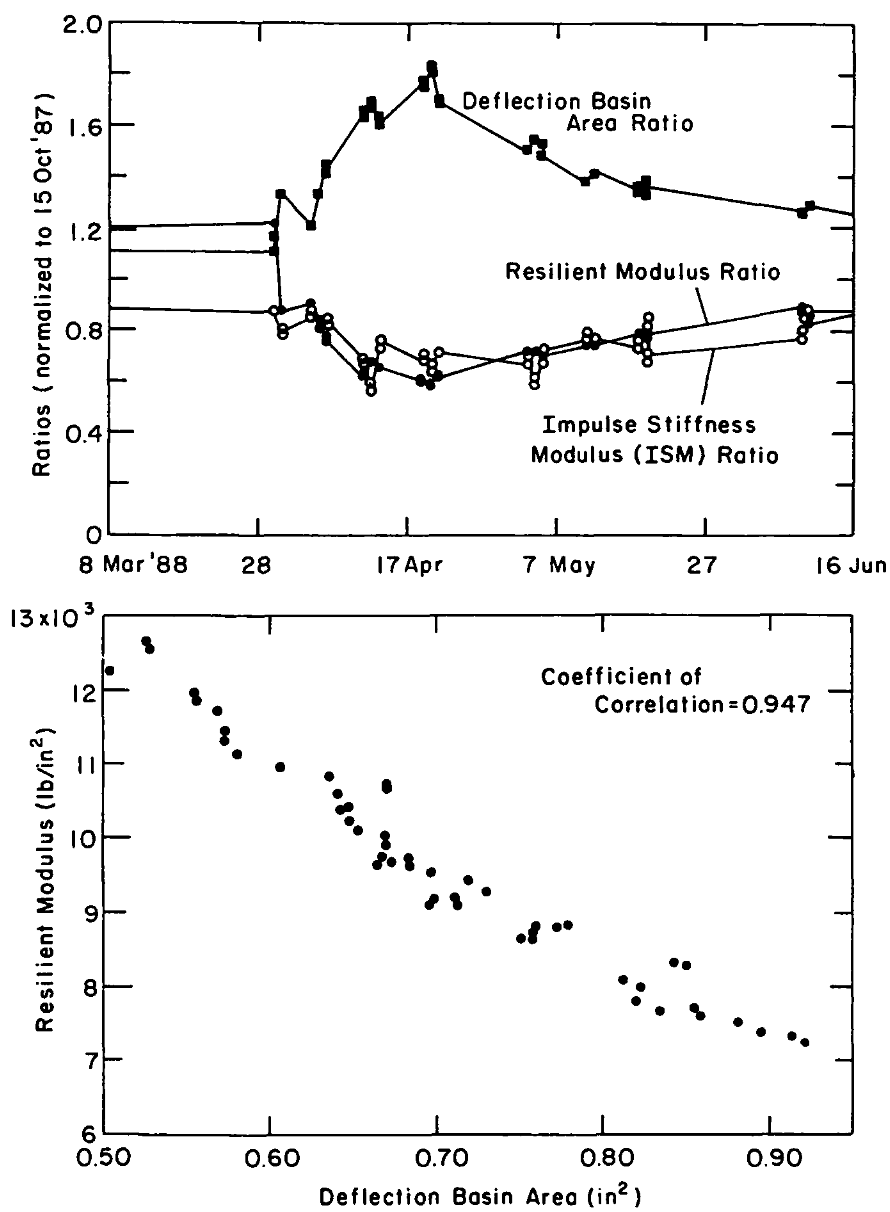
b. Correlation between deflection basin area and resilient modulus.

