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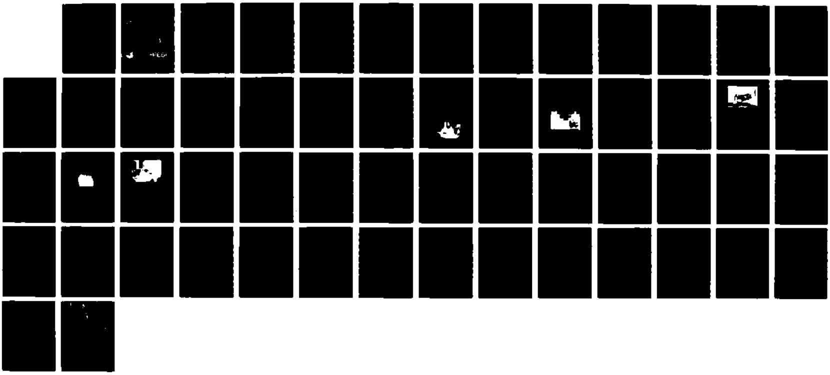

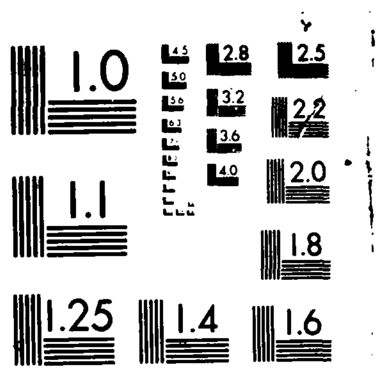

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(4)

OPTIMIZE FREEZE-THAW DESIGN OF ALRS

R.L. BERG

U.S. ARMY COLD REGION RESEARCH
AND ENGINEERING LABORATORY
72 LYME ROAD
HANOVER NH 03755

AUGUST 1987

FINAL REPORT

OCTOBER 1983 - NOVEMBER 1984

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This report documents research to develop concepts for structurally designing ALRS for use in areas that experience seasonal frost. Three test sections were designed, constructed, and traffic-tested to failure. One test section, constructed using about 3.3 inches of asphaltic concrete pavement, 10.5 inches of crushed stone base course, and 10.5 inches of crushed shoulder stone subbase above the clay subgrade, sustained about 10 percent more coverage with the F-15 loadcart than the minimum operational requirements. The other two test sections failed to meet the minimum operational requirements. The traffic tests were conducted after the test sections had undergone two freeze-thaw cycles. Frost penetrated into the clay subgrade about 15 inches during both freezing cycles.

A revised design procedure for ALRS pavements in cold regions is recommended. The revised procedure will cause thicker ALRS pavements than previously planned.

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EXECUTIVE SUMMARY

ALRS test sections were designed, constructed, and traffic-tested to failure. The test sections were designed using the reduced subgrade strength method of conventional airfield pavements, but Frost Area Soil Support Indexes (FASSI) lower than those normally used were assumed for the lean clay subgrade during thaw-weakened conditions. This assumption resulted in increased total thickness of pavement, base course, and subbase course above the subgrade. Results from simulated aircraft traffic indicate that ALRS pavements designed according to procedures presently used to design conventional airfield pavements have a high probability of premature failure if the design traffic is applied during thaw-weakened conditions. One test section, which was constructed using 3 inches of asphaltic concrete pavement, 10.5 inches of crushed stone base course, and 10.5 inches of crushed shoulder stone subbase above the subgrade, failed due to excessive rutting after 166 passes with the F-15 loadcart. A second test section was constructed using 3 inches of asphaltic concrete pavement, 8 inches of crushed stone base course, and 8 inches of crushed shoulder stone subbase above the subgrade; it failed after 88 passes with the F-15 loadcart.

A revised design procedure for ALRS pavements in cold regions is recommended. The revised procedure causes thicker ALRS pavement systems in seasonal frost areas. The designer can choose a California Bearing Ratio (CBR), depending upon soil type and probability of premature failure, to establish the total thickness of the ALRS pavement. The author recommends that designers use a probability of failure of 5 percent to design ALRS pavements. Total pavement thickness over a clay subgrade would be 19.5 inches using this criterion, and the total pavement thickness over a silt subgrade would be 23.0 inches. For a probability of failure of less than 1 percent the minimum pavement thickness over a clay subgrade would be 24.5 inches and the minimum thickness over a silt subgrade would be 30.0 inches.

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PREFACE

This report was prepared by the US Army Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL), 72 Lyme Road, Hanover NH 03755. The work was funded by the Engineering and Services Laboratory of the Air Force Engineering and Services Center, Tyndall Air Force Base, Florida.

The performance period for this effort was October 1983 to November 1984. The HQ AFESC/RDCP project officer was Maj George E. Walrond. The CRREL principal investigator was Dr Richard L. Berg.

This report has been reviewed by the Public Affairs Office (PA) and is releasable to the National Technical Information Service (NTIS). At NTIS, it will be available to the general public, including foreign nationals.

This technical report has been reviewed and is approved for publication.

GEORGE E. WALROND, Maj, USAF
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SECTION I

INTRODUCTION

A. OBJECTIVE

The object of this research was to construct and evaluate Alternate Launch and Recovery Surface (ALRS) test sections subjected to subgrade thaw weakening. The evaluation would be conducted by subjecting the test sections to simulated aircraft traffic during a thaw-weakened period.

B. BACKGROUND

The U.S. Air Force needs Alternate Launch and Recovery Surfaces (ALRSs) at foreign bases. Modern fighter dependency on high-quality pavement surfaces has made primary airfield pavements (runways and taxiways) very attractive targets for enemy air attack. The desirability of attacking airfield pavements has been further enhanced by the widespread construction of hardened aircraft shelters that greatly reduce the vulnerability of the aircraft while on the ground. An enemy may neutralize allied air power by concentrating his initial attack effort on damaging and/or destroying the runways and taxiways, thus, grounding the aircraft. The time the airfield must be closed for minimum essential pavement repairs is crucial to the outcome of the early stages of the conflict.

To counter the threat of unusable runways and taxiways, ALRSs built and in place long before the conflict begins will provide a greater targeting problem, thereby increasing the probability that an undamaged minimum operating area will exist after an attack. The ALRSs will be used to support limited aircraft operations until the primary airfield pavements are ready to support sustained aircraft operations.

The ALRSs will be used only in contingency situations when the runways are destroyed, but they will be designed for a 20-year life. Airbases where ALRSs may be installed are located in areas where the design air-freezing indexes range from 300^{o}F-days to 1000^{o}F-days, where the average annual rainfall is 25-30 inches, and where the average annual snowfall is 14-36 inches (Reference 1). The ALRSs will be constructed at a specific number of bases and must meet the following criteria: (1) be relatively inexpensive to construct in comparison with permanent airfield pavements, (2) be easily maintained, (3) support the imposed loads at any time of the year, and (4) provide an adequate surface for the design aircraft (Reference 1).

A preliminary study to evaluate the design of alternate launch and recovery surfaces for environmental effects was completed by Bush et al. (Reference 1). A recommendation of that study was:

> "A comprehensive field and laboratory study should be conducted to determine the extent and duration of thaw-weakened conditions. Results from such a study would provide information about the length of the severely weakened pavement condition, i.e., a few days or a few weeks, and provide definitive estimates of the loss of a substantial strength."

A Development Test and Evaluation of a full-size ALRS (75 by 8000 feet) was to be conducted at Spangdahlem Air Base, Germany, in FY 1984 and FY 1985. This study was needed to estimate the freeze-thaw effects on the subgrade at Spangdahlem and to assure that the final ALRS design was as accurate as possible, to reduce construction costs to their lowest, yet provide an ALRS that would be functional for its design life.

C. SCOPE

The objective was attained by designing, constructing, testing, and evaluating three ALRS test sections. The test sections were designed and constructed at the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL), New Hampshire, using methods, procedures, and materials similar to those expected to be used in the full-size ALRS at Spangdahlem Air Base, Germany. Subgrade soils from Spangdahlem Air Base were provided by the Air Force Engineering and Services Center (AFESC), and laboratory frost-susceptibility and hydraulic property tests were conducted on these soils.

SECTION II

SPANGDAHLEM AIR BASE SUBGRADE SOILS

Three typical subgrade materials from Spangdahlem Air Base, Germany, were sent to CRREL by AFESC for examination. A laboratory frost-susceptibility test was conducted on one of the samples.

Grain-size distributions and Unified Soil Classification system symbols for the three materials are shown in Figure 1. Sample 1, which was classified as a low-plasticity clay, CL, was selected for the laboratory frost-susceptibility test. Results from two laboratory frost-susceptibility tests are shown in Table 1. The results exhibited an average rate of heave of 0.27 mm/day, resulting in a frost-susceptibility classification of "negligible."

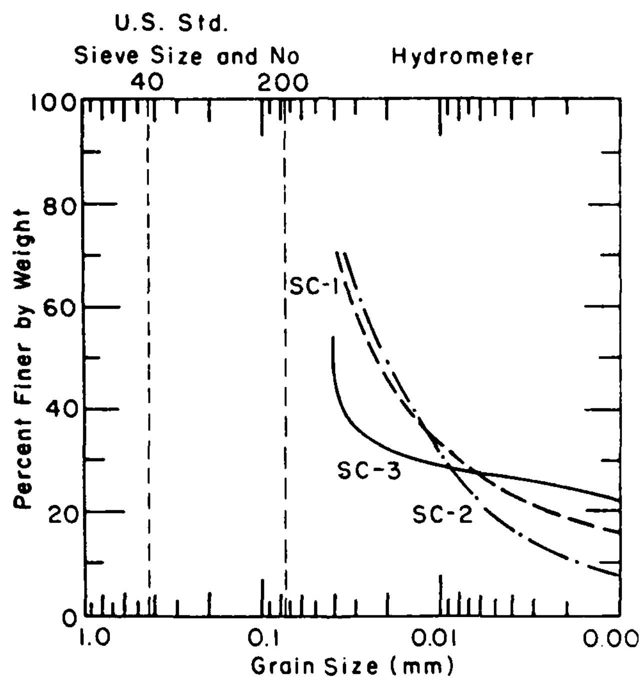

Figure 1. Grain-Size Distributions And Classifications of Three Subgrade Materials from Spangdahlem AFB, Germany.

TABLE 1. LABORATORY FROST-SUSCEPTIBILITY TESTS RESULTS.

Material	Dry Unit weight (lb/ft^3)	Average water content before test (% dry wt.)	Rate of heave (mm/day)	Frost susceptibility classification
Spangdahlem clay	117.4	13.8	0.19	Negligible
Spangdahlem clay	119.6	13.0	0.36	Negligible
Gonic A clay	119.7	14.0	1.25	Low
Gonic A clay	121.1	13.7	0.83	Very low
Sand subbase	124.5	10.4	0.02	Negligible
Sand subbase	124.4	11.5	0.06	Negligible
Crushed shoulder stone	--	6.3	1.52	Low
Crushed shoulder stone	--	6.5	1.23	Low

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SECTION III

TEST SECTIONS

A. DESIGN

Three test sections were designed according to methods and criteria presented by the Departments of the Navy, Army, and Air Force (References 2, 3, and 4). The reduced subgrade strength method was used to compute the total thickness of pavement, base, and subbase necessary above the subgrade. The subgrade Frost Area Soil Support Index (FASSI) from Reference 3 was used to obtain the minimum required thickness. The test sections were designed to withstand at least 150 passes of an F-15 aircraft with a gross load of 68,000 pounds and a tire pressure of 355 lb/in^2. Two of the test sections used unbound layers of crushed rock as the base course and subbase course. The third test section used a cement-stabilized sand base course and an unbound sand subbase above the subgrade. Layer thicknesses for the test section with cement-stabilized sand were obtained using equivalency factors presented in Reference 2.

An asphalt concrete pavement was placed on the base course of all three test sections. The pavement served as the wearing surface during the traffic tests. The design criteria (Reference 2) required at least 3 inches of asphalt surfacing, depending on the strength (CBR) of the base course. A 3-inch-thick pavement was used in the design of these tests.

To approximate expected seasonal variations in strength of the pavement and to simulate the performance of the ALRS at Spangdahlem Air Base, a low-plasticity clay, CL, was located and used as the subgrade soil. A minimum of 34 inches of the clay subgrade was placed over the natural silt, ML, subgrade in the test area. Gonic A clay was selected after searching for about 2 weeks. The pit containing the Gonic A clay was located approximately 75 miles from CRREL. The in situ material occurred in natural deposits at an average moisture content slightly above the optimum value for the CE-12 compactive effort.

The grain-size distribution and Atterberg limits of the Gonic A clay subgrade are shown in Figure 2; Figures 3 and 4 contain compaction and CBR data for the material, respectively. Laboratory frost-susceptibility test data are shown in Table 1; the soil was classified as an F3 material using grain-size and Atterberg limit test results. Since the average rate of frost heave from the standard laboratory frost-susceptibility test was only 1.04 mm/day, the material was classified to possess a "very low to low" potential for frost heave.

Two subbase materials were used. Grain-size distributions of the two materials are shown in Figure 5 and 6. Figures 7 and 8 contain compaction and laboratory CBR data for the sand, and Figures 9 and 10 contain similar data for the crushed shoulder stone. The sand was classified as an SW-SM under the unified Soil Classification System and in Frost Group S2 in the frost design soil classification system. Results from the laboratory frost-susceptibility tests on the sand (Table 1) indicated an average frost heave rate of 0.04 mm/day, so the sand had a negligible potential for frost heave. The crushed shoulder stone subbase of crushed granite; was classified as GW in the unified soil classification system. The frost design soil classification is S1. The laboratory frost-susceptibility test (Table 1) indicated an average frost heave rate of 1.38 mm/day, so it had a low potential for frost heave.

