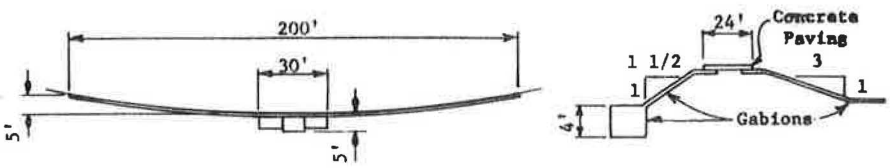
Figure 7. Elevation view of weir (left) and road cross section at weir (right).



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Ten-Year Performance Report on Asphalt-Stabilized Sand Road with Instrumentation

EUGENE L. SKOK, JR., TEJ S. MATHUR, NORMAN G. WENCK, AND NEIL RAMSEY

In 1972, a test road that consisted of 12 sections of sand stabilized with bituminous materials was built in the Chippewa National Forest near Cass Lake, Minnesota. The local outwash sand is a very clean and one-sized material. One-half of the sections were stabilized with a 200-300 penetration asphalt cement and the other half with a medium-curing (MC) 800 cutback. Thicknesses were determined by using criteria based on strains calculated with the elastic-layered system. The permanent deformation criteria were shown to be critical for the predicted timber and recreational traffic. Thicknesses of 3.5, 5.0, and 8.0 in were constructed with each stabilizing material. These thicknesses represented design lives of 5, 8, and 15 years for the predicted traffic, respectively. Mixing of the materials was done in an asphalt batch plant and lay down with a standard paver for uniformity and control. Compaction was based on laboratory densities. Strain-measuring devices composed of induction coils were placed in the sections to monitor static (long-term) and dynamic deformations in the pavement layers. These have been compared with calculated values. It was found that the strains in the asphalt-cement-stabilized sections were close to the calculated values, but that the strains in the MC-stabilized sections were higher, probably because of incomplete curing. Pavement condition has been evaluated by using present serviceability index, rut depth, cracking, and deflections. After 10 years of service, the sections have all performed well under the applied traffic with no maintenance overlay, surface treatment, or seal coat. Design comparisons are made to evaluate how this performance information can be used to improve current design methods.

The Forest Service, U.S. Department of Agriculture, in cooperation with the University of Minnesota, has sponsored the design, construction, and evaluation of a test road in the Chippewa National Forest. The test road was designed and constructed on a timber access and recreational road called the Third River, which is located northeast of Cass Lake, Minnesota, north of US-2 off of Cass County Road 10. It crosses the Mississippi River between Cass and Winnibigoshish Lakes. The Third River Road is divided into three major segments--designated A, B, and C. Segment A includes the first twelve 1000-ft test sections constructed in 1972. Six of these have asphalt-cement-stabilized sand and six have medium curing (MC) cutback-stabilized sand of various thicknesses as pavement structures. The six 2000-ft segment C test sections were constructed in 1975 and are composed of the same outwash sand stabilized with asphalt emulsion. Segment B is a short portion of the road near the Mississippi River constructed in 1970 with an emulsion-treated sand base and a conventional asphalt concrete (AC) surface. The sections have been evaluated based on their structural condition, ride, and strength. The strength has been evaluated by using a number of techniques: strain sensors in the pavement, Benkelman beam deflection device, and road rater.

Because there is not much information available for designing stabilized-sand pavements, a number of different design procedures and values were adapted from standard pavement design procedures. In order to verify the designs, the test road was instrumented to monitor strains and deformations that occur within the pavement to relate these parameters to the actual performance of the sections.

The criteria used for evaluating the performance of the test sections include (a) present serviceability rating of the section over a period of time, (b) rut depth measured at the surface, (c) cracking, and (d) surface condition. Static deformations have also been measured electronically to compare them with the measured rut depths. Dynamic deformations under moving axle loads have also been determined, and an attempt is made to relate these to the performance of the sections. The design and performance evaluation must take into consideration the effects of traffic and the environment on the road. Traffic has been analyzed in terms of equivalent 18 000-lb single-axle loads. The traffic on the sections has been a mixture of loaded and unloaded timber trucks, recreational vehicles, and cars.

By any measure of performance, the sections stabilized with asphalt cement and MC cutback are in good condition and have performed better than anticipated over the 10-year period. There was no surface treatment put on the sections as planned after construction, and there has not been any major maintenance expenditures on the pavement surface of the section A test areas. This, plus the fact that the original construction cost less than a gravel road with surface treatment, makes this type of construction a good alternative for recreational and timber-haul roads.

The original design of the test road was based on elastic theory calculations of strains in the pavement section. In order to check the levels of strain actually developed, dynamic strain measurements have been made periodically. These are summarized and compared with the calculated values used for design. Recommendations are made for use of this concept of pavement design.

DESIGN OF MATERIALS AND PAVEMENT SECTIONS

Description of Materials

The test road was originally set up to determine a mixture and thickness design for low-volume rural roads by using local materials as much as possible. Initially, it included two asphalt materials, two percentages of each of these materials, and three thicknesses of stabilized material based on predicted design lives of 2, 5, and 10 years. The naturally occurring material at the test site is a sand. Stabilization of this material was considered a good design alternative. Asphalt cement and an MC liquid asphalt were chosen as stabilizing materials. Details of the material properties and design are included elsewhere (1-5). A brief summary of the materials and design parameters are included in this section.