Figure 26. Deflection basin area, station 4+50, Newton Field, spring 1989.

values. The normalized deflection basin area is referred to as the de-flection basin area ratio (Janoo and Berg 1990). Resilient moduli were approximated using deflections from the third sensor and assuming a homogeneous, linearly elastic, one-layer system (Yoder and Witczak 1975). ISM is a stiffness indicator defined as the ratio of the applied load to the center deflection (Alexander et al. 1989). For insulated pavements, it is a poor indicator of stiffness because, again, the higher temperatures (recorded during FWD testing) attained by an insulated pavement result in lower asphalt concrete (A/C) moduli, and these lower moduli result in higher deflections. So the ISM can disproportionately serve as an indicator of the A/C stiffness rather than the pavement stiffness. Existing temperature corrections apply only to conventional noninsulated pavements. A reasonable correlation between deflection basin area and resilient modulus is seen in Figure 26b.

Analogous curves for the noninsulated pavement in Nichols Road for winter–spring 1987–1988 are shown in Figure 27. Of the three relationships shown, the area ratio most clearly defines the periods of thaw weakening and recovery for the noninsulated pavement. The correlation between the deflection basin area and approximated resilient modulus for the noninsulated pavement is quite high.

While conventional pavements typically peak and recover with time, as shown in Figure 26b, the deflection basin area of the insulated pavement remains larger than the conventional pavement, yet relatively constant



a. Deflection basin area ratio, resilient modulus ratio, and ISM ratio.

b. Correlation between deflection basin area and resilient modulus.

Figure 27. Deflection basin area, Nichols Road, spring 1988.

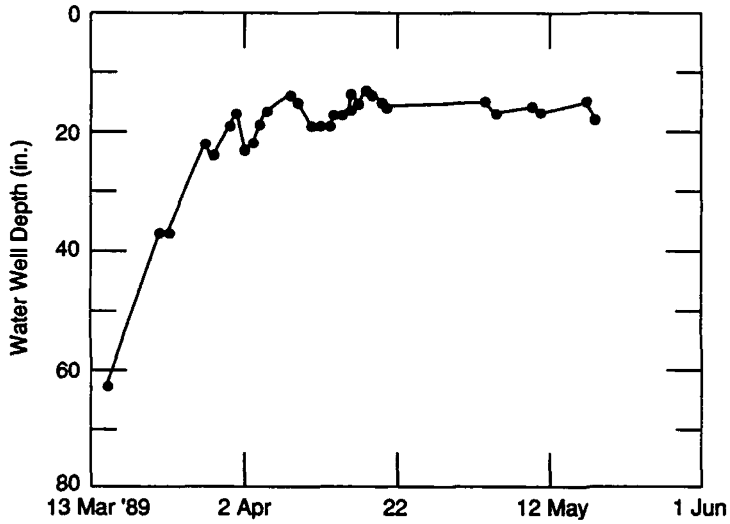


Figure 28. Water well depth at Newton Field, spring 1989.

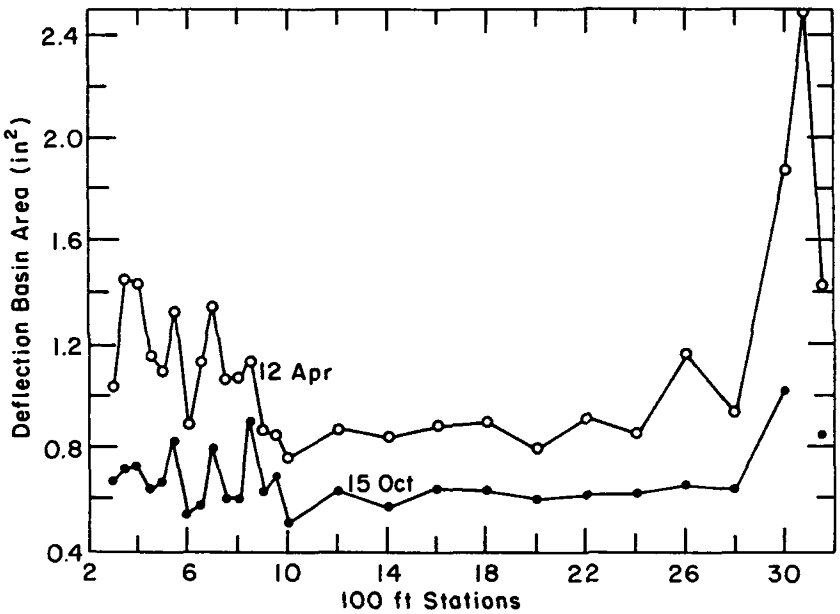


Figure 29. Deflection basin areas along the entire length of Newton Field, spring 1988.

