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MAY 1980

FINAL REPORT

PERFORMANCE OF SOIL-AGGREGATE-FABRIC SYSTEMS IN FROST-SUSCEPTIBLE ROADS, LINN COUNTY, IOWA

ISU-ERI-AMES-80211
Project 1269

ENGINEERING RESEARCH INSTITUTE
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Final Report

PERFORMANCE OF SOIL-AGGREGATE-FABRIC SYSTEMS IN FROST-SUSCEPTIBLE SECONDARY ROADS, LINN COUNTY, IOWA

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DEPARTMENT OF CIVIL ENGINEERING
ENGINEERING RESEARCH INSTITUTE
IOWA STATE UNIVERSITY, AMES

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1. INTRODUCTION

Thousands of miles of unimproved secondary roads in the upper midwest are annually plagued by frost action in their subgrades. Among the economic losses incurred are costs of repair and maintenance of the damaged roadway. Considerable loss of skid resistant surface aggregate occurs because of intrusion into the thaw-softened subgrade. Economic implications also occur to highway users if a weight limit embargo is imposed, or more severely if complete closure of the roadway is dictated by thaw-induced lack of traffic support capacity.

Frost action in subgrade soils occurs in two phases. In the first phase, soil heaves with the growth of ice layers or lenses as the freezing front moves into the subgrade. Freezing of the pore water essentially "dries" the soil, making the pore water unavailable for soil attraction and thereby decreasing its capillary potential (or increasing the surface attraction for water). More water then moves upward from an area of higher capillary potential, increasing the volume of frozen water and the soil's water content, creating additional ice lensing, ultimately accompanied by heaving or vertical displacement of the roadway.

The second phase of frost action occurs in the spring of the year when thawing of ice lenses occurs with the downward advancing thaw front. Melting of ice lenses causes a supersaturated condition in the soil, with the diminishing layer of ice impeding gravitational drainage.

During this period of time the roadway is vulnerable to severe traffic rutting; aggregate is pushed into the supersaturated region, and any subgrade that is displaced is pumped to the surface, extending to the shoulder region, or both.

It was the purpose of this investigation to evaluate the in-situ performance of Mirafi 140 fabric in the construction of soil-aggregate and granular surfaced roadways overlying frost-susceptible fine grained subgrades. The primary function of the fabric was as an interlayer reinforcement between the subgrade and either a soil-aggregate surface or a base course material.

2. TEST SECTIONS

Under the supervision of representatives of Celanese, Mirafi 140 fabric was placed in seven test sections, located at two sites in Linn County, Iowa. Five were on a county road north of Alburnett and two were northwest of Fairfax (Figure 1). Each test section was paired with an adjacent control section constructed in the same manner as the test section, except lacking fabric. The areas of frost-susceptible subgrades were established by personal communications and site inspections with Linn County engineering and maintenance personnel and local residents.

One commonly used method to combat frost action is to core out the frost-susceptible material, replacing it with a coarse granular backfill. Sections 1A, 1B, 2A, and 2B at the Alburnett site (Figure 2) were constructed October 21-22, 1976 in this manner. The existing soil-aggregate surface course was removed; the frost-susceptible subgrade was undercut about two feet and backfilled with a coarse aggregate; and the soil-aggregate surface course was replaced and compacted. Removal and replacement of materials was done with a self-loading scraper. Section 1A employed a layer of fabric between the granular backfill and the soil-aggregate surface course. In sections 1B and 2A the granular backfill was located within a trough of fabric produced by placing the fabric along the bottom and sides of the core-out. A layer of fabric was placed between the granular backfill and surface course of section 1B, while in section 2A, the soil-aggregate was

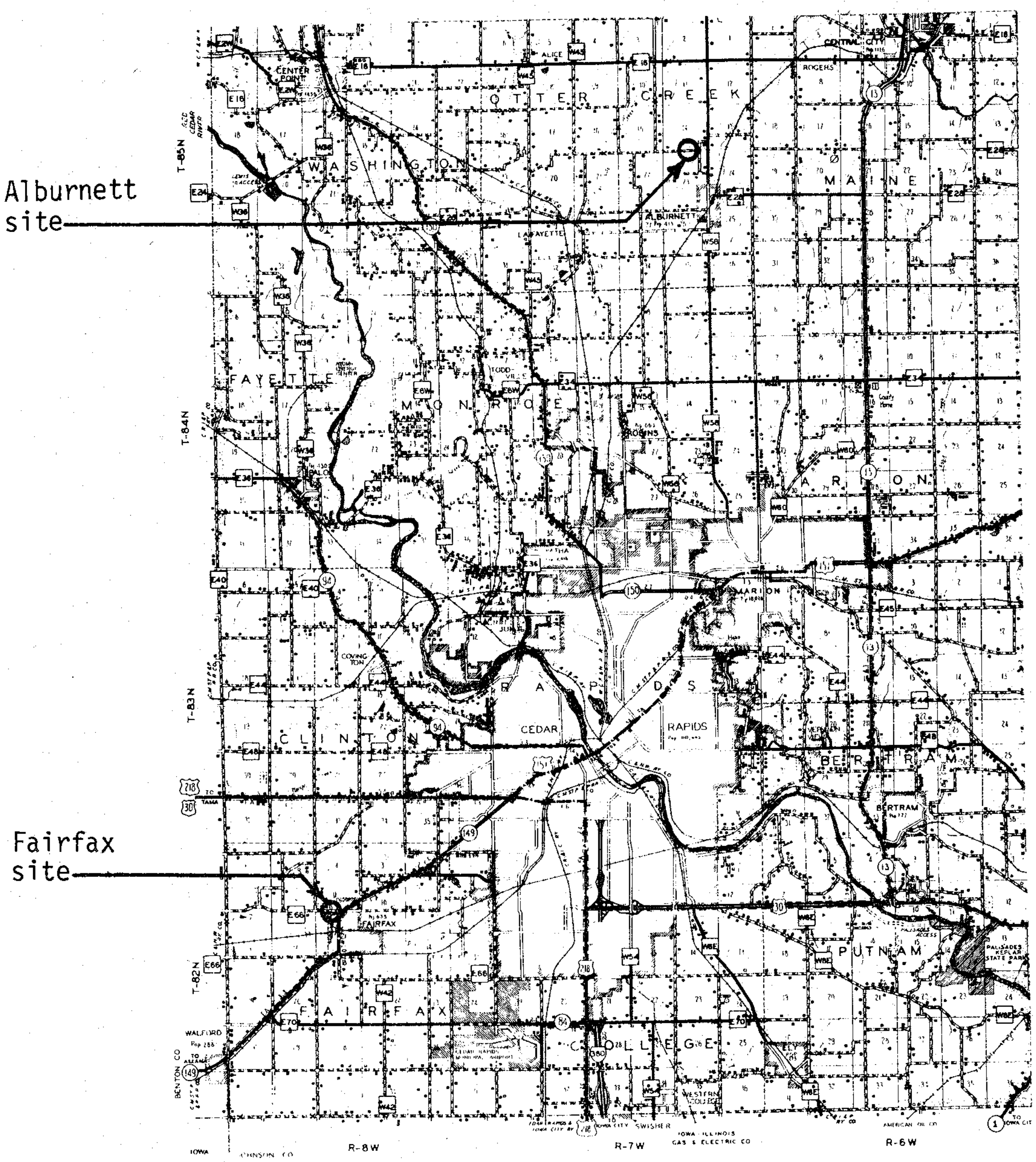

Figure 1. Map of Cedar Rapids, Iowa and surrounding area with locations of test section sites.

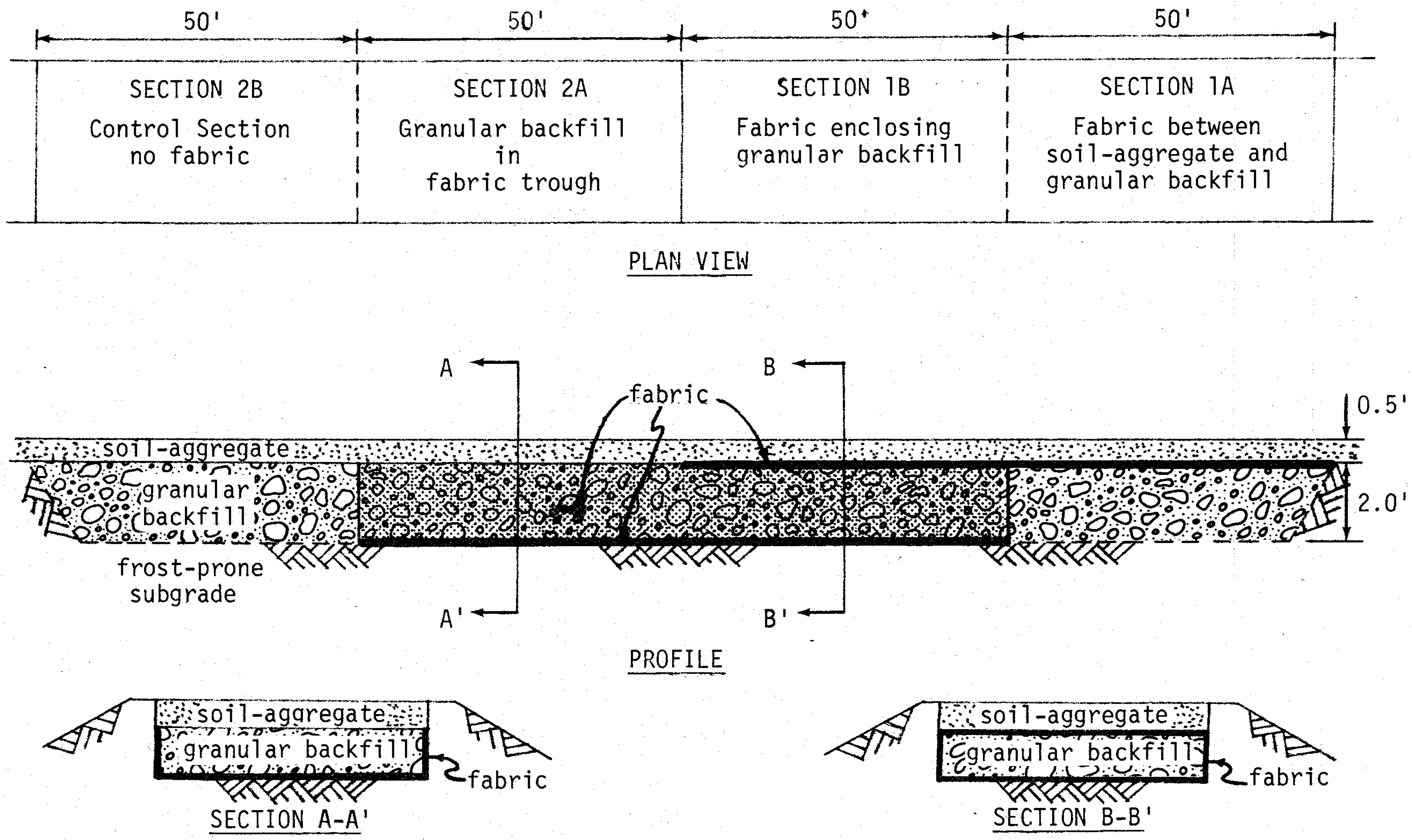

Figure 2. Mirafi fabric test sections 1A, 1B, 2A and 2B at Alburnett, Iowa.

placed directly on the backfill. Section 2B became the comparative control for sections 1A, 1B, and 2A since no fabric was used in its construction.

Alburnett sections 3 and 4 were constructed October 14, 1976 (Figure 3). The frost-susceptible subgrade was exposed by stripping off the soil-aggregate surface. The subgrade was then shaped with a blade grader and compacted with a sheep's-foot roller. In section 3 a layer of fabric was placed on the subgrade and the soil-aggregate was replaced on top of the fabric. Section 4 became the comparative control for section 3, as no fabric was used; the soil-aggregate was replaced on the subgrade in a manner identical to that of section 3. Following subgrade compaction, recording thermometer leads were installed in a one-inch deep trench in the subgrades of sections 3 and 4. A third thermometer lead was installed in such a manner as to record ambient air temperatures. Purpose of the thermometers was (a) to record any insulating effect the fabric may have had on the subgrade, and (b) to determine the number of freeze-thaw cycles occurring within the upper portion of the subgrade.

Sections 5 and 6 at Alburnett were constructed October 13, 1976 on frost stable subgrades as a means of overall comparative control between stable and frost-prone subgrades, Mirafi treated and non-treated systems (Figures 4, 5 and 6). These sections were constructed in the same manner as sections 3 and 4 where the exposed subgrade was shaped and rolled. A layer of fabric was placed on the subgrade of section 5,

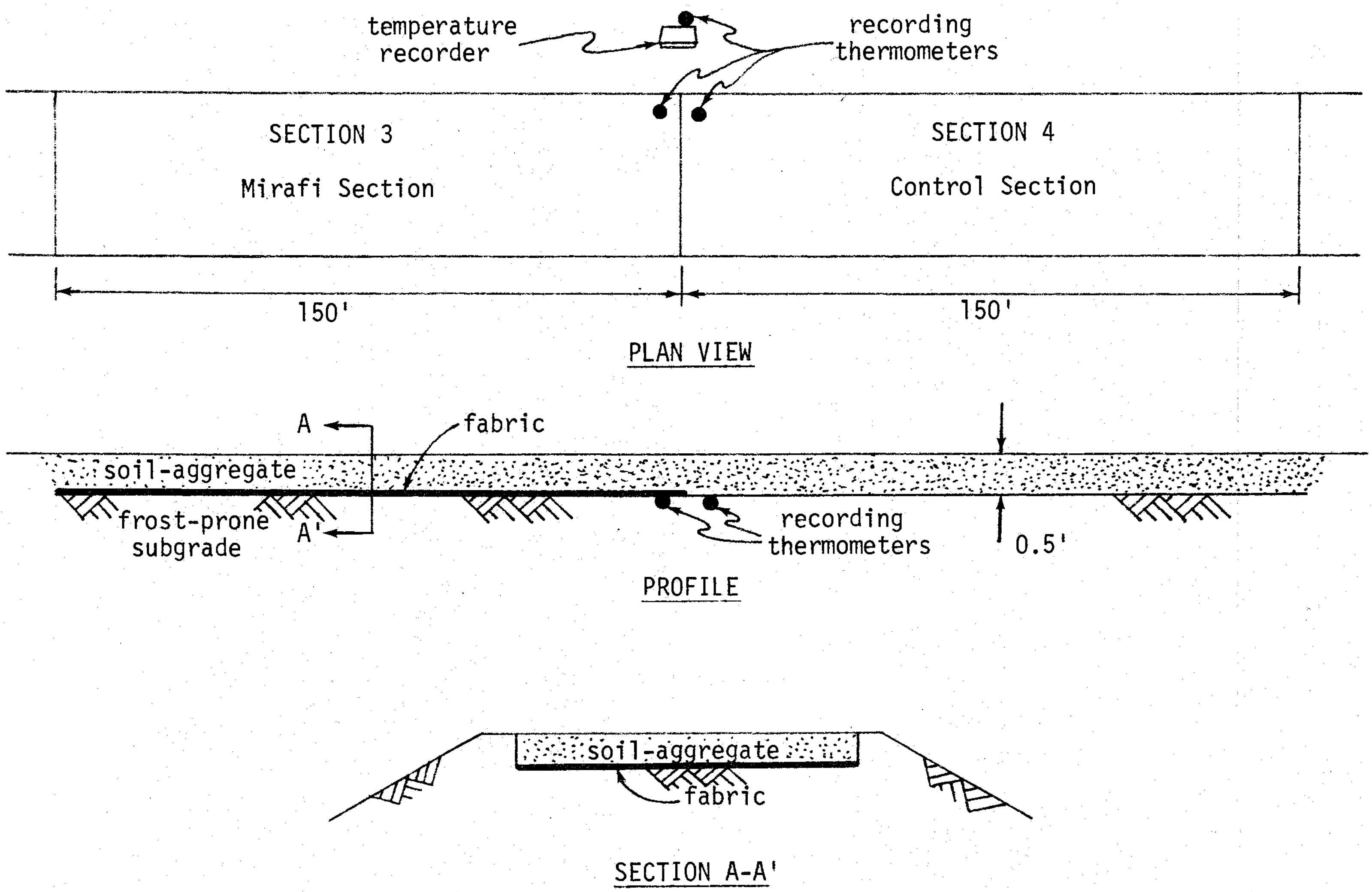

Figure 3. Mirafi fabric test sections 3 and 4 at Alburnett, Iowa.

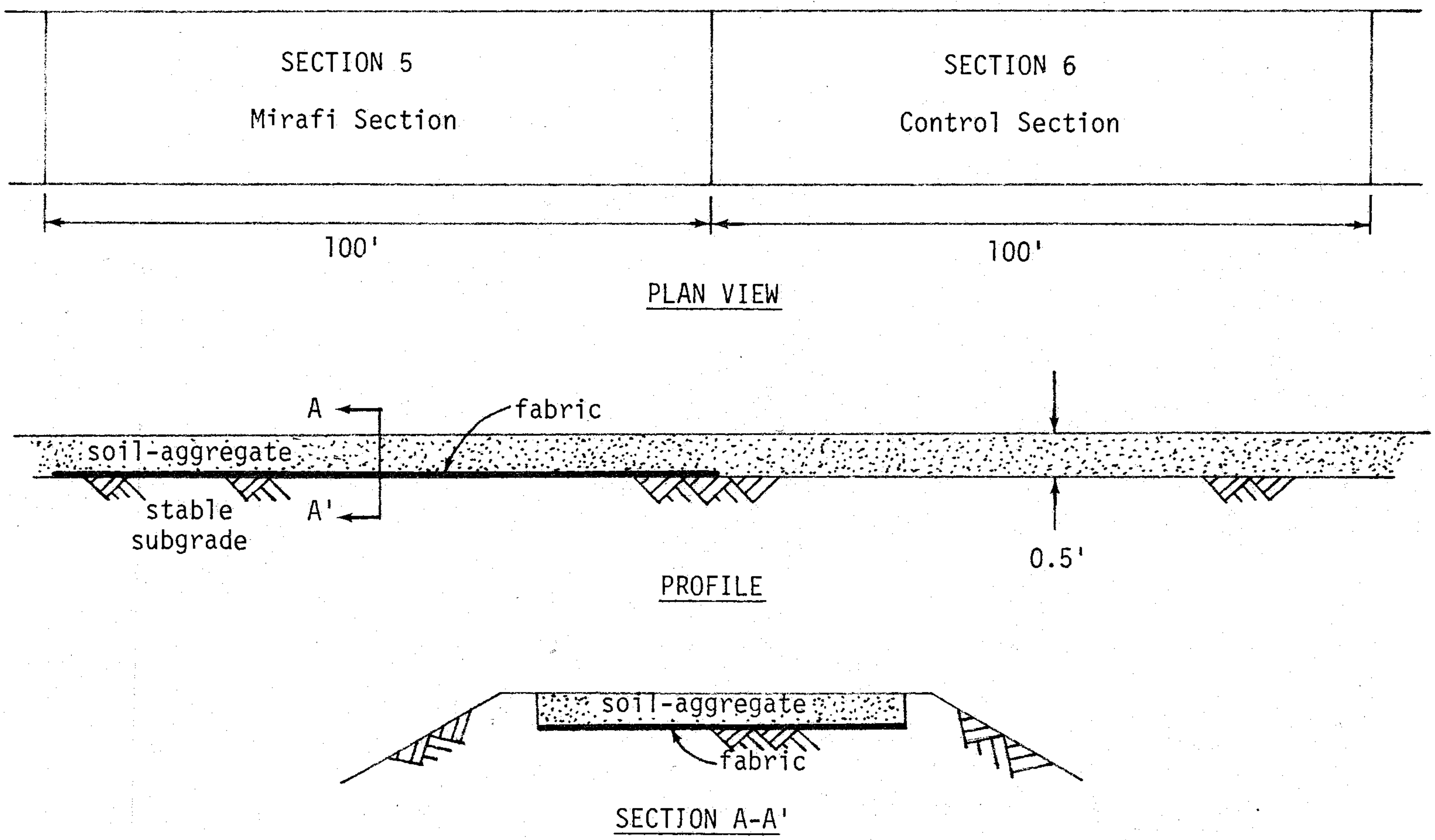

Figure 4. Mirafi fabric test sections 5 and 6 at Alburnett, Iowa.

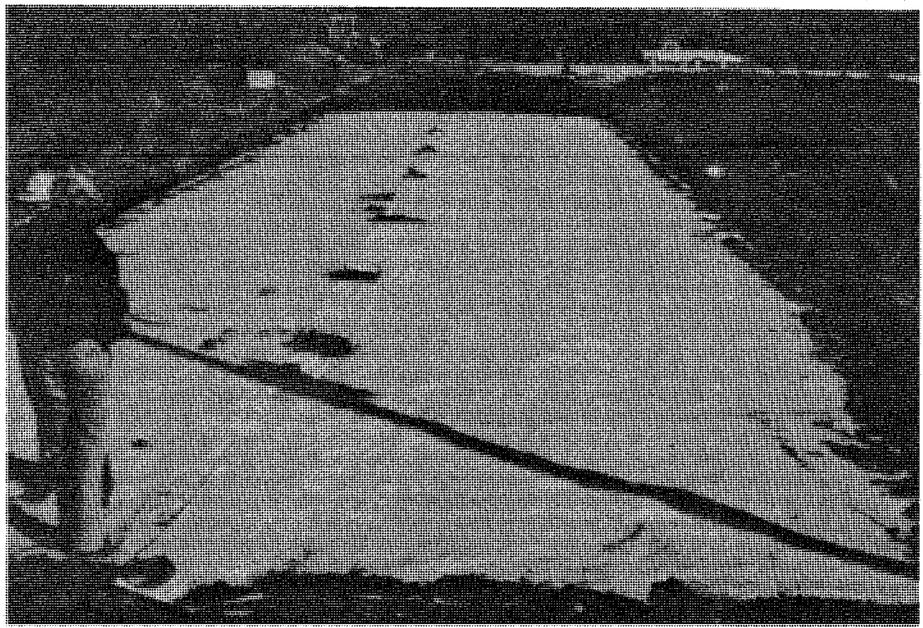

Figure 5. Mirafi 140 fabric placed on subgrade of Alburnett Section 5.

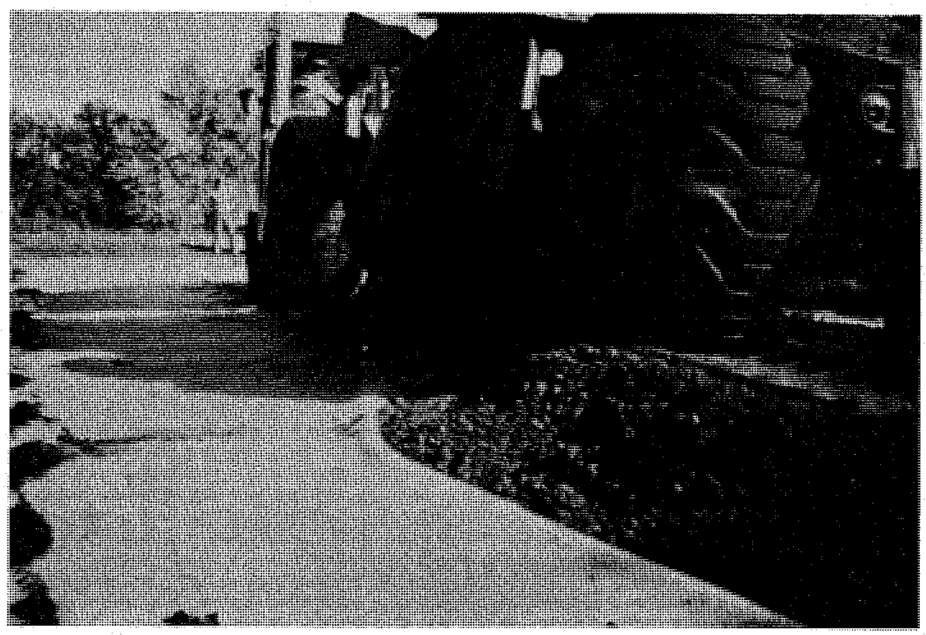

Figure 6. Soil-aggregate surface course being replaced on top of fabric.

and the soil-aggregate surface course was replaced and compacted on both sections. Section 6 also became the control for section 5.