The naturally occurring material at the Third River Road is a very uniform fine outwash sand. The table below gives the gradations:

Sieve No.	Total Percentage Passing
4	97
10	94
40	59
80	15
200	1.3

The maximum density is 118 lb/ft^3, and the optimum moisture content is 9 percent (according to AASHTO T-180). The American Association of State Highway and Transportation Officials (AASHTO) classification is an A-3; the measured R-value and California bearing ratio (CBR) are 70 and 10, respectively; and the specific gravity of the particles is 2.60. The outwash sand is an AASHTO A-3 material and has a specific gravity of 2.60.

An AC 250-300 was selected rather than a lower penetration asphalt to reduce the probability of thermal cracking. The MC-800 material was selected to allow a comparison of asphalt type with construction practices and performance in the test road. Standard tests were run on the asphalt materials to determine the amount of hardening with heating plus the percentage of volatiles in the liquid asphalt. The percentage of the original penetration after the standard thin-film oven test (TFOT) was 55 percent, which is well above the minimum of 37 percent specified by the Forest Service for 200-300 penetration asphalts (2). The results of the loss-of-heating test for the MC-800 show that 14 percent of the original weight was lost after 5 h in the oven at 325°F. The penetration of the residual asphalt ranged from 37 to 40, which indicates a base asphalt harder than the AC 250-300.

Design of Stabilized Mixture

The design of the stabilized mixes is covered in Skok (1) and Root and others (4). To determine the appropriate amount of asphalt material to mix with the sand, repeated-load triaxial tests were made on samples of the sand material mixed with various percentages of the AC 250-300 and the MC-800 materials.

In addition to resilient modulus, the slope of the permanent strain with time was determined. This value makes it possible to obtain a comparison between the various asphalt contents with respect to the resistance to permanent deformation (3).

From the results of these tests, 5 and 7 percent asphalt contents were chosen for the sand stabilized with asphalt cement.

The MC-800 specimens were cured after mixing to simulate curing in the field. The average loss due to the curing procedure was 11.4 percent. The 7 and 9 percent MC-800 mixtures were selected for the liquid-asphalt-stabilized sections based on resilient and permanent strain measurements (3,6). The percentage of MC includes the diluent.

Thickness Design Procedure

In order to design asphalt pavements by using the elastic theory, three items are required. First, it is necessary to determine an appropriate failure criteria that will predict the failure of the pavement section. Second, it is necessary to determine the critical strains used in the pavement section as defined by the failure criteria with the various thicknesses of the pavement section. Third, the number and magnitude of the axle loads to use the proposed pavement section are needed in order to predict the life of various pavement sections by using the failure criteria.

From analysis of the materials and mixtures it was determined that permanent deformation or rutting would be the failure criteria (3). The other possible cause of failure (fatigue cracking) in the stabilized material could also occur as a result of repeated axle loading.

The allowable values of vertical strain on the embankment were based on an analysis of the American Association of State Highway Officials (AASHO) Road Test by Dorman and Metcalf (7). For that analysis, the level of vertical strain on the embankment for a section with different levels of traffic was determined (3).

A series of repeated-load triaxial tests were made on the embankment materials from the three locations. A number of investigators have shown that the resilient modulus of a granular material is dependent on such factors as deviator stress level and lateral stress, in addition to a number of other factors (8,9).

More than 150 repeated-load triaxial tests were run on the stabilized embankment materials. In general, the results indicate that the resilient modulus was independent of vertical and lateral stresses at this temperature for the test ranges studied. The results of these tests and analyses are discussed in Gardner and Skok (3) and Root and others (4).

The values of vertical strain on the embankment and tensile strain on the stabilized layer were calculated by using the Chevron five-layer computer program (10). A 9000-lb single-axle load was used in the calculation. A resilient modulus of 70 000 psi was used for the stabilized sand material and 20 000 psi was used for the sand subgrade soil.

Computed thicknesses that use this information can be developed into design curves for stabilized materials of various moduli, as shown in Miyaska (2), Gardner and Skok (3), and Root and others (4).

Test Section Layout

Based on the results of the laboratory tests and analyses, bituminous content and thickness of the test section have been determined. One bituminous content was chosen for each stabilizing material because of the limited size of the test road. Bituminous contents of 6.0 percent for the AC and 6.5 percent for the MC were used.

The test road was divided into twelve 1000-ft sections that had thicknesses of 3.5, 5.0, and 8.0 in and one of the two types of bituminous-treated layers. The layout of the 12 sections is shown in the table below. The sections are arranged in random order so that no bias is built into either the construction or performance of the sections.

Section No.	Bituminous Type	Thickness (in)
1	MC	8
2	MC	5
3	AC	3.5
4	MC	3.5
5	AC	5
6	AC	8
7	AC	5
8	AC	8
9	MC	8
10	MC	5
11	MC	3.5
12	AC	3.5

From information provided by the Chippewa National Forest, a traffic analysis was made for the pavement thickness design. The design traffic in terms of equivalent 18 000-lb single-axle loads was calculated. Originally, the design traffic, in terms of equivalent 18 000-lb single-axle loads, was determined to be 2200, 6700, and 19 000 for design periods of 2, 5, and 10 years. Traffic analysis is covered in more detail in Miyaska (2). By using the design curves shown in the reports, thicknesses of 3.5, 5.0, and 8.0 in result for the design periods for 2, 5, and 10 years. These thicknesses are based on limiting vertical strain in the embankment because this criterion was shown to be critical compared with the fatigue-cracking criteria based on tensile strain in the stabilized layer.