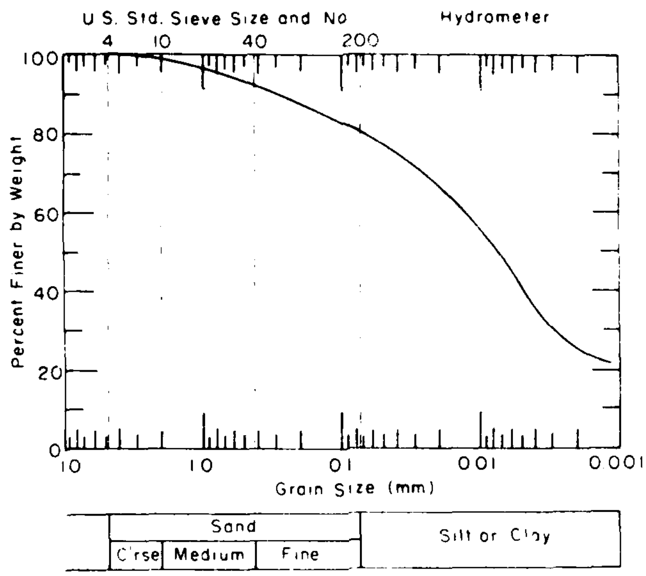


Figure 2. Grain-Size Distribution and Classification of Clay Subgrade.

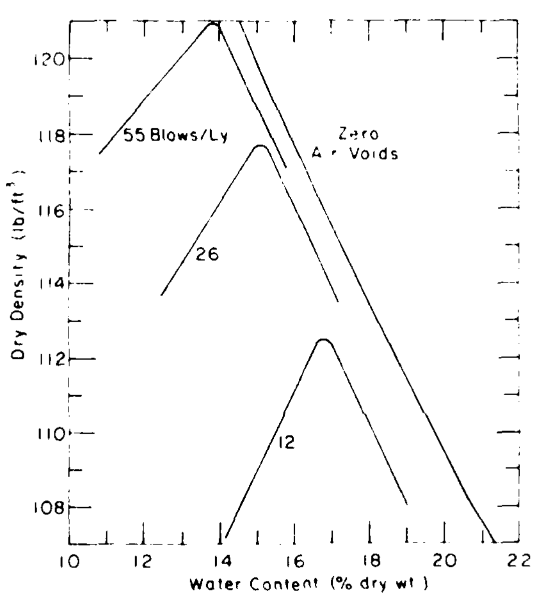


Figure 3. Laboratory Compaction Data for the Clay Subgrade.

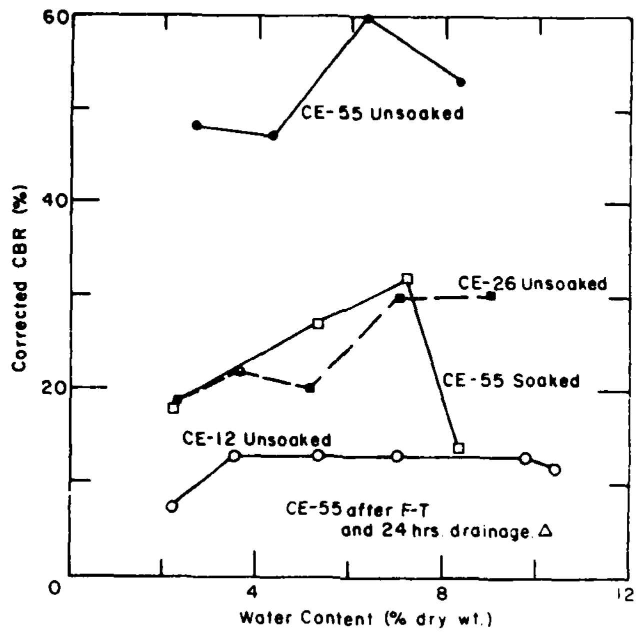


Figure 4. Laboratory CBR Data for the Clay Subgrade.

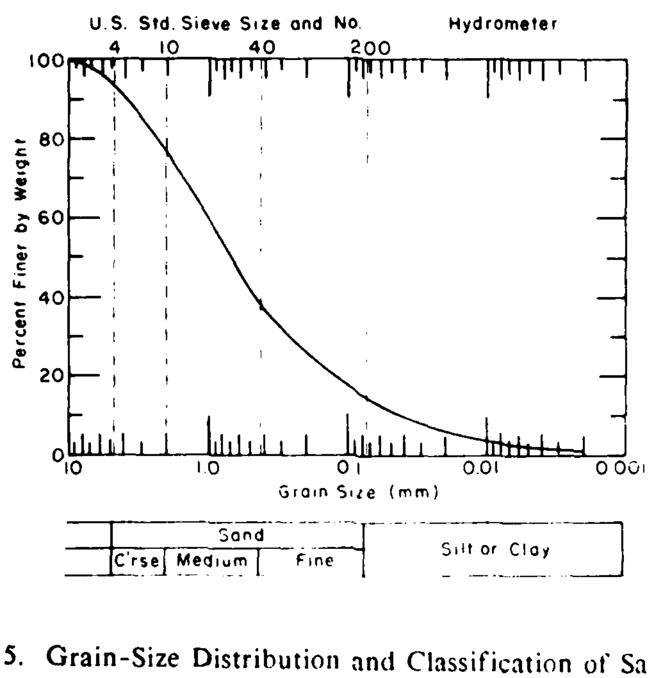


Figure 5. Grain-Size Distribution and Classification of Sand Subbase.

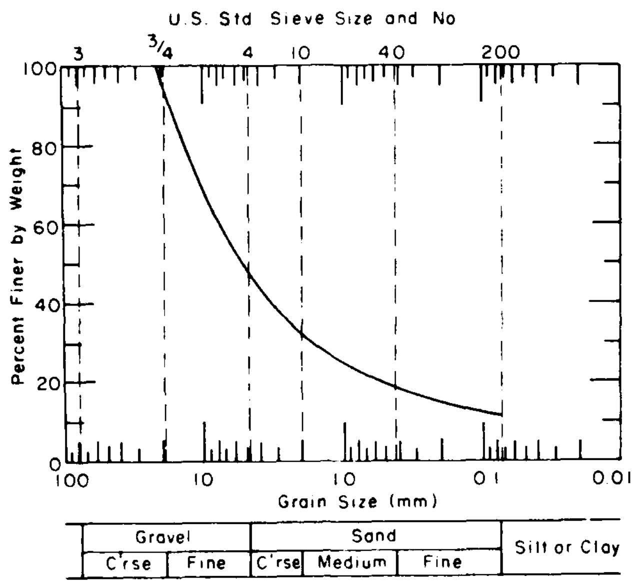


Figure 6. Grain-Size Distribution and Classification of the Crushed Shoulder Stone Subbase.

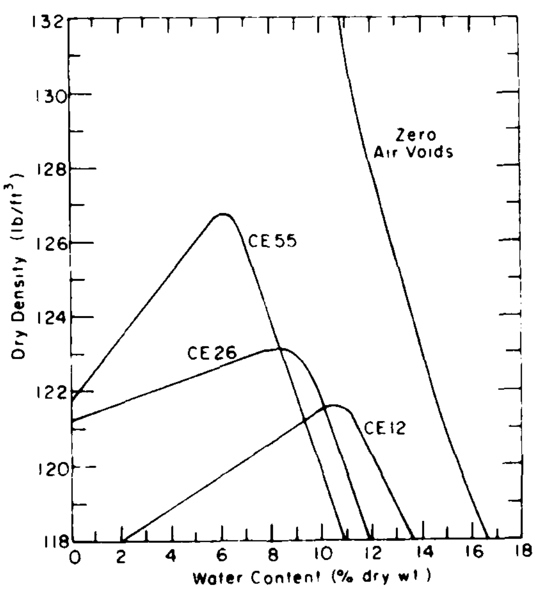


Figure 7. Laboratory Compaction Data for the Sand Subbase.

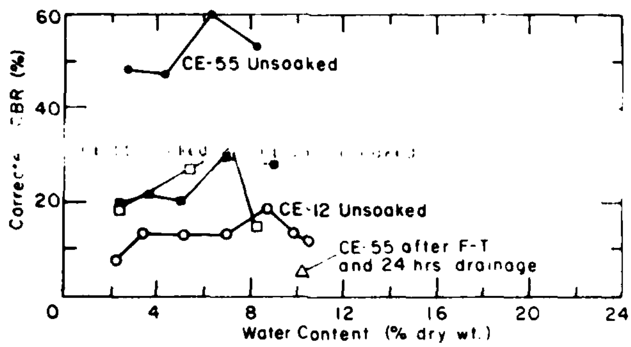


Figure 8. Laboratory CBR Data for the Sand Subbase.

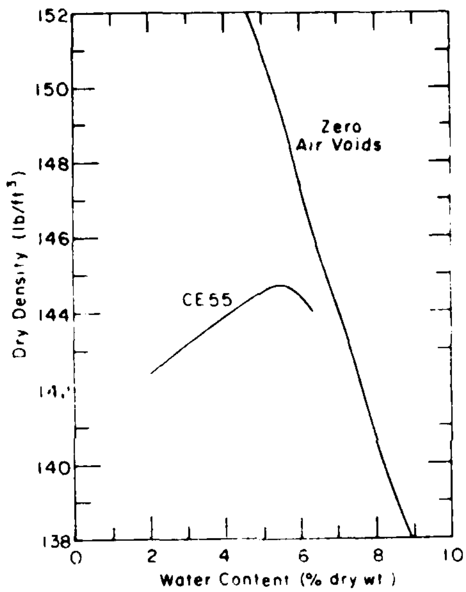


Figure 9. Laboratory Compaction Data for the Crushed Shoulder Stone Subbase.

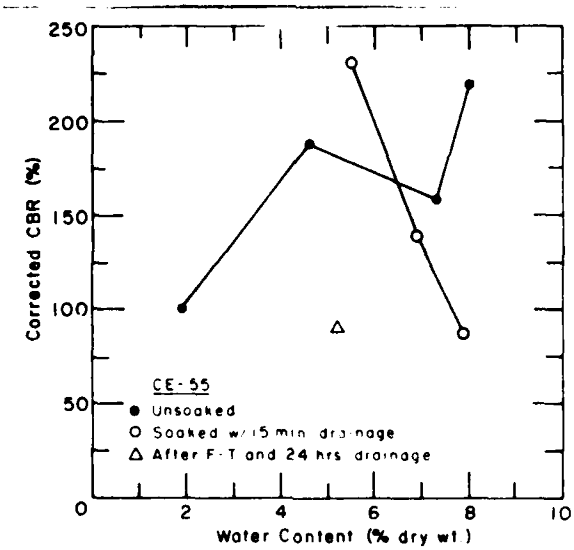


Figure 10. Laboratory CBR Data for the Crushed Shoulder Stone Subbase.

The base course material was crushed granite for two of the test sections and cement-stabilized sand for the third. The grain-size distribution and classification of the crushed granite are shown in Figure 11; laboratory compaction and CBR data for the material are shown in Table 2. Due to the extremely small percentage of particles finer than 0.02 mm, the crushed granite base course material was classified as non-frost-susceptible (NFS) and laboratory frost-susceptibility tests were not conducted on the material. The cement-stabilized layer used the same sand that was used as a subbase material (Figures 5 through 8). Procedures outlined in Reference 4 were used to determine the optimum cement content -- 8 percent by dry weight -- for the sand. Type I Portland cement was purchased in bags and spread over the surface of the sand by hand. Several passes of a tractor-mounted rototiller were used to mix the cement with the sand. After the tests were completed it was determined that the cement was not mixed to the design depth using this procedure and equipment.

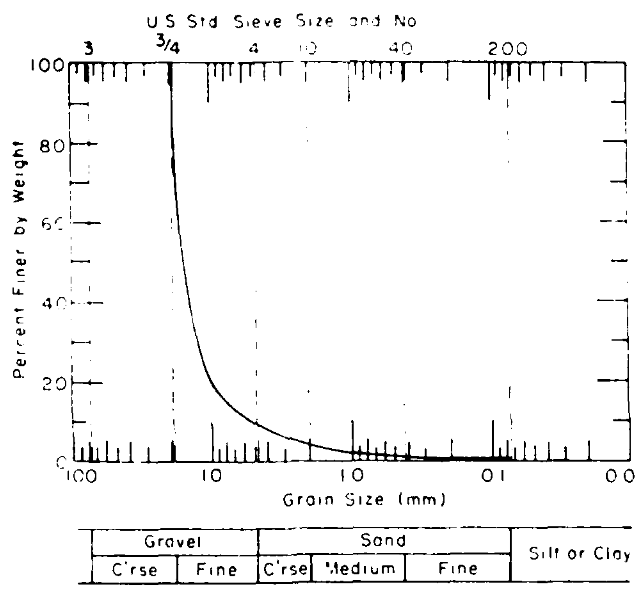


Figure 11. Grain-Size Distribution and Classification of the Crushed Granite Base Course.

TABLE 2. LABORATORY COMPACTION AND CBR DATA FOR CRUSHED GRANITE BASE COURSE.

Condition[a]	Dry unit weight (lb/ft^3)	Water content (% dry wt.)	CBR
Dry	122.3	0.1	98
Surface wet	124.8	1.3	67

[a] All samples were compacted using the CE-55 effort and procedure.

The asphalt concrete pavement was placed in a single 3-inch-thick layer using a standard State of New Hampshire mix for the wearing course of highway pavement. The maximum aggregate size was 1/2 inch and the asphalt content was approximately 6.1 percent. The bitumen was classified as an AC-10. The State of New Hampshire's specifications and the gradations furnished by the paving contractor are shown in Table 3. The coarse and fine aggregates were both crushed granitic rock from the same source as the crushed rock base and subbase courses.

TABLE 3. JOB MIX FOR THE BITUMINOUS CONCRETE WEARING SURFACE.

U.S. standard sieve size	Percent Passing: Aggregates Course	Fine	Sand	Specified limits	Job mix formula
3/4 in.	100	-	-	100	100
1/2 in.	97	-	-	95-100	97
3/8 in.	38	100	100	83-97	94
No. 4	5	38	95	58-72	63
No. 10	-	4	84	44-52	48
No. 20	-	-	63	28-36	32
No. 40	-	-	36	15-23	18
No. 80	-	-	15	4-12	9
No. 200	-	-	4	1-15	2
	Percent to obtain job mix				
	10	40	50		
Asphalt				5.7-6.5	6.1
	Grade of bitumen				AC-10

Reference 2 requires that the asphaltic concrete pavement meet additional criteria. These requirements and results from core samples obtained from the pavement after traffic tests are shown in Table 4.