All test sections at the Alburnett site were constructed by Linn County maintenance personnel using conventional County-owned equipment.

Test sections at the Fairfax site were constructed October 21, 1976 following a contracted geometrical change of the embankment that consisted primarily of ditch and shoulder widening, i.e., little or no change in longitudinal profile or elevations. Mirafi fabric was incorporated between the subgrade and a contracted macadam base/surface course by Linn County, Iowa State University, and Celanese personnel. Test sections 1 and 2, using the method of coring-out frost-susceptible subgrade followed by replacement with granular backfill, were eliminated by Linn County because of the additional expense that would have been incurred from force accounting a nearly completed contract.

Fairfax sections 3 and 4 were built in an area presumably containing frost-susceptible subgrade soils (Figures 7 and 8). The subgrade had been previously graded and compacted by the contractor. A layer of Mirafi 140 fabric was placed on the subgrade in section 3, and each section was then overlaid with 8 inches of an open graded macadam stone of 4 inch top size, section 4 being used as the control section. The macadam base was topped off with 4 inches of choke, consisting of 3/4 inch maximum size Class A road stone. Both the macadam base and choke stone were compacted by vibratory roller.

Sections 5 and 6 at the Fairfax site were built on frost-stable subgrades previously blade graded and rolled by the contractor,

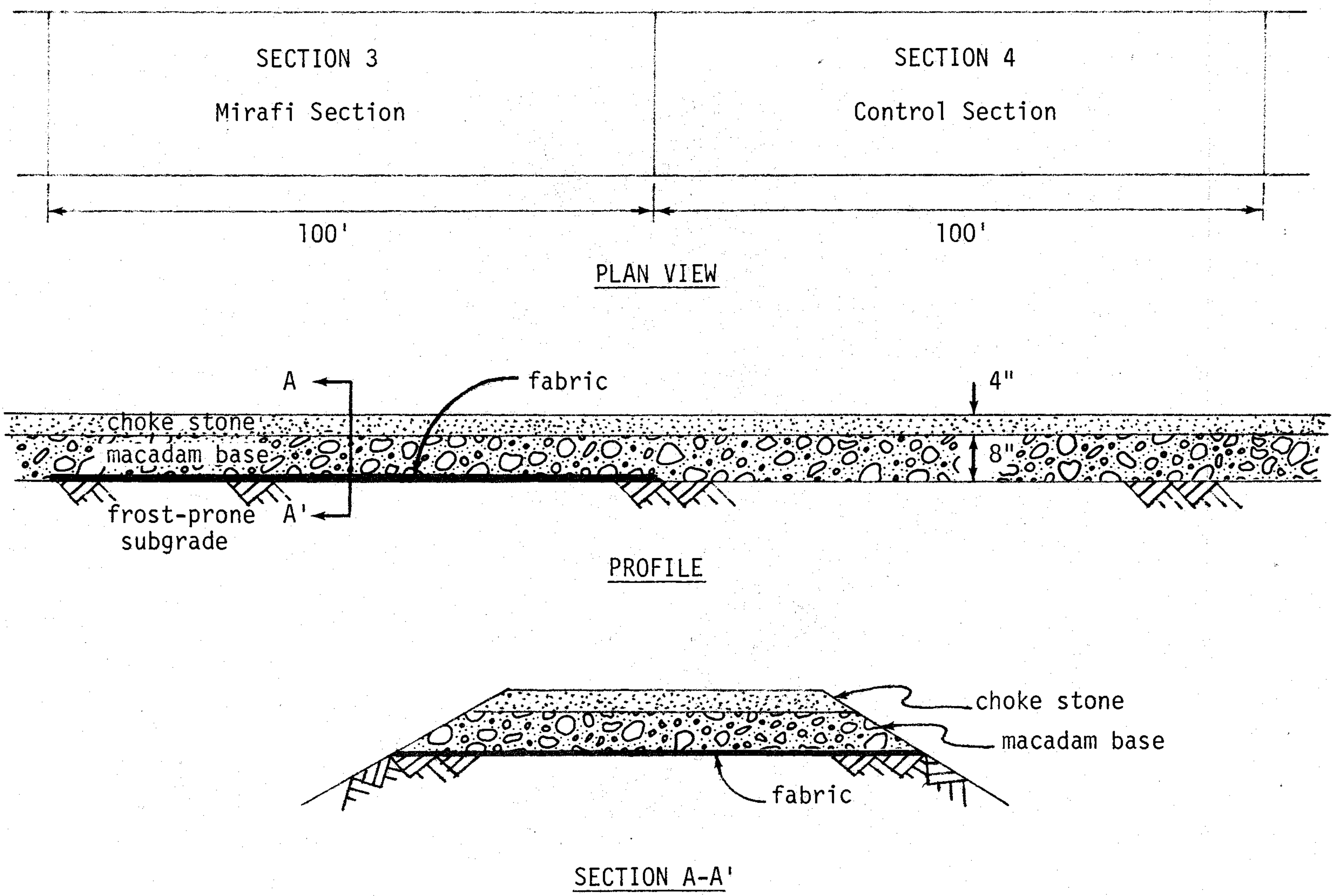

Figure 7. Mirafi fabric test sections 3 and 4 at Fairfax, Iowa.

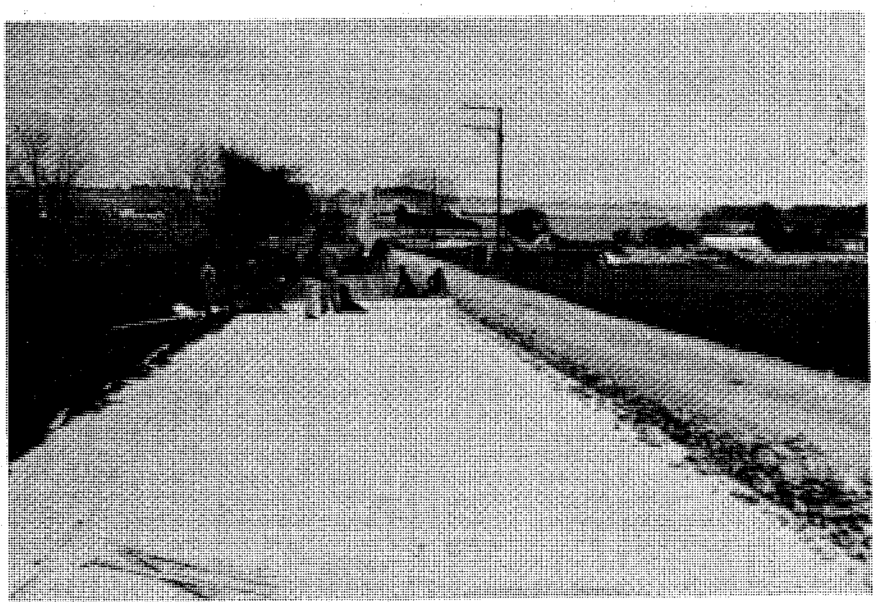

Figure 8. Mirafi 140 fabric being placed on subgrade of Fairfax Section 3.

Figures 9, 10, and 11). A layer of fabric was placed on the subgrade of section 5 and the open graded macadam base, with choke stone, was overlaid on both sections, section 6 being the comparative control for section 5. Sections 5 and 6 were thus the comparative control for stable and frost-prone subgrades, Mirafi treated and non-treated systems.

In June 1977, a seal coat wearing surface was applied to all Fairfax sections. In the Spring of 1978, a thin asphaltic concrete overlay was applied to each section at Fairfax.

The Alburnett and Fairfax sections provided a range of subgrade test and control sections underlying (1) a commonly used soil-aggregate surface or (2) a higher type base/surface system, respectively. For example, Alburnett sections 1A through 2B compared selected backfill treatment of the subgrade soil, without Mirafi, to sections containing the Mirafi as a membrane separation between surface and backfill, membrane separation between backfill and probable stable subgrade, as well as an encapsulated or Mirafi wrapped backfill. Sections 3 and 4 compare less costly methods of dealing with the frost-prone subgrade soil, with and without Mirafi, yet comparison of sections 1A through 4 entertained the potential comparison that normal backfill treatment may be unnecessary. Sections 5 and 6 provided the data needed to objectively analyze sections 1A through 4. Similar comparisons were available in the Fairfax sections, with the exception of the granular backfill process.

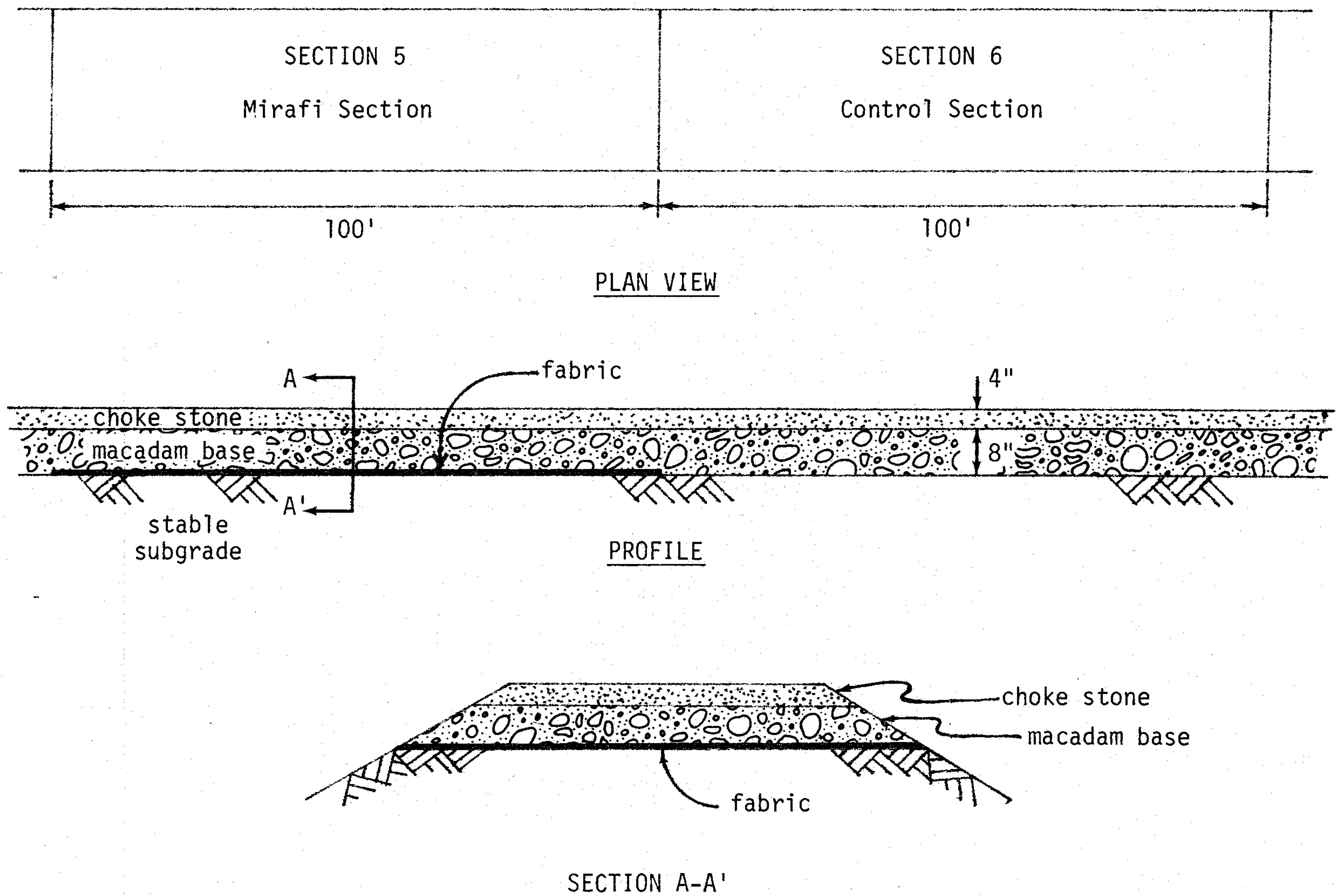

Figure 9. Mirafi fabric test sections 5 and 6 at Fairfax, Iowa.

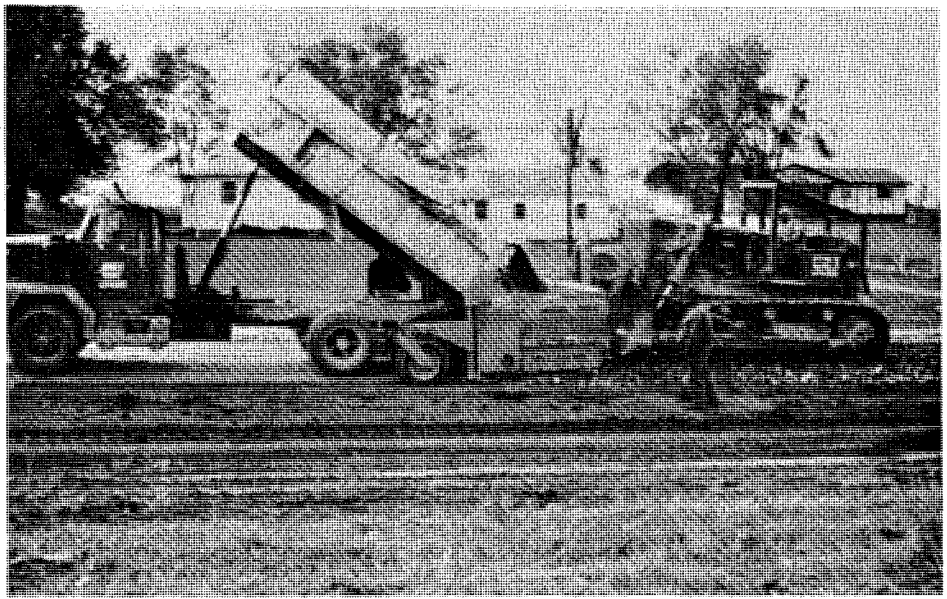

Figure 10. Procedure used to lay and shape macadam base.

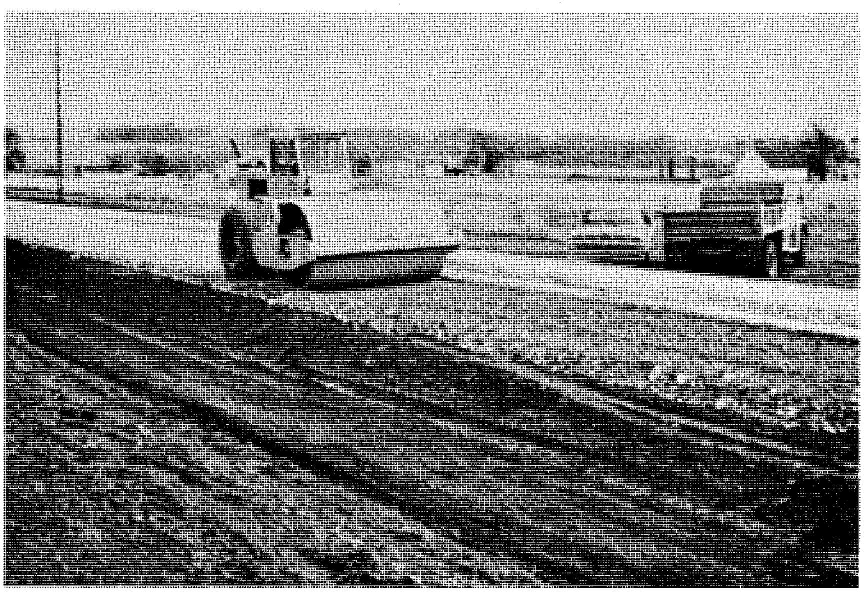

Figure 11. Vibratory roller compacting macadam base.

3. INVESTIGATIONS

Though the investigation reported herein was primarily a field performance evaluation, both laboratory and in-situ tests were conducted on all materials and within each test section location.

3.1. Properties of Mirafi 140 Fabric

The nonwoven fabric used in this study is trademarked as mirafi 140. The fabric is composed of two types of continuous artificial fibers, one a polypropylene, the other a polypropylene core sheathed in nylon [1].* The fibers are heat bonded into a random arrangement by fusion of the nylon sheathed fibers at their contact points. According to the manufacturers literature, the random structure thus produces a fabric having equal strength in all directions.

Table 1 is a summary of information on the fabric as obtained from the manufacturer [1]. The grab test procedure for determining the breaking load and elongation of fabrics is presented in ASTM standard D-1682 [2]. Data presented for the grab test are the maximum load sustained and the elongation at the breaking load. Grab test specimens are 4 inches in width and are clamped in jaw faces 3 inches apart, each having a width of 1 inch. Therefore the test determines the

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*Numbers in brackets indicate references listed at the end of the report.

Table 1. Information summary on Mirafi 140 fabric (from Celanese [1]).

Grab strength, wet	120 lb
Retention at $-70^{o}F$	100 %
Grab elongation, wet	130 %
Retention at $-70^{o}F$	40 %
Trapezoid tear strength	65 lb
Air permeability	250 cfm/ft^2
Minimum weight	4.1 oz/yd^2
Average thickness	30 mils
Fabric width	14 ft 9 in
Length per roll	328 ft
Average weight per roll	170 lb

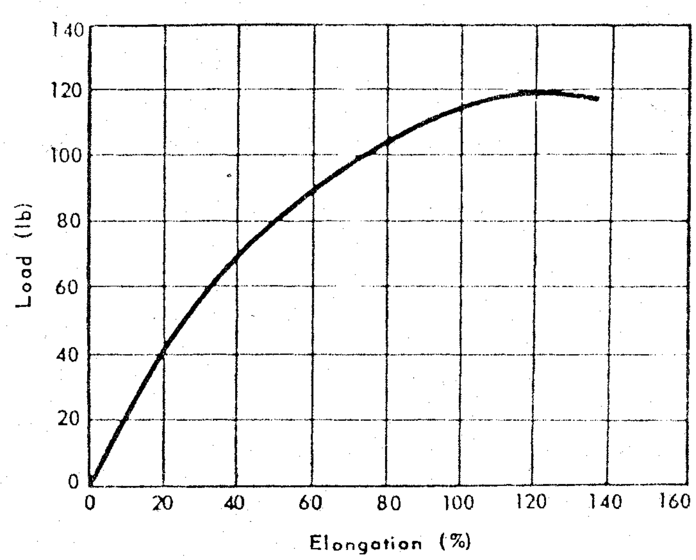

Figure 12. Load-elongation curve for Mirafi 140 fabric in grab test (from Celanese [1]).

"effective strength" of the fibers in a specific width, together with the additional strength contributed by adjacent fibers. Tested at -70°F, the fabric retains 100% of its grab strength, yet elongates only 40% of what it does when tested at the standard temperature. Figure 12 presents a typical load-elongation curve for Mirafi 140 in this test [1].

The trapezoid tear test procedure is presented in ASTM standard D-2263 [2]. Trapezoid tear strength of a fabric is determined primarily by the individual fibers actually gripped in the clamps, rather than by the full fabric structure. The force required to successively break individual fibers is thus the trapezoid tear strength. This test is useful in estimating relative tearability of various fibers.

The fabric is claimed to be rot-proof, mildew-proof, and insect-and-rodent-proof, and chemicals normally encountered in civil engineering applications produce no noticeable effect on the fabric [1].

3.2. Laboratory Investigation

Samples of the subgrade and soil-aggregate surface of each Alburnett section were obtained immediately prior to and during construction, sufficient for classification study purposes. Particle size, AASHTO and Unified classifications of the Alburnett samples are presented in Table 2, along with results of Atterberg Limits tests. Table 3 presents similar data obtained on base and subgrade samples from the Fairfax sections, all samples again being obtained either prior to or during

Table 2. Classification of Alburnett Test Sections Soil-Aggregate Surfaces and Subgrades, October, 1976.

	Subgrade			Surface		
Physical Properties	Sections 1 and 2	Sections 3 and 4	Sections 5 and 6	Sections 1 and 2	Sections 3 and 4	Sections 5 and 6
% Gravel (76.2-4.76 mm)	0	0	0	18	28	20
% Sand (4.76-0.074 mm)	52	34	39	63	56	59
% Silt (0.074-0.005 mm)	32	41	42	14	10	14
% Clay (<0.005 mm)	16	25	19	5	6	7
Atterberg Limits:						
Liquid Limit	26.0	37.1	29.1	-	-	-
Plastic Limit	17.4	19.5	18.1	-	-	-
Plasticity Index	8.6	17.6	11.0	NP[a]	NP[a]	NP[a]
AASHTO Classification	A-4(1)	A-6(9)	A-6(4)	A-1-b	A-1-b	A-1-b
Unified Classification	SC	CL	CL	SW	SP	SW
Uniformity Coefficient, C_u	170	410	78	55	95	92

[a]NP = Nonplastic.

Table 3. Classification of Fairfax Test Sections Base and Subgrades, October, 1976.

Physical Properties	Subgrades: Sections 3 and 4	Subgrades: Sections 5 and 6	Macadam Base
% Gravel (76.2-4.76 mm)	0	0	86 (101.6-4.76 mm)
% Sand (4.76-0.074 mm)	44	40	7
% Silt (0.074-0.005 mm)	39	19	7 Silt and Clay
% Clay (<0.005 mm)	17	21	
Atterberg Limits:			
Liquid Limit	22.8	33.6	-
Plastic Limit	15.6	18.6	-
Plasticity Index	7.2	15.0	NP[a]
AASHTO Classification	A-4(1)	A-6(6)	A-1-a
Unified Classification	CL	CL	GP
Uniformity Coefficient, C_u	131	114	17

[a]NP = Nonplastic.

actual construction.

As may be noted from Tables 2 and 3, properties of each surface or subgrade section show some variability. This is most evident in the Alburnett soil-aggregate surface materials and stems from localized variations within the surface course due to annual spreading and mixing operations, as well as to more frequent spot spread of aggregate within a deteriorating surface.

The frost susceptibility of soils is often related to silt content, silts being of such a particle size as to provide maximum capillary conductivity of water to the freezing front. However, other particle sizes play an important role in capillary moisture movement in that they may provide a poorly graded to well-graded particle size distribution, or mixture of particle sizes. The well-graded distribution retards capillarity, the poorly graded assists it. The subgrade soils at both sites could be classified as non-uniform due to their high uniformity coefficients. Also, more than three percent of the particle sizes of each subgrade soil was smaller than 0.02 mm, a particle size distribution criterion set by Casagrande [3] for considerable ice segregation to occur in a non-uniform soil, assuming natural freezing conditions with a sufficient water supply.

All of the subgrade soils at both sites classified as either A-4 or A-6 in the AASHTO system. Group A-4 represents frost-prone silty soils having a high and rapid capillary action. Alburnett sections 1A through 2B and Fairfax sections 3 and 4 each classified in Group A-4, and each was identified as frost-susceptible on the basis of past performance. Alburnett sections 3 and 4 classified as A-6, but were also identified by past performance as frost prone. An A-6 classification represents a group of soils in which clay content reduces the rapidity of capillary action from that of an A-4. The high group index and uniformity coefficient of Alburnett sections 3 and 4 soils, however, indicates a definite frost susceptibility.