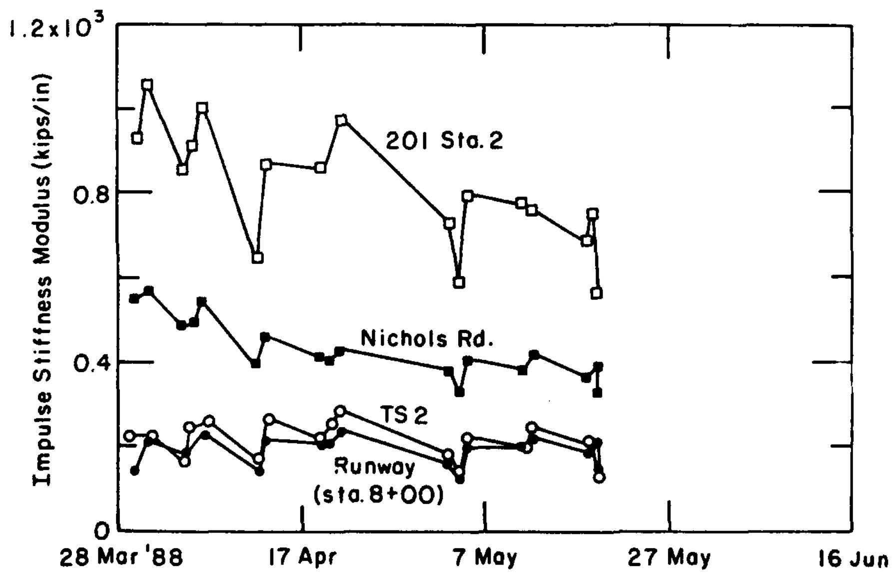


Figure 30. Impulse stiffness modulus at each of the FWD test sites, spring 1988.

through the spring and summer, with fluctuations caused primarily by changes in pavement temperature. This is probably due to a combination of reasons:

1. The insulation prevents frost from penetrating into the subgrade, so the subgrade never undergoes thaw weakening.
2. The high groundwater table (Fig. 28) causes the subgrade to remain quite weak throughout the year.
3. The insulating layer has a much lower strength than the other layers.

The localized peaks in basin area occurred on days when the pavement temperature was high. Since insulated pavements can attain higher temperatures than noninsulated pavements, the reduced A/C moduli are reflected in larger deflection basin areas. No temperature corrections were applied to the deflections.

Deflection basin areas along the entire length of the runway are shown in Figure 29 for winter–spring 1987–1988. The largest deflection basin areas coincide with locations that experienced excessive frost heave and areas where the subbase was generally around 6 in. thick.

Figure 30 shows the ISM at each of the FWD test sites and also on U.S. Route 201. U.S. Route 201 is a conventional, noninsulated pavement, but the A/C and the base course layers are thicker than at Nichols Road. Although the pavement strength of the insulated pavements is less than that of the noninsulated pavements, the insulated pavements did not exhibit significant loss of strength during spring thaw.

THERMAL DESIGN OF INSULATION

For a design air freezing index of 2570°F days, the Departments of the Army and Air Force (1985) require an insulation thickness of approximately 3 in. Both the general rule-of-thumb of $^1/_2$-in. of insulation for each 500°F days of design freezing index and Ontario's old approximation of 25 mm for each 555°C days result in $2^1/_2$ in. Ontario's updated design chart, developed following an extensive study at Val Gagne Experimental Site in Ontario (MacMaster and Wrong 1986), also yields an insulation thickness of approximately $2^1/_2$ in. Each of these methods ensures designs that prevent frost penetration into the subgrade. During the three winters of observation at the test sections, the 32°F isotherm penetrated through the bottom of the 2-in.-thick insulation in test section 1 but not into the subgrade. From these data, it appears that the test section field results conformed well to design thicknesses. During a design winter, frost is expected to penetrate the 2-in. insulation, but probably not the 3-in. insulation. Although winters in Jackman during the observation period ranged from average to colder than average, the design freezing index was never attained.