CONSTRUCTION

The Third River test road construction started in November 1971 and was completed in September 1972. Although the majority of the road is located on the A-3 outwash sand subgrade, there are portions of two sections on clayey and silty sands. For uniformity, a minimum sand subcut 2 ft thick was placed over the natural subgrade. The construction specifications required the compaction of the upper 2 ft of embankment to 95 percent of AASHTO T-180 and the remaining embankment to 85 percent.

The specifications called for a sprayed prime of MC-70 at 0.4 gal/yd^2 over the finished subgrade for the full length. After observing that the construction traffic was having difficulty traveling through the first mile (test sections 1 through 4) of the road, it was decided to use a mixed-in-place prime of RC-250 in this portion. It was applied at the rate of 0.5 gal/yd^2 and mixed to an approximate depth of 3 in.

Once the subgrade was prepared, the stabilized-sand layer was mixed in a batch plant and laid down with a standard paver. A chip-seal coat was considered initially but was later dropped to expedite curing of MC sections and because the mix looked okay without it. The asphalt content remained within the ±0.25 percent tolerance limits for both the 5 percent AC and 7 percent MC mixtures. Temperatures of the asphalt just before mixing did cool below specification values occasionally, but not for any significant periods of time.

Specifications based on laboratory tests required a minimum density of 120 lb/ft^3 for the stabilized sand. Densities monitored by using nuclear gauges in the MC sections were generally more than 120 lb/ft^3, but in some AC sections they were mostly less than 120 lb/ft^3. A greater compactive effort was applied without much effect. When the remolded sample densities from these sections were found to be consistently lower than those obtained initially, the work was accepted, assuming that sufficient compactive effort had been applied.

Difficulty was also encountered in placing and compacting MC sections due to the plastic nature of the material on hot summer days. It was overcome by placing the entire 8-in thickness as quickly as possible. This resulted in a short curing time. The construction was finished on schedule and the road opened to traffic in September 1972.

DESCRIPTION OF INSTRUMENTATION

Instruments and Layout

The instrumentation system was designed to measure three variables: strain, temperature, and traffic. The strain measurement system measures both long-term static (permanent deformation) and transient (dynamic strains) in the subgrade, subbase, and stabilized-sand pavement. The basic components for these measurements are wire-wound induction coil strain sensors that use the principle of an electromagnetic couple between sensor pairs. This type of sensor requires no mechanical linkage, thereby minimizing the disturbance of the material to be measured. A sensor pair consists of two disk-shaped coils that can be placed either coplanar or coaxial. The sensor layout is shown in Figure 1. Each of the 12 test sections of section A have identical layouts. The dynamic pattern was designed to measure

1. Vertical strain in the stabilized pavement,
2. Transverse and longitudinal strains at the interface between the pavement and the subbase,
3. Vertical strains between this interface and a point 7.5 ft into the subgrade, and
4. Strains between the 7.5-ft point and a point approximately 13 ft below (this lower point is considered to be below the expected frost line).

Although the primary purpose of this pattern is to measure dynamic strains, it also has the capability of measuring long-term static deformations. By using five strain gages specially wired together, all dynamic readings could be taken simultaneously.

The temperature measurement system consists of temperature sensors embedded in the middle of the pavement at each test section. The sensors are capable of measuring temperatures between -50°F and 180°F. These sensors are located in one of the thickest and one of the thinnest test sections.

The traffic-monitoring system consists of conventional pneumatic-tube traffic counters that have the ability to print the date and time each vehicle passes over the point. Two such devices were located to measure traffic in both directions. The result of the counters were supplemented with a photographic detection system.

A total of 408 strain sensors and 12 temperature sensors were installed. The system is working satisfactorily except for four pairs of sensors (all 2 in in diameter) that have failed. (Pictures and slides of the construction of the road and installation of the instruments are available from the authors.)

Static measurements were made weekly for the first several weeks after the pavement was completed to assess the initial strains. The objective of the static measurements are to measure the long-term strains and deformations. The procedure for reduction of the data is presented in Skok (1, Appendix I).

Dynamic measurements are taken less frequently than the static measurements. Dynamic measurements have been taken 12 times over the 10-year period. These measurements were taken over a one-week period of intensive test runs. Typical measurements taken at one station include the static measurements of the dynamic pattern, dynamic measurements with a known wheel load with no magnetic parts, and dynamic measurements with a test vehicle (loaded truck) at various speeds.