All surface course materials as well as the Fairfax macadam base were classified in the A-1 grouping, generally ranging from coarse sand to stone fragments or gravel as the predominant material. Such materials have very low capillary conductivities and thus contribute little or nothing to roadway frost action.

Laboratory classification testing therefore correlated with observed freeze-thaw performances of each roadway section.

3.2.1. Freeze-Thaw Tests

The use of Mirafi 140 fabric for stabilizing frost-susceptible roads is based on the concept of strengthening of the soil materials to withstand thaw-softened support, rather than on reducing heave that occurs during freezing of a subgrade. The amount of heave occurring in a soil provides an indication of strength of the thawed soil. The more a soil heaves, the more water is imbibed during freezing, and more water present during thawing lowers the soils' thawed strength. Therefore a limited investigation was undertaken to determine if inclusion of Mirafi in a soil reduced the amount of heave which might occur during freezing.

A modification of the Iowa freeze-thaw test, developed by George and Davidson [4] for determining freeze-thaw durability of stabilized soils, was used to measure heave of both untreated and Mirafi-treated soil specimens. Basically this test duplicates field conditions of freezing from the top while water is available at the bottom of the specimen for upward capillary moisture movement. In this procedure, 4 inch diameter by 4.56 inch high soil specimens (1/30 cu ft) are molded in three equal layers at standard ASTM D698 [2] density and optimum moisture content. For the Mirafi-treated specimens, single-thickness 4 inch diameter discs of fabric were inserted between compacted layers during molding. Following molding, soil specimens were placed in Plexiglas holders which fit inside insulated thermos flasks, filled to a predetermined depth of water for contact with the specimen base, providing an available source of capillary water. All specimen height (heave) measurements were performed with the specimen in its holder and the holder in its flask in order to avoid disturbing test components. Initial height measurements served as a datum for all succeeding measurements following freezing or thawing.

Flasks containing each specimen were placed in a freezer maintained at 20 ± 2°F, a temperature equivalent to the average minimum air temperature in Iowa during winter months. To simulate in-situ freezing conditions, the specimens were frozen from the top down, while the water in contact with the specimen base was maintained at approximately 35°F by insulation of the flask; the temperature was controlled with a small light bulb attached to the base of the specimen holder, regulated by a variable voltage source. After 16 hours of freezing, specimen flasks were removed from the freezer, height measurements taken, and thawing was allowed for 8 hours at room temperature. This constituted one cycle, and was repeated at least 10 times. Elongation, or change in specimen height following freeze or thaw, was expressed as a percentage of the original 4.56 inch height.

The soil used in this series of tests was an S.C.S. identified Shelby series derived from glacial till, like the soils of Linn County. Linn County test section soils were not used in these tests in order to conserve limited samples and because in freeze-thaw tests more drastic heaves are known to occur with the Shelby than initial testing indicated would occur with the test site samples. Physical and mineralogical properties of this soil are noted in Table 4.

In the first test series, a disc of Mirafi fabric was molded beneath the top compacted layer of the specimen. A control specimen (no fabric) was also produced. Percent elongation measured during eleven freeze-thaw cycles on both specimens is shown in Figure 13. Each specimen expanded similarly during the first two freeze-thaw cycles. This was apparently the result of capillary water seeking the freezing front during freezing, coupled with continued elongation during thawing due to swelling of the montmorillonitic clay upon absorption of water. Additional elongation occurred during the third freeze, but both specimens contracted during the thaw cycle, indicating the

Table 4. Physical and mineralogical properties of Shelby series soil

Property	Shelby Soil Knoxville, Iowa
Textural Composition, %	
Gravel (4.76 mm)	0.0
Sand (4.76 - 0.074 mm)	2.6
Silt (0.074 - 0.005 mm)	57.4
Clay (0.005 mm)	40.0
Physical Properties:	
Liquid Limit, %	54
Plastic Limit, %	32
Plasticity Index, %	22
Specific Gravity	2.70
Standard Dry Density, pcf	100.0
Standard Optimum Moisture Content, %	20.9
Classification:	
Textural	Silty Clay
AASHTO	A-7-5 (15)
Unified	MH
Predominant Clay Mineral:	Montmorillonite

probability of complete expansion of the clay lattices coupled with partial gravitational drainage during thawing. Through cycle eleven, the untreated specimen continued elongating, generally heaving 8 to 10% each freeze cycle. Maximum elongation of the untreated control

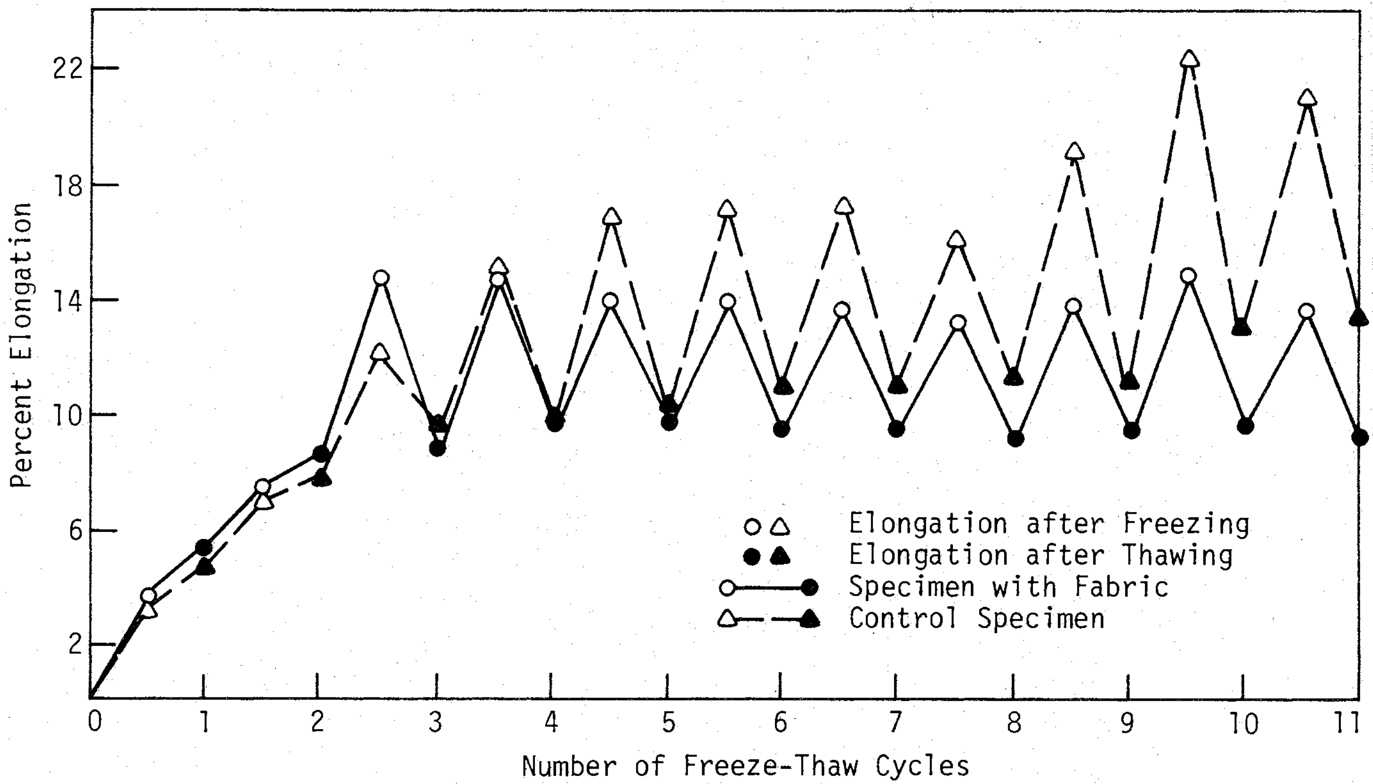

Figure 13. Freeze-thaw elongation test results, Shelby soil.

specimen was 23% of its original height. Through cycle eleven, the specimen containing Mirafi stabilized at about 14% elongation after freezing, and between 9 and 10% after thawing. The treated specimen thus heaved slightly greater than 4%, or about one-half as much as the untreated.

A second test series was performed on the Shelby soil using techniques identical to those described in the previous paragraph. Results confirmed the same relative relationships between control and treated specimens.

Since location of the fabric within a specimen profile might influence elongation, a third series of freeze-thaw tests was conducted on the Shelby soil. The series included (1) a Mirafi disc molded between the top and mid-layers, (2) a Mirafi disc between the mid and bottom layers, (3) a disc of fabric at each of the third-point locations, and (4) control. Figure 14 presents the average elongation results of this series. All specimens expanded through the fifth freeze-thaw cycle. During the sixth thaw, all treated specimens indicated the advent of gravitational drainage, while the clay in the control specimen continued to expand. Expansion of the control specimen clay lattice appeared complete following freeze cycle seven, since a decrease in elongation appeared at the seventh thaw. Although differences between Mirafi-treated and control specimens were evident, magnitude of heave was generally not as pronounced as that observed in the first and second test series.

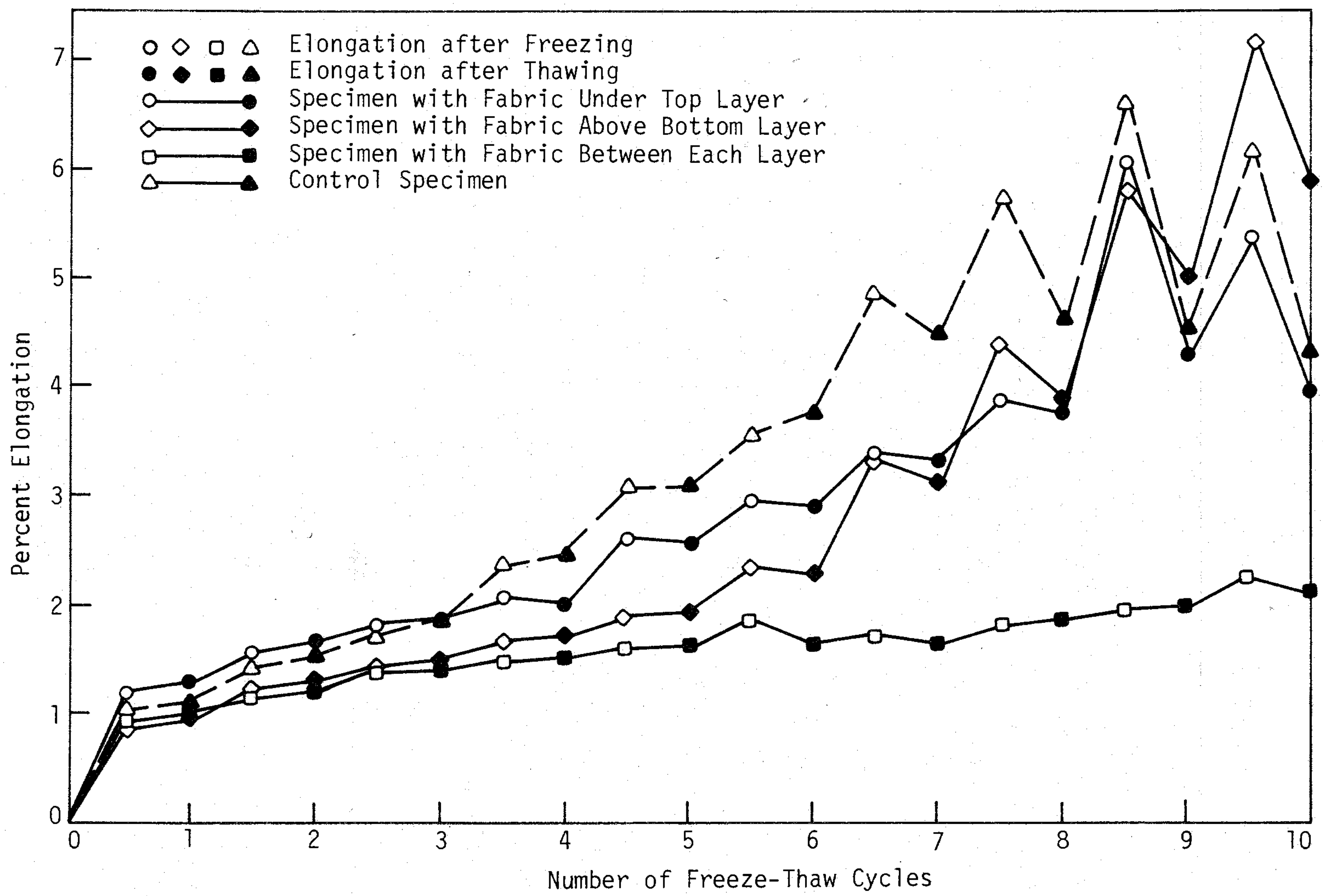

Figure 14. Freeze-thaw elongation test results, Shelby soil.

Regardless of location of the single disc treatment, similar elongation characteristics were observed for specimens containing only one layer of fabric. Two fabric discs, each placed at the specimen third points, significantly reduced expansion as compared with any of the other treatments and/or control. After an initial elongation of about 2%, additional expansion during continued freeze-thaw cycles amounted to less than 0.5%. All specimens, except that with the fabric under the top layer, were recovered after the freeze-thaw tests for use in stability evaluation.

When compared with the control, the lower heaving observed with Mirafi-layered specimens may have resulted from the fabric providing a partial cutoff of capillary water to the remainder of the specimen. Since Mirafi 140 fabric was designed for use as a filter, its average pore size is equivalent to that of a medium fine sand, a material generally anticipated to have a lower capillary conductivity than the silty clay Shelby series soil used in this study. In addition, surface attraction between the fabric and water should be considerably less than that between the Shelby soil and water, thus inhibiting capillary water rise.

A small amount of lateral bulging may occur during the growth of ice lenses within a container of the size used in this freeze-thaw study. With the inclusion of fabric in a specimen, any reinforcement provided may prevent part of this bulging and inhibit growth of ice lenses, which in turn may tend to result in smaller elongations.

3.2.2. Stability Tests

A limited investigation was undertaken to evaluate a soil's strength parameters and stress-strain characteristics due to inclusion of Mirafi within a soil specimen. The patented Iowa K-Test allows determination of various shear and stability parameters from a single specimen [5]. The test, which has been used extensively in other research efforts [6, 7], utilizes standard Proctor 1/30 cu ft. [4] laboratory specimens normally thrown away after density measurements. Such specimens are unacceptable for triaxial tests since they emphasize end restraint due to a height-to-diameter ratio approaching 1.1. Unconfined compression tests of Proctor specimens are also somewhat unrealistic, especially for sandy soils. However, these same specimens may be utilized in the Iowa K-Test apparatus, which provides a continuous measure of lateral stress as a function of vertical stress. One of the important characteristics of the test is that as the vertical load increases, the lateral stress continuously increases through a variable horizontal restraint - a characteristic similar to some forms of a far more costly stress-path triaxial test, yet much more realistic than commonly used unconfined compression testing. In addition, the test can be performed on each specimen from a moisture-density and/or freeze-thaw test for evaluation of effect of moisture. The time required for testing and calculation of elastoplastic parameters is about the same as that needed to run and evaluate an unconfined compression test, yet the Iowa K-Test provides a very high degree of correlation with more sophisticated triaxial shear tests.

Shelby series soil specimens were prepared in a manner identical to that used for the freeze-thaw tests. Discs of Mirafi 140 fabric were inserted between the compacted layers during the standard ASTM D698 compaction process [2].

To conduct the K-Test, the 4 inch diameter soil specimen is placed in a split mold and subjected to applied vertical loads. Vertical displacement of the specimen and lateral expansion of the mold are measured. Vertical loads are applied through a proving ring (or other load measuring devices) and the radial stresses produced on the soil specimen are related to the mold's expansion through a calibration factor. Vertical and lateral (radial) stresses and strains of the soil specimen are thus obtained. By monitoring these values during a test, continuous evaluation of k, the ratio of horizontal to vertical stress, ϕ, internal friction angle of soil, c, the soil's cohesion parameter, and E_v, the soil's vertical deformation modulus, may be obtained.

Results of the Iowa K-Tests are presented in Table 5. Three specimens were tested with two layers of fabric, one each at the specimen third points; six specimens were tested with one disc of fabric molded between the top and middle compacted layers; and, for comparative purposes, seven control specimens were tested. In addition, the K-Test was also performed on similiar specimens from the third series of freeze-thaw tests, following completion of 10 freeze-thaw cycles. For the sake of brevity, Table 5 presents only the mean values of all tests.

Data indicated that incorporation of fabric during specimen compaction created a slight reduction in dry density, even with very similar

Table 5. Moisture-density and Iowa K-Test parameters for Shelby series soil specimens, with and without Mirafi 140 fabric.

Treatment	Moisture Content, % Dry Soil Wt.	Dry Density, pcf	Vertical Deformation Modulus, E_v, psi.	Cohesion c, psi	Angle of Internal Friction, ϕ, Degrees	Lateral Stress Ratio, k
Control - Untreated:						
Near standard M-D	20.6	98.2	2138	14.3	22.7	0.337
After freeze-thaw	30.9	90.7	1752	8.2	0.0	0.777
One fabric layer:						
Near Standard M-D	20.8	96.3	2032	14.8	23.7	0.317
After freeze-thaw	31.9	89.8	1850	9.9	5.1	0.648
Two fabric layers:						
Near standard M-D	21.0	95.7	1622	16.4	23.4	0.317
After freeze-thaw	29.3	91.8	1784	10.1	8.7	0.574

specimen moisture contents. This condition may result from the load spreading capability of the fabric, creating a more even distribution of the compacting ram force, as was also evidenced in the Benkelman beam field tests. In addition, a soil must be sheared to be compacted; i.e., it must develop internal shear surfaces. if the fabric tended to confine these surfaces, it would thus contribute to the lower densities.

Since the quantity of K-Test data in this study is limited to only one soil, and although any conclusions drawn herein may be somewhat speculative, even with the obvious reduction in densities and increase in moisture content following freeze-thaw, it is evident that the fabric provided at least some improvement in most of the stability properties.

Vertical deformation modulus, E_v, is the ratio of vertical stress to vertical strain. Under standard moisture-density conditions only, values of E_v were slightly reduced through inclusion of one horizontally layered Mirafi disc; yet two layers of fabric markedly decreased E_v. The latter was apparently due to the high values of vertical strain associated with the two-layer fabric specimens during testing, thus contributing to the higher compressibility and accompanying lower values of E_v for these specimens. After 10 cycles of freeze-thaw, relatively little change of E_v was noted in any of the specimens.

Near standard optimum moisture and density conditions, there was a minor improvement in both cohesion and angle of internal friction of the treated specimens. Following freeze-thaw, slight improvements in cohesion were evident, coupled with a good increase in friction angle, though densities were reduced and moisture contents had increased significantly. The increase in cohesion may be a result of the tensile strength added by the fabric, with two layers showing the greatest average improvement. The zero friction angle obtained with the control is typical of saturated clays; all specimens, including the treated ones, were near 100% saturation at the conclusion of freeze-thaw testing. The higher friction angles obtained with the treated specimens appear to indicate that the fabric tended to confine the propagation of continuous shear planes within the specimens. Use of the $c - \phi$ parameters in a bearing capacity analysis would generally indicate improved support, particularly where two fabric layers were utilized. Bearing capacity of a single fabric layer would probably be significantly improved through use of a fabric having greater load versus lesser elongation properties than the Mirafi 140.

Lateral stress ratio, K, is an indicator quality of roadway material rutting potential. Values of K should never exceed 1.000, and the smaller the K value, the less rutting potential and greater lateral stability are available within a material. Only a slight reduction in K was obtained within the Mirafi treated specimens near standard moisture-density conditions. A larger magnitude of reduced K was obtained with increased number of Mirafi discs following freeze-thaw. Both test and treatment conditions reflect radial reinforcement provided by the horizontal discs. However, the magnitude of reinforcement may also be indicative of the large amount of elongation provided by the Mirafi 140; a fabric having greater load versus lesser elongation properties would probably produce lower values of K.

In analyzing the composite effect of fabric-reinforced Shelby soil, properties of the fabric should be considered alone. It is a composite of thermo-plastic fibers that are not very resilient and have a low tensile deformation modulus. The tangent modulus at 120% elongation is only about 3,000 psi. Since slippage may occur at the soil-fabric contact, the effective modulus of the fabric is probably lower than 3000 psi.

Figure 15 indicates that the vertical modulus, E_v, decreased with an increased number of fabric layers for specimens not subjected to freeze-thaw. After the soil had been degraded through capillary moisture absorption during freeze-thaw, the presence of fabric provided a slight improvement, indicating the importance of the properties of the composite constituents. First, the effective fabric modulus, which includes its ability to bond, may have been small compared to that of the soil, creating a situation where the soil may have reinforced the fabric. When the soil modulus was reduced by freeze-thaw action, presence of the fabric caused a slight improvement, indicating the roles of the individual moduli may have been reversed, with the net result being some reinforcement of the composite.

Another factor that plays an undetermined though important role in reinforcement of the composite is the degree to which fabric properties

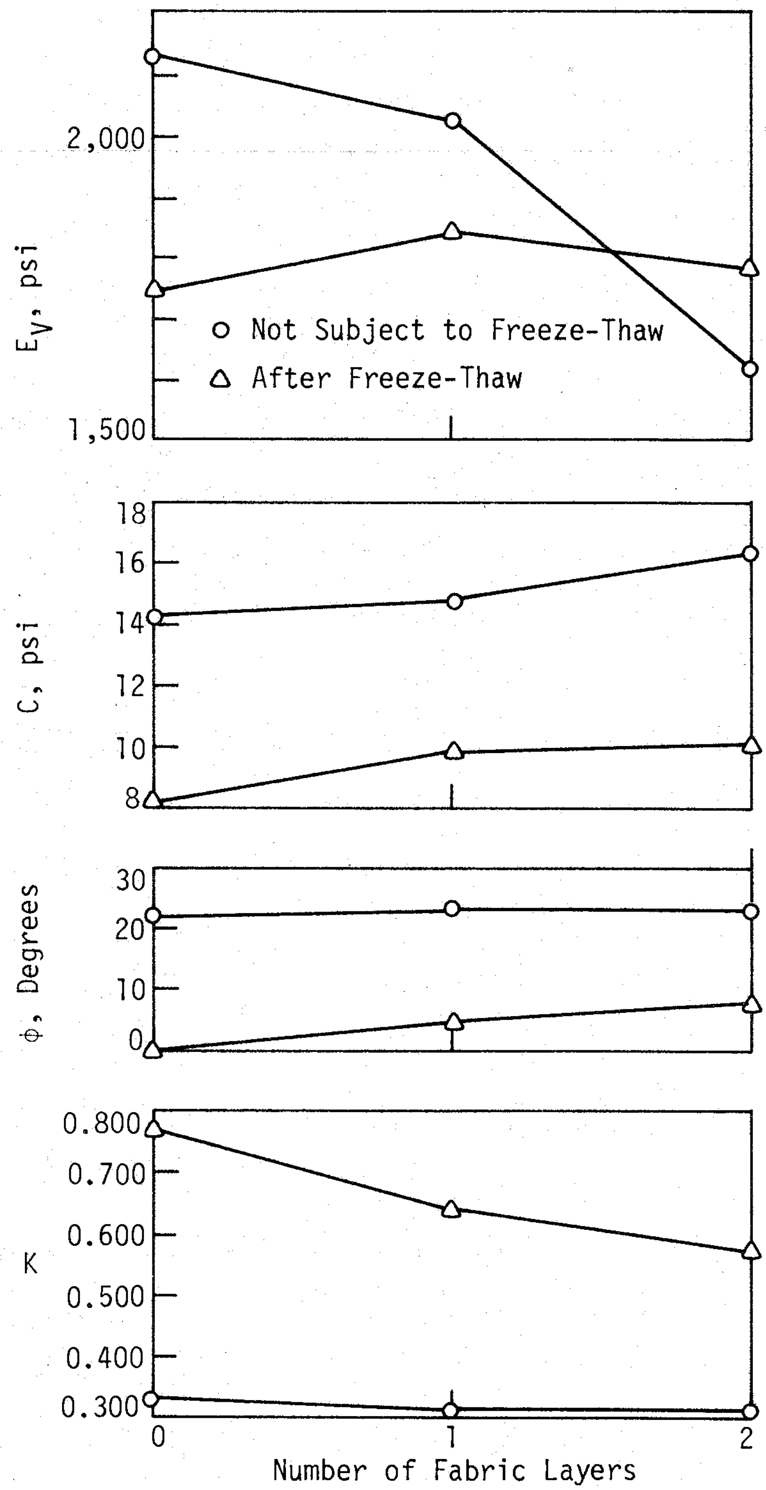

Figure 15. Material properties vs. fabric content.

are transferred to the soil. This of course depends on the bond achieved at the interface between the two materials. Casual observation indicated that at least for the fine-grained Shelby soil, bonding may have been achieved through partial intrusion of the soil into the fabric mesh. The more plastic the soil, the better the probable bond. This may explain why the values of E_v were nearly the same for both the one- and two-fabric layer specimens following freeze-thaw; some level of bonding was established, and hence a limiting amount of reinforcement was provided.