Table 6. Locations where air and pavement surface temperatures were monitored, winter 1990–1991.

Location	*Type of pavement*	*Condition of insulation*
Nichols Road	Noninsulated	—
Test sections	Insulated	Intact
Newton Field		
Station 4+50	Insulated	Assumed to be damaged
Station 18+00	Insulated	Assumed to be intact

To minimize surface icing, insulation is typically required to be at a minimum depth of 18 in. beneath the pavement surface. Both the questionable integrity of the insulation panels beneath the runway and the apparent range in depth to the insulating layer could promote differential icing under certain environmental conditions. Since visual observations have been limited to periodic visits of CRREL personnel to the site, temperature sensors were installed in 1990 at a variety of locations to monitor both air and pavement surface temperatures during winter 1990–1991 (Table 6).

CONCLUSIONS

1. During the four winters of observation, frost heave at the insulated test sections was comparable to that of the conventional, noninsulated pavement at Nichols Road; however, frost heave in localized areas of the insulated runway appreciably exceeded that in Nichols Road.

2. Although the design air freezing index of 2570°F days was never attained during the four winters of observation, the winter of 1986–1987 was an average winter, and the following three winters have been progressively colder. Since the 32°F isotherm penetrated through the 2-in.-thick insulation at the test sections, but never deeper than 5 in. into the subbase, it appears that, as long as the continuity and integrity of the insulation are maintained, field results conform well to design thickness (Kestler and Berg 1989).

3. Evidence of insulation irregularities or discontinuities exists at both ends of the runway:

- Damaged and overlapped insulation was uncovered and replaced when a section of the pavement at station 30+00 was removed in July 1987.
- Data from the thermocouple assemblies at station 4+50 show a distinct temperature discontinuity immediately beneath the insulation.
- Each winter, both ends of the runway exhibited substantial frost heave with localized areas of substantial differential frost heave.
- Ground-penetrating radar results showed an extremely irregular insulation surface with an apparent range in depth to the insulation/subbase interface of approximately 5 to 24 in. Depths to the top

of the insulation immediately adjacent to the edge of the runway at 25 (hand-excavated) random locations ranged from 6 to 16 in. One of the two larger hand-excavated trenches alongside the runway revealed a $2^1/_2$-in. gap between insulation panels.

- Based upon results of FWD tests, both ends of the runway exhibited considerably larger deflection basin areas than the middle of the runway.

4. Although pavement stiffness of the insulated pavement was less than that of the conventional noninsulated pavement, the insulated pavement did not exhibit any significant loss of stiffness during spring thaw.

5. For an insulated pavement, the modulus of asphalt concrete, and therefore the center deflection, are particularly sensitive to the pavement temperature.

6. The insulated pavement can perform well, as was demonstrated by both the insulated test sections and areas along the runway with a thick granular subbase. Pavement performance does, however, appear to be quite dependent upon construction quality control and upon the presence of a stable working platform during construction as was demonstrated by both ends of the runway.

RECOMMENDATIONS

1. Field investigations confirm that gaps in excess of 2 in. have developed between the buried insulation panels in the shoulder area immediately adjacent to the asphalt pavement, indicating that similar discontinuities have probably developed beneath the asphalt pavement itself. Further studies are recommended to assess the thermal effects of panel spacing on both the thermal regime and frost heaving of insulated pavements.

2. Extreme pavement distress is exhibited by cracks up to 9 in. deep and by severe secondary cracking. It is recommended that both the pavement surface and the underlying insulation be monitored for horizontal displacement. From this it could be determined whether the extreme crack-related failures are due to differential movement caused by the extreme temperature fluctuations experienced by insulated pavements, movement of the asphalt concrete and frozen base, or shrinkage of the asphalt concrete due to accelerated aging.

3. A design procedure for insulated pavements using CRREL's FROST1 model should be developed.

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