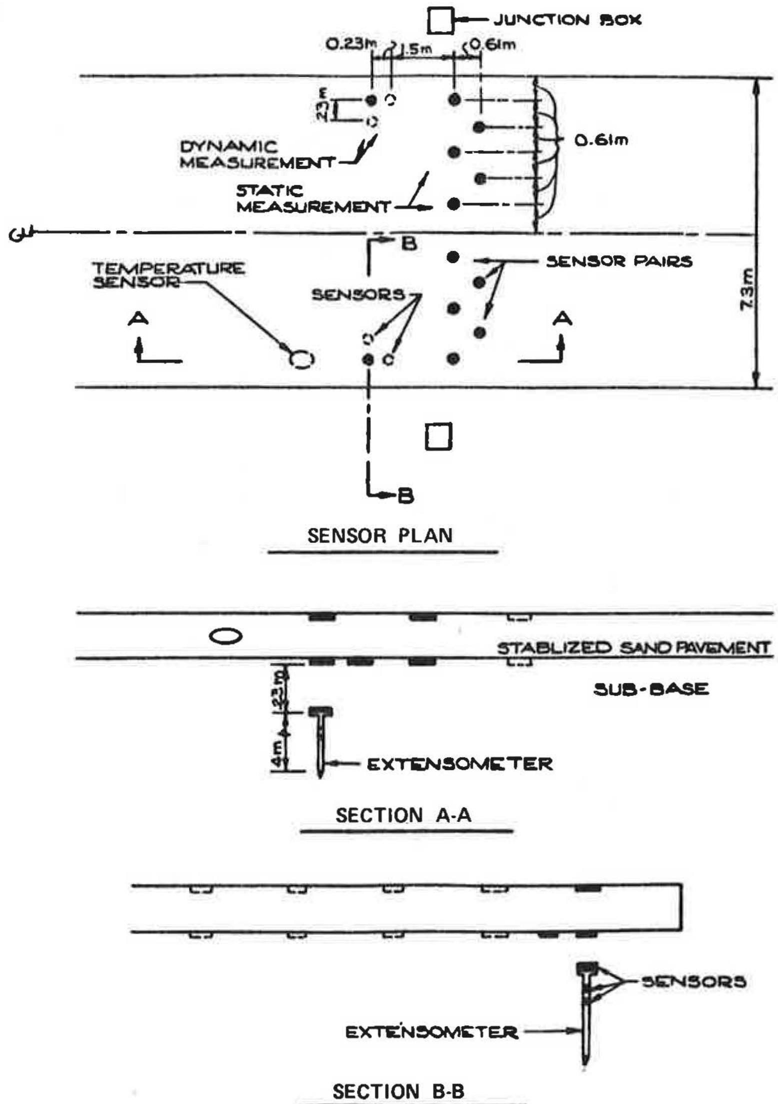
Temperature measurements were made continuously at two stations and were made at other stations whenever either static or dynamic measurements were taken. Traffic measurements are taken on a continuous basis. A visual traffic count and distribution determination were taken during the summer of 1973 to supplement the counter and camera data. Other data observations made on the road include road condition surveys, mechanical surface roughness measurements, and deflection measurements.

Evaluation of Strain Data

Initially, an analog computer system was connected to a digital computer (CDC 3300) at the University of Minnesota Health Computer Science Center. A fully automated data-processing procedure was developed by using a computer program to interpret all the data and to produce all the necessary digitized outputs such as maximum strains, loading rate, and truck speed. However, it was found that because of the diversity and the complexity of the data pattern, manual interpretation was easier and cheaper. The results have been plotted by using the calibration signal as the unit scale. This scale makes it easier and possible to read and calculate the strain manually.

Two types of static test results were obtained. One is the data from the static sensor pattern and the other is from the dynamic sensor pattern. A calibration curve, which gives the distance between the sensors, is used in both cases. The distances between sensors obtained from the first measurement

Figure 1. Sensor layout.



(just after construction) are used as the initial distances. Permanent deformation can be measured by repeating the measurement after a period of time. Five static sensor pairs and the two top pairs of the dynamic sensors provide the long-term deformation of the stabilized layer.

Data Analysis

Table 1 is a summary of dynamic test results for the 18 000-lb loaded single axle. The average values of four sets of data (replications of two sections and two lanes) are listed in the table.

Inspection of Table 1 shows the following:

1. Significantly larger strains and deflections were obtained for the MC sections than for the corresponding thicknesses of AC sections for each type of measurement.
2. A smaller vertical strain of the upper part of the subgrade was obtained with the increase in thickness of the stabilized layer. This relation was observed for both the ACand MC-stabilized sections.
3. Vertical strain of the stabilized layer decreased as thickness increased.
4. Both longitudinal and transverse tensile strains at the bottom of the stabilized layer decreased as thickness increased.
5. For the AC-stabilized sections, surface deflections decreased as the thickness of the stabilized layer increased. However, this tendency was reversed for the MC-stabilized section. The softness of the MC-stabilized sand most likely caused this result.
6. Deflection of the subgrade decreased as thickness of the stabilized layer increased. This was more pronounced in the AC sections than in the MC sections.

Comparison of Measured and Theoretical Strains and Deflections

Measured strains and deflections were compared with computed values. The N-layer elastic system program was used for calculating strains. From the repeated-load triaxial test results, elastic moduli of 55 000 psi for stabilized layers (both AC and MC) and 20 000 psi for the subgrade material have been used. A Poisson's ratio of 0.4 was used for both layers. Because very large strains and deflections were measured in the MC sections, the computed results are not shown for the MC sections. Plots and analyses of the measured strains are presented elsewhere (1,2).

The measured surface deflections are somewhat scattered and cannot be compared directly with the computer results for the MC sections. However, as the thickness of the stabilized layer increases, the surface deflections tend to decrease for the AC sections and tend to increase for the MC sections. For the AC sections, the measured subgrade deflections agree fairly well with the computed results. For the MC sections, extremely large values were measured, especially for surface deflection. Although surface deflections for the MC sections tended to increase with thickness, the subgrade deflection decreases with thickness. This shows that a significant portion of the deflection is occurring within the MC-stabilized layers themselves.