As might be expected from elastic theory, the influence of horizontally oriented fabric reinforcement on the vertical deformation modulus would be reduced or tempered by the effect of Poisson's ratio (ν). Since the fabric is oriented to resist only lateral deformations, its effect manifested in the vertical direction would be reduced by twice Poisson's ratio, times the lateral strain (i.e., for $\nu = 0.25$, the effect of horizontal reinforcement would be reduced by a factor of 0.5 when measured vertically). This might explain, at least in part, the lack of complete elongation control realized from the fabric in the freeze-thaw specimens.

With one exception, the parameters indicating shear strength at limiting equilibrium (c and ϕ) were more sensitive to the effect of reinforcement both before and after subjection to freeze-thaw. Keeping in mind that c and ϕ represent the shear stress on a plane oriented at an angle of $45 + \phi/2$ degrees with the horizontal at impending failure and large deformation, the fabric is oriented so that it can have considerable influence on these parameters (Figure 16). The one inconsistency in the results of Table 5 is that fabric reinforcement did not appreciably influence the friction angle for the control specimens not subjected to freeze-thaw, a condition which may relate to soil-fabric bonding.

Lateral stress ratio, k, can be derived from both c and ϕ and therefore represents a combination of both factors. Thus, the experimental results showing a decrease in k with reinforcement, indicating less load transfer in the lateral direction, are consistent with the preceeding analysis. Even though the lateral stress ratio is a reflection of basic plasticity parameters, it appears to offer a convenient analytical representation of rutting potential of soil and road materials containing a fabric.

3.3. Field Investigations

3.3.1. In-Situ Moisture and Density Tests

In-situ moisture and density tests were performed in October 1976 within the subgrade of each test and control section after the soil-aggregate surface had been removed and the subgrade had been shaped and compacted, but before placement of the fabric and/or replacement of the surface course. Some difficulties were encountered in keeping ahead of construction operations, resulting in only one test rather than the minimum of two initially prescribed for several of the sections.

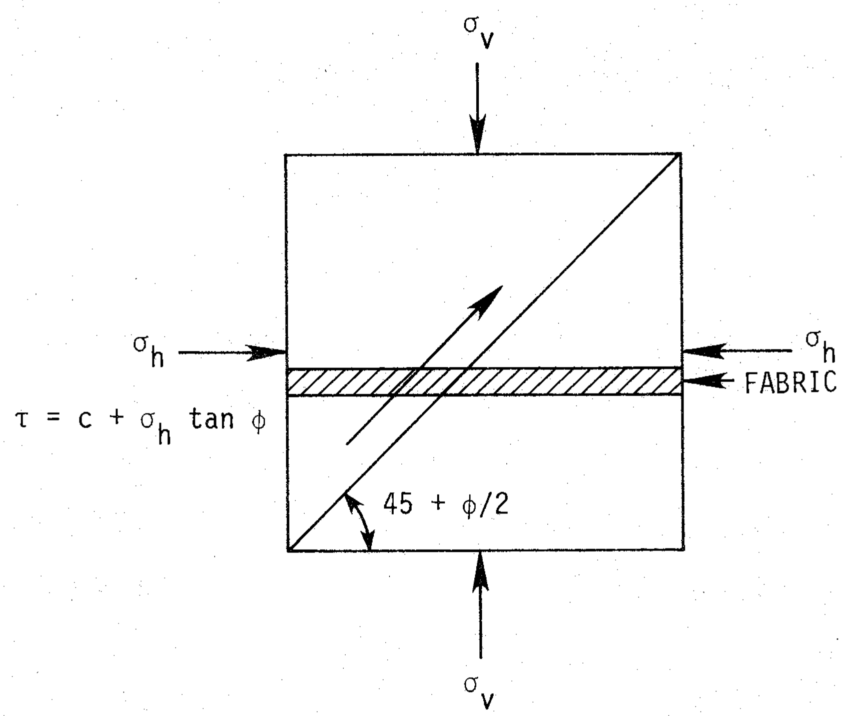

Figure 16. Stress diagram for element of fabric reinforced soil.

Assistance was provided by Linn County personnel in order to hasten construction operations.

All density tests were performed with a rubber balloon volumeter in accordance with ASTM designation D-216 [2]. Moisture content was measured by the oven-dry moisture-weight loss method, ASTM procedure D-2216 [2].

Tables 6 and 7 present results of the in-place moisture-density tests at the Alburnett and Fairfax sites respectively. As might be anticipated, considerable variation of both density and moisture content existed within the subgrades of both sites. Such variations occur in most county or local roads because of variability in transportation and evaporation rates of materials, in subgrade drainage, and in material properties due to use of locally available materials only.

Clayey soils retain moisture longer than coarser grained soils. This is reflected in moisture contents obtained for the subgrade materials of Alburnett sections 3 and 4. As noted in Tables 2 and 6, this material had the highest clay and moisture contents of any of the test section subgrades.

Subgrade drainage is highly dependent on ditch conditions. For example, the Alburnett site is fairly flat with shallow ditches. Moisture contents of the Alburnett site subgrades varied from 7.2 to 27.6% by dry soil weight, accompanied by similarly wide variations in dry densities. At the Fairfax site, ditches had been widened and deepened as previously noted. In addition, the Fairfax site slopes

Table 6. Dry Densities and Moisture Contents of Alburnett Test Section Subgrades at Time of Construction, October 1976.

Location	Dry Density, pcf	Moisture Content, % dry wt.	Remarks
Mirafi Section 1A	107.5	17.7	Linn Co. data
Mirafi Section 1B	113.3	9.1	Linn Co. data
Control Section 2B	90.8	24.6	Linn Co. data
Mirafi Section 3	102.8	15.3	
	99.4	27.6	
Control Section 4	105.3	18.8	
	112.1	13.8	
Mirafi Section 5	97.2	17.1	
	86.2	12.3	
Control Section 6	92.0	7.2	
	102.4	15.1	

Table 7. Dry Densities and Moisture Contents of Fairfax Test Section Subgrades at Time of Construction, October 1976.

Location	Dry Density, pcf	Moisture Content, % dry wt.	Remarks
Mirafi Section 3	133.3	11.9	
	119.5	11.8	Linn Co. data
Control Section 4	114.3	8.6	
Mirafi Section 5	132.1	10.7	
	141.1	6.8	Linn Co. data
Control Section 6	128.8	13.7	

from west to east within the test section locations. Moisture contents of the Fairfax subgrades varied from 6.8 to only 13.7% by dry soil weight, thus showing much less variation than the Alburnett site.

Subgrades of the Fairfax site, consisting predominantly of the old soil-aggregate surface course, were generally less plastic and contained somewhat larger particle sizes than the Alburnett site subgrades, as indicated in Tables 2 and 3. Such soil properties tend to account for the higher subgrade densities within the Fairfax site.

Lower dry densities were obtained for sections 5 and 6 than for other sections of the Alburnett site, yet much lower moisture contents were also observed, possibly accounting for the apparent stability of these sections. At the Fairfax site, comparable moisture contents were obtained for all sections, yet much higher dry densities were obtained in sections 5 and 6, than in sections 3 and 4, a condition potentially contributing to the stability visually observed in the past within the area of sections 5 and 6.

In-situ moisture and density tests were again performed within the subgrades of each test and control section in early June 1978, approximately 20 months after construction. This point of time was selected for the following reasons: (a) the preceeding winter was relatively severe in regard to depth of frost penetration; (b) all frost was out of the subgrade; (c) late spring/early summer temperatures had not yet set in to provide rapid transpiration and evaporation of roadway moisture;

and (d) the ground water table was still relatively high, preventing any excesses of gravitational flow from the subgrades. Because of those conditions, frost boiling had been extremely severe throughout April and May 1978 and was just beginning its healing process.

Results of these tests are presented in Tables 8 and 9. With the exception of Alburnett sections 3 and 4 and Fairfax sections 5 and 6, no consistent trends between fabric and control sections or between times of testing were observable. Comparison of Alburnett sections 3 and 4, Tables 6 and 8, indicate at least a limited degree of moisture-density stability within Mirafi section 3 over the 20 months since construction and following two seasonal frost heave/boil cycles. However, control section 4 indicated a significant decrease in subgrade density, coupled with a severe increase in moisture content following the two seasonal cycles.

As previously noted, Fairfax sections 5 and 6 were presumably on frost-stable subgrades. Comparison of data for these sections within Tables 7 and 9, however, indicate significant decreases in density with accompanying increases in moisture contents. Comparison of the in-situ data between sections 3-4 and 5-6 indicate a probable loss of stability within sections 5 and 6. This loss, however, was only partially reflected in other in-situ data performed on the roadway surface overlying the macadam base and may be due at least in part to the rigidity of the base bridging a weakened subgrade. As noted in Table 9, control

Table 8. Dry densities and moisture contents of Alburnett subgrades, June 1978.

Location	Dry Density pcf	Moisture Content % dry wt.	Remarks
Mirafi Section 1A	99.5	18.0	Top of stone
Mirafi Section 2A	115.0	11.3	Base of stone
Control Section 2B	112.8	15.7	
Mirafi Section 3	107.8	15.8	
Control Section 4	74.0	45.5	
Mirafi Section 5	105.1	17.7	
Control Section 6	110.0	16.2	

Table 9. Dry densities and moisture contents of Fairfax subgrades, June 1978.

Location	Dry Density pcf	Moisture Content % dry wt.	Remarks
Mirafi Section 3	114.2	12.8	North Lane
Control Section 4	110.6	14.5	North Lane
Mirafi Section 5	95.0	27.1	South Lane
Control Section 6	89.6	32.1	North Lane

sections 4 and 6 showed slightly lower density and greater moisture content than their respective Mirafi sections.

3.3.2. Fine-Grained Particle Movement

Particle size and/or hydrometer tests were performed on soil samples removed in June 1978 from the Mirafi-subgrade contact zone, or from an equivalent depth in the control sections, for both Alburnett and Fairfax sites. These tests were performed to examine fabric capabilities in prohibiting movement of fines into, or out of, the subgrade under the following conditions: (1) into the subgrade due to percolation through the soil-aggregate surface course; or (2) from the lower reaches of the subgrade only, due to capillary movement toward the fabric contact where presumably fines might be trapped.

Though the data were somewhat inconclusive in relation to the intended objective, several interesting observations were apparent. Tables 10 and 11 present the particle size classification of the fines only (passing no. 200 U.S. Standard Sieve) of the June 1978 fabric contact zone soil samples. As noted in the discussion of Tables 2 and 3, silts provide maximum capillary conductivity of water to the freezing front while clays retard and/or prevent such movement. From Table 10 all Alburnett sections showed similar silt contents. In a like manner, sections 1A through 5 indicated similar clay contents, but section 6 showed an increased percentage of clay, possibly illustrating its past non-frost-susceptible characteristics due to capillary moisture retardation.

Table 10. Partial particle size classifications of Alburnett subgrades, June 1978.

Particle Size Classification	Sect. 1A Mirafi	Sect. 2A Mirafi	Sect. 2B Control	Sect. 3 Mirafi	Sect. 4 Control	Sect. 5 Mirafi	Sect. 6 Control
% Silt (0.074-0.005 mm)	40	40	39	40	41	41	45
% Clay (<0.005 mm)	20	19	20	24	26	22	32

Table 11. Partial particle size classifications of Fairfax subgrades, June 1978.

Particle Size Classification	Sect. 3 Mirafi	Sect. 4 Control	Sect. 5 Mirafi	Sect. 6 Control
% Silt (0.074-0.005 mm)	29	35	35	42
% Clay (<0.005 mm)	30	22	33	24

Fairfax section 6 (Table 11) indicated a change of silt and clay contents from those noted in sections 3, 4 and 5. The variation of increased silt and reduced clay content of section 6, may illustrate at least a portion of the reason for reduced density and increased moisture content observed in the preceeding discussion of Table 9.

Comparison of the June 1978 particle size classifications, Tables 10 and 11, to those obtained prior to construction, Tables 2 and 3, shows higher percentages of clay fraction at the contact zone than in the subgrade prior to construction. The exceptions to this observation were Alburnett sections 3 and 4 where clay contents were nearly identical over the 20-month period. Silt contents appeared about the same in Alburnett sections 3, 4, 5 and 6, about the same or slightly reduced in Fairfax sections 3 and 4, but increased in Alburnett 1 and 2 and Fairfax 5 and 6, which involved, respectively, the granular backfill and macadam base.

Table 12 presents a comparison of total fines content (silt plus clay fractions) of the subgrade soils prior to construction in October 1976 versus the Mirafi-subgrade contact zone of June 1978. Between comparative fabric and control sections, little or no variation of subgrade fines occurred during the 20-month period, with a possible exception of Alburnett sections 5 and 6 where control section 6 indicated an increase in fines at the potential contact zone. Some movement and/or entrapment of fines from either the soil-aggregate surface or subgrade was observable with Alburnett sections 1 and 2 and Fairfax sections 5

Table 12. Comparison of Fines Content, Subgrade Prior to Construction, October 1976, versus Mirafi-Subgrade Contact Zone, June 1978.

Location	Section No.	Percent Fines, % Passing No. 200 Sieve: Oct. 76 Subgrade	Percent Fines, % Passing No. 200 Sieve: June '78 Mirafi-Subgrade Contact Zone	Remarks
Alburnett	1A	48	60	Fabric between soil-aggregate surface and granular backfill.
	2A	48	59	Granular backfill in fabric trough
	2B	48	59	Control-granular backfill only
	3	66	64	Fabric over frost-prone subgrade
	4	66	67	Control-no fabric over frost-prone subgrade
	5	61	63	Fabric over stable subgrade
	6	61	77	Control-no fabric over stable subgrade
Fairfax	3	56	59	Fabric over frost-prone subgrade
	4	56	55	Control-no fabric over frost-prone subgrade
	5	41	68	Fabric over stable subgrade
	6	41	66	Control-no fabric over stable subgrade

and 6. The frost-prone subgrades, both Alburnett and Fairfax sections 3 and 4, showed little variation of fines content over the 20-month period.

As may be noted from the above discussion, no evidence was obtained which would suggest that the Mirafi fabric prevented migration of fines due to capillary and/or percolation water movement. However, the data indicates that a movement of fines did occur within several of the test sections during a period of 20 months.

In addition, the data do not suggest any greater entrapment of fines through use of the fabric than through use of either the granular backfill at Alburnett or the macadam base at Fairfax.

Of those sections indicating a potential of silt/clay movement, a question thus remained as to whether the movement was from the subgrades, the soil-aggregate surface, or the macadam base. Samples of each were subjected to x-ray diffraction analysis in an effort to discern any variability of mineralogy in the fabric-subgrade contact samples due to intrusion particularly of the soil-aggregate surface or the macadam base minerals. For example, had the Alburnett fabric-subgrade contact samples shown an increase in calcium carbonate ($CaCO_3$) content, it might be concluded that a major percolation movement had occurred from the soil-aggregate surface toward the subgrade, since $CaCO_3$ is a major constituent of the crushed limestone surfacing material. No minerals characteristic of either the Alburnett soil-aggregate surface or the Fairfax macadam base were observed in the x-ray traces of the fabric subgrade contact samples. The traces were characteristic only of the two site subgrade materials. Therefore it may be assumed that movement of fines was predominantly from the subgrades only, through capillary activity.

3.3.3. Climatological Data

Climate of Linn County is characterized by frequent and often rapid changes in weather throughout the year. Summers are warm and winters cold, but prolonged periods of intense heat or extreme cold are relatively rare. Climatic conditions of both temperature and precipitation that existed within or near both the Alburnett and Fairfax sites were obtained as an aid to performance evaluation of the test sections.

3.3.3.1. Temperatures. Ambient air and subgrade temperatures were recorded at Alburnett sections 3 and 4, the objective being to observe any insulating effect attributable to the Mirafi fabric and a probable determination of average number of yearly freeze-thaw cycles of the soil-aggregate surface course, and upper surface of the subgrade. Placement of the three-channel recorder and thermocouples is noted on Figure 3. The recorded ambient air temperature was generally applicable to all sections at both Alburnett and Fairfax. Figures 17 and 18 are representations of the average daily temperatures recorded during the fall, winter, and early spring months of 1976-77 and 1977-78 respectively. Recorder operational problems occurred during the 1977-78 season, resulting in several data gaps which are obvious within the plots of Figure 18.

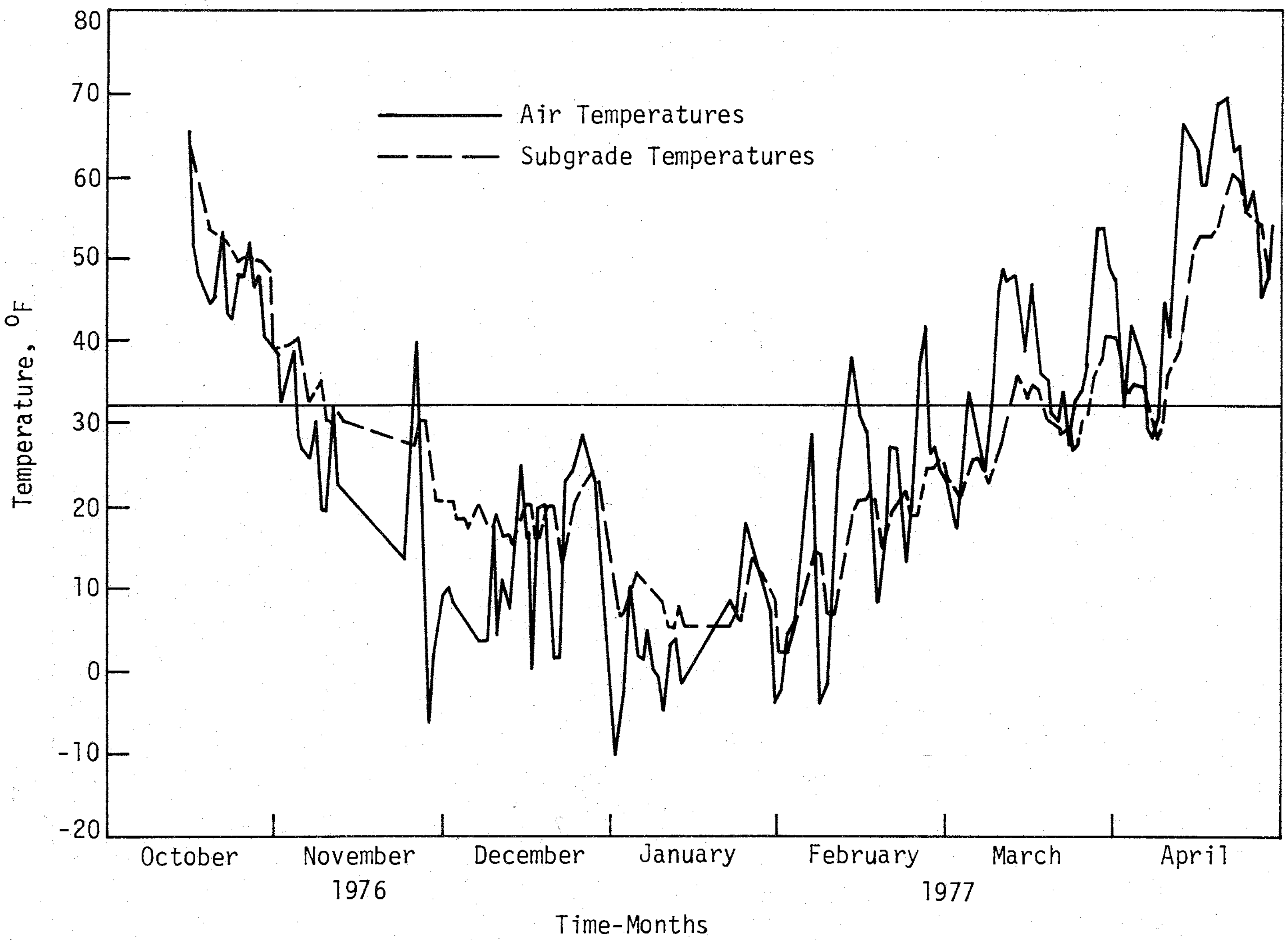

Figure 17. Average daily temperatures, October 1976 through April 1977.

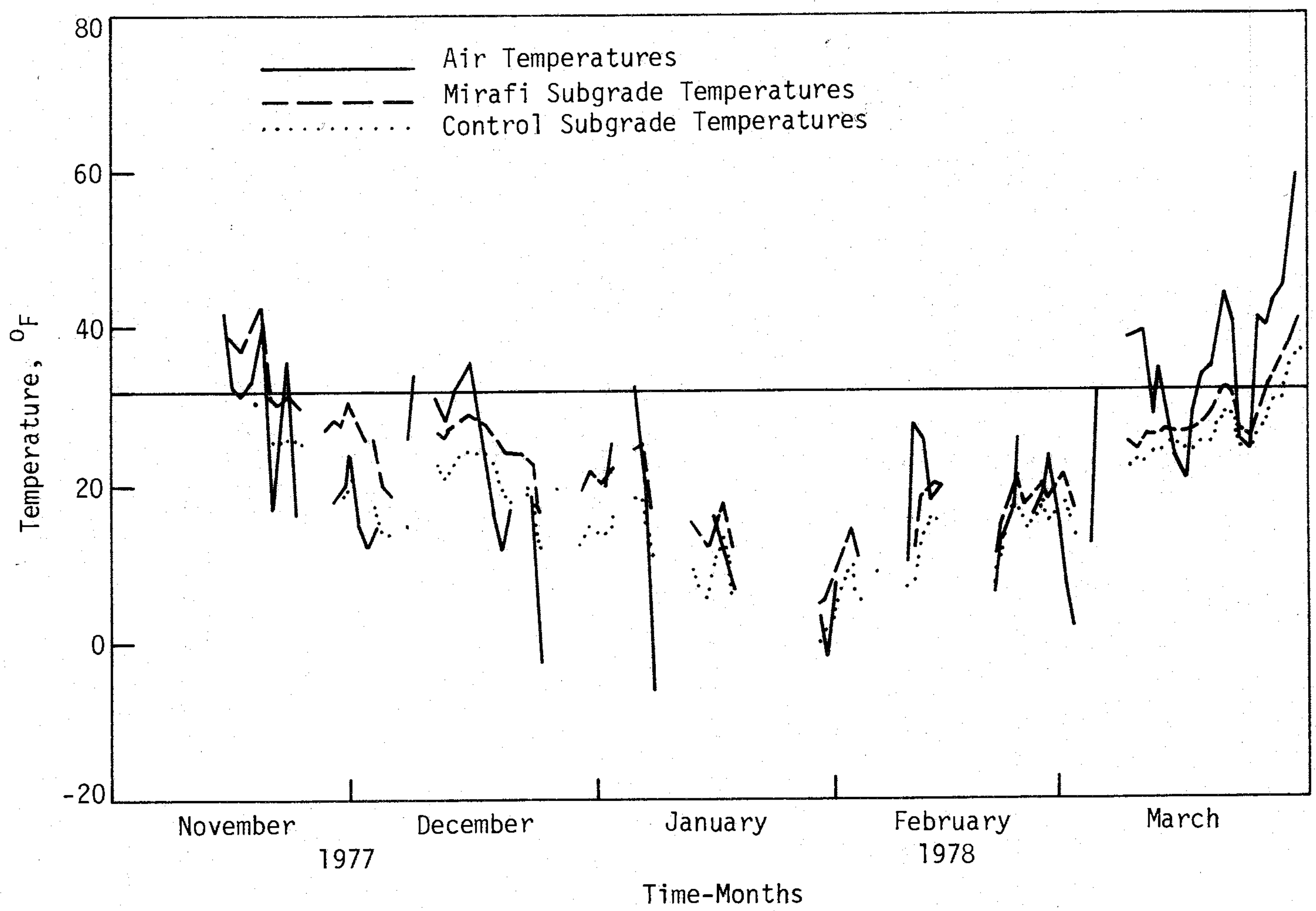

Figure 18. Average daily temperatures, November 1977 through March 1978.