TABLE 4. PROPERTIES OF ASPHALT CONCRETE PAVEMENT AND ASPHALT CEMENT.

Test	Specified limits	Test Section 1[a] Traffic	1[a] No traffic	2[a] Traffic	2[a] No traffic	3 Traffic	3 No traffic	Original asphalt
Marshall stability (lb)	> 1800	1821	2277	2097	1506	1186	941	-
Flow, 0.01 in.	≤ 16	21	22	23	22	22	24	-
Percent voids in total mix	3-5	4.7	3.3	2.7	4.5	6.9	8.3	-
Percent voids filled	70-80	76.9	82.3	83.7	75.2	63.3	63.5	-
Thickness (in.)	3	3.9	3.8	3.4	3.3	3.1	3.1	-
Density (lb/ft³)	--	156.0	157.9	159.1	156.1	154.6	151.7	-
Asphalt (%)	5.7-6.5	6.2		5.5		6.1		-
Penetration, 0.1 mm	> 70	49		84		58		85
Viscosity at 140°F, poise	800-1200	[illegible]		[illegible]		3075		1411
Viscosity at 275°F, centistokes	> 150	[illegible]		379		462		342
Pen-Vis No.	--	-0.6		-0.5		-0.5		-

[a] Asphalt recycled through Porta-Patcher*.
All data shown are an average of 3 or 4 samples.

Instrumentation in each of the test sections included: (a) four thermocouple psychrometers for indirectly monitoring changes in moisture content in the subgrade during freezing and thawing; (b) four tensiometers for monitoring changes in pore pressure as well as indirectly monitoring changes in moisture content in the subgrade during freezing and thawing; (c) two thermocouple assemblies, each with sensors placed at 14 locations between the pavement surface and the bottom of the clay subgrade; (d) two electrical resistivity gages in the subgrade to monitor the position of the freezing and/or thawing fronts; and, (e) two sets of seven 2-inch diameter Bison gages to measure deflections due to frost heave and caused by loading at the surface. Layouts of the test sections and vertical locations of the instrumentation are shown in Appendix A.

B. CONSTRUCTION

Removal of the previous test surfaces and construction of the three test sections for this study occurred between 19 October 1983 and 17 November 1983. Layer thickness and properties of the layers are shown in Tables 5 through 7. In November and December 1983, two sets of tests were conducted on Test Sections 1 and 2 with CRREL's Falling Weight Deflectometer (FWD) (Figure 12). Deformations observed due to the applied loads, approximately 9000 pounds, indicated that the test sections probably would not sustain the desired number of passes with an F-4 aircraft loading. This conclusion was based on a relationship between passes to failure and plate deflection presented in Reference 1. Since loads imposed by the F-15 would be more severe than those of the F-4 aircraft, Test Sections 1 and 2 were constructed.

TABLE 5. MATERIALS AND LAYER THICKNESS (OCT-NOV 1983 CONSTRUCTION).

Material	Test Section 1	Test Section 2	Test Section 3
Pavement	3 in. AC	3 in. AC	3 in. AC
Base	6 in. CS	8.5 in. CS	6.4 in. CSB
Subbase	6 in. S	8.5 in. S	4 in. S
Filter	- F	- F	- F
Subgrade	39 in. C	34 in. C	40.6 in. C

AC - Asphalt concrete
CS - Crushed stone (max. size 1-1/2 in.)
CSB - Cement-stabilized base
S - Sand
F - Geotechnical fabric
C - Clay

Reconstruction began on 19 January 1984 and was completed on 13 February 1984. Revised design thicknesses were developed after consulting with A. J. Bush and others at WES as well as the Program Manager at AFESC. Instrumentation and materials were [illegible]

[illegible] reconstructed test sections (Table 6). Gradations, compaction data and laboratory CBR values for the materials are shown in Figures 5, 6, 9, and 10 and Table 2.

The following discussion applies only to the final three test sections; no further discussion of the original base and subbase courses on Test Sections 1 and 2 will be presented.

TABLE 6. MATERIALS AND LAYER THICKNESS (AFTER RECONSTRUCTION).

Material	Test Section 1	Test Section 2[a]
Pavement	3 in. AC	3 in. AC
Base Course	8 in. CS	10.5 in. CS
Subbase	8 in. SS	10.5 in. SS
Silter	- F	- F
Subgrade	39 in. C	34 in. C

[a] Test Section 3 was not reconstructed.
AC - Asphalt concrete
CS - Crushed stone (max. size 3/4 in.)
SS - Crushed shoulder stone
F - Geotechnical fabric
C - Clay

1. Subgrade and Geotechnical Filter

When excavation and removal of the previous test sections were completed, the in situ silt subgrade was smoothed by hand and a geotechnical fabric was placed over it. A 6-inch-thick layer of 3/4-inch (maximum size) crushed stone was placed to serve as a water distribution medium if it was decided to provide a high water table during freezing of the test sections. Another layer of geotechnical fabric was placed over the crushed stone to keep clay from being forced into the voids in the crushed stone during compaction and traffic-testing of the test sections. A layer of clay approximately 8 inches thick was placed on the geotextile. A small bulldozer was used to spread the clay. A 15-ton vibratory steel-wheeled roller was used to compact this and each subsequent clay layer to approximately 6 inches thick. The roller vibration caused water to move toward the surface of the layer being compacted, so vibrations were discontinued after two passes on the first layer of clay subgrade.

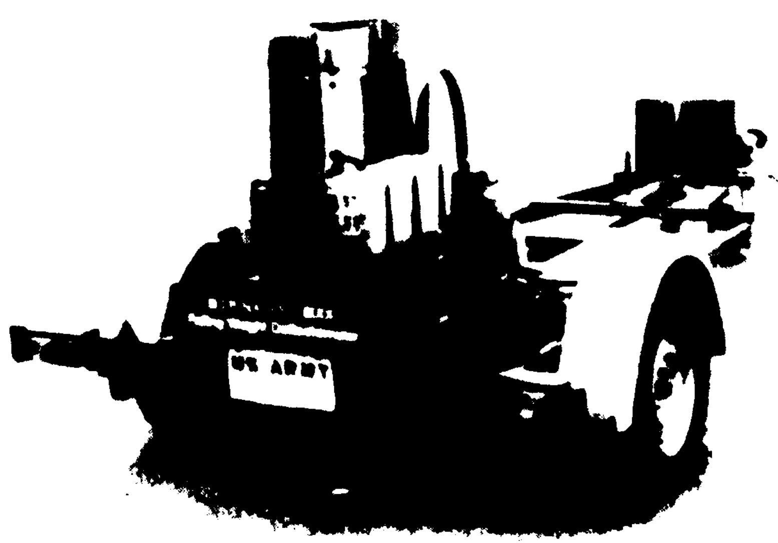


Figure 12. CRREL Falling Weight Deflectometer (FWD).

Table 7 contains the CBR values, water contents, and densities of the clay subgrade during construction. The optimum moisture content, optimum density, and CBR at optimum conditions for the CE-12 compaction effort were 16.8 percent, 112.5 lb/ft^3, and 10 percent (Figures 3 and 4), respectively. A geotechnical fabric was placed at the top of the clay subgrade to act as a filter. The fabric was used instead of a sand filter, which could also have been used. A fabric or sand filter is required to prevent contamination of the subbase course by the subgrade (Reference 3). As required by the reference, no structural advantage was allowed for the geotechnical fabric.

TABLE 7. SUMMARY OF TESTS TAKEN ON THE CLAY SUBGRADE AND SAND SUBBASE DURING CONSTRUCTION.

Material	Approx. depth below pavement surface (in.)	Layer	CBR[a] (%)	Moisture content (% dry wt.)	Dry density[b] (lb/ft^3)
		Test Section 1			
Clay subgrade	51	1	6.9	16.5	106.3
	43	2	6.0	16.8	107.8
	35	3	4.1	19.4	-
	27	4	9.4	-	103.5
	19	5	6.3	-	-
	19	5	7.3	15.6	101.9
	19	5	7.2	-	-
		Average	6.7	17.1	104.9
		Test Section 2			
Clay subgrade	52	1	10.0	15.8	105.5
	45	2	8.6	17.2	108.5
	38	3	6.8	18.3	105.7
	31	4	4.6	18.5	106.0
	31	4	4.4	-	-
	24	5	4.8	-	-
	24	5	4.5	17.0	102.6
	24	5	4.5	-	-
		Average	6.0	17.4	105.7
		Test Section 3			
Clay subgrade	46	1	4.8	17.1	98.7
	38	2	15.6	20.1	99.5
	30	3	9.2	18.2	101.3
	22	4	5.7	18.9	102.5
	22	4	6.4	-	-
	14	5	2.6	-	-
	14	5	6.6	16.6	105.2
	14	5	7.0	-	-
		Average	7.1	18.2	101.4
Sand subbase		2	-	6.3	123.7

[a] Average of 2 to 5 samples in each layer.

[b] Sand cone procedure used.

2. Subbase and Base Courses

Properties of the sand subbase placed in Test Section 3 are shown in Figures 5 and 7. It was classified as an SW-SM under the Unified Soil Classification System (USCS) and as an S2 material under the Frost Design Soil Classification System. The as-placed average moisture content and density of the material were 6.3 percent and 123.7 lb/ft^3 (Table 7), respectively. No field CBRs were conducted on the sand during construction. The sand was placed and compacted with the same equipment that had been used to place the subgrade. The vibrator on the roller was off during compaction so that water would not be drawn from the subgrade.

The subbase material placed in Test Sections 1 and 2 was crushed shoulder stone classified as GW-GM, using the USCS, and as an S1, using the Frost Design Classification System; it was manufactured by crushing blasted granite. Results from laboratory frost-susceptibility tests are shown in Table 1, and the average grain-size distribution curve is shown in Figure 6. Laboratory compaction test results and results from laboratory CBR tests are shown in Figure 9 and 10. The shoulder stone subbase was placed and compacted in two layers, each about one-half the thickness of the final course thickness. A hand-operated, dual-drum vibratory roller (Figure 13) was used to compact each layer. The roller consisted of two steel drums with a water reservoir for each. It was self-propelled and weighed about 2000 pounds with the water tanks full. This roller was much lighter than a roller that would be used on an actual construction project; however, because relatively thin layers (6 inches or less) were placed, compaction approximated what would have been attained with larger equipment.

Figure 13. Dual-Drum Vibratory Roller Used to Compact Base and Subbase During Reconstruction.

Test Section 3, containing the cement-stabilized sand layer, was designed to withstand the same volume of traffic as the original Test Section 1 (Table 5). Equivalency factors of 1.15 for an unstabilized base course and 2.30 for the unstabilized sand subbase were used with the thickness design procedure outlined for stabilized soil layers in Reference 2. The optimum cement content was determined using procedures outlined by the Portland Cement Association (Reference 5), which is similar to the Department of the Army procedure (Reference 4). The optimum cement content was determined to be 8 percent by dry weight of soil. The optimum water content (8.8 percent) and maximum dry density (130.4 lb/ft^3) of the cement-sand-water mixture are shown in Figure 14. Results from laboratory freeze-thaw durability tests are shown in Table 8 as are results from unconfined compressive strength tests conducted on samples removed after the traffic tests. Some of the laboratory durability tests were conducted on specimens containing much higher than optimum cement contents. Tests using these high cement contents were conducted because the cement-stabilized layer was found to be only about one-half of the design thickness when the test pit was installed after the traffic tests. The total amount of cement used, however, was sufficient to have stabilized the design thickness.

A tractor-mounted rototiller was used to mix the cement into the sand. Bags of cement were opened by hand and raked onto the surface of the sand; the rototiller then made several passes to blend the cement into the sand. After the cement was mixed with the sand, the 15-ton truck was used to compact the mixture.

The base course in Test Sections 1 and 2 was 3/4-inch maximum size crushed rock. The grain-size distribution of the material is shown in Figure 11, and Table 2 contains the laboratory compaction and CBR data for the material. It was placed and compacted in two layers, each layer being approximately one-half the final thickness of the base course.

The 2000-pound dual-drum roller was used to compact each layer of the base course. Because the material contained very few fines and was very coarse, representative CBR tests or field density measurements with the sand cone could not be obtained.

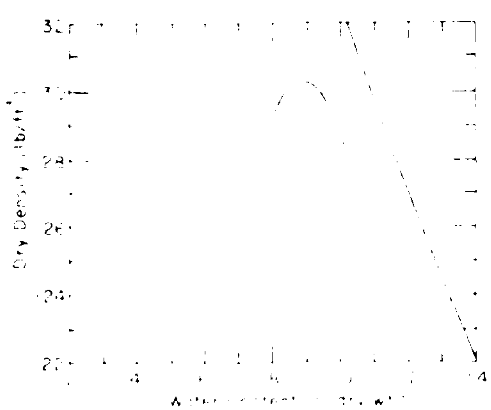


Figure 14. Compaction Test Results on the Cement-Stabilized Sand Subbase

TABLE 8. RESULTS OF TESTS ON THE CEMENT-STABILZED SAND.