In summary, the analysis of the measured data indicates the following:

1. The strains and deflections measured for the AC-stabilized sections agree relatively well with the computed values.
2. The results prove that the necessary strains for a theoretical pavement design can be approximated by using repeated-load triaxial test results and the elastic-layered system can be used to simulate the pavement section.
3. The strains and deflections measured for the MC-stabilized sections were much larger than expected, especially in the stabilized layer. The reason for this is that the elastic modulus measured in the laboratory was significantly higher than what was actually achieved in the field. This is because the MC material did not cure out properly during the construction and subsequent weathering on the test road. Referring back to the test results on the MC material, it was found that the resulting material was harder than the AC material after the TFOTs. Therefore, it is felt that with time, as the sections harden, the measured strains will tend to decrease in the MC sections.

By using the same material property values for design as mentioned previously, the design strains for the various stabilized-sand thicknesses are shown in Table 1. These strain values are used as the design values for correlation with measured field strains.

Vertical and tensile strain data that have been collected during the 10 years of observation on the Third River Road can best be compared with the design strains.

The data values for a temperature range of 65°F to 80°F were measured in the stabilized sand at the time of testing and represent truck speeds of 15-35 mph. These limits are put on the data values so that the effects on the strains are reduced, which gives a more uniform range of strain values for comparison with the design strains.

The correlation between the field measurements and predicted failure strains reveal that the design of the test road was conservative. According to the

Table 1. Summary of typical dynamic test data for 9000-lb wheel load (creep speed).

Item	AC Sections 3.5 in	AC Sections 5.0 in	AC Sections 8.0 in	MC Sections 3.5 in	MC Sections 5.0 in	MC Sections 8.0 in
Vertical strain						
Stabilized layer (measured 10^{-3})	0.90	1.52	1.42	9.80	9.69	7.20
Subgrade (calculated 10^{-3})	1.28	1.18	0.80	1.37	1.27	0.96
Tensile strain						
Longitudinal (10^{-3})	0.45	0.47	0.32	1.63	1.68	1.10
Transverse (10^{-3})	0.56	0.45	0.23	1.32	0.77	0.59
Surface deflection (10^{-3} in)	23	29	25	72	66	82
Subgrade deflection (10^{-3} in)	23	21	17	27	17	26
Design failure strains						
Vertical strain (10^{-3})	1.88	1.58	1.08	1.88	1.58	1.08
Tensile strain (10^{-3})	0.65	0.63	0.46	0.65	0.63	0.46

vertical strain on the embankment, criterion failure should be occurring in the 3.5-in sections after seven years at the present time, with failure of the 5.0-in sections imminent. Fatigue cracking should be causing some failure in the 5.0-in sections at eight years as a result of the failure tensile strain being reached through the accumulation of equivalent loads. However, both observations of the road surfaces and the strain data show no indication of surface failure. Neither cracking nor rutting failures have occurred in the stabilized sand. In the next section, observations of data on rutting and surface cracking show that these measures of deterioration are minimal when compared with levels considered as failure.

Probably the two most significant factors that affect strains are the curing of the MC mixture and the truck speed as it passes over the sensors. The 8-in MC sections were placed without allowing the intermediate lifts of material to cure as is normal practice. The strain data reveal that the MC sections are softer than expected, most likely due to lack of curing, which would reduce the resilient modulus of the mixture. The other factor likely affecting the strains in a distinguishable way is the speed of the truck. With the truck moving at 20-30 mph across the sensors, which is probably slower than most of the traffic, a loading time of 0.02-0.03 s can be expected on the sensors. This agrees with projections from the AASHO Road Test (11). However, during the triaxial tests on the stabilized-sand mixture, a loading time of 1 s was used. Thus, the laboratory loading may be 50 times greater than is applied in the field, and thus the moduli predicted from laboratory tests are very likely to be less than those actually attained during field tests. Van der Poel's nomograph (12) for determining bitumen stiffness illustrates this effect. For using the nomograph, the bitumen stiffness, which relates directly to the resilient modulus, decreases very noticeably when loading time is slightly increased. The lower moduli obtained would predict higher strains for failure than would be measured. In some cases, the measured field strains are only 10-20 percent of those predicted in the design. The tensile strains are closer to the predicted values than the vertical subgrade strains at the lesser thicknesses. This implies that failure of all the sections will likely occur because of fatigue cracking rather than rutting. With the strain data gathered thus far, it is evident that a modification factor either needs to be applied to the stabilized-sand modulus or to the predicted strains. The value of such a factor cannot be determined until failure begins to occur in the sections.

The report by Skok (1, Appendix II) has a listing of the dynamic test results in June 1978. The level of dynamic deflections and strains has remained essentially the same over the six-year period.

STATIC STRAIN MEASUREMENTS

Static deformation readings were taken from four to six times a year early in the test and once or twice per year the past few years. Dynamic strain data were collected annually through 1982.

Examples of the data obtained from the static tests are shown graphically in Skok (1). The following are a number of the observations made from the static deformation data:

1. Similar deformations occur in the thinner sections in both the MC and the AC sections, while in the 8-in sections the MC shows greater permanent deformation than the corresponding AC sections.
2. Greater deformations occur on the edges of the driving lanes than in the middle.
3. The southwest-bound lanes (SWBL), with greater traffic equivalent loads, generally show greater deformations than the sections in the northeast-bound lanes (NEBL).