During the 1976-77 season, no differences were observed in temperature recorded at the surface of the two subgrades; thus only one set of dashed lines represents average daily subgrade temperatures (Figure 17). It was not feasible to determine a temperature profile at the site because only one recording unit was available, so it was assumed that freezing or thawing occurred only when the average daily temperatures were below or above 32°F respectively. Thus only six subgrade freeze-thaw cycles were observed. The average daily subgrade temperature dipped below 32°F on November 8, 1976 and remained below freezing until March 9, 1977. This four month period coincided with the coldest fall experienced in Iowa in this century, and the coldest winter since 1936 [8]. The upper portion of the soil-aggregate surface undoubtedly underwent more freeze-thaw cycles than indicated by the average daily subgrade temperatures, because of shallow daytime thaw followed by nighttime freezing as observed from more than 15 average daily ambient air temperatures.

Unlike the previous winter when no temperature differences were observed in the subgrades of sections 3 and 4, slight temperature variations appeared during the winter of 1977-78, with the Mirafi section 3 generally higher (Figure 18). At the beginning of winter this temperature difference was of the order of 6°F, decreasing to a difference of about 3°F in the spring, with the most rapid decrease occurring near the end of January 1978 when subgrade temperatures reached their lowest values.

Data generated during the 1977-78 winter seem to indicate a subgrade insulating effect due to the Mirafi fabric, in contradiction to the previous winter's readings. Whether or not such insulating effect was valid is relatively inconclusive. First, operational problems occurred with the recorder during this period. The recorder and thermocouples were removed during the summer of 1978, checked, and found to produce some slight variation in temperatures, the magnitude of variation, however, being less than the 3 to 6°F difference noted in the data. Unfortunately the recording system could not be reinstalled for the 1978-79 season because of a greater need on another research project. Second, lower than normal precipitation was received prior to the first winter, with the opposite occurring prior to the second winter. Effect of precipitation on subgrade conditions is evaluated in succeeding sections. Third, the behavioral patterns of the fabric, particularly potential deterioration and plugging of the fabric voids with soil fines during moisture movement, is an unknown factor. It was noted at the Fairfax site, however, where several inches of the fabric were exposed on the shoulder foreslope, that limited amounts of deterioration had occurred during the two years of observation.

3.3.3.2. Precipitation. Strength and performance of soil-aggregate surfaced roadways, as well as amount of frost action, is greatly influenced by moisture conditions within the subgrade. Moisture is available to the subgrade in the form of precipitation, gravitational movement, and capillary water rise from a groundwater table. Each are interrelated, in that the amount of precipitation influences groundwater movement, which in turn influences the elevation to which capillary water will rise. Thus precipitation received within an area is an indicator of moisture conditions within a subgrade.

Figure 19 presents the monthly precipitation received near the test section sites. Rainfall recorded at the National Weather Service Cedar Rapids 1 Station was closest to the Alburnett sections, while data recorded at the Cedar Rapids Municipal Airport were used in conjunction with the Fairfax sections. Normal monthly data were available only for the recording station nearest Alburnett.

As may be noted in the bar chart of Figure 19, there was some variability in rainfall near the two sites. The 10-month period from May 1976 through February 1977 was the driest on National Weather Service records for Iowa, with precipitation only 56 percent of normal. November 1976 was the driest month ever recorded in Iowa with 0.11 and 0.16 inches near Alburnett and Fairfax respectively. By the end of February 1977, drought had reduced available subsoil moisture to an average 98 percent short of normal, lowering water tables, drying up wells, and placing some Iowa communities on water rationing during the summer of 1977. Above-normal rainfall was recorded in March 1977; it remained below normal until July, was significantly greater than normal through October, and by December 1977 available subsoil moisture was only 12 percent short of normal. Through September 1978, precipitation was nearly normal.

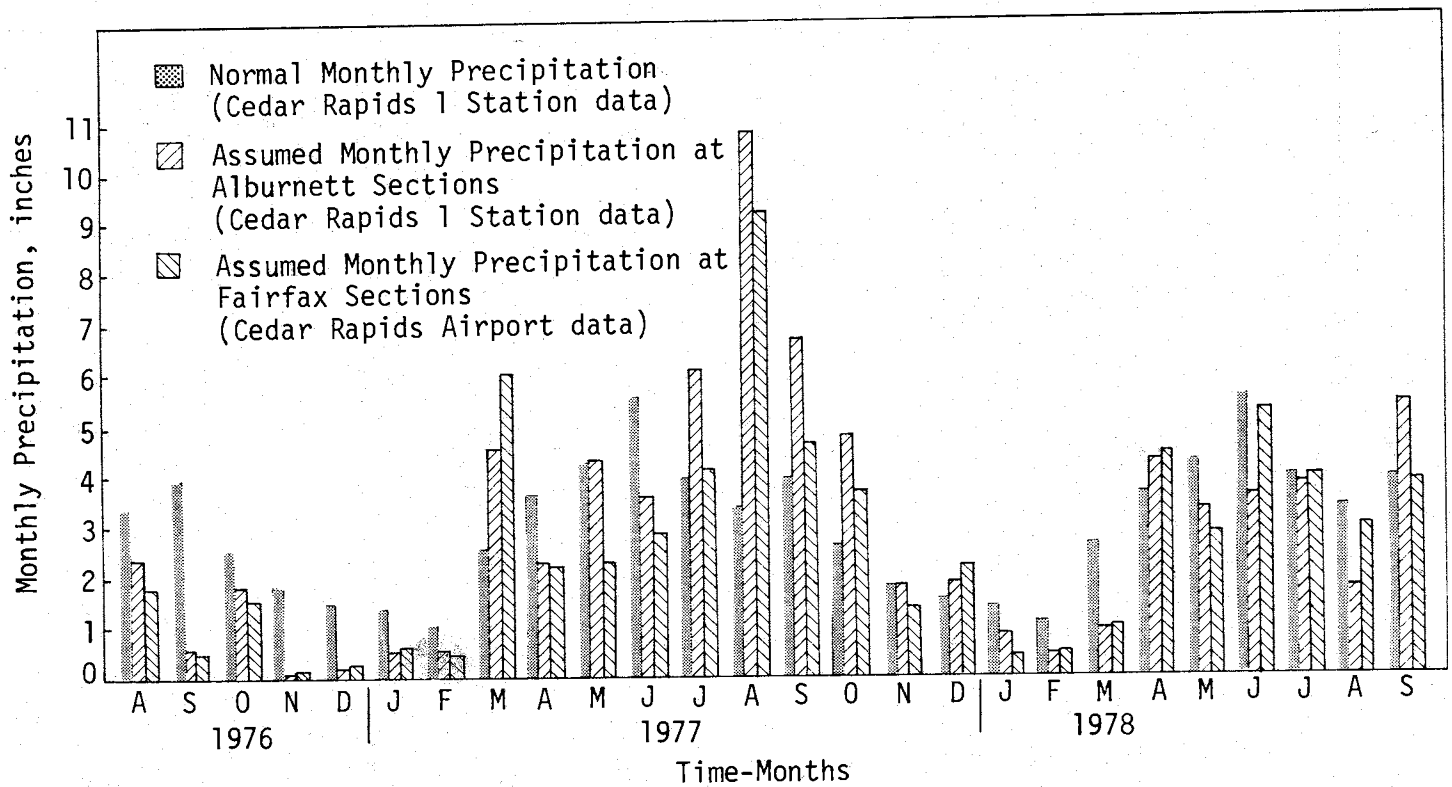

Figure 19. Monthly precipitation data.

The above discussion makes evident the extreme variability which may exist in central midwestern climatic conditions from year to year, especially as related to subsurface moisture. Extended periods of sub-freezing temperatures occur during the winter months. Yet with dry subgrade moisture conditions, these temperatures may not create detrimental frost action in a roadway. Such conditions existed during the winter of 1976-77. Only a small amount of surface heave was observed in an area west of, but adjacent to, Alburnett section 2B, while no heaving or boiling was visible in any of the test or control sections at either site.

Precipitation received during the fall of 1977 provided subgrade moisture conditions which created visual identification of both heave and boil softening in the second winter following fabric placement. Figure 20 illustrates an extremely severe frost boil condition which occurred in April 1978 at a location several miles southwest of the Alburnett sections, along an alluvial area adjacent to a small stream.

Both the Alburnett and Fairfax sites, however, are more upland regions than that illustrated in Figure 20. No heave or boils were evident at the Fairfax site. At the Alburnett site, only limited amounts of heaving and/or boil softening were observed within the test sections. Figure 21 shows the boil softened area immediately adjacent to the west end of section 2B, as of early April 1978. This region produced greater heave during the 1977-78 season than was noted the previous year, and was coupled with considerable boil softening. Sections

Figure 20. Severe frost boil within an alluvial area several miles southwest of Alburnett sections, April 1978.

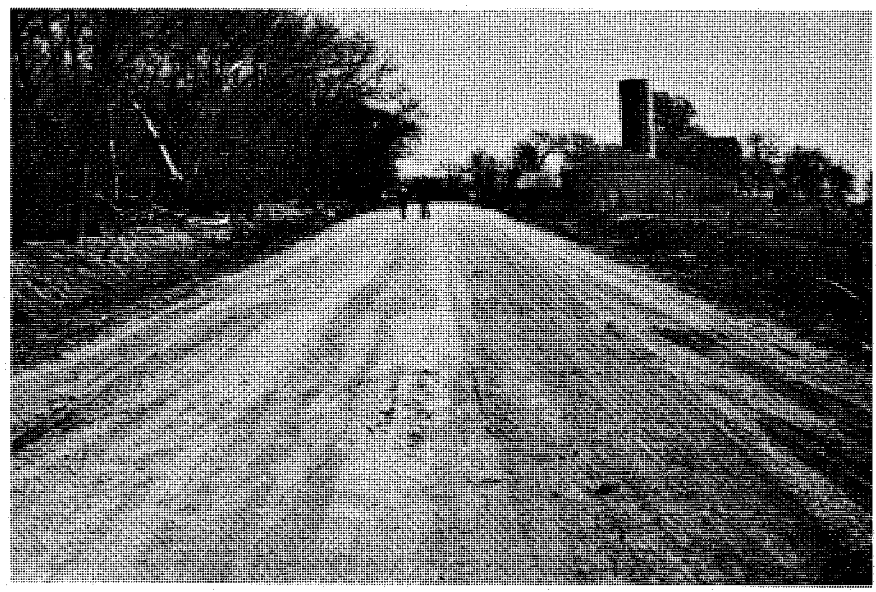

Figure 21. Boil softened area (foreground) immediately adjacent to Alburnett Section 2B (note demarcation transverse to centerline), April 1978. Sections 2A, 1B and 1A in background.

1A and 1B showed little or no surface moistening. Section 2A showed some slight darkening due to surface moisture, whereas section 2B showed definite discoloration of the surface due to internal moisture. If such discoloration through moisture may be attributable to the lack of fabric within control section 2B, it may be assumed either (1) that fabric test sections 1A and 1B were somewhat superior to section 2A in the control of capillary moisture movement to the surface, and/or (2) that they provided more rapid capillary movement resulting in quicker transpiration and evaporation of moisture.

During April 1978, Alburnett section 3 showed some alligator cracking and checking within the soil-aggregate surface. Considerable alligatoring and checking were visible within control section 4, coupled with indications of slight rutting and shoving of the surface course. Thus there were visible indications of at least a limited quantity of heave/boil control due to the fabric within section 3. No surface signs of boil softening were visible within the presumed non-frost-susceptible sections 5 and 6.

In April 1979, visible indications of heave/boil conditions were nonexistent at Fairfax and similar to those of 1978 within Alburnett sections 1A through 2B. The area immediately adjacent to the west end of section 2B was significantly softened and rutted. Alligator cracking, checking, rutting and shoving were quite visible in both sections 3 and 4. Sections 5 and 6 also showed very slight evidence of heave/boil conditions.

3.3.4. Field Performance Evaluations

In-situ performance of the soil-aggregate-fabric and control sections was evaluated by conducting periodic Benkelman beam deflection and Spherical Bearing Value tests through 1978. A series of Benkelman beam and plate bearing tests was conducted in the spring and summer of 1979 on the Alburnett sections only.

3.3.4.1. Benkelman Beam. Quantity of deflection of a road surface under either moving or static loads is an indication of the roadway's structural capacity. Deflection is made up of both elastic and plastic strains. Elastic deflections rebound upon removal of a load, whereas no rebound occurs with plastic deformations. Amount of deflection is influenced by gross load, tire contact pressure, repetition of load, thickness and quality of the various roadway structural courses or components, and the elastic properties of such components, particularly the subgrade soil. Possible failures may result from excessive stresses, fatigue, accumulated plastic deformations, or some combination of factors.

Several methods of flexible pavement design are based on limiting deflection criteria [9], on the principle that repetition of deflection causes progressive deterioration until at some stage the road becomes unacceptable for carrying traffic. Limiting design deflections normally range from 0.05 to 0.2 inch for flexible pavements. The Benkelman beam measures roadway surface deflections under a slowly moving wheel load, essentially evaluating the flexural capabilities of the composite vertically profiled components.

Figure 22 illustrates the Benkelman beam test. Each rear tire of the load test truck was maintained at 75 psi air pressure, supporting a 17,300 lb single rear axle wheel load. Since the maximum allowable single axle loading in Iowa is 18,000 lbs, deflections were thus determined near the legal loading.

Deflection measurements were taken at the one-third points of both traffic lanes in each fabric and control section. In addition, measurements were made at both the inside (IWT) and outside (OWT) wheel track of the load truck traveling within the normal traffic lane. All deflection measurements were averaged for both IWT and OWT conditions within each lane, as well as for each individual section (Tables 13 and 14). Preconstruction tests were performed on the Alburnett sections but not the Fairfax sections, since the latter involved complete reconstruction of the roadway structure.

As a qualitative measure of flexibility of the vertical profile of each section, a relative stiffness factor was computed by dividing the load on one set of dual wheels (one-half of the axle load) in thousands of pounds (kips), by the average maximum deflection. The more flexible the profile components, the greater the deflection but the lower the relative stiffness. Tables 15 and 16 present the computed relative stiffness values, while Figures 23 through 27 graphically illustrate the variability of stiffness versus time for both site sections.

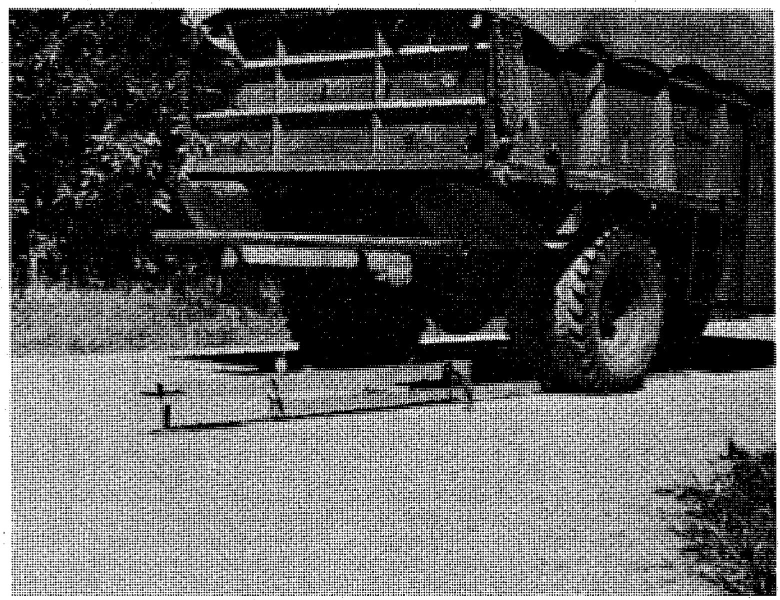

Figure 22. Benkelman beam inserted between rear dual wheels of load truck.

Table 13. Average Maximum Deflections from Benkelman Beam Tests, Alburnett Sections.

Section & Location	Maximum Deflections, in. Pre-Const. Oct. 1976	Post-Const. Oct. 1976	6 mo. after Const. April 1977	7 mo. after Const. May 1977	8 mo. after Const. June 1977	10 mo. after Const. Aug. 1977	19 mo. after Const. June 1978	21 mo.[a] after Const. July 1978	22 mo. after Const. Aug. 1978	30 mo. after Const. April 1979	32 mo. after Const. June 1979
Mirafi Section 1A											
North Lane	0.008	0.055	0.033	0.028	0.023	0.024	0.038	0.022	0.020	0.073	0.017
South Lane			0.032	0.023	0.018	0.026	0.088	0.026	0.022	0.024	0.039
(Average)	(0.008)	(0.055)	(0.033)	(0.026)	(0.021)	(0.025)	(0.063)	(0.024)	(0.021)	(0.049)	(0.028)
Mirafi Section 1B											
North Lane			0.024	0.034	0.022	0.025	0.036	0.052	0.029	0.078	0.036
South Lane	0.040	0.097	0.065	0.030	0.025	0.025	0.030	0.024	0.028	0.049	0.066
(Average)	(0.040)	(0.097)	(0.045)	(0.032)	(0.024)	(0.025)	(0.033)	(0.038)	(0.028)	(0.064)	(0.051)
Mirafi Section 2A											
North Lane	0.032	0.052	0.037	0.035	0.017	0.023	0.072	0.100	0.035	0.106	0.030
South Lane			0.020	0.027	0.024	0.025	0.044	0.032	0.025	0.059	0.074
(Average)	(0.032)	(0.052)	(0.029)	(0.031)	(0.021)	(0.024)	(0.058)	(0.066)	(0.030)	(0.083)	(0.052)
Control Section 2B											
North Lane			0.056	0.043	0.026	0.033		0.042	0.031	0.226	0.033
South Lane	0.080	0.063	0.028	0.034	0.022	0.027		0.050	0.028	0.068	0.053
(Average)	(0.080)	(0.063)	(0.042)	(0.039)	(0.024)	(0.030)		(0.046)	(0.030)	(0.147)	(0.043)
Mirafi Section 3											
North Lane	0.073	0.113	0.113	0.128	0.088	0.094	0.168	0.134	0.135	0.368	0.206
South Lane	0.043	0.140	0.132	0.098	0.087	0.015	0.146	0.136	0.109	0.172	0.320
(Average)	(0.058)	(0.127)	(0.123)	(0.113)	(0.088)	(0.055)	(0.157)	(0.135)	(0.122)	(0.270)	(0.263)
Control Section 4											
North Lane	0.123	0.100	0.145	0.165	0.102	0.109	0.238	0.166	0.184	0.403	0.362
South Lane	0.084	0.130	0.151	0.128	0.111	0.106	0.220	0.126	0.102	0.499	0.532
(Average)	(0.104)	(0.115)	(0.148)	(0.147)	(0.107)	(0.108)	(0.229)	(0.146)	(0.143)	(0.451)	(0.447)
Mirafi Section 5											
North Lane	0.042	0.046	0.063	0.054	0.039	0.043	0.198	0.064	0.031	0.049	0.073
South Lane	0.069	0.054	0.050	0.048	0.065	0.043	0.058	0.048	0.054	0.114	0.085
(Average)	(0.056)	(0.050)	(0.057)	(0.051)	(0.052)	(0.043)	(0.128)	(0.056)	(0.042)	(0.082)	(0.079)
Control Section 6											
North Lane	0.040	0.041	0.072	0.049	0.038	0.041	0.070	0.012	0.054	0.154	0.068
South Lane	0.100	0.067	0.058	0.055	0.082	0.040	0.058	0.016	0.050	0.122	0.078
(Average)	(0.070)	(0.054)	(0.065)	(0.052)	(0.060)	(0.041)	(0.064)	(0.014)	(0.052)	(0.138)	(0.073)

[a]Additional surface aggregate added since previous test.

Table 14. Average Maximum Deflections from Benkelman Beam Tests, Fairfax Sections

Section & Location	Maximum Deflections, in.							
	Post-Const. Oct. 1976	5 mo. after Const. March 1977	7 mo. after Const. May 1977	8 mo. after Const. June 1977	10 mo.[a] after Const. Aug. 1977	19 mo. after Const. June 1978	21 mo. after Const. July 1978	22 mo. after Const. Aug. 1978
Mirafi Section 3								
North Lane	0.058	0.053	0.034	0.047	0.022	0.059	0.049	0.054
South Lane	0.032	0.046	0.043	0.032	0.019	0.047	0.033	0.045
(Average)	(0.045)	(0.050)	(0.039)	(0.040)	(0.021)	(0.053)	(0.041)	(0.050)
Control Section 4								
North Lane	0.049	0.049	0.022	0.023	0.020	0.057	0.048	0.034
South Lane	0.029	0.049	0.032	0.015	0.022	0.047	0.037	0.040
(Average)	(0.039)	(0.049)	(0.027)	(0.019)	(0.021)	(0.052)	(0.043)	(0.037)
Mirafi Section 5								
North Lane	0.066	0.067	0.026	0.039	0.034	0.068	0.051	0.059
South Lane	0.040	0.054	0.026	0.036	0.031	0.056	0.046	0.046
(Average)	(0.053)	(0.061)	(0.026)	(0.036)	(0.031)	(0.062)	(0.049)	(0.052)
Control Section 6								
North Lane	0.094	0.054	0.051	0.019	0.022	0.049	0.055	0.053
South Lane	0.034	0.040	0.029	0.015	0.023	0.039	0.041	0.038
(Average)	(0.064)	(0.047)	(0.040)	(0.017)	(0.023)	(0.044)	(0.048)[b]	(0.046)

[a]Tests conducted after seal coat surface added to roadway.

[b]This series taken following 3" rain and 1.5 hr delay.