Freeze-Thaw Durability Tests - 12 cycles

Dry unit weight (lb/ft^3)	Moisture content (%)	Cement content (%)	Soil-cement loss (%)
129.2	6.1	7.0	25.8
130.3	6.2	11.0	3.2
128.9	5.5	15.0	3.8

14-Day Flexural and Compressive Strengths

	Moisture content (%)	Cement content (%)	Flexural strength ($lb/in.^2$)	Compressive strength ($lb/in.^2$)
3-1/2 in. x 4-3/4 in. x 16 in. beams:				
	7.9	7.0	14.8	301
				277
	6.9	11.0	15.3	258
				239
	6.8	15.0	29.7	375
				385
14-Day Compressive Strengths				
3 in. diam. x 6 in. high cylinders:				
	6.9	11.0	--	190
				236
	6.8	15.0	--	93
				94

3. Pavement

The central 12-foot-wide portion of the initial asphalt concrete pavement was placed with a paver. The outside 3 feet on each side of the mat was placed by hand. Compaction was obtained by rolling with an 8-ton steel-wheeled roller. This pavement remained in place on Test Section 3. When Test Sections 1 and 2 were reconstructed, all of the local plants manufacturing asphalt concrete were closed for the winter, so the original pavement was recycled through a Porta-Patcher® (Figure 15) manufactured by Brown Equipment Co. Inc., which was developed for recycling asphaltic concrete to patch potholes in pavements. According to the manufacturer's literature, the recycling capacity of the unit is 300 lb/min (9 tn/hr). However, the average rate of recycling on this job was about 80 lb/min or about 2.5 tn/hr. The recycled pavement on Test Sections 1 and 2 was dumped from wheelbarrows and spread by hand. The recycled mix exited from the Porta-Patcher® at a temperature of from 275°F to 290°F. The temperature outside of the building was about -12°F and the heat inside the building was from the Porta-Patcher® and the cooling asphalt concrete mat. The inside temperature was generally slightly above freezing. The recycled mix was compacted using a hand-operated vibratory plate compactor with a plate size of 21 by 24 inches and an overall weight of about 170 pounds. Data in Table 4 indicate that the Marshall stability tests on the recycled material exceeded the minimum allowable value in areas covered by traffic. Densities of the recycled materials were also high, indicating that the material was high-quality pavement.

Figure 15. Porta-Patcher® Used to Recycle Asphalt Concrete Pavement During Reconstruction of Test Sections.

Table 4 contains properties of the asphalt concrete and the bitumen determined from core and chunk samples obtained after the traffic tests were completed. Properties of the recycled asphalt cement in Test Sections 1 and 2 did not vary appreciably from the unrecycled material in Test Section 3, indicating that the recycling process caused very little damage to the asphalt pavement. In fact, Marshall stabilities and densities of the recycled materials were significantly greater than those of the unrecycled material in Test Section 3.

C. FREEZING HISTORY

Our initial proposal included freezing the test sections individually over a period of about 6 months. This concept would have allowed use of the 5 ton refrigeration unit available at the test site. The major drawbacks to this concept were that the loadcart would need to be transported back and forth between CRREL and AFESC three times, and CRREL and AFESC project personnel and equipment would be involved for a long time.

Ultimately CRREL purchased additional freezing panels using Corps of Engineers funds. Funds from this project were used to rent a larger-capacity refrigeration unit. All of the equipment was delivered and assembled in early April 1984. The first freezing cycle started on 10 April 1984 and was completed on 15 May 1984. Our goals were to cause 18 inches of frost penetration into the subgrade beneath each of the test sections and to freeze the subgrade at the rate of 2 in./day. Table 9 shows the total frost penetration depth beneath the surface of the test sections and the amount of frost penetration into the subgrade in each test section. The frost depths in Test Sections 1 and 2 were slightly less than our goal. The primary reason for greater frost penetration into Test Section 3 was inadequate control of the volume of chilled coolant through the refrigeration panels. The average rate of frost penetration into the subgrade was 0.83 in./day for all three test sections. Test Section 3 had the highest rate, 0.92 in./day, and Test Section 1 had the lowest rate, 0.71 in./day.

TABLE 9. MAXIMUM FROST PENETRATION INTO THE TEST SECTIONS.

Test section	First Freeze Cycle: Total depth (in.)	First Freeze Cycle: Depth into subgrade (in.)	Second Freeze Cycle: Total depth (in.)	Second Freeze Cycle: Depth into subgrade (in.)
1	34	15	28	9
2	39	15	39	15
3	45	31	43	29

To simulate the desired rate of frost penetration into the subgrade more closely and to provide more uniform frost penetration beneath the three test sections, the rented refrigeration unit was used to freeze Test Sections 1 and 2 on the second freezing cycle and the small CRREL unit was used to freeze Test Section 3. Rates of frost penetration into the clay did increase slightly, but the rented refrigeration unit was damaged during a thunderstorm and the system was off for about 1 week (7-14 June) while parts were being obtained and installed. During this 1-week period the circulating fluid temperature on Test Section 3 was warmed to reduce the rate of frost penetration. Cooling of the deeper soils continued in Test Section 3, although the surface temperature was warmed. Therefore, when the temperatures were again lowered after the refrigeration unit was repaired, frost penetrated into Test Section 3 (Figure 18) more rapidly, and the total frost penetration when the refrigeration units were turned off on 1 July 1984 was again significantly deeper than our target depth. Frost penetrations beneath Test Sections 1 and 2 during the second freezing cycle were slightly less than the target depths (Figures 16 and 17).

Frost penetration rates and maximum frost penetration depths slightly different from the target conditions did not significantly impact the results from these tests. Natural frost penetration rates in actual field conditions generally range from 0.25 to 1.0 in./day. The faster rate was expected to reduce the amount of time for conducting the test. The maximum frost penetration into the subgrade was chosen primarily to allow time to conduct tests as thawing progressed after the freeze cycle. Whether the maximum frost penetration into the clay was a few inches less than the target or several inches more would have no major bearing on the results from these tests.

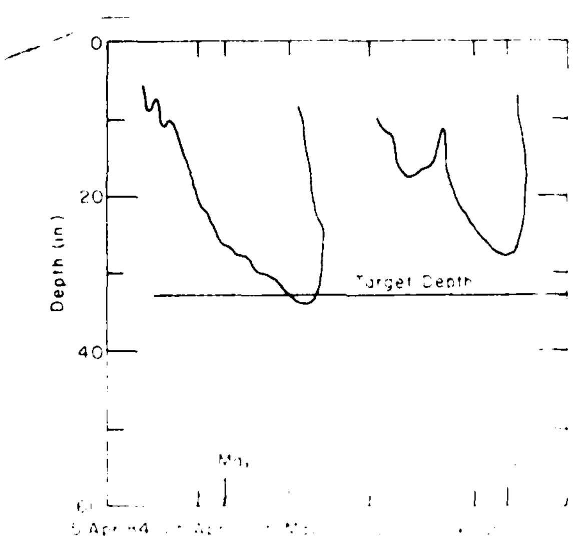


Figure 16. Frost and Thaw Penetration with Time in Test Section 1

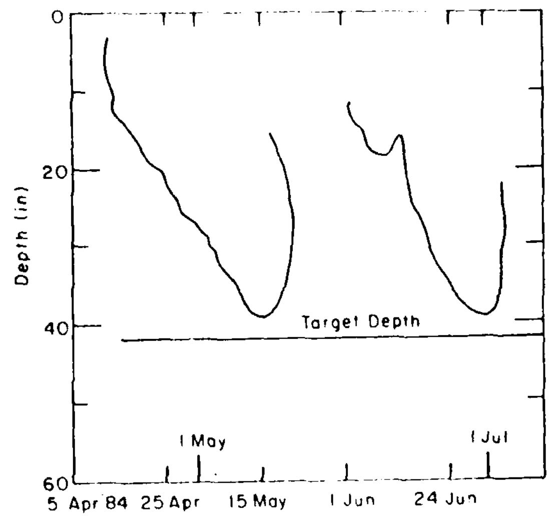


Figure 17. Frost and Thaw Penetration with Time in Test Section 2.

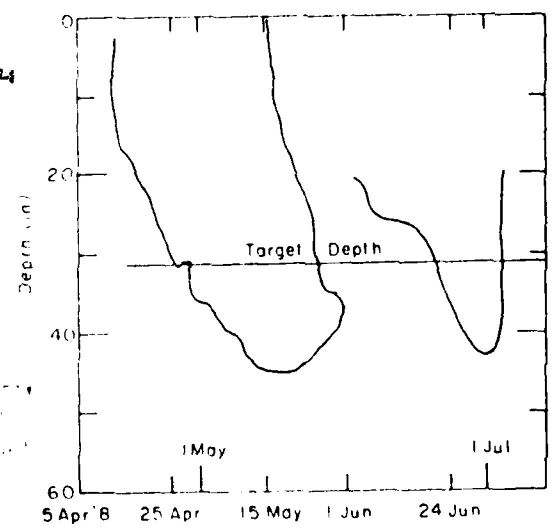


Figure 18. Frost and Thaw Penetration with Time in Test Section 3.

Elevations of the pavement surface of each section were measured before and after the first freeze cycle. The average amount of frost heave in each of the test sections was:

Test Section 1	0.03 feet = 0.36 inches
Test Section 2	0.01 feet = 0.12 inches
Test Section 3	0.06 feet = 0.72 inches

Surface elevations were not measured immediately after the second freeze cycle but, on visual examination, frost heave did not appear to be significantly greater than after the first cycle.

The subgrade of these test sections was a low-plasticity clay, so very little frost heave was expected due to the small amount of heave exhibited in the laboratory frost-susceptibility test and the low unsaturated hydraulic conductivity of the clay. While clays are freezing, water moves only short distances and frost heave caused by the formation of an ice lens in one location may be offset, or nearly so, by consolidation due to desiccation when the water moved to the ice lenses formed in the clay. Figure 19 shows a core of the subgrade removed from the untrafficked area of Test Section 2 during the test pit operation. The horizontal voids were created by the formation of ice lenses during freezing.

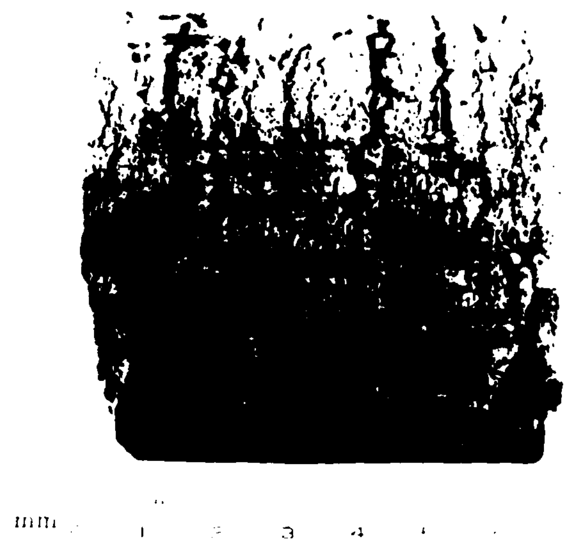


Figure 19. A Core of the Clay Subgrade from the Untrafficked Area of Test Section 2.

D. TRAFFIC TESTS

Traffic was applied to the test sections using an F-15 loadcart (Figure 20). The inflation pressure on the test tire was 355 lb/in.^{2}. The total weight of the loadcart and lead weights was 41,000 pounds with approximately 32,500 pounds of the load applied through the test tire. A normal channelized traffic distribution for the F-15 aircraft was used during the traffic tests. The traffic pattern is shown in Figure 21.

To fulfill the objectives of this study, it was necessary to apply traffic when the pavement was near or at its weakest condition, because the ALRS may be needed at any time of the year. Our experience with laboratory tests on low plasticity clay soils indicated that two freeze-thaw cycles of the material would alter its load-carrying capacity significantly, and additional freeze-thaw cycles would not reduce the capacity significantly more. We estimated that the greatest reduction in load-carrying capacity of the subgrade would occur at the time when most of the previously frozen layer was thawed. To establish the optimum time to apply the loads, tests were conducted using a falling-weight deflectometer (FWD) during the first thawing cycle.

Figure 20. F-15 Loadcart used to apply Moving Loads to the Test Sections.

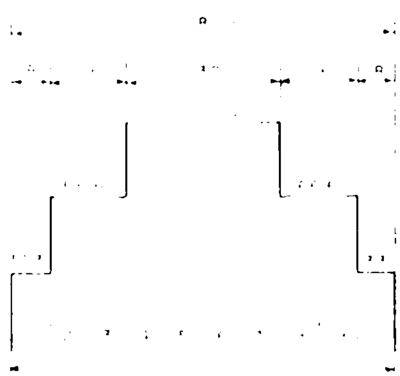


Figure 21. Traffic Distribution Pattern for the F-15 Loadcart.

The initial freezing cycle was completed on 15 May 1984 and the freezing panels were removed from the pavement on that day. Surface elevations were obtained on that date and tests were conducted with the FWD. Surface and subsurface temperatures were monitored daily during freezing and thawing of the test sections and, during thawing, tests were conducted daily with the FWD to determine pavement deflections.

The resilient stiffness, determined by dividing the applied load by the measured plate deflection for each test section, is plotted versus time in Figure 22. Resilient stiffnesses were used rather than measured plate deflections because identical loads were not applied each day. All three test sections lost strength rapidly as thawing progressed. Their weakest period was at the time when the last part of the frozen soil became thawed. Therefore, the traffic testing was scheduled to take place when nearly all of the frozen soil was thawed.

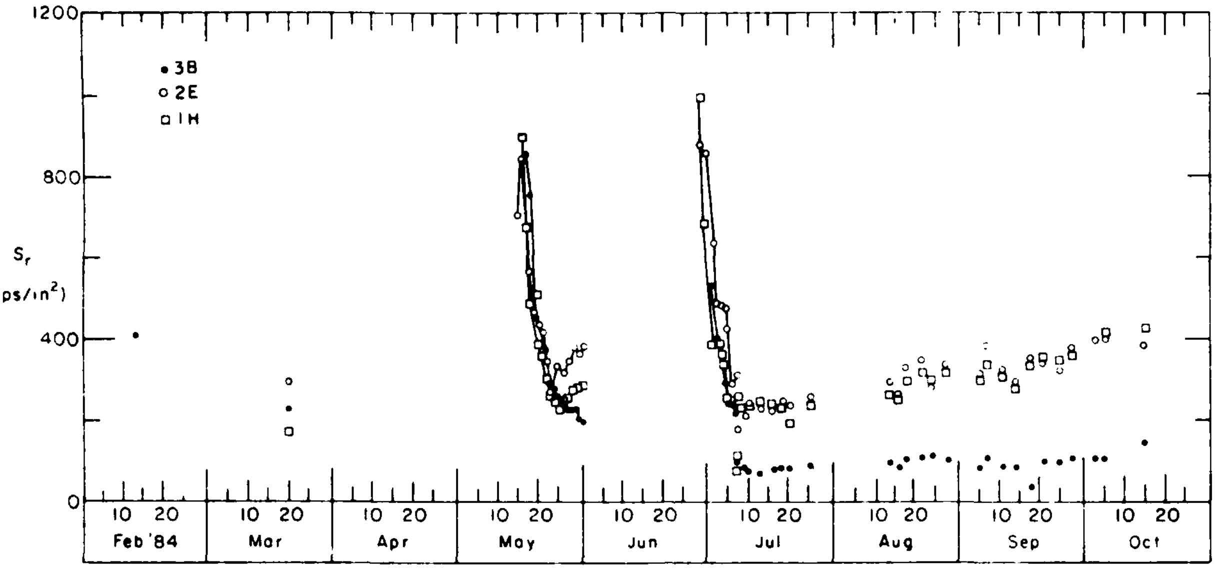


Figure 22. Resilient Stiffness with Time for Each of the Test Sections.