These observations are expected and are supported by the bulk of the data.

The static-dynamic data, which are data obtained from the dynamic sensor pattern evaluated in a static mode, provide additional deformation information. These show that

1. Greater consolidation and variability is generally apparent in the MC sections than in the AC sections, except the 5-in-thick sections. In the 5-in sections, there has been little difference between the AC- and MC-stabilized materials.
2. As stabilized-sand thickness increases, the surface elevation variability decreases.
3. The 5-in sections show seasonal variations, such as swelling in winter.
4. Greater traffic loading in the SWBL generally results in greater variability of elevation values. This is more obvious the AC than the MC sections.
5. Subgrade elevation changes closely parallel the stabilized-layer changes, which indicate that a great proportion of the movement occurs in the subgrade. Some consolidation of the stabilized material is apparent.

The only unusual observation from these data is the greater impact of traffic on the AC sections. Although the MC sections are softer and may consolidate more, the deformations of the AC sections seem to be more traffic dependent.

TRAFFIC EVALUATION

An important element in the design of a pavement, and therefore for the test road, is the prediction of the traffic loading. The method used to represent the traffic loading for design is a determination of equivalent 18 000-lb single-axle loads. The predicted accumulated number of equivalent loads forms the basis for describing the fatigue effect of the structural or functional deterioration of a pavement. The structural deterioration is in the form of fatigue cracking or rut depth, and the functional deterioration is in terms of decrease in present serviceability rating (PSR).

Determining the equivalent axle loads that a road is sustaining is dependent on the average daily traffic (ADT), distribution of vehicle types within the traffic stream, and the damage effect of individual vehicle types. The traffic monitoring system is designed to provide a prediction of these parameters for the Third River Road. The system consists of two traffic counters on the road and a battery-operated 8-mm camera that was actuated by the movement of vehicles. All vehicles that pass along the road are recorded by the two conventional pneumatic-tube traffic counters. The types of vehicles that use the road have been observed with the movie camera. An infrared sensor activates the camera for two or three frames when movement occurs in front of the sensor. The different kinds of vehicles and the relative percentage of each kind are determined by viewing the developed film. Due to various problems, this photographic detection system has not worked well consistently. However, adequate data have been collected to provide a good determination of the distribution of vehicle types. The other parameter necessary to determine equivalent loads is the N18 vehicle factor or average vehicle effect. This factor varies from one vehicle type to another and is dependent mainly on the weight of the vehicle and axle type. The relative damage effect of a vehicle is measured by considering the weight and type of individual axles. The results of the AASHO Road Test have been used along with typical empty and loaded timber truck weights to add up total effects of each truck. Predictions of what traffic would use the road were used to design the pavements. The monitoring system is being used to check the predictions. The method of summing up the equivalent loads is presented in Skok (1, Appendix III). A table that shows the different vehicle types and their N18 factors is found in the appendix along with more detailed procedures for calculating the equivalent loads for any given time period.

The accumulations of equivalent loads are shown for both counter stations in Figure 2. This shows that the SWBL carries several times the number of equivalent loads as the NEBL. This is because most of the timber trucks in the SWBL are loaded while they are unloaded in the NEBL. The total traffic for both lanes taken together has been increasing at a rate greater than predicted. Examining Figure 2, it is seen that the NEBL has axle loadings less than predicted while the SWBL carries loads at approximately those anticipated. Thus, the SWBL would be expected to fail sooner than the NEBL. However, as shown in the next section, there is not much deterioration in terms of any of the performance parameters as of yet.

CONDITION OF TEST SECTIONS

The condition of pavement is defined by a number of parameters. The overall functional condition is defined by present serviceability index (PSI), which is a measure of how well the pavement rides. Other parameters employed to determine the condition of the pavement are rut depth, cracking (transverse, longitudinal, and patterned cracking), wear, and weathering. Observations were made during the period 1973-1982 and are discussed in this section. Average values of all parameters, except longitudinal cracking, of AC and MC sections for two years are given in Table 2 for comparing performance. Observations of longitudinal cracking made in 1982 are given in Table 3.

Present Serviceability Rating

The PSR is a measure of how well a pavement rides or

Figure 2. Summation of equivalent 18 000-lb single-axle loads, Third River Road.