Table 15. Average Relative Stiffness from Benkelman Beam Tests, Alburnett Section

Section & Location	Relative Stiffness, kips/in. Pre-Const. Oct. 1976	Post-Const. Oct. 1976	6 mo. after Const. April 1977	7 mo. after Const. May 1977	8 mo. after Const. June 1977	10 mo. after Const. Aug. 1977	19 mo. after Const. June 1978	21 mo.[a] after Const. July 1978	22 mo. after Const. Aug. 1978	30 mo. after Const. April 1979	32 mo. after Const. June 1979
Mirafi Section 1A											
North Lane	1081.3	157.3	262.1	308.9	376.1	360.4	227.6	393.2	432.5	118.5	508.8
South Lane			270.3	376.1	480.6	332.7	98.3	332.7	393.2	360.4	221.8
(Average)	(1081.3)	(157.3)	(266.2)	(342.5)	(428.3)	(346.6)	(163.0)	(362.9)	(412.8)	(239.4)	(365.3)
Mirafi Section 1B											
North Lane			360.4	254.4	393.2	346.0	240.3	166.3	298.3	110.9	240.3
South Lane	216.3	89.2	133.1	288.3	346.0	346.0	288.3	360.4	308.9	176.5	131.0
(Average)	(216.3)	(89.2)	(246.7)	(271.4)	(369.6)	(346.0)	(264.3)	(263.4)	(303.6)	(143.7)	(185.6)
Mirafi Section 2A											
North Lane	270.3	166.3	233.8	247.1	508.8	376.1	120.1	86.5	247.1	81.6	288.3
South Lane			432.5	320.4	360.4	346.0	196.6	270.3	346.0	146.6	116.9
(Average)	(270.3)	(166.3)	(333.1)	(283.8)	(434.6)	(361.0)	(158.4)	(178.4)	(296.6)	(114.1)	(202.6)
Control Section 2B											
North Lane			154.5	201.2	332.7	262.1		206.0	279.0	38.3	262.1
South Lane	108.1	137.3	308.9	254.4	393.2	320.4		173.0	308.9	127.2	163.2
(Average)	(108.1)	(137.3)	(231.7)	(227.8)	(362.9)	(291.2)		(189.5)	(294.0)	(82.8)	(212.6)
Mirafi Section 3											
North Lane	118.5	76.5	76.5	67.6	98.3	92.0	51.5	64.6	64.1	23.5	42.0
South Lane	201.2	61.8	65.5	88.3	99.4	576.7	59.3	63.6	79.4	50.3	27.0
(Average)	(159.8)	(69.2)	(71.0)	(77.9)	(98.9)	(334.3)	(55.4)	(64.1)	(71.7)	(36.9)	(34.5)
Control Section 4											
North Lane	70.3	86.6	59.7	52.4	84.8	79.4	36.3	52.1	47.0	21.5	23.9
South Lane	103.0	66.5	57.3	67.6	77.9	77.9	39.3	68.7	84.8	17.3	16.3
(Average)	(86.7)	(76.6)	(58.5)	(60.0)	(81.4)	(78.6)	(37.8)	(60.4)	(65.9)	(19.4)	(20.1)
Mirafi Section 5											
North Lane	206.0	188.0	137.3	160.2	221.8	201.2	43.7	135.2	279.0	176.5	118.5
South Lane	126.3	160.2	173.0	180.2	133.1	201.2	149.1	180.2	160.2	75.9	101.8
(Average)	(166.1)	(174.1)	(155.2)	(170.2)	(177.4)	(201.2)	(96.4)	(157.7)	(219.6)	(126.2)	(110.2)
Control Section 6											
North Lane	216.3	211.0	120.1	176.5	227.6	211.0	123.6	720.8	160.2	56.2	127.2
South Lane	86.5	129.1	149.1	157.3	105.5	216.3	149.1	540.6	173.0	70.9	110.9
(Average)	(151.4)	(170.0)	(134.6)	(166.9)	(166.6)	(213.6)	(136.4)	(630.7)	(166.6)	(63.6)	(119.0)

[a]Additional surface aggregate added since previous test.

Table 16. Average Relative Stiffness from Benkelman Beam Tests, Fairfax Sections

Section & Location	Relative Stiffness, kips/in. Post-Const. Oct. 1976	5 mo. after Const. March 1977	7 mo. after Const. May 1977	8 mo. after Const. June 1977	10 mo.[a] after Const. Aug. 1977	19 mo. after Const. June 1978	21 mo. after Const. July 1978	22 mo. after Const. Aug. 1978
Mirafi Section 3								
North Lane	149.1	163.2	254.4	184.0	393.2	146.6	176.5	160.2
South Lane	270.3	188.1	201.2	270.3	455.3	184.0	262.1	192.2
(Average)	(209.7)	(175.6)	(227.8)	(227.2)	(424.2)	(165.3)	(219.3)	(176.2)
Control Section 4								
North Lane	176.5	176.5	393.2	376.1	432.5	151.8	180.2	254.4
South Lane	298.3	176.5	270.3	576.7	393.2	184.0	233.8	216.2
(Average)	(237.4)	(176.5)	(331.7)	(476.4)	(412.8)	(167.9)	(207.0)	(235.3)
Mirafi Section 5								
North Lane	131.1	129.1	332.7	221.8	254.4	127.2	169.6	146.6
South Lane	216.2	160.2	332.7	262.1	320.4	154.5	188.0	188.0
(Average)	(173.7)	(144.6)	(332.7)	(242.0)	(287.4)	(140.8)	(178.8)	(167.3)
Control Section 6								
North Lane	92.1	160.2	169.6	455.3	393.2	176.5	157.3	163.2
South Lane	254.4	216.3	298.3	576.7	376.1	221.8	211.0	227.6
(Average)	(173.2)	(188.2)	(233.9)	(516.0)	(384.6)	(199.2)	(184.1)[b]	(195.4)

[a]Tests conducted after seal coat surface added to roadway.

[b]This series taken following 3" rain and 1.5 hr delay.

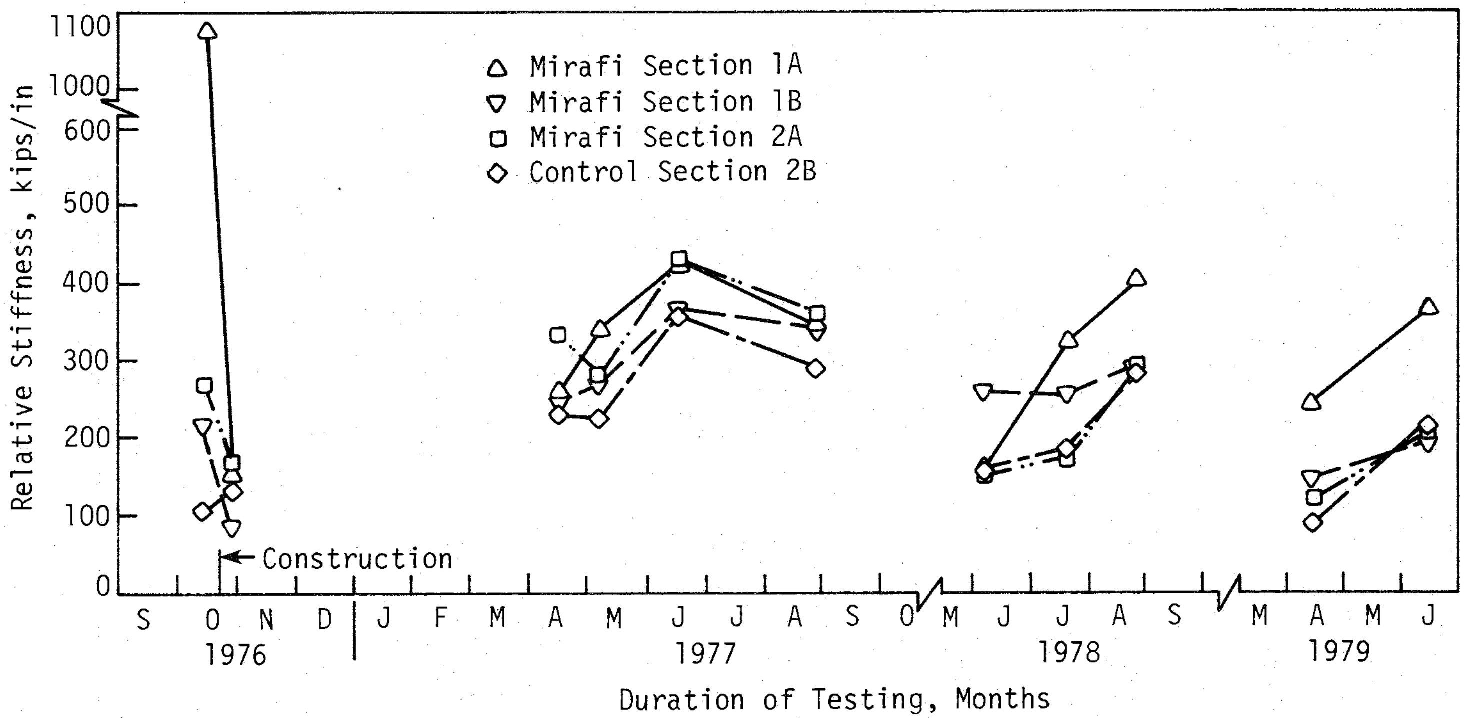

Figure 23. Benkelman beam relative stiffness vs. time, Alburnett Sections 1A-2B.

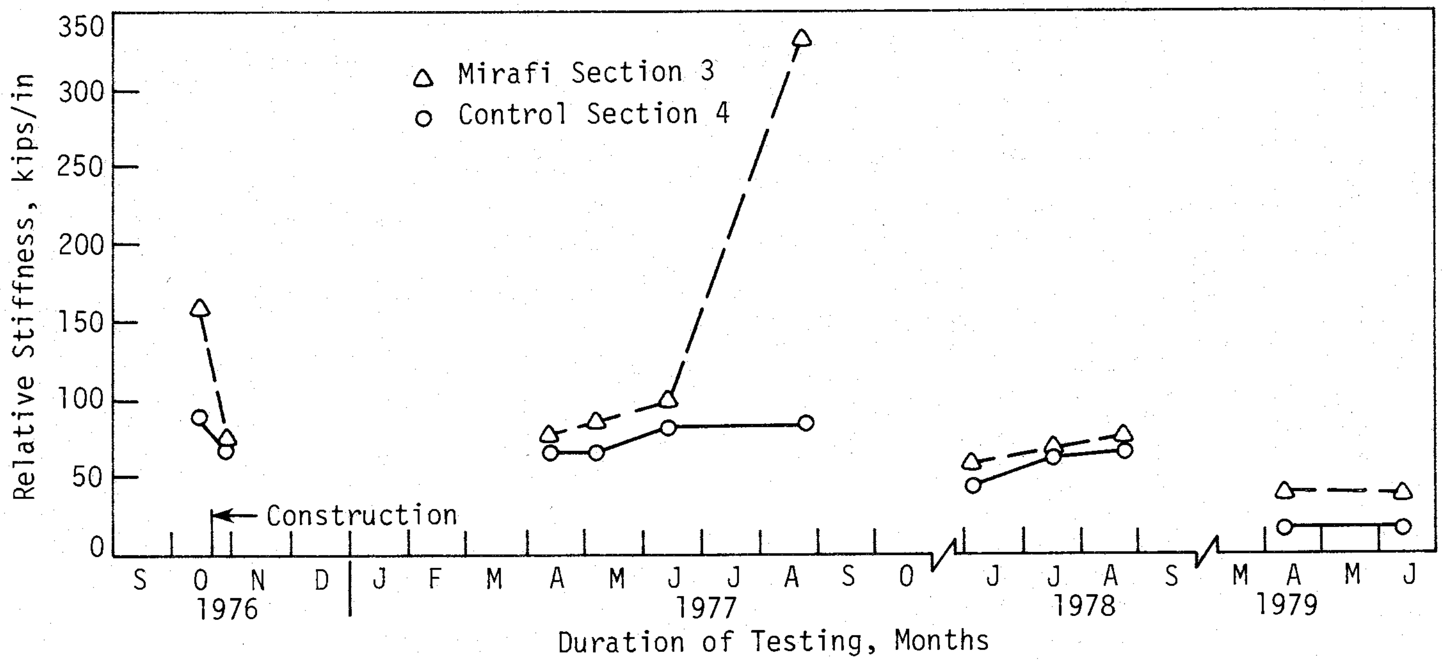

Figure 24. Benkelman beam relative stiffness vs. time, Alburnett Sections 3 and 4.

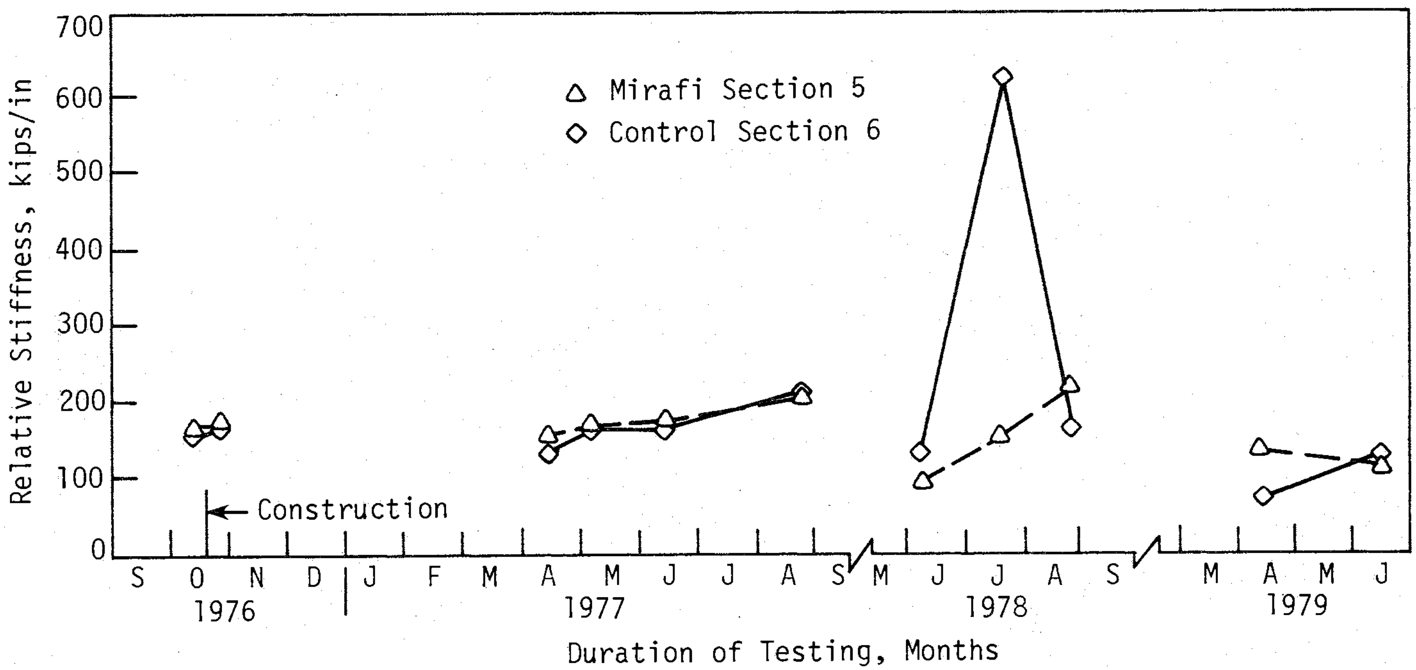

Figure 25. Benkelman beam relative stiffness vs. time, Alburnett Sections 5 and 6.

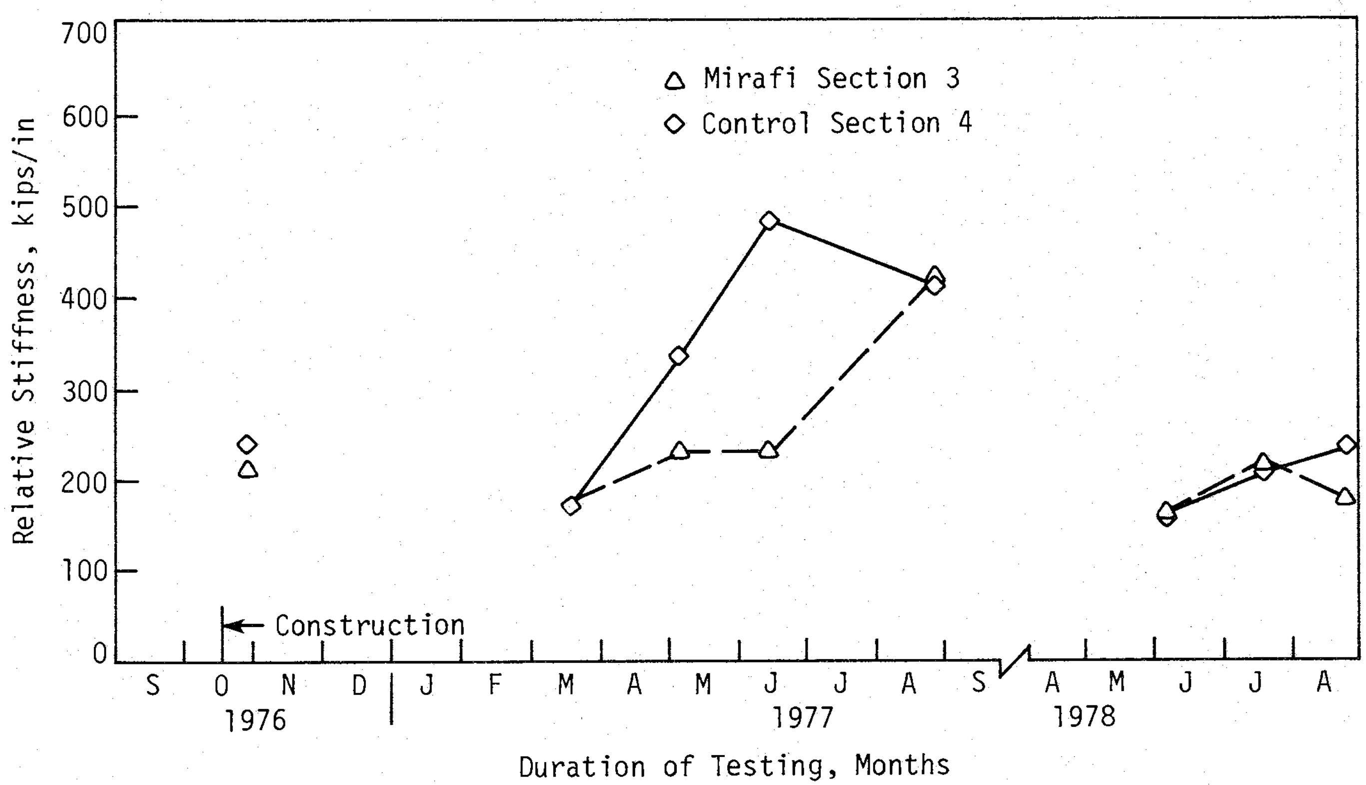

Figure 26. Benkelman beam relative stiffness vs. time, Fairfax Sections 3 and 4.

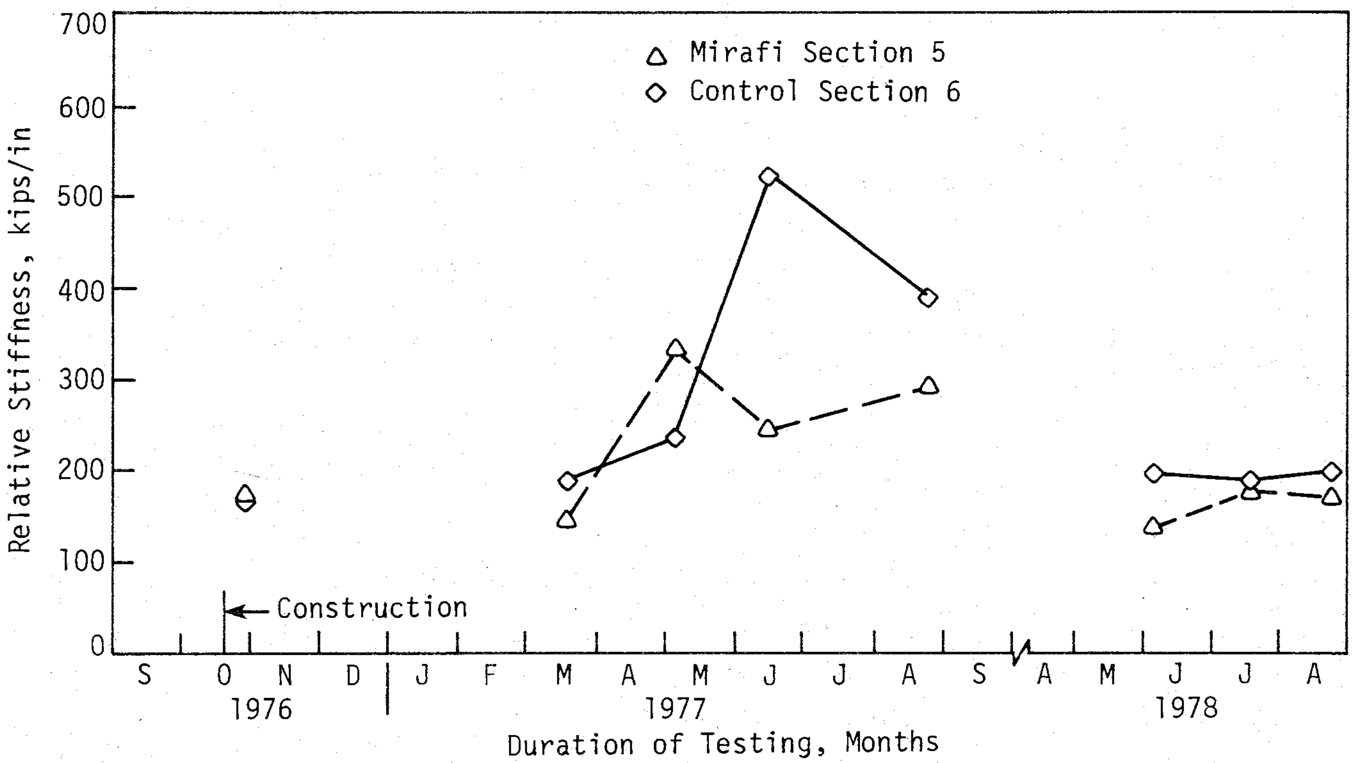

Figure 27. Benkelman beam relative stiffness vs. time, Fairfax Sections 5 and 6.

Considerable variation in average deflections and relative stiffness existed in the Alburnett sections prior to construction. However, for similar subgrades the values were reasonably close. Similarity of post-construction values was important since they served as the initial datum for subsequent comparison of performance versus time. In general, post-construction values were less than pre-construction, due to disturbance of natural in-place stabilities inherent with most soil materials. Following construction remolding, most roadway materials will tend to regain a stability through natural processes as well as traffic densification. In general, Figures 23, 26, and 27 illustrate the stability regained within Alburnett sections 1A through 2B and Fairfax sections 3 through 6 during the period from construction to spring and/or summer 1977. However, Alburnett sections 3, 4, 5 and 6 (Figures 24 and 25) illustrate little stability regained during the same period of time.

Deflections and stiffnesses of sections 1A through 2B were consistently lower and higher, respectively, than of other Alburnett site sections (Figures 23, 24, and 25), indicating the added strength provided by the two-foot thick granular backfill. Deflections within sections 5 and 6 were consistently lower than those in sections 3 and 4, a condition compatible with the prior assumption that sections 5 and 6 were founded on less frost-susceptible subgrades. If it is assumed that sections 5 and 6 were thus a datum for comparison with all other sections, it is then apparent that the granular backfill, and/or backfill coupled with fabric, increased stability and performance characteristics of the frost susceptible subgrade above that attainable within the non-frost-susceptible subgrade. Likewise it is also apparent that use of the fabric within frost susceptible sections 3 and 4 did not improve deflection and stiffness performance to that of the non-frost-susceptible sections 5 and 6.