The second freeze cycle was started on 1 June 1984 and the test sections were frozen to the desired depths on 27 June. The freezing panels again were removed and daily measurements with the FWD were made; daily subsurface temperature measurements were also conducted.

The loadcart and weights had arrived from Tyndall AFB on 8 May 1984. On 5 July 1984 it was estimated that the desired thawing would occur on 7 or 8 July 1984. A group of CRREL engineers and technicians was contacted to be available on those dates. Major Walrond of the Air Force Engineering and Services Center was present during the testing; he brought a high-pressure tire gage to check the tire pressure on the loadcart.

Traffic testing commenced on the morning of Saturday, 7 July 1984. While the lead weights were being placed on the loadcart, FWD tests were conducted on the test sections.

Tables 10 and 11 contain resilient stiffnesses of the test sections based on measurements with the FWD. The resilient stiffness and plate deflections were used along with the equation developed by Bush (Reference 1) to estimate the traffic capacity of the test sections without the effects of freezing and thawing. The equation developed by WES for the F-4 loadcart is:

$$\text{Passes} = 1250 - 20.7 \text{ (deflection in mils)}$$

Using this equation and data obtained on 7 July (Table 11), before starting traffic, estimates were made of passes-to-failure of each test section if the F-4 loadcart had been used and if the test sections were unaffected by freeze-thaw cycles. The estimates were: Test Section 1, 525 passes; Test Section 2, 650 passes, and Test Section 3, 380 passes. Since the loading of the F-15 simulated in these tests was greater than that of the F-4, we expected that the test sections would not sustain the amount of traffic estimated from the equation. The equation had been developed for the test sections that did not include cement-stabilized base courses, therefore, it would not accurately reflect the capacity of Test Section 3, which contained a cement-stabilized sand base course.

After the weights were placed on the loadcart, it was backed onto Test Section 3 and parked while guide arms were installed. As this work was being done, the loaded wheel began to settle into the pavement. The loadcart could not be moved by the vehicle itself nor with the assistance of a large front-end loader, so the weights were removed and the loadcart was towed from the

TABLE 10. STRENGTHS OF TEST SECTIONS AS DETERMINED FROM MEASUREMENTS WITH THE FWD BEFORE AND AFTER THE FIRST FREEZING CYCLE.

Test point	Date	Before Freezing: S_r [a] (k/in.^{2})	Before Freezing: Δ [b] (mils)	Date	After First Freeze: S_r [a] (k/in.^{2})	After First Freeze: Δ [b] (mils)
1H max.	20 Mar	174.0	52.0	15 May	9896.0	1.0
1H min.				26 May	232.0	39.0
2E max.	20 Mar	293.0	31.0	15 May	8678.0	1.0
2E min.				23 May	272.0	33.0
3B max.	20 Mar	226.0	38.0	15 May	5002.0	2.0
3B min.				31 May	198.0	45.0

[a] Resilient stiffness determined by dividing the actually applied load by the plate deflection.

[b] Plate deflection for a 9000-pound load. Determined by dividing 9 kips by the resilient stiffness.

TABLE 11. STRENGTHS OF TEST SECTIONS AS DETERMINED FROM MEASUREMENTS WITH THE FWD BEFORE AND AFTER THE SECOND FREEZING CYCLE.

Test point	After Second Freeze: S_r [a] (k/in.^{2})	After Second Freeze: Δ [b] (mils)	Date	During Load Applications: S_r [a] (k/in.^{2})	During Load Applications: Δ [b] (mils)	passes
1H max.	1065.0	8.0	27 June			
1H min.	[c]258.0	[c]35.0	7 July	73.0	123.0	44
2E max.	1277.0	7.0	27 June			
2E min.	[c]310.0	[c]29.0	7 July	142.0	63.0	44
3B max.	5042.0	2.0	27 June			
3B min.	[c]216.0	[c]42.0	7 July	98.0	92.0	5

[a] Resilient stiffness determined by dividing the actually applied load by the plate deflection.

[b] Plate deflection for a 9000-pound load. Determined by dividing 9 kips by the resilient stiffness.

[c] Data obtained prior to applying traffic with the loadcart.

depression. We estimate that the loadcart settled into the pavement about 6 inches within 1 minutes after it was parked. From the time the loadcart was parked until it was removed, about 1 hour later, the depth of the rut had reached about 12 inches. Fortunately, the loadcart was not damaged and traffic testing was completed after the cart was reloaded. We did not park the loadcart on any of the test sections after this experience.

The number of passes to failure of each of the test sections is listed in Table 12. Only Test Section 2 exceeded the minimum number of passes (150) extablished by the Air Force for these facilities. Figures 23 through 25 illustrate rut development in each of the Test Sections. Figure 23, at Station 0+10, is Test Section 1; Figure 24, at Station 0+40, is for Test Section 2; and Figure 25, at Station 0+75, is for Test Section 3.

Test Sections 1 and 2 failed due to the development of a rut more than 3 inches deep, and Test Section 3 failed because of excessive resilient deformation.

TABLE 12. NUMBER OF PASSES TO FAILURE OF EACH TEST SECTION.

Test section	Passes to failure	Failure mode
1	88	3 in. rut
2	168	3 in. rut
3	5	Excessive resilient deflection[a]

[a] Additional traffic was not applied to preclude the loadcart being stuck or damaged.

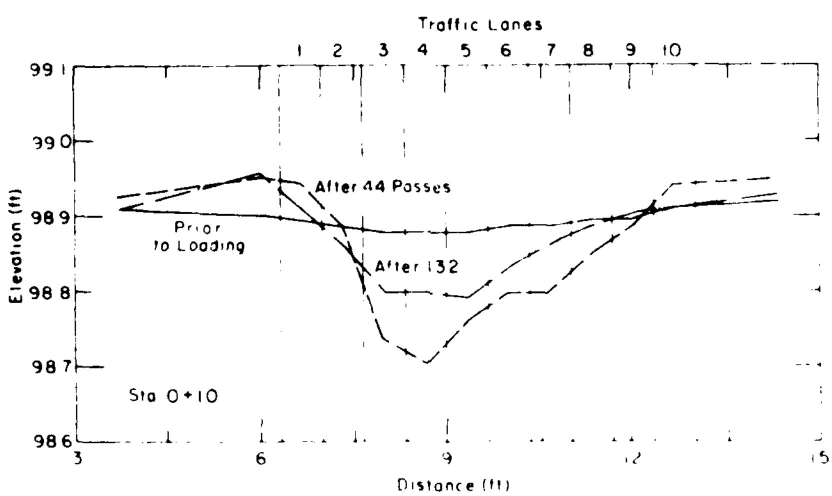


Figure 23. Rut Development at Station 0+10 in Test Section 1.

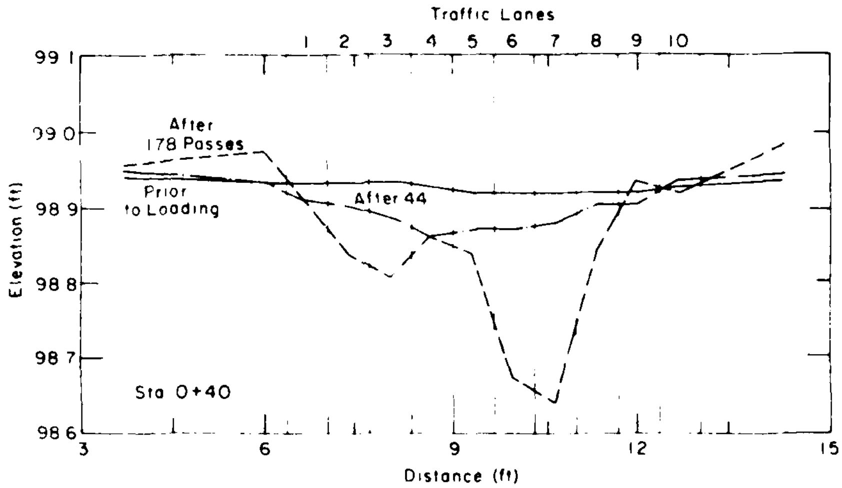


Figure 24. Rut Development at Station 0+40 in Test Section 2.

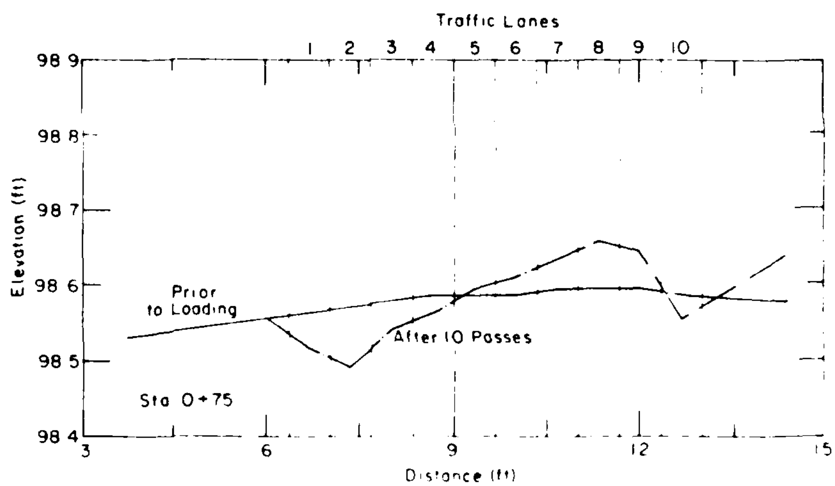


Figure 25. Rut Development at Station 0+75 in Test Section 3.

E. TEST PITS

When the traffic tests were completed, test pits were excavated in each of the test sections. The test pits were located so that a set of tests was conducted in the trafficked area and another set in the untrafficked area of each test section. The results are shown in Tables 13 and 14.

Figure A-2 and Tables A-1 through A-3 in Appendix A show the locations of the pits, as well as the locations of the instrumentation.

TABLE 13. SUMMARY OF TESTS ON THE CLAY SUBGRADE AFTER TRAFFIC TESTS.

Test section	Depth into clay (in.)	Depth below pavt. surface (in.)	CBR (%)	Moisture content (% dry wt.)	Dry density[b] (lb/ft³)
1	0	19	[a]5.5	[a]13.7	[a]112.9
			6.5	14.6	119.1
2	0	24	[a]6.9	[a]14.7	[a]115.1
			8.0	14.7	118.7
3	0	14	2.5	16.6	104.6
			2.4	17.9	108.6
			2.0	18.9	
1	6	24	[a]2.5	[a]14.9	[a]109.0
			4.9	16.7	113.1
2	6	30	[a]5.4	[a]15.7	[a]108.3
			5.6	16.2	108.9
3	6	20	2.0	19.2	101.8
			1.4	19.3	106.2
			1.1	19.8	
1	12	30	7.8	15.2	111.4
			[a]2.5	[a]15.4	[a]114.2
2	12	36	[a]5.4	[a]16.1	[a]111.8
			3.9	16.7	114.0
3	12	26	2.0	20.3	101.9
			1.3	20.4	108.7
			1.9	21.2	
1	18	36	4.3	15.9	108.3
			[a]5.5	[a]16.1	[a]110.2
2	18	42	5.2	[a]16.6	[a]107.4
			[a]4.7	16.8	112.7
3	18	32	4.8	18.4	97.5
			2.3	19.1	105.0
			1.4	21.7	
1	24	42	--	15.8	109.8
2	24	48	--	16.7	114.3
3	24	38	--	18.8	102.3
1	30	48	--	15.0	111.2
2	30	54	--	18.1	106.5
3	30	44	--	20.4	103.1
1	36	54	--	20.4	107.7
2	36	60	--	20.2	105.6
3	36	50	--	20.9	94.4

[a] Measurements in trafficked areas.

[b] Using sand cone procedure.

TABLE 14. DENSITIES, MOISTURE CONTENTS, AND CBRs OF BASE AND SUBBASE COURSES AFTER TRAFFIC TESTS.

Material	Depth below pavt. surface (in.)	Test section	CBR (%)	Moisture content (% dry wt.)	Dry density (lb/ft³)
Cement-stabilized sand	0-3	3	19.9		
		3	25.9	6.9	114.8
		3	30.9		
West Hartford sand		3	13.8		
	3-12	3	10.0	6.85	112.3
		3	8.7		
Crushed stone base course[a]	3-6	2	8.8	0.50	111.3
		2	8.6		
		2	24.3	0.97	124.1
		2	23.6		
		1	19.0	0.76	107.5
		1	14.8		
Shoulder stone subbase course	12-15	2	30.7	3.40	122.0
		2	51.3		
	9-12	1	69.0	3.61	146.3
		1	81.3		
		1	10.0	4.62	109.2
		1	26.8		

[a] Due to the open-graded nature of the crushed stone base course, the CBR values and dry densities shown for that material are not representative of the actual values. Due to the test methods, both values are substantially lower than the true values.