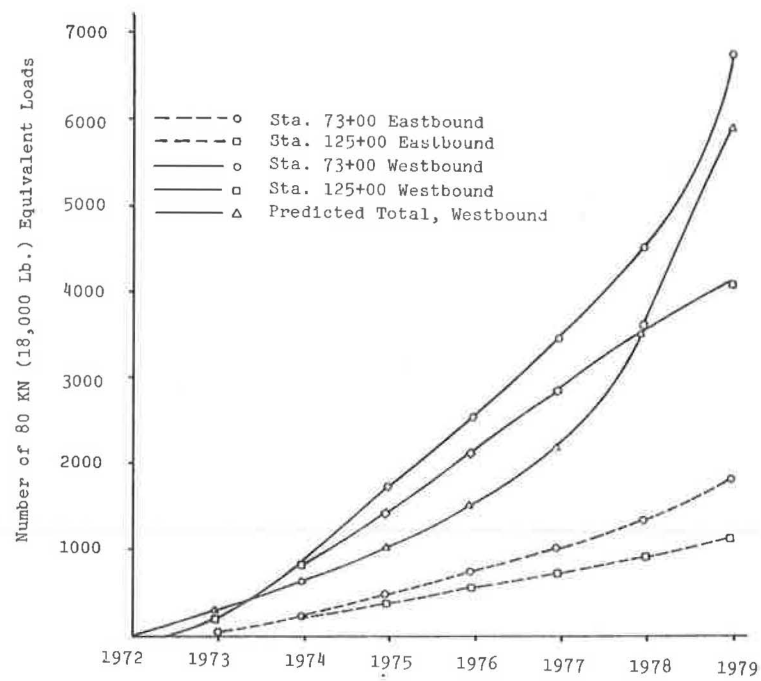


Table 2. Condition of Third River Road test sections.

Asphalt Type	Thickness (in)	Lane	PSR 1975	PSR 1982	Rut Depth (in) 1974	Rut Depth (in) 1982	No. of Transverse Cracks 1974	No. of Transverse Cracks 1982	Wear Rating 1976	Wear Rating 1982	Weathering Rating 1976	Weathering Rating 1982
MC	3.5	NEB	3.04	3.17	0.094	0.090			3.25	2.20	3.35	2.80
		SWB	2.75	2.96	0.078	0.078	2.5	12.5	3.75	2.40	3.60	2.90
	5.0	NEB	3.18	3.23	0.082	0.094			3.20	2.30	3.50	2.70
		SWB	2.98	2.95	0.047	0.078	1.5	7.5	3.20	2.40	3.60	2.80
	8.0	NEB	3.80	3.67	0.078	0.074			3.65	2.10	3.30	2.50
		SWB	3.48	3.46	0.098	0.074	2.0	8.0	3.50	2.30	3.55	2.60
AC	3.5	NEB	3.68	3.48	0.035	0.066			3.10	2.50	3.35	2.90
		SWB	3.46	3.23	0.047	0.070	2.5	23.5	3.10	2.80	3.80	2.90
	5.0	NEB	3.83	3.67	0.051	0.066			3.00	2.20	3.20	2.55
		SWB	3.96	3.76	0.047	0.086	7.0	19.0	3.45	2.55	3.60	2.80
	8.0	NEB	4.85	4.48	0.039	0.059			2.80	2.30	3.20	2.50
		SWB	4.50	4.51	0.055	0.094	3.0	12.5	3.25	2.60	3.90	2.90

Table 3. Length of longitudinal cracking on Third River Road test section A, 1982.

Section	Stabilizing Material	Longitudinal Cracking
1	MC	Continuous 15- to 120-ft cracks across NEBL; continuous up to 30-ft cracks across SWBL
2	MC	900 ft plus 10-15 ft across near beginning
3	AC	General subsidence in 150- to 200-ft length; 1150-ft total longitudinal cracking
4	MC	620 ft
5	AC	385 ft
6	AC	0
7	AC	65 ft
8	AC	0
9	MC	Many fine cracks: 150-ft NEBL, 450-ft SWBL
10	MC	200 ft
11	MC	30 ft
12	AC	0

how well it serves the public (11,13). It is an average of the ratings given by a group of people as to how well a set of pavements rides. These ratings are then correlated with a measure of the roughness of the pavement. A scale of 1 to 5 is used. The longitudinal profile of the road has the greatest effect on this index. On this project, up until 1980, a Minnesota Department of Transportation (DOT) roadmeter was used to measure this profile in inches per mile. In 1982 a Maysmeter was used.

The PSRs of AC and MC sections during the period 1973-1982 did not change significantly. The values for AC and MC sections are 2.2 to 4.0+. Generally, the thicker sections and lanes with unloaded trucks (NEBL) have higher values, as would be expected. Lower PSR values for sections 2, 3, and 4 may be attributed to the presence of some silt in the subgrade.

Sections 13 through 18, which are stabilized with asphalt emulsions, were constructed in 1975. Their PSRs were lower than those of the AC and MC sections. This is due to nonuniformity of the surface and some slippage that has occurred between the two surface layers that have no tack coat in between. Sections 13, 14, and 15 were repaired in 1980.

Rut Depth

The rut depth has been measured annually on the test sections by using an A-frame device from the Minnesota DOT. It measures maximum deformation between two points 3.5 ft apart. Observations were taken for both the outer wheel path (OWP) and the inner wheel path (IWP) of each lane near the beginning, the end, and adjacent to the strain sensor patterns for each test section. Averages of these values are reported in Table 2.

Rut depth (permanent deformation) was considered as the critical design criterion on this project. However, in the first 10 years, the pavement has shown no tendency toward this type of failure. As the sections get older, they will tend to cure out and get harder, thus resulting in less permanent deformation with time. The rut depths range from 0.059 to 0.094 in with an average of 0.089 in for both AC and MC sections. These values are minimal compared with the AASHTO failure condition--1.0-in rut depth.

One would expect relatively lesser rut depths for (a) AC sections versus MC sections, (b) thick sections versus thin sections, (c) IWP versus OWP, and (d) unloaded lanes (NEBL) versus loaded lanes (SWBL). The observations do not indicate any such variation consistently. This is partly due to the fact that these deformations are small and are influenced by some unidentified factors. However, AC sections, in general, have less rut depth than in the MC sections. Also, the load-carrying lanes (SWBL) in the AC sections have less rut depth compared with the NEBL.