Immediately following a spring thaw, deflections of a roadway will generally be high while stiffness values are low, each characteristic tending to reverse and improve with time as gravitational moisture movement and/or capillary transpiration/evaporation occurs, a condition often referred to as healing. In general, such conditions were noted each season within each kind of site material (Figures 23 through 27). The principal exception to this generality occurred in August 1977 when exceptionally high rainfall finally broke the preceeding drought (Figure 19). At this time, deflections increased and stiffness decreased in Alburnett sections 1A through 2B and Fairfax sections 4 and 6. Alburnett sections 3 through 6 and Fairfax sections 3 and 5 were basically unaffected.

Performance of a roadway will usually be increased following any subgrade, base, or surface improvement, which then slowly diminishes with time due to various combinations of traffic density, loading, environmental factors, and material fatigue and deterioration. In general, Figures 23 through 27 intimate such a diminution of support performance. For example, a linear regression of relative stiffness versus time data within the Alburnett sections would indicate a slight slope down and to the right with the data of each succeeding season beginning in the spring of 1977. It cannot be concluded, however, that the seasonal decrease in performance was due strictly to the previously noted material factors, since the effect of both low and high subgrade moisture conditions during the study period may not have provided a normal data base equilibrium of such conditions.

During the three seasons of study, deflection/stiffness performance within Alburnett sections 1A through 2B indicates a general superiority of sections 1A and 1B, decreasing with section 2A to control section 2B (Figure 23). Section 1A contained fabric between the soil-aggregate surface and granular backfill, while fabric encapsulated the granular backfill within section 1B. If it may be assumed that the 1977 season was atypical relative to subgrade moisture, section 1A could be looked upon as generally providing the best cost versus stability benefits for the granular backfill treatment of this frost-prone subgrade soil.

It is apparent from Figure 24 that use of fabric between the soil-aggregate surface and this frost-susceptible subgrade improved the deflection/stiffness characteristics over each of the three seasons of study. Disregarding the outlier data of August 1977, improvement of deflection/stiffness values through fabric usage ranged from a low of about 6% in July 1978 to a high of 90% in April 1979. While the April 1979 percentage improvement in Mirafi section 3 as compared to control section 4 is significant, it should be noted that average maximum deflections recorded at this time (Table 13) were in excess of 0.25

inch for section 3 and approached 0.5 inch for section 4 -- values higher than normally accepted design deflections.

Disregarding the outlier data of July 1978, Alburnett sections 5 and 6 basically showed no variability during the 1977 boil/heal season, a slight variability during 1978, and some definite variation in 1979. Particularly in April immediately following thawing. Healing occurred rather quickly, however, within both the 1978 and 1979 seasons.

Relative stiffness values for each Fairfax section versus time (Figures 26 and 27), were in a range similar to that for values obtained for Alburnett sections 1A through 2B and may be sttributed to a similarity between the Alburnett granular backfill and the Fairfax macadam base.

Unlike the granular backfill Alburnett sections, however, use of Mirafi fabric between the Fairfax subgrade and macadam base provided no basic deflection and stiffness performance improvement, particularly at the critical stage of frost boil development, i.e., following the spring thaws of both 1977 and 1978. Figures 26 and 27 illustrate that initial relative stiffnesses were similar to one another at the beginning of each season and also were similar to the post-construction values.

Fairfax sections 5 and 6 were founded on what was assumed to be a stable subgrade, yet slightly greater post-construction deflections were observed there as compared to those of section 3 and 4 which were founded in the presumed frost-susceptible area (Table 14). With widening and deepening of ditches, the subgrade of sections 3 and 4 may have become much better drained than it was prior to construction. Sections 3 and 4 are also on a slightly steeper grade than sections 5 and 6, thus encouraging more rapid runoff and less infiltration of precipitation.

In general Fairfax control sections 4 and 6 showed a more rapid healing than Mirafi sections 3 and 5 during the 1977 season. Post-boil healing of any of the sections was basically nonexistent during the 1978 season. None of the Fairfax sections was as greatly affected by the heavy August 1977 rains as were the Alburnett sections, a fact that may be at least partially attributable to the seal coat surfacing in the spring of 1977 which would limit surface infiltration.

It is thus apparent from Figures 26 and 27 that use of Mirafi 140 fabric within the Fairfax site provided no improvement in deflection and stiffness performance of the roadway. The observations noted may reflect a tendency of the macadam base to reach equilibrium stability throughout the Fairfax site regardless of subgrade variations or the presence of Mirafi fabric. In addition, any beneficial performance effects due to fabric may be overshadowed by the quality of the entire roadway as created by the macadam base only.

With the primary exception of Alburnett sections 3 and 4, most Benkelman beam rebound measurements at both sites were nearly equivalent to maximum deflections, indicating that each roadway was undergoing predominantly elastic strains. Use of elastic theory, in which deformations are directly proportional to applied stresses, thus appears reasonably valid in analysis of deflection test results. In evaluating loading capabilities, comparisons should be made only with sections having similar subgrade, surface and/or base course material properties in order that any supportive differences might be attributable to the fabric. Using the 1977 Alburnett data, fabric sections 1A, 1B and 2A decreased average deflection by 10% as compared to control section 2B, indicating that the stress at the subgrade surface was about 10% less in these sections. Alburnett sections 3 and 4 indicated about 25% reduction in potential subgrade stress with use of the fabric, while sections 5 and 6 indicated only a 7% stress reduction with use of the fabric on a stable subgrade.

By making various assumptions of Poisson's ratio and configuration of wheel loadings, values of the moduli of elasticity could be back-calculated for each of the above sections through use of Boussinesq theory. Thus a Poisson's ratio of 0.3 was assumed for the Alburnett granular backfill, while 0.4 was assumed for those sections with no granular backfill. Contact area of the tire was assumed as circular and was obtained by dividing the rear axle load by four (number of tires) and the constant tire pressure of 75 psi. Since the measured deflection was between two loaded areas and superposition is valid in elastic theory, one-half of the deflection area was assumed to be caused by one tire loading.

Moduli of elasticity values thus calculated for the 1977 Alburnett sections ranged from about 3500 to 15,600 psi, the latter being obtained in the granular backfill sections. The higher value is within a typical range for dense sand and gravel, while the lower value is typical of silty or sandy clays, each of which made up a basic subgrade soil of the

Alburnett sections. Through this process of back-calculation of moduli from deflection values, a modulus of elasticity of each section would exhibit exactly the same comparative relationship as all deflections of Tables 13 and 14. Since the calculated moduli were in a range of typical soil values, they add validity to use of Benkelman beam deflections for a roadway system performance evaluation and load carrying capabilities.

Figures 23 and 24 indicate some test section variability. Laboratory K-Tests were conducted on a similar subgrade soil, because of insufficient site samples, thus making direct correlations between laboratory and field responses somewhat conjectural. However, some pertinent trends regarding laboratory tests and fabric usage as a structural reinforcement can be seen. The K-Test vertical deformation modulus E_v and lateral stress ratio k (Table 5) are, in principle, related to relative stiffness obtained from the Benkelman beam test. Laboratory stability tests indicated that addition of fabric should produce a slight reduction in composite stiffness prior to degradation by capillary moisture absorption during freeze-thaw. Figures 23 and 24 indicate a decrease in relative stiffness following construction. After subjection to freeze-thaw, laboratory tests predicted a slight increase in E_v; field tests made in April 1977 showed some nominal improvements in relative stiffness. Incorporation of fabric predicted an improved c-ϕ relationship following freeze-thaw, which should then tend to improve as moisture content is reduced through drainage, transpiration, and/or evaporation, (healing). However, portions of the soil matrix and/or soil-matrix-fabric bond might be expected to gradually reduce through continued freeze-thaw cycling and moisture absorption, reducing the stress ratio and c-ϕ capabilities; this action is not unlike the gradual reduction of field relative stiffness values versus time. Fluctuations of relative stiffness due to field moisture content changes could also be anticipated on the basis of the data of Table 5. When coupled with the discussion of freeze-thaw elongation and stability previously presented, the purpose in citing such trends is primarily to emphasize that simple laboratory testing methods may, and should, be developed, which will provide predictive criteria for designed in-situ usage of a soil-aggregate-fabric system. From all indications herein, the Iowa K-Test could potentially be developed for such a purpose.

3.3.4.2. Spherical Bearing Values. During 1977 and 1978, relative bearing capacities of the surface of each field test section were evaluated through useage of the Spherical Bearing Value (SBV) test [10, 11]. The SBV is somewhat similar to, though less destructive than, a CBR. SBV data may be converted to such values as CBR, or Westergaard's modulus of subgrade reaction, K.

SBV testing consists of measuring road surface penetration of a 6 inch diameter spherical penetrometer during controlled hydraulic jacking against a loaded frame. Penetration deflections are independently measured through a dial gage supported from an arm on the roadway surface, well away from the deflection basin created during the test. Penetration deflections are taken at 200 psi hydraulic pressure gage intervals to at least 2000 psi, or until a maximum dial gage reading of 1.0 inch is achieved. Spherical penetrometer contact area within the roadway is obtained from πDh, where D is the sphere diameter and h is the penetration. Since the area of the hydraulic piston was 2.234 sq in., gage pressures were converted to pounds of load on the spherical penetrometer. A plot of these loads versus their respective contact areas may be produced, and the slope of a linear regression line through the points is defined as the SBV, in psi. The larger the SBV, the greater the relative bearing capacity of the roadway system.

SBV's were obtained at the one-third points of both traffic lanes in every section. As with the Benkelman beam tests, pre-construction SBV tests were run on the Alburnett but not on the Fairfax sections. Average SBV results for each section are presented in Tables 17 and 18.

Results from the two-year series of tests were relatively disappointing in regard to defining significant variations in bearing capacities between comparable sections. Plots of SBV data versus time indicated some variation in Alburnett sections 1A through 2B, both before and after construction, but became more uniform after about 6 months of traffic possibly because of more uniform densities of the materials after repeated traffic loadings. SBV's versus time for sections 1A through 2B were consistently higher than those obtained for sections 3 through 6, probably because of inclusion of the granular backfill.

Table 17. Average Spherical Bearing Values, Alburnett Sections

	Spherical Bearing Values, psi								
Section & Location	Pre-Const. Oct. 1976	Post-Const. Oct. 1976	6 mo. after Const. April 1977	7 mo. after Const. May 1977	8 mo. after Const. June 1977	10 mo. after Const. Aug. 1977	19 mo. after Const. June 1978	21 mo.[b] after Const. July 1978	22 mo. after Const. Aug. 1978
Mirafi Section 1A									
North Lane	2025	815	3720	1405	4286	2448	a		964
South Lane			1960	1321	3064	1373	a	1231	4299
(Average)	(2025)	(815)	(2840)	(1363)	(3675)	(1911)		(1231)	(2631)
Mirafi Section 1B									
North Lane			3510	625	4315	1123	a		574
South Lane	2371	1201	2960	2077	8729	2018	a	1070	1225
(Average)	(2371)	(1201)	(3235)	(1351)	(6522)	(1571)		(1070)	(900)
Mirafi Section 2A									
North Lane	1598	374	2740	810	4834	3033	a		1477
South Lane			1715	1760	3700	1417	a	1397	2014
(Average)	(1598)	(374)	(2227)	(1285)	(4267)	(2225)		(1397)	(1746)
Control Section 2B									
North Lane			1160	840	3788	1816	a		682
South Lane	929	657	1590	1345	4170	1805	a	1448	909
(Average)	(929)	(657)	(1375)	(1092)	(3979)	(1811)		(1448)	(795)
Mirafi Section 3									
North Lane	420	355	280	378	1239	531	464		559
South Lane	730	225	520	474	1047	507	a	472	460
(Average)	(575)	(290)	(400)	(426)	(1143)	(519)	(464)	(472)	(509)

Control Section 4									
North Lane	455	325	330	518	686	493	a		537
South Lane	1050	245	690	349	638	524	a	274	328
(Average)	(752)	(285)	(510)	(433)	(662)	(509)		(274)	(432)
Mirafi Section 5									
North Lane	2435	280	580	401	1300	359	a	a	413
South Lane	660	260	410	407	987	354	a	a	921
(Average)	(1547)	(270)	(495)	(404)	(1143)	(357)			(583)
Control Section 6									
North Lane	1890	360	460	364	1660	412	a	a	838
South Lane	875	265	380	297	737	301	a	a	436
(Average)	(1382)	(312)	(420)	(330)	(1198)	(357)			(704)

[a] Surface materials too soft to test.

[b] Additional surface aggregate added since previous test.

Table 18. Average Spherical Bearing Values, Fairfax Sections

Section & Location	Post-Const. Oct. 1976	5 mo. after Const. March 1977	7 mo. after Const. May 1977	8 mo. after Const. June 1977	10 mo.[a] after Const. Aug. 1977	19 mo. after Const. June 1978	21 mo. after Const. July 1978	22 mo. after Const. Aug. 1978
	Spherical Bearing Value, psi							
Mirafi Section 3								
North Lane	455	635	2671	3113	1541	497	938	1250
South Lane	715	1600	1916	3976	1425	340	967	1026
(Average)	(585)	(1117)	(2293)	(3544)	(1483)	(418)	(952)	(1138)
Control Section 4								
North Lane	505	1215	2390	4302	1539	358	944	1301
South Lane	740	1080	2947	2391	1139	372	1135	912
(Average)	(622)	(1147)	(2668)	(3346)	(1339)	(365)	(1040)	(1107)
Mirafi Section 5								
North Lane	415	585	3000	2246	1358	919	1037	1195
South Lane	530	720	2842	1704	1219	1457	1041	1174
(Average)	(472)	(652)	(2921)	(1975)	(1289)	(1188)	(1039)	(1185)
Control Section 6								
North Lane	610	465	2993	8922	1138	881	1212	1273
South Lane	595	735	3308	2103	1099	1453	1313	1254
(Average)	(602)	(600)	(3150)	(5512)	(1119)	(1167)	(1262)	(1263)

[a]Tests conducted after seal coat surface added to roadway.

Comparative plots of SBV versus time for sections 3, 4, 5 and 6 show a generally equal trend regardless of time or use of Mirafi fabric, though average SBV's of sections 5 and 6 were slightly larger than those of sections 3 and 4, thus partially relating to the more stable subgrade.

Plots of SBV versus time for the Fairfax sections also indicated a generally equal trend regardless of time or use of fabric. However, average SBV's of the stable subgrade sections 5 and 6 were slightly lower than those of sections 3 and 4, a reversed condition similar to that noted within the Benkelman beam discussion.

Portions of the indistinct variability of SBV results between comparable sections may be due in part to the ratio of soil or aggregate particle sizes to diameter of contact area of the spherical penetrometer approaching unity with the macadam stone. If such a condition were occurring, SBV properties of the full vertically profiled roadway systems courses were not being tested. Butt et al. [10, 11] observed that the deformation mechanism operating during penetration of a soil mass by a spherical penetrometer consists of compression and consolidation of the soil, followed by rupture and plastic flow, thus eliminating usage of any elastic theory of evaluation. Deformations under the spherical penetrometer, at least within the Alburnett sections, were similar to those observed by Butt et al. for several secondary roads, indicating a combination of local and general shear during bearing capacity failure. When a bearing capacity failure occurs, shearing takes place along a curved surface emanating down and out from opposite sides of the loading device, then bending toward the surface. Depth of the failure surface may be dependent on the contact area of the device itself, which in turn is dependent on the depth of penetration (in the case of the sphere) and the strength of the various courses of the roadway. Thus the spherical penetrometer may not have produced a depth of shearing into or through the Mirafi-fabric contact zone, yet the values were affected by strengths of the surface courses, base, granular backfill and subgrades.

A parallel can be drawn between the laboratory-derived shear strength parameters (Table 5) and the spherical bearing test results. SBV may be looked upon as a punching shear test involving an undefined combination of c and ϕ. From the laboratory tests it was predicted that these parameters would be influenced at least to some degree by the fabric, particularly after freeze-thaw. In general, the predictions proved accurate. However, of the various sets of data taken after construction, three primary exceptions to the laboratory predictions were noted. These data were from tests performed in May and August 1977 on Alburnett sections 1A through 2B, in May and August 1977 on sections 3 and 4, and in August 1977 on sections 5 and 6. In these cases, the presence of fabric appeared to have little or no influence on the bearing values, suggesting either a divergence between laboratory and field test mechanisms, or the significant role of other unmeasured variables present in the field, or both. Because of its small contact area, the spherical bearing value test subjects material to only a limited shear depth. It is therefore possible that the limited test zone may not have included the fabric-reinforced material, and failure could have occurred within the 6-in. soil-aggregate layer above the fabric. If this were the case, the next question might ask why the SBV test reflected some strength difference between fabric and control sections for a majority of the tests. One possible answer may be found in the effect of field moisture content on the fabric bond at time of testing. The laboratory K-Tests indicated a structural continuity was enhanced after freeze-thaw, or at relatively high soil moisture contents. The coincidence that non-differentiating SBV values were obtained on the same test dates suggest that field soil moisture may be a highly relevant factor in soil-fabric bonding characteristics and should be further studied in much more detail.

3.3.4.3. Plate-Loading Tests. Since spherical bearing value tests did not adequately define performance differences between comparable fabric and non-fabric sections, conventional plate-loading tests were used in conjunction with the Benkelman beam during the latter phases of performance evaluation of the Alburnett sections. Although plate-loading is a somewhat cumbersome and costly test, it has the advantages (1) of being the basis for subgrade evaluation used in several pavement design methods, (2) of providing a subsurface influence configuration roughly equivalent to that of prototype tire loads, and (3) of being readily associated with the modulus of deformation of elastic theory, a basic material parameter which can be back-computed.

A rigid, 12-inch diameter circular plate, consistent with the contact area of the dual wheel tires, was hydraulically loaded, while vertical plate displacements were measured. Uniform plate contact with the roadway was achieved through use of a thin layer of fine sand between the plate and the road surface. Incremental loads, each approximating 10 percent of the maximum anticipated wheel load, were applied and held until the settlement rate was less than 0.002 inch per minute. Figures 28 and 29 are representative plate stress versus deformation plots taken April 13 and June 14, 1979, respectively, on Alburnett Section 5. The April 1979 test, Figure 28, shows a response where appreciable initial deformations were observed at each stress level until settlement stability of 0.002 in./min. or less was achieved, thus giving rise to a second stable deformation-load curve. For the June 1979 test, Figure 29, measurable unstable deformations did not occur at each stress level; thus a single load curve resulted.

Some degree of subjectivity must be applied to the interpretation of plate-loading results. Normally, the modulus of subgrade reaction, K, is defined by

$$K = \frac{p}{\Delta},$$

where p = plate stress at 10 psi and Δ = the corresponding stable deformation value [9]. Values for modulus of subgrade reaction presented in

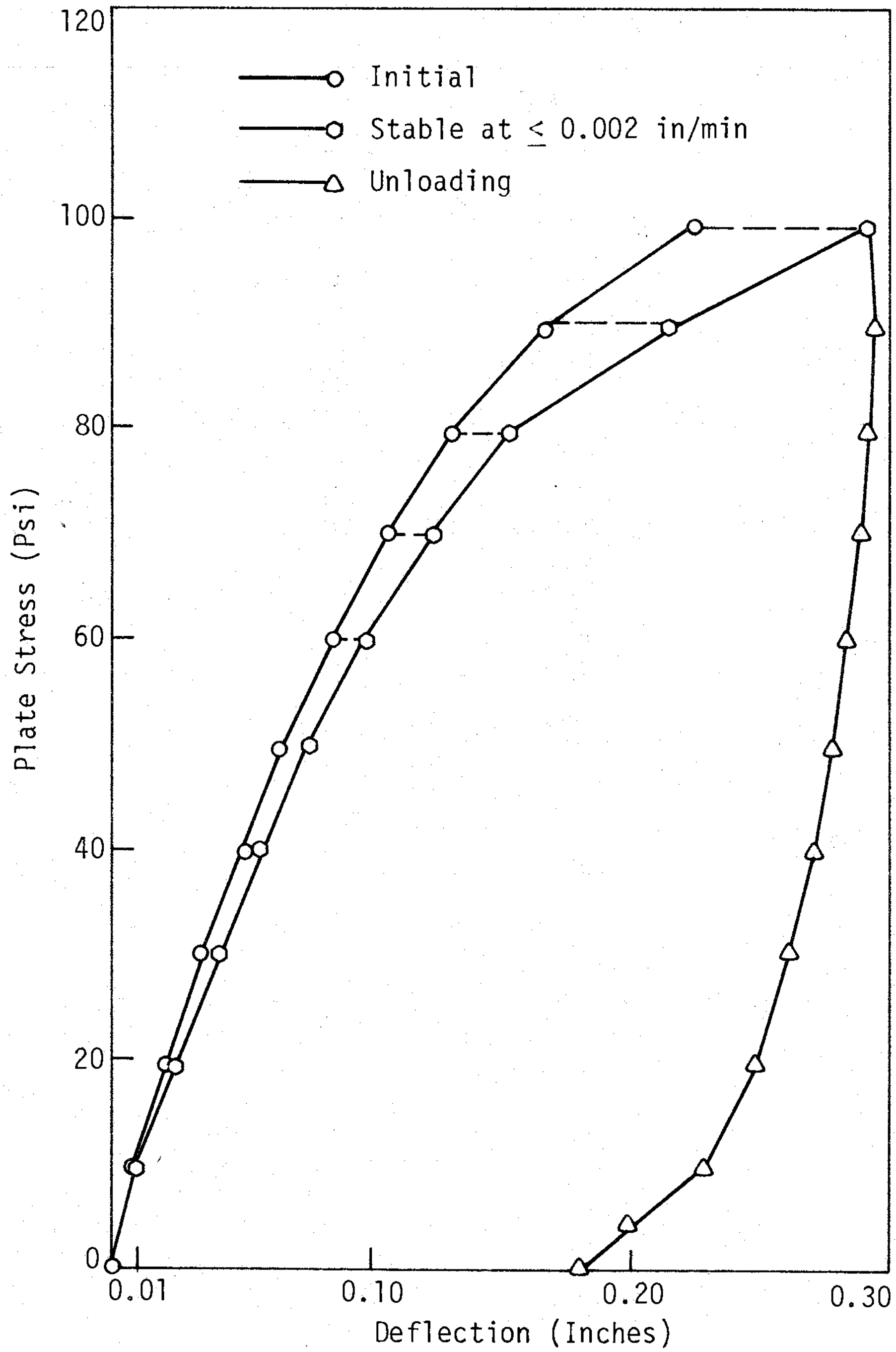

Figure 28. Plate stress versus vertical deflection, Alburnett Section 5, April 1979.