As indicated in Table 5, the design thickness of the cement-stabilized sand was 6.4 inches. The test pit placed in this section after the traffic tests indicated that the cement-stabilized layer averaged only 2.6 inches thick. CBR tests were performed on the cement-stabilized sand after the traffic tests; the results are shown in Table 7 along with results from measurements of the in situ moisture content and density. Although the CBR of the cement-stabilized sand was 2.5 times greater than that of the unstabilized sand, the value was less than 50 percent of that required for the top of a granular base course. The in situ density was also only slightly greater than that of the unstabilized sand and only about 88 percent of the optimum dry density based on the laboratory test results (Figure 14). The low density of the cement-stabilized sand, coupled with its inadequate thickness, caused the pavement on this test section to fail after only a few passes with the loadcart.

The test pit in Test Section 3 was excavated 18-20 July 1984. In Test Section 2, the excavation took place on 24-26 July 1984, and Test Section 1 was excavated on 30 July through 2 August 1984. CBR tests were conducted to a depth of about 18 inches into the subgrade in all the test pits. Moisture content and dry density samples were obtained to depths of about 36 inches into the subgrade in all three test sections.

A comparison of pre- and posttraffic soil properties was made by comparing average test values in the upper 18 inches of subgrade in each test section. Data from various depths are shown in Table 7 (pretraffic) and Table 13 (posttraffic). The average pretraffic CBR in Test Section 1 was 6.9; after traffic the average CBR was 4.0 in the traffic lane and 5.9 outside of the lane. In Test Section 2 the average pretraffic CBR was 4.9, and after traffic the CBR values were 5.6 in the trafficked area and 5.7 outside of the traffic lane. In Test Section 3, the pretraffic CBR was 6.1, but it was only 2.1 when the test pit was excavated. The moisture content and dry density conditions before and after the test sections were subjected to traffic are summarized in Table 15.

TABLE 15. MOISTURE CONTENT AND DRY DENSITY BEFORE AND AFTER TRAFFIC.

	Test Section 1	Test Section 2	Test Section 3
Moisture Content (% dry wt.)			
Before traffic	17.5	17.9	17.9
Untrafficked area	15.6	16.1	19.4
Trafficked area	15.0	15.8	--
Dry Density (lb/ft^3)			
Before traffic	102.7	104.8	103.0
Untrafficked area	113.0	113.6	104.3
Trafficked area	111.6	110.6	--

In Test Sections 1 and 2, the subgrade dry density increased after traffic was applied, but Test Section 3, which received essentially no traffic, did not show this increase. The average subgrade moisture contents in Test Sections 1 and 2 decreased after they were subjected to traffic, whereas the moisture content in Test Section 3 increased after the freeze-thaw cycles and after essentially no traffic.

F. DISCUSSION OF RESULTS

Equipment used to construct the test sections was that normally used in roadway or airfield construction. Although a small roller was used to compact the crushed stone base and crushed shoulder stone subbase, properties of these materials obtained after the traffic tests indicate that the strengths of these materials did not contribute significantly to the failures of Test Sections 1 and 2

Recycling of the pavement on Test Sections 1 and 2 did not cause a significant change in the anticipated behavior of the test sections. The recycled pavement had higher-than-required Marshall stabilities and contained approximately the specified amount of asphalt. Somewhat surprisingly, the recycled asphalt cement properties did not differ significantly from the unrecycled material (Table 4). No substantial cracking occurred in the recycled pavement until large deformations occurred, indicating that it was not excessively brittle due to the recycling process.

Results from Test Section 3, which contained the cement-stabilized sand base course, certainly did substantiate the fact that a thick AFRS pavement will be required over a wet, thaw-weakened subgrade. Results from Test Section 3 also indicate that heavily loaded aircraft could become stuck on an underdesigned pavement over a severely thaw-weakened subgrade, if they must stop even for relatively short periods of time, i.e., 5 minutes or less.

Results from these test sections indicate ALRS pavements designed according to procedures outlined by the Departments of the Army and Air Force (Reference 3) have a high probability of premature failure if the design traffic is applied during the most severely thaw-weakened period. If the ALRS pavements must be designed for full capacity at any time of the year, a revised design procedure must be developed. That procedure is contained in the following section.

SECTION IV

REVISED FROST DESIGN PROCEDURE

Figure 26 illustrates vertical deflections at the surface of a pavement when it is subjected to wheel loadings at different times of the year. The data are idealized, but illustrate several important points: (a) deflections are small during the winter when the pavement, base, and upper portions of the subgrade are frozen; (b) the frozen soil thaws from the bottom and from the top; (c) deflections begin to increase as the pavement system begins to thaw from the surface; (d) pavement deflections increase as thaw penetration increases and additional water is released from previously frozen soil; (e) the maximum deflection occurs at the time the soil is just completely thawed; and,(f) deflections decrease rapidly for a period of time after thawing is completed and then the rate of decrease is significantly reduced.

Although no comprehensive study of the behavior of airfield pavements during freezing and thawing has been conducted in the last 30 years, a few studies have been conducted on roadway pavements in seasonal frost areas. One of the early studies of highway pavements used results of plate-bearing tests during the winter, spring, and summer to monitor pavement behavior during the frost melting period, i.e., spring breakup or spring thaw. Figure 27 illustrates the behavior of a good road and a poor road in Iowa. The results in Figure 27 are presented relative to the strength obtained from the plate bearing test prior to freezing. The good road retained 30 to 60 percent of its strength during the frost melting-period, but the poor road retained only 15 to 25 percent of its prefrozen strength.

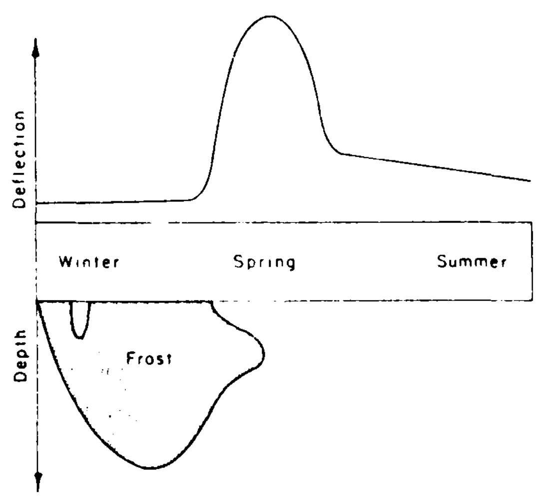


Figure 26. Deflection of a Pavement Surface due to an Applied Wheel Load at Different Seasons of the Year.

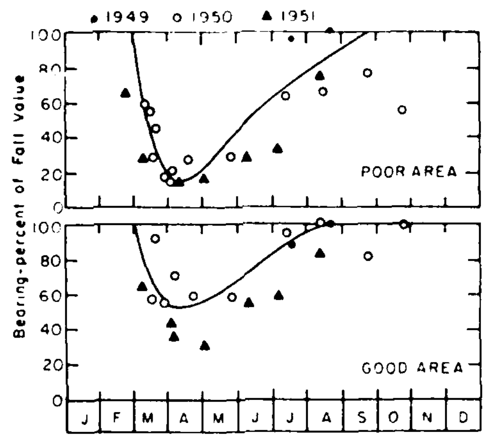


Figure 27. Reduction of Bearing Capacity of Roadway Pavements near Grand Junction, Iowa.

When the AASHO Road Test was conducted in 1959-1961, the Present Serviceability Index was used to evaluate the condition of the pavements tested. The Present Serviceability Index is determined from an empirical equation:

$$P_t = P_o e^{-bW_t}$$

where

P_t = Present Serviceability Index at time t

P_o = the same index at start of traffic (t = O)

b = deterioration rate parameter

W_t = millions of accumulated load applications to time t.

The Present Serviceability Index was initially a qualitative evaluation of the pavements made by a team of judges who considered longitudinal and transverse roughness and the extent of cracking and patching in their evaluation. Referring to the behavior of flexible pavements, Painter (Reference 8) states:

> "In the equation, b is the deterioration rate of the pavement, dependent on thickness and strength of surface, base and subbase, subgrade soil strength, and the load applied to the pavement. Analysis of the Road Test data shows that, for a given pavement, b is constant throughout the year, except for the spring thaw periods. During these times, the deterioration rate increases to some higher value and then returns gradually to its original, nonspring thaw, value. This increase of the deterioration rate is the result of a loss of strength in the pavement structure, most likely in the subgrade soil and the granular layers."

Figure 28 illustrates the change in Present Serviceability Index of two test items at the AASHO Road Test; note the two periods during which the Present Serviceability Index decreased abruptly on each test item. These two intervals corresponded to the frost melting periods in 1959 and 1960. The elapsed time between each determination of the Present Serviceability Index was about 2 weeks at the AASHO Road Test. Therefore, the duration of the intervals when the pavements deteriorated most rapidly was about 6 weeks. Although only a small percentage of the total traffic occurred during the frost-melting periods, most of the reduction in Present Serviceability Index took place during these times.

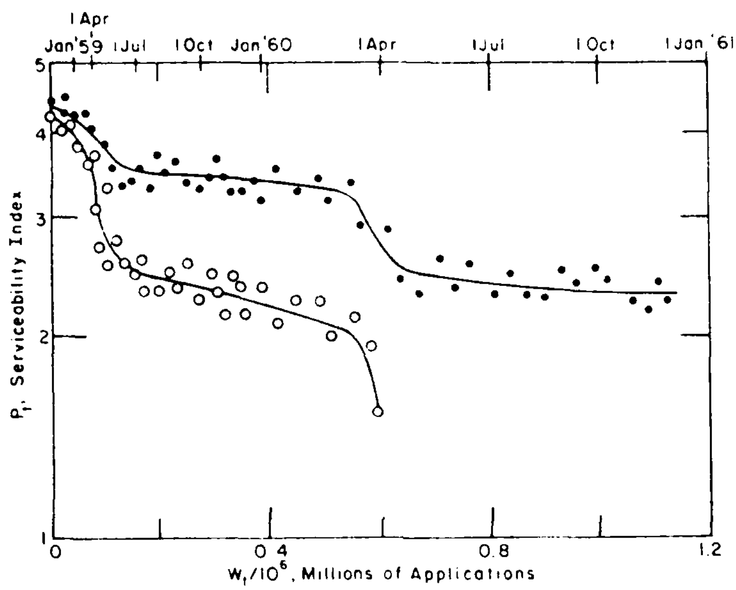


Figure 28. Performance of Typical Asphalt Concrete Pavements at the AASHO Road Test.

During the last 20 years nondestructive testing devices, i.e., the Benkleman beam, the Dynaflect, and the falling weight deflectometer (FWD), have been used more frequently to evaluate and predict the performance of pavements. Figure 29 illustrates the idealized variation in resilient modulus (defined as the deviator stress divided by the recoverable strain) of a typical silt subgrade soil through a year in a seasonal frost area. Computer simulations are used to obtain the resilient modulus. When modulus values are determined several times per year, a relationship similar to that shown in Figure 29 can be prepared. If freezing and thawing depths are measured during the winter and spring, relationships between reduction of the resilient modulus and thaw penetration can also be prepared.

Reference 9 divided the annual strength variation of flexible pavements subject to "deep" seasonal freezing into segments. Areas of deep seasonal freezing were defined as locations where the freezing index was approximately 500°F-days or more. Dynaflect was used to apply loads and measure surface deflections on the pavements tested. Twenty-four pavements located between central Illinois (mean freezing index of about 100°F-days) and northern Minnesota (mean freezing index of about 2100°F-days) were used in the study and ranged in thickness from 8 to 24 inches. The subgrade soils for all but one of the test sites are classified as F3 or F4 materials according to the Departments of the Army and Air Force frost classification system (Reference 3).

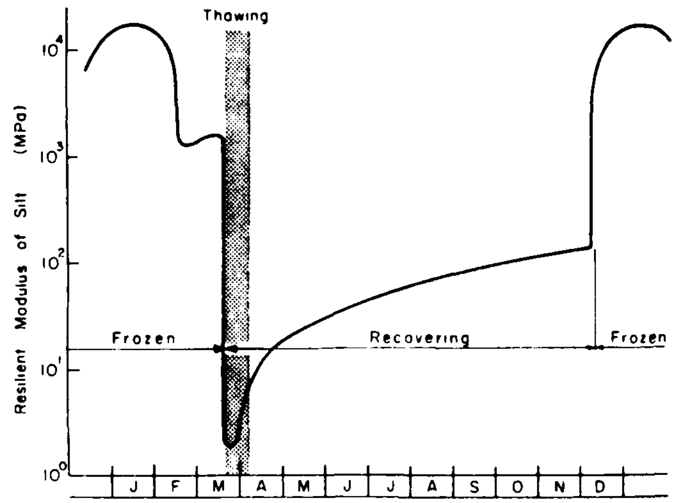


Figure 29. Seasonal Variation of Resilient Modulus of a Typical Silt Subgrade Soil.

Figure 30 illustrates the typical behavior; the annual cycle has been divided into four fairly well-defined periods (Reference 9), as follows:

Designation	Description
A	Period of deep frost
B	Period of rapid strength loss
C	Period of rapid strength recovery
D	Period of slow strength recovery

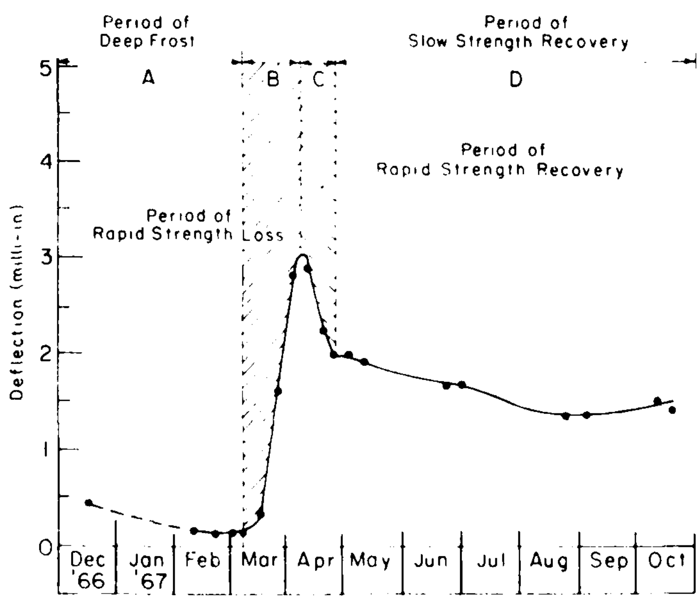


Figure 30. Typical Seasonal Variations in Depth of the Deflection Basin.