Cracking

Crack surveys have been taken each year as part of the visual evaluation of the performance of the Third River Road test sections. The survey includes the number of transverse cracks, the length of longitudinal cracks, and the area of pattern cracking. The condition of the cracks is also evaluated by using the width of crack openings, abrasion of the crack edges, and the associated multiple cracks with the transverse or longitudinal cracks. The rating system is on a scale of 1 to 5 (bad to good condition) for each of these conditions. The rating adopted is one that was developed for National Cooperative Highway Research Program (NCHRP) Project 10-9 (14).

The number of transverse cracks increases with time. The thicker sections have fewer cracks, as would be expected. The MC sections have more than double the number of cracks in the AC sections. It is of interest to note that, in May 1976, there generally were more cracks than in October of the same year for the MC sections. This indicates that there can be some healing effect due to heat during the summer period. The winter of 1982 was particularly severe in the Cass Lake area with temperatures as low as -40°F. This may have accounted for the rapid increase in the number of transverse cracks in the AC sections.

Longitudinal cracking has started to develop in some of the sections, especially the MC-stabilized sections, in the last few years. These are the result of some shrinkage as the mix cured out and, in some cases, slight movements of the subgrade. The greatest quantity of longitudinal cracks is in the west end of the project (sections 1 and 2). Quantities of longitudinal cracks (in feet per 1000-ft section) are listed in Table 3. These cracks are generally very fine and do not affect the ridability of the pavement.

Wear and Weathering

Another visual survey condition is the determination of the wear and weathering of the pavement. The meaning and methods of determining the rating for a given surface are presented in the NCHRP report (14). The wear ratings refer to the erosion of the surface, especially in the wheel path, caused by the action of tires. Weathering refers to the amount of erosion across the whole pavement surface due to environmental effects. These two ratings are helpful in evaluating when the pavement needs a surface treatment or other type of nonstructural maintenance.

On a typical AC surface that has a significant quantity of coarse and fine aggregate, wear will usually show up as erosion of the sand matrix of a mix. However, on the Third River test road, the mix is primarily sand; thus, there is not a significant amount of deterioration that can be observed between the coarse aggregate particles. It is evident, however, that some aggregate material is eroding at the surface both with the action of traffic and due to weather. The wearing is small and should not be detrimental to the performance of the pavement. However, there is a need to add a sand seal or a fog seal to the surface in the next year or so to help bind it together.

DESIGN COMPARISONS

The original design of the Third River test road section A pavements is presented in Root and others (4). The traffic is calculated based on the number of equivalent 18 000-lb single axles over the design period. The calculations are presented in Stuart and others (5). Initially, design periods of 2, 5, and 10 years were set up for the study. The equivalent loads for these periods were predicted to be 2110, 6700 and 19 000. By using the design chart developed, thicknesses of 1.5, 3.5, and 7.0 in resulted. It was felt that 1.5 in of stabilized sand was too thin because of the low stability of the mix and the variations possible in construction. With a design value of 1.5 in, some areas could be less than 1 in thick. Thicknesses of 3.5, 5.0, and 8.0 in were therefore chosen. These represent design lives of 5, 8, and 15 years, respectively. The 15-year design (8-in) is for 28 000 equivalent 18 000-lb single-axle loads.

The thicknesses are based on limiting the vertical strain on the subgrade (ε_v) or the tensile strain in the stabilized layer (ε_t). Below a thickness of 5 in the vertical strain criteria controls the design, which limits rut depth, and above the 5-in thickness the tensile strain controls, which limits cracking in the stabilized layer.

Design comparisons have been made for the estimated traffic on the Third River test road and the traffic to actually use the road. The objective is to compare the design procedures used for the Third River Road with other design procedures commonly in use for flexible pavements. The procedures used for comparison are the Asphalt Institute method, the California method, the Minnesota method, and the AASHTO Interim Guide. To make these designs for the test pavement sections, the accumulated equivalent 18 000-lb single-axle loads were used for the design period originally determined: 5, 8, and 15 years. The required pavement thicknesses according to the various procedures are then determined.

The results are summarized in the table below:

Nominal Life (years)	Design Thickness (in) Elastic Theory	Asphalt Institute	AASHTO Guide	Minnesota	California
5	3.5	5.9	4.0	6.7	5.0
8	5.0	6.1	4.7	6.7	5.5
15	8.0	6.2	5.3	6.7	6.0

The table shows that, for the 3.5-in sections, which are assumed to be for a five-year life, all four methods result in a more conservative estimate of thickness. For the 5-in (eight-year) design, the Asphalt Institute and Minnesota procedures are somewhat conservative and the California and AASHTO methods are within 0.5 in of the Third River designs. For the 8-in thickness, all of the design procedures result in thicknesses less than 8 in. Therefore, in terms of expected life, we would expect, according to these design procedures, the 3.5-in sections to fail before the five-year design period, which did not occur. The Asphalt Institute and Minnesota methods indicate that a longer life would be anticipated for a 5-in section. For all the 8-in sections, a longer design life than 15 years would be expected.

Continued observation of the section A sections should be made so that additional comparisons between thicknesses calculated with the various design procedures and observed performance can be made.

SUMMARY AND RECOMMENDATIONS

In 1972, the Third River