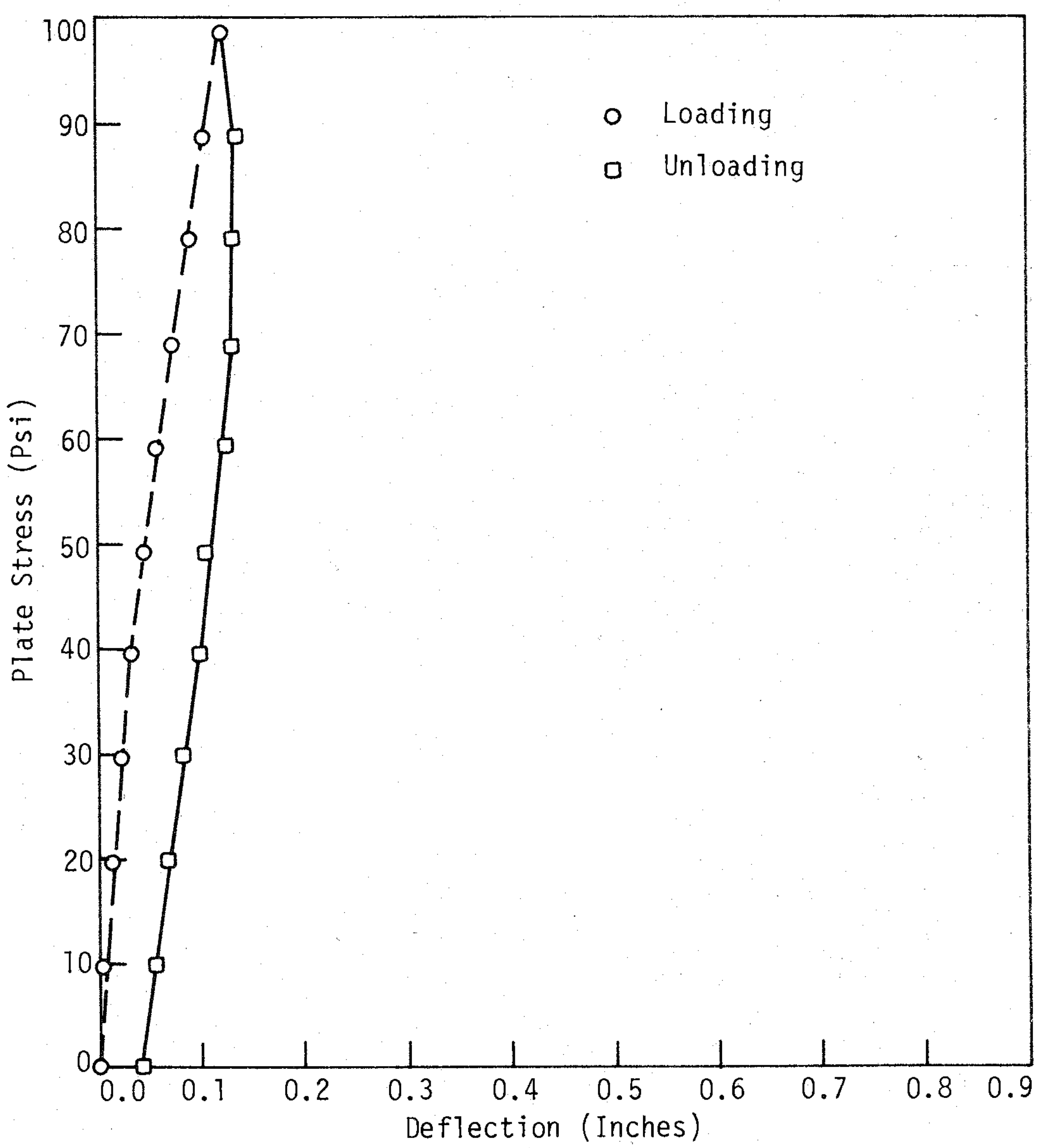

Figure 29. Plate stress versus vertical deflection, Alburnett Section 5, June 1979.

Table 19 were determined through this convention.

Deformation moduli, in column 4 of Table 19, were computed using the relation

$$E = \frac{\pi\ p\ D\ (1-\nu^2)}{4W},$$

where p = plate stress, D = plate diameter, ν = Poission's ratio*, and W = plate settlement. This expression was developed by Burmister [12] for a rigid plate on a homogeneous material. In a strict sense, this equation may not apply to the internal response of the soil-aggregate-fabric composite. However, the resulting deformation modulus is thought to be a valid indicator of net response of the composite system. Where the option existed, plate settlements were taken from the initial loading curves at 75 psi because of the potential for correlation with the Benkelman beam tests.

Loading rate is known to influence deformation response, particularly for nonelastic systems and the Benkelman beam test involves a fairly rapid load application rate. Though loading rates were thus obviously different for both tests, Table 19 indicates a high degree of correlation between test values of deformation at 75 psi stress and modulus of deformation, E, and between the calculated plate bearing and beam values. Three exceptions to this correlation are noted, specifically for three plate bearing tests in which deformations exceeded the capacity of the measuring

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*For comparative purposes ν was estimated at 0.33. This is a value commonly used for unsaturated soils.

Table 19. Comparison of Plate Bearing and Benkelman Beam Test Results

		Plate Bearing				Benkelman Beam		
(1) Section	(2) Date	(3) Subgrade Reaction K (pci)	(4) Deformation Modulus, E (psi)	(5) Deformation at 75 psi	(6) Permanent Deformation (in)	(7) Deformation Modulus, E (psi)	(8) Deformation at 75 psi	(9) Comments
3	4/13/79	189	1050	0.60[a]	0.25	3658	0.172	Plate bearing E determined at 60 psi[a]
4	"	67	373	>0.80[a]	0.40	1261	0.499	Failure at 20 psi plate bearing E determined at 20 psi[a]
5	"	1000	5250	0.120	0.18	5519	0.114	
6	"	714	4375	0.144	0.28	4559	0.122	
1A	6/14/79	2,222	18,000	0.035	0.05	16,134	0.039	
1B	"	1,429	11,050	0.057	0.05	9,534	0.066	
2A	"	1,429	10,500	0.060	0.05	8,503	0.074	
2B	"	2,000	13,700	0.046	0.01	11,873	0.053	
3	"	333	1,115	0.565	0.45	1,523	0.413	
4	"	153	512	0.82[a]	0.50	1,203	0.532	Plate bearing E determined at 50 psi[a]
5	"	2,000	7,500	0.084	0.04	7,402	0.085	
6	"	2,000	8,400	0.075	0.06	8,067	0.078	

[a]These are tests in which deformations exceeded the measurement capabilities of the field plate bearing apparatus.

device before a 75 psi stress could be achieved; i.e., the deformation moduli were computed at the stresses corresponding to the maximum measurable deformation. It should be noted that in one test (section 4, April 13, 1979), stress levels above 20 psi could not be attained. The road failed, thus achieving a state of limiting equilibrium.

A third parameter derived from a plate-bearing test is the permanent deformation occurring after unloading. Values presented in column 6 of Table 19 represent the intersection of the unloading curve with the abscissa of the stress-deformation plots, for example, Figures 28 and 29. The magnitude of permanent deformation is thought to be an important factor in evaluation of secondary roads since economics dictate the use of soil-aggregate systems which will nearly always undergo permanent or plastic deformation. Although the question of acceptable permanent deformation magnitudes has not been universally resolved, experience with many Iowa secondary roads suggests that one-half inch of permanent set represents an upper limit. The April test results for sections 3 and 4 tend to substantiate this estimate. At the time of test, section 3 was performing adequately, while section 4 was apparently near the threshold of unsatisfactory performance, in that permanent deformation observed at section 3 was 0.25 inches, while it was 0.4 inches for section 4.

The relative performance of comparable fabric versus nonfabric sections is indicated by nearly all parameters developed from the plate-loading tests. The April tests on frost-prone subgrade sections 3 and 4 show a 2.8 improvement factor for both K and E where fabric was present (section 3). Tests on strips constructed over the less frost-susceptible subgrade sections 5 and 6 indicate that presence of fabric increased K by a factor of 1.4 and E by 1.2. This supports the previously presented contention that any benefit from fabric may not be realized until the soil is rendered exceptionally weak through some natural process such as frost action or lack of drainage. Further support for this concept is evident from test results for sections 3 through 6 obtained on June 14, 1979 (Table 19). Bearing in mind that the April tests were performed at what is probably the most critical point in the spring thaw period, the June tests represent performance after the frost boils have at least partially healed. Improvement factors for both K and E on the frost-prone subgrade sections 3 and 4 are reduced to 2.2. For sections 5 and 6, over the non-frost-prone subgrade, the K parameter indicates no improvement, while E values suggest that including fabric actually decreased the system stiffness. A similar analysis can be made through consideration of permanent deformations. Specifically, permanent deformations were smaller when fabric was present and a more pronounced difference in corresponding section occurred when the system was soft.

Plate-loading results for section 1A indicate that only slight improvement factors of 1.1 for K and 1.3 for E were achieved when a single fabric layer was placed between the soil-aggregate surface and granular fill. K and E values for sections 1B and 2A were lower than those for the comparable nonfabric section. Thus as of the time of testing, observed performance and field tests suggest that little was gained by using fabric in conjunction with a thick granular backfill under subgrade and environmental conditions similar to those prevalent for this field study.

An alternative approach to the evaluation of the relative merit associated with using fabric to improve secondary roads is an illustrative example of pavement thickness using plate-loading parameters. This is a realistic approach, since county engineers are often faced with upgrading farm-to-market roads through a staged construction process. Grades are improved during one construction season, and when funds become available, the road is paved. Typical traffic data for such a road may be as follows:

Traffic count: 200 vehicles/day

Maximum total gross load: 40,000 lb

Single axle load limit: 18,000 lb

Percent trucks: 30%

One flexible pavement design technique based on plate-loading tests is prescribed by the Asphalt Institute [13] and uses the bearing value on a 12-inch diameter plate at 0.2 inch deformation after 10 repetitions. Since bearing values for a single repetition are all that were performed at Alburnett, the resulting example design would be slightly underconservative and valid for comparative purposes only. Considering the critical Alburnett sections 3 and 4, bearing values taken from the plate-loading curves were 25.5 and 16.3 psi respectively. Section 3 containing fabric would require a 9 inch asphaltic concrete pavement. By the same design method, untreated section 4 would require in excess of 11 inches of pavement. Since this design method has actually been correlated to a minimum bearing value of only 21 psi, in the case of section 3, fabric would thus permit use of a pavement without employing potentially costly subgrade improvement alternatives such as drainage structures or extensive fill.

A similar analysis can be made for a rigid pavement. Assuming the concrete modulus of rupture at 300 psi and the previously indicated traffic information, and using the modulus of subgrade reactions reported for sections 3 and 4 (Table 19), the AASHTO rigid pavement design method [9] indicates a 7.5-inch pavement would be required for section 4, while the fabric-treated section 3 would require 6.5 inches. This difference could be economically significant.

3.3.4.4. Benkelman Beam/Plate-Loading Correlation. Since the Benkelman beam test comprises a large portion of the analyses for this investigation, and since the plate-loading test was used only in the latter phases, a correlation between the two tests was thought to be useful, in that any inferences drawn from one technique should also apply to the other. The obvious approach to such a correlation is through the deformation modulus, as back-computed using the Burmister relation. It is known that both plate and tire contact areas were nearly equal. If it is assumed that surface configurations of each are equivalent, deformation moduli can be computed using the Benkelman beam deflection. In Table 19, column 8, beam deflections measured at approximately the same location at which plate-loading tests were performed are reported, and the computed deformation moduli, assuming $\nu = 0.33$, are listed in column 7. The resulting equivalency between the two parameters is illustrated in Figure 30, the diagonal representing equality. With the exception of one point, the correlation is quite good for E values less than 9000 psi, beyond which the Benkelman beam yields somewhat smaller values than does the plate-load test. Differences in load rate or possibly surface contact configuration (i.e., two tires versus a continuous plate) could account for some of this variation. It is interesting to note, however, that the four comparisons showing greatest deviation from equality occurred within sections having thick granular backfill, Alburnett 1A through 2B. Another speculation is that spacing of the truck dual tires may create a composite elliptical contact area having a major dimension greater than that of the 12-inch plate. This suggests that the Benkelman beam influence could extend farther into the soft subgrade, hence lower moduli. Actually, divergence from the equality line of Figure 30 is insignificant when variations normally experienced between test techniques used for soils and a detailed analysis of the phenomenon are not considered important. The fact that there appears to be a consistent relationship supports the validity of the Benkelman beam test thus used as the primary performance evaluation technique in this report.

3.3.4.5. K-Test Results From In-Situ Samples. Laboratory K-Tests were performed on specimens cut from 12-inch long, thin-walled tube

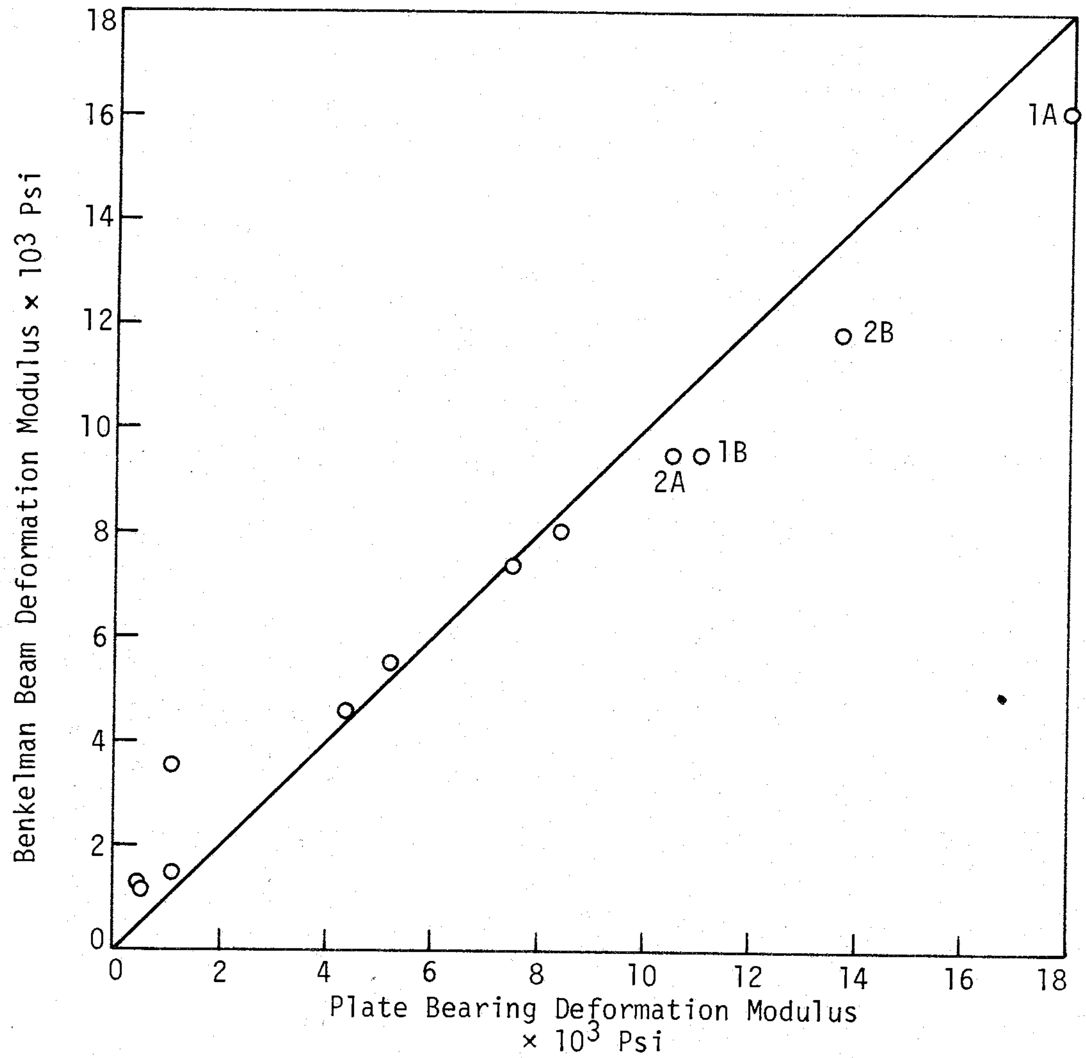

Figure 30. Relationship between deformation moduli as determined from Plate bearing and Benkelman beam tests.

samples randomly removed from the Alburnett sections on April 13, 1979. Table 20 presents the results of these K-Tests. It should be noted that while samples were taken from sections 1A through 2B, the granular nature of the material made extrusion of such specimens unsuitable for testing.

The purpose of this study was to provide a comparison of laboratory derived moduli of deformation from the K-Tests, with those derived in-situ from either the plate bearing and/or Benkelman beam tests. In addition, it was hoped that c-ϕ values from the presumably undisturbed roadway samples would provide data relative to the ultimate bearing capacities of the individual sections. As may be noted from Table 20, deformation moduli are quite inconsistent with the field derived values of Table 19. Sample disturbance appeared to account for at least a portion of the inconsistencies noted. In addition, only the upper portion of each tube, consisting of the soil-aggregate surfacing, was of adequate length to permit proper trimming to a length required for the K-Test. The laboratory c-ϕ values however provide insight to one factor relevant to evaluation of the test sections, i.e., considerable variability of near surface materials, probably due to the nonuniform spot additions of surface aggregate noted during early summer 1978.

Each c-ϕ value of section 4 was used for computation of ultimate bearing capacity in the hope that a correlation could be obtained with the plate-loading failure of 20 psi. However, the high c-ϕ values

Table 20. K-Test Results from In-Situ Samples of April 13, 1979, Alburnett Sections.

Section	Deformation Modulus E_{75} (psi)	Friction Angle ϕ (degrees)	Cohesion c (psi)
3	2405	52.3	15.6
	1500	38.7	10.7
	2676	52.8	12.2
	2519	40.1	19.6
	1970	35.8	13.5
	$\bar{E}$ = 2214	$\bar{\phi}$ = 43.9	$\bar{c}$ = 14.3
	s = $\pm$ 478	s = $\pm$ 8.0	s = $\pm$ 3.5
4	2135	48.2	12.9
	1507	26.7	10.4
	1592	32.8	13.2
	1668	38.4	13.7
	3260	38.8	24.2
	$\bar{E}$ = 2032	$\bar{\phi}$ = 37.0	$\bar{c}$ = 14.9
	s = $\pm$ 728	s = $\pm$ 8.0	s = $\pm$ 5.4
5	1527	40.7	6.4
6	976	34.5	8.6
	2855	44.9	11.8
	$\bar{E}$ = 1915	$\bar{\phi}$ = 39.7	$\bar{c}$ = 10.2
	s = $\pm$ 1328	s = $\pm$ 7.4	s = $\pm$ 2.3

$\bar{E}$, $\bar{\phi}$, $\bar{c}$ represent mean values

s = standard deviation

obtained on the soil-aggregate surfacing resulted in calculated bearing capacities far in excess of the 20 psi bearing failure in-situ, indicating that the failure occurred predominantly within the weakened subgrade materials.

Table 20 also presents the median and one standard deviation of each parameter. The large standard deviations not only indicate the near surface materials variability previously noted, but lend support to the view that field tests such as Benkelman beam and plate bearing are considerably more appropriate for performance evaluations. Such in-situ tests provide a more adequate performance measurement of the composite surface-fabric-subgrade system.

4. CONCLUSIONS

Many secondary roads in the upper Midwest are vulnerable to loss of traffic support capacity each spring, particularly as induced by excessive moisture in thaw-softened subgrades. It was the purpose of this investigation to evaluate in-situ performance of Mirafi 140 fabric as an interlayer reinforcement between a frost-susceptible fine grained subgrade and either a soil-aggregate surfaced or granular based roadway.

Seven Mirafi test sections were constructed in October 1976 in Linn County, Iowa, five on a soil-aggregate surfaced road north of Alburnett and two on a macadam base roadway northwest of Fairfax. Each test section was paired with an adjacent control section constructed in the same manner as the test section, but lacking fabric. Following is a brief description of each section:

1. Alburnett sections 1A through 2B consisted of a 2 ft. thick granular backfill placed in an undercut of a frost-susceptible subgrade and overlaid with a 6-inch soil-aggregate surface. In section 1A the fabric was placed between the surface course and granular backfill. Fabric encapsulated the granular backfill in section 1B. Granular backfill was placed in a trough of fabric for construction of section 2A. Section 2B was the control, in that it contained no fabric.

2. Alburnett sections 3 and 4 were constructed as a 6-inch thick soil-aggregate surface overlying a frost-susceptible subgrade. Fabric was interlayered between the surface and subgrade materials in section 3.

3. Alburnett sections 5 and 6 consisted of a 6-inch thick soil-aggregate surface overlying a stable, non-frost-susceptible subgrade. Fabric was interlayered between the surface and subgrade in section 5.

4. Due to necessities outlined within the text of this report, Fairfax sections 1 and 2, similar to Alburnett sections 1 and 2, were not constructed.

5. Fairfax sections 3 and 4 consisted of an 8-inch thick macadam base overlying a frost-susceptible subgrade. The base was topped with a 4-inch layer of choke stone and ultimately a thin asphaltic concrete wearing surface. Section 3 contained a layer of fabric between the base and frost-prone subgrade.

6. Fairfax sections 5 and 6 were constructed in the same manner as sections 3 and 4, with the exception that the subgrade was considered stable, or non-frost susceptible. Section 5 contained a layer of fabric between the base and stable subgrade.

Though the investigation reported herein was primarily a field performance evaluation, both laboratory and in-situ tests were conducted on the various materials and within each test and control section location. The following general conclusions are thus based on parameters obtained through evaluation of such items as freeze-thaw tests, stability/strength analyses, in-situ moisture and density tests, climatological data, Benkelman beam, and plate-bearing tests.

1. Laboratory and in-situ data within this study indicate a reinforcement capability when Mirafi 140 is interlayered between a soil

aggregate surface and a frost-susceptible subgrade. Further reinforcing improvements can probably be achieved in frost-susceptible secondary roads utilizing fabrics having greater elastic characteristics than the 140.

2. Laboratory freeze-thaw tests of a frost-susceptible fine grained soil interlayered with one or more Mirafi 140 fabric discs indicated a reasonable control of freeze-thaw elongation.

3. Laboratory stability/strength tests of a frost-susceptible fine grained soil interlayered with one or more Mirafi 140 fabric discs indicated some degree of improvement of parameters due to reinforcement both prior to and after subjection to freeze-thaw. Two layers of fabric appeared more beneficial than a single layer.

4. Over the three years of this study, Iowa experienced the coldest winter (1976-1977) since 1936. November 1976 was the driest month ever recorded in Iowa. By the end of February 1977, drought had reduced subsoil moisture to an average of 98 percent short of normal. By December 1977, subsoil moisture was still 12 percent short of normal, and it was not until 1978 that normal subsoil moisture conditions existed. The combination of these extremes of climatological conditions, particularly subsoil moisture feed of the frost-susceptible subgrades, tended to complicate field performance evaluations until the 1977-78 heave/boil season.

5. Twenty months after construction, and following two seasonal frost cycles, a limited degree of moisture-density stability was noted only within the subgrade of Alburnett section 3. A significant reduction

in density and increase in moisture content were observed in the subgrade of Fairfax sections 5 and 6, but were only partially reflected in the in-situ performance data due in part to the macadam base rigidity bridging the weakened subgrade.

6. On the basis of in-situ Benkelman beam deflections and relative stiffness values, plus moduli of subgrade reactions and deformations determined from plate-bearing tests, Mirafi 140 fabric performed most favorably during the three-year study as a reinforcement between a soil-aggregate surface and frost-prone subgrade (Alburnett section 3).

7. Based on the in-situ Benkelman beam and plate bearing parameters, use of Mirafi 140 fabric in conjunction with granular backfill in an undercut frost-susceptible subgrade soil does not appear significantly justifiable.

8. Based on the various performance parameters, use of Mirafi 140 fabric does not appear justifiable as a reinforcement between (1) a soil-aggregate surface and stable non-frost-susceptible subgrade, or (2) between a macadam base and either frost or non-frost-susceptible subgrades.

9. Use of Mirafi 140 fabric as a reinforcement for frost-susceptible secondary roads should probably be limited to apparent and/or suspected areas of frost-prone subgrades underlying base and/or surface materials having less rigidity than a macadam base. Such fabric reinforcement appears primarily desireable within limited areas of a soil-aggregate surface overlying a frost-prone subgrade and can be easily constructed by county maintenance personnel.

10. Field soil moisture may be a highly relevant factor in soil-fabric bonding characteristics and should be studied in much greater detail.

11. Laboratory investigations, similar to the freeze-thaw and Iowa K-Tests performed on a limited basis in this study, should be expanded to a wider range of subgrade soils, in order to increase the reliability and predictability of in-situ fabric effectiveness in frost-susceptible secondary roads.

5. ACKNOWLEDGMENTS

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