Period A begins with the first appearance of deep (subgrade) frost penetration in the late fall or winter. Period B begins with the abrupt upturn of the deflection curve coinciding with the beginning of thawing from the pavement surface in the spring. Period C begins at the peak of the deflection curve. Period D begins at the point where the deflection curve levels off following the spring peak.

The total interval encompassing Periods B and C is referred to as the "critical period." Duration of the critical period ranged from 37 to 53 days and averaged nearly 43 days.

Although data in Figure 27 do not clearly exhibit the period of rapid strength recovery, data in Figures 28 and 29 do. The periods of rapid pavement deterioration at the AASHO Road Test lasted about 42 days and the data in Figure 29 indicate that the period of rapid change is about 35 days. The combined data from these three sources indicate that the critical period may be about 42 days (6 weeks). However, data in Figures 29 and 30 also indicate that the duration of the most severely weakened period is only about 7 days.

The current design procedure used for ALRS is given in Reference 1. Either the reduced subgrade strength method or the limited subgrade frost penetration method may be used to design ALRS pavements, but the reduced subgrade strength method was suggested as probably providing the most economical pavements in most areas. The reduced subgrade strength method was used to design the ALRS pavements tested at CRREL in 1984. After the test sections were constructed, data from tests with the FWD on the pavement surface indicated that they would all fail before experiencing the design traffic. Therefore, two of the three test sections were reconstructed using estimates of the minimum subgrade CBR that would be experienced during thawing periods. The design CBR values used were 2.0 for Test Section 1 and 1.2 for Test Section 2. Tables 6 and 9 contain layer thickness and depths of frost penetration experienced by each test section during the CRREL studies. Table 12 gives the number of passes to failure and the mode of failure in each test section.

Results from the 1984 CRREL tests indicate that ALRS pavements designed using the reduced subgrade strength method will fail if the loads are applied when the subgrade is in a thaw-weakened condition. These results were not unexpected, because Reference 1 states:

> "It is extremely important to note, however, that the FASSIs are effective weighed values averaged over the entire life of a pavement. <u>The FASSIs are generally not the minimum CBR values esperienced during frost-melting periods</u>. Since all the traffic on an ALRS could occur over a time span of only a few hours or a few days, the frost area soil support indices may not provide pavement thicknesses sufficient to sustain the design traffic if it should occur during periods of subgrade thawing in the winter and spring. For example, a lean clay subgrade classified as an F4 soil under the frost classification system may exhibit a normal-period CBR of 6. The frost area soil support index for this material is 3.5, but if its CBR were only 2.0 after thawing, pavements designed with the FASSI may fail due to inadequate thickness [if all of the traffic occurred during frost-melting periods]. The pavement thickness for a frost area soil support index of 3.5 is 14.5 in. when the aircraft gross weight is 60,000 lbs and the design traffic is 150 passes. For the same aircraft and amount of traffic, a pavement thickness of 18.5 in. is required over a subgrade having a CBR of 2.0."
>
> "A question which may be raised is that if ALRS pavements are

designed using the Frost Area Soil Support Index for a particular soil and the CBR of the soil during application of the design traffic is less than the FASSI, what is the risk of pavement failure prior to sustaining the desired amount of traffic? The answer is that the probability of premature failure of the pavement is high unless the gross weight of the aircraft is reduced. In the example given on the previous page, the gross load of the aircraft would have to be reduced to about 38,000 lbs if the CBR of the subgrade were 2.0 rather than the 3.5 for which the pavement was designed. The number of passes of the F-4 aircraft, having a gross weight of 60,000 lbs, before failure of the 14.5 in.-thick pavement is about 50."

"Chamberlain (Ref. [6]) stated that the thaw period for in-service highway pavements studied by Scrivner (Ref. [9]) ranged from a few days to two weeks but the time for the pavement to reach a deflection which was only 20% greater than the fall deflection was 35 to 60 days. Unfortunately, no data are available to estimate the period of time when the CBR of the subgrade may be less than the FASSI. Also, no extensive correlations between laboratory and field CBR values after freezing and thawing have been conducted. The period of recovery from a thaw weakened condition is influenced by the hydraulic properties of the soil. For example, a sandy silt will probably drain excess water and recover its modulus values are determined several times per strength more rapidly than a highly plastic clay because the clay has a lower permeability. When road or airfield pavements are underdesigned, the road and airport managers must restrict traffic loads or, in extreme cases, close facilities to traffic for a period of a few days to several weeks in the spring. Typically, these periods are 2 to 6 weeks long. It seems highly unlikely that a design premise of closure or restriction of traffic on the ALRS pavements during such periods could be acceptable to the Air Force."

Figure 31 illustrates the possible behavior of a subgrade soil subjected to freezing and thawing. The soil illustrated is classified as an F4 material according to the frost classification system. Four periods similar to those described in Reference 9 are shown on the figures, as are the "nonfrost" CBR value, the FASSI value that would be used for reduced subgrade strength designs, and the minimum CBR that may be experienced by this soil during the thawing period. For the ensuing discussion, the end of the critical period is shown in Figure 31 as the time when the CBR of the subgrade recovers to the FASSI value. This could logically be the time shown in Figure 28 when the rate of pavement deterioration decreases dramatically after the spring thaw. Stated in different words, when the subgrade strength is less than the design value, pavement deterioration will occur much more rapidly than when the subgrade strength is at or greater than the design value.

The only pavement design that would support all of the design traffic on any day of its life would be based on the minimum CBR value shown in Figure 31. However, pavements designed using the CBR would be overdesigned for about 51 weeks (98 percent) of the year. If we assume that the critical period is 42 days (6 weeks) in length and if we further assume that the subgrade CBR is less than the FASSI during this entire period, then ALRS pavements with thickness designs based on the FASSI would be underdesigned for approximately 12 percent of the year. (The assumption that the subgrade CBR is less than the FASSI during this entire 42-day period is reasonable because most pavement deterioration occurs during this interval, as shown in Figures 28, 29, and 30.)

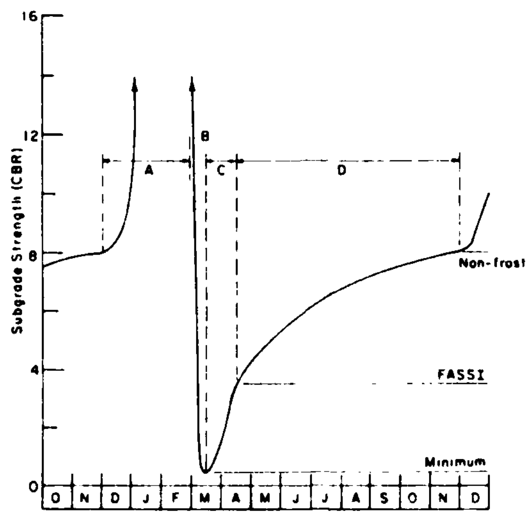


Figure 31. Change in CBR of an F4 Subgrade Soil that Freezes in Winter.

The question of the reliability of the design or the probability of failure must be addressed. Is a design that will be adequate 100 percent of the year necessary? Is a design that will fail if the traffic occurs during a period equal to about 12 percent of the design year adequate?

Data from the AASHO road test (References 6, 8, 9, and 11) indicate that a thaw-weakened period of about 42 days is reasonable. Data in these reports suggest that the thaw-weakened period, i.e., the period when the subgrade CBR is less than or equal to the FASSI, can be approximated as follows:

1. The subgrade CBR becomes less than the FASSI about 2 days before reaching the minimum subgrade CBR;

2. The mimimum subgrade CBR occurs for a period of 7 days; and

3. The subgrade recovers rapidly to a CBR equal to the FASSI after 33 additional days.

This approximation divides the periods of rapid strength loss and rapid strength gain in Figures 29 and 30 into three segments rather than only two, as shown in the figures.

To complete this approach, it is necessary to establish reasonable estimates of the minimum CBR values experienced by each type of subgrade soil. The low-plasticity clay soil used in this study exhibited a CBR of approximately 1.2 during traffic testing. This value will be used for clay soils.

Since silt soils generally exhibit greater frost heave than clay soils, a lower minimum CBR value will be used for silty soils. Johnson et al. (Reference 10) indicated that the silt soil in their study lost nearly all of its strength upon thawing. Laboratory frost susceptibility tests on silt soils frequently caused large volumes of excess ice, which result in positive pore pressures upon thawing. Therefore, a CBR of 0.5 is the minimum value recommended for silty soils. An analysis of data from tests conducted in the early 1950s (Reference 12) suggested minimum CBR values of 6.0 for F1 soils and 4.3 for F2 subgrade soils. Reference 3 recommends measuring the CBR on S1

and S2 soils, after conducting a laboratory frost-susceptibility test, and using that value for pavement design. The same procedure is also suggested for S1 and S2 subgrade soils for ALRS pavements.

Figure 32 illustrates approximate CBR values throughout the thaw-weakened period for different subgrade soils with frost classifications F1 through F4. Data in Figure 32 should be used with the reduced subgrade strength procedure to design ALRS pavements for seasonal frost areas. It is probable that soils within the same frost classification type will behave differently when subjected to the effects of frost action. It is also probable that the same pavement will behave differently after nearly every winter. However, very limited laboratory or field test data are currently available to verify these concepts.

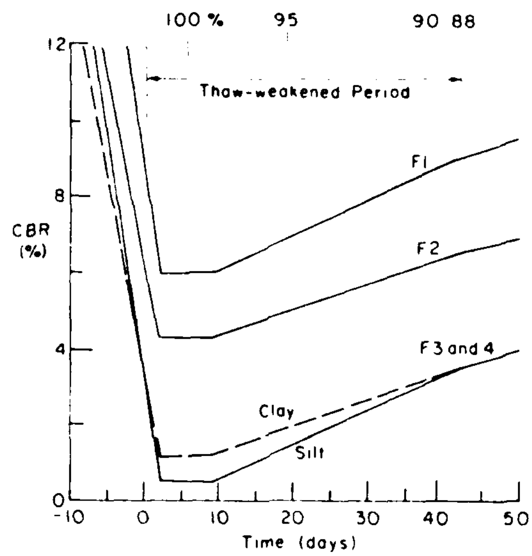


Figure 32. Estimated CBR Values of Subgrade Soils During Thaw-Weakened Periods.

As stated above, the minimum value for any soil can be chosen to provide a design adequate to carry the loads during any time of the year. If the FASSI is chosen for design, ALRS pavements are very likely to fail if the design traffic is applied during that 12 percent of the year when the subgrade CBR is less than the FASSI. If a subgrade CBR is chosen that will be adequate for 95 percent of the year, ALRS pavements would be very likely to fail if they were subject to traffic during the critical 5 percent of the year, an 18-day period at the end of the winter. A 90-percent design would give a period of approximately 37 days when ALRS pavements were likely to fail. The combined thicknesses of pavement and base for each of these conditions are shown in Table 16. It is suggested that the 95 percent protection level shown in Table 16 be used to design ALRS pavements where the subgrade is subject to seasonal freezing.

A question arises about designing pavements in areas with low-design freezing indices so that the combined thicknesses for the limited subgrade frost penetration method design would be less than those required in Table 16. Allowing no frost penetration into the subgrade (full protection from frost effects) would be overly conservative, but applying the limited subgrade frost penetration method used to design conventional pavements could be unsafe for pavements that may be subjected to all traffic during the frost-melting period. A modified limited subgrade frost penetration design method is recommended as an alternative procedure for ALRS pavements in areas with low-design freezing indices.

TABLE 16. COMBINED THICKNESS OF PAVEMENT AND BASE FOR REDUCED SUBGRADE STRENGTH DESIGNS FOR ALRS PAVEMENTS.[a]

Protection level	Soil Type							
					F3 and F4			
	F1		F2		Silts		Clays	
	CBR	D[b]	CBR	D[b]	CBR	D[b]	CBR	D[b]
FASSI (88%)	9.0	8.5	6.5	10.5	3.5	14.5	3.5	14.5
90%	8.5	9.0	6.2	11.0	3.0	15.0	3.2	15.0
95%	6.9	10.0	5.0	12.0	1.4	23.0	1.9	19.5
100%	6.0	11.0	4.3	13.0	0.5	30.0	1.2	24.5

[a] These thicknesses are adequate for 150 passes of the F-15 aircraft loaded to 32,500 lb on each main gear and with tire pressures of 355 lb/in.^{2}.

[b] Minimum combined thickness of pavement and base required above subgrade (inches).

With this alternative procedure, design thicknesses must be checked for adequate bearing capacity based on normal-period CBR values. The modified procedure allows about one-half of the amount of frost penetration into subgrade soils that is allowed for conventional pavements. For fine-grained subgrade soils, i.e., Frost Classes F3 and F4, the required base course thickness is 85 percent of the base course thickness necessary to prevent subgrade frost penetration (full protection). For F1 and F2 subgrades, a value of 75 percent is recommended. Figure 33 may be used to determine the design base course thickness in areas that have low freezing indices. Caution is required when using the "modified" limited subgrade frost penetration design method. It was not evaluated during this study, and the results from this method have not been subjected to ALRS-type traffic loads and repetitions in the thaw-weakened condition. The modified limited subgrade frost penetration method should only be used when it requires less pavement and base thickness than are required by the reduced subgrade strength method for ALRS pavements, but thicknesses less than those required by the normal-period CBR values should never be used.

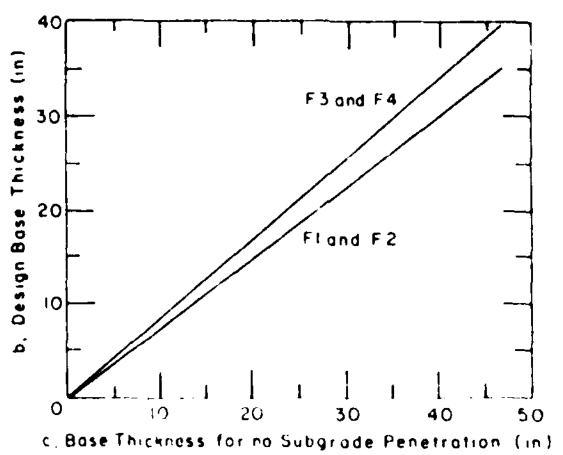


Figure 33. Design Thickness of Non-Frost-Susceptible Base Course Using the Limited Subgrade Frost Penetration Method for ALRS Pavements.

Figure 34 shows the combined thicknesses of asphalt pavements and base courses required to prevent subgrade freezing. The maximum air freezing index show in the figure is 500°F-days. In general, design freezing indices less than 400°F-days will require thinner pavements by the modified limited subgrade frost penetration method than by the reduced subgrade strength method for ALRS pavements.

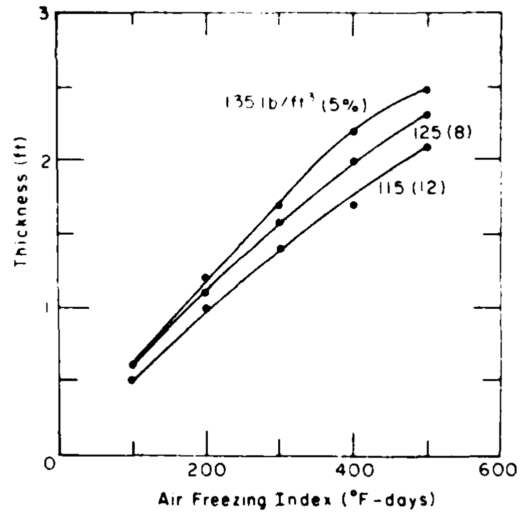
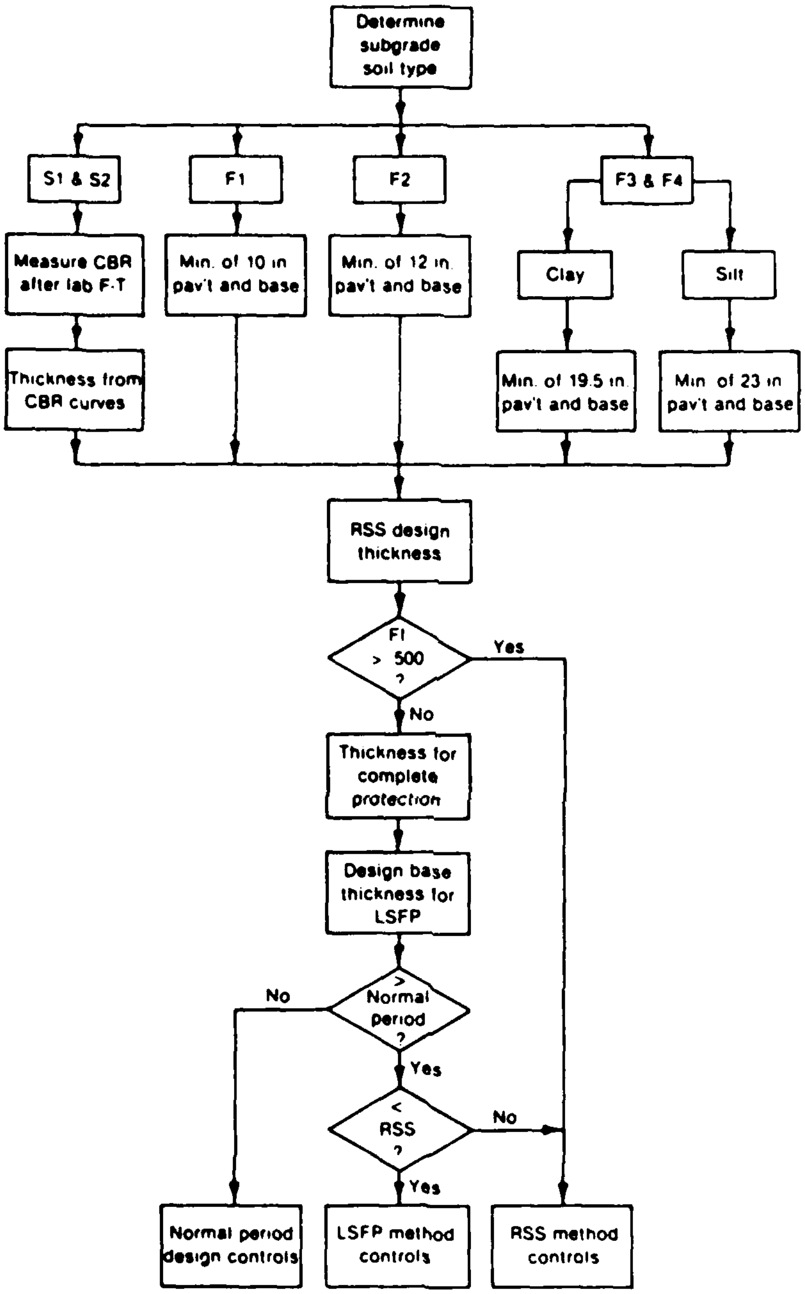


Figure 34. Thickness of Asphalt Pavement (3 in.) and Non-Frost-Susceptible Base Course to Prevent Subgrade Freezing.

Figure 35 is a summary and flowchart for determining the recommended thickness of pavement and base course for ALRS pavements in cold regions.



FI = Freezing Index, °F-days
RSS = Reduced subgrade strength for ALRS
LSFP = Limited subgrade frost penetration for ALRS

Figure 35. Recommended Design Procedure for Asphalt Concrete ALRS Pavements in Cold Regions.

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SECTION V

CONCLUSIONS

Results from the test sections constructed and subjected to simulated aircraft traffic indicated the ALRS pavements designed according to procedures outlined in Reference 6 posses a high probability of premature failure if the design traffic is applied during severely thaw-weakened periods of the year.

The revised frost-design procedure developed in Section IV of this report should be used to design flexible pavements for ALRS in areas exposed to seasonal freezing.

No guidance is provided in this report for the design of rigid ALRS pavements in seasonal frost areas. Construction and testing of full-scale test sections will be required to prepare reliable and economical design methods for rigid ALRS pavements.

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SECTION VI

RECOMMENDATIONS FOR FUTURE RESEARCH

Some concepts may be considered that reduce the initial cost or increase the useful life of ALRSs in seasonal fr . areas. The first concept is to review winter and spring conditions regionally in areas where ALRSs will be installed. Mean and design frost penetration depths should be estimated as well as the onset of thawing in the spring. If bases in a region can be chosen so that aircraft could be moved from one base to another during thawing, pavements of reduced thickness may be used. When pavements are severely weakened at the first base to thaw, aircraft would be based where thawing had not started. The aircraft could be moved to a sequence of bases; when thawing commenced at the last one, they would be moved back to the first. The pavement strength at that base would have increased from its weakest condition. Probabilistic methods should be used to establish the dates of initial thawing and the possible variations from year to year and location to location.

The second concept would not reduce the initial cost, but would allow for greater use of the facility than the 150 passes for the ALRS. For example, the pavements could be designed for 5000 passes over a period of 20 years. Thicknesses would be determined by using frost design procedures for conventional pavements. This procedure would allow pavements to be used for training and other missions rather than for only a one-time ALRS application.

Traffic testing of pavements designed using the "conventional" and "modified" limited subgrade frost penetration method may allow reduced thickness of pavements for ALRS (and for conventional) pavements.

A desk, laboratory, and field study should be conducted to determine the minimum CBR experienced by subgrade soils. The study would also determine the length of the weakest period and develop a method of estimating the minimum CBR using a laboratory test. Perhaps a CBR test immediately after thawing a laboratory frost-susceptibility test specimen would be useful. The field study could be conducted over a period of several years by collecting data from several different Air Force bases or over a shorter period by using test sections in the Frost Effects Research Facility recently completed at CRREL.

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APPENDIX A

LAYOUT OF TEST SECTIONS AND INSTRUMENTATION

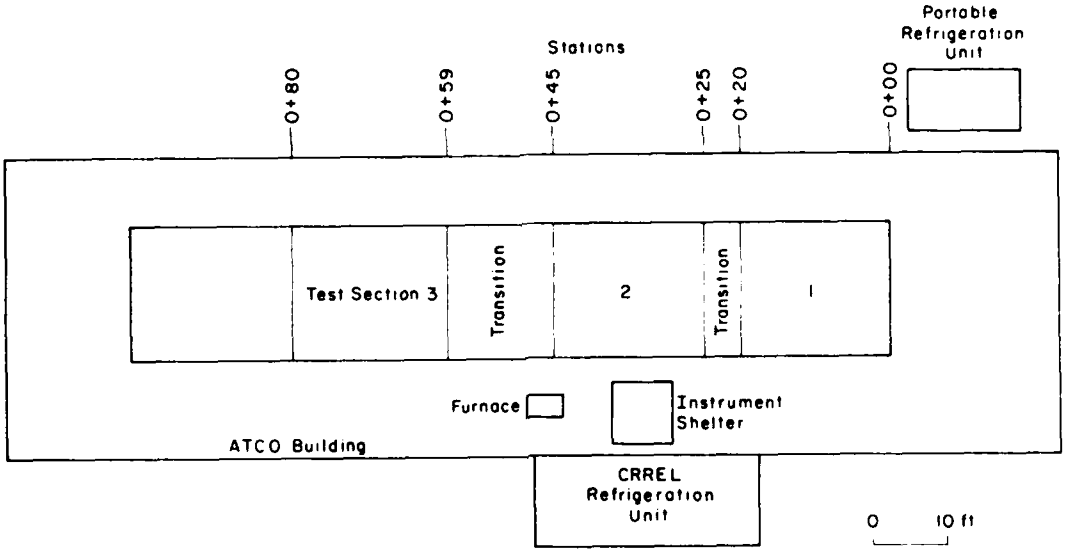

Figure A-1. Layout of Test Sections and Surrounding Area.

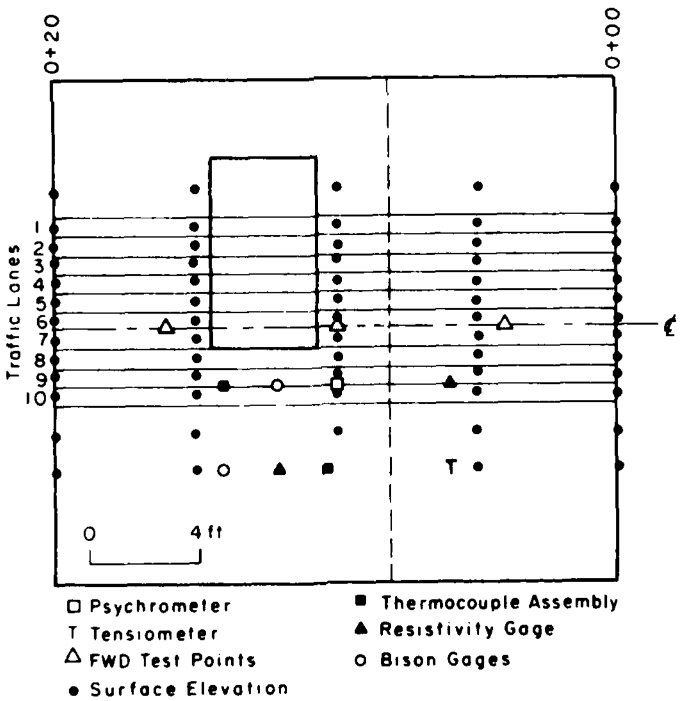

Figure A-2. Layout of Instrumentation and Test Pit in the Test Sections.

TABLE A-1. INSTRUMENTATION LOCATIONS IN TEST SECTION 1.

	Thermocouple assembly		Resistivity gages		Bison gages		Psychro-meter	Tensio-meter
Station	0+10	0+14	0+06	0+12	0+12	0+14	0+10	0+05
Offset	5'N	2'N	2'N	5'N	2'N	5'N	2'N	5'N
Sensor			Depth below pavement surface (in.)					
1	0		19		11		25	25
2	3		21		15		31	31
3	6		23		19		43	37
4	9		25		23			43
5	12		27		27			55
6	15		29		31			
7	18		31		35			
8	21		33					
9	24		35					
10	27		37					
11	33		39					
12	39		41					
13	45		43					
14	51		45					
15			47					
16			49					
17			51					
18			53					
19			55					

TABLE A-2. INSTRUMENTATION LOCATIONS IN TEST SECTION 2.

	Thermocouple assembly		Resistivity gages		Bison gages		Psychro-meter	Tensio-meter
Station	0+35	0+39	0+31	0+37	0+37	0+39	0+35	0+31
Offset	5'N	2'N	2'N	5'N	2'N	5'N	2'N	5'N
Sensor			Depth below pavement surface (in.)					
1	0		24		12		30	30
2	3		26		16		36	36
3	7.5		28		20		48	42
4	11.5		30		24			48
5	15.5		32		28			60
6	20		34		32			
7	24		36		36			
8	27		38					
9	30		40					
10	33		42					
11	36		44					
12	39		46					
13	45		48					
14	51		50					
15			52					
16			54					
17			56					
18			58					
19			60					

TABLE A-3. INSTRUMENTATION LOCATIONS IN TEST SECTION 3.

	Thermocouple assembly	Resistivity gages	Bison gages	Psychrometer	Tensiometer
Station	0+10 / 0+14	0+06 / 0+12	0+12 / 0+14	0+10	0+05
Offset	5'N / 2'N	2'N / 5'N	2'N / 5'N	2'N	5'N
Sensor	**Depth below pavement surface (in.)**				
1	0	19	11	25	25
2	3	21	15	31	31
3	6	23	19	43	37
4	9	25	23		43
5	12	27	27		55
6	15	29	31		
7	18	31	35		
8	21	33			
9	24	35			
10	27	37			
11	33	39			
12	39	41			
13	45	43			
14	51	45			
15		47			
16		49			
17		51			
18		53			
19		55			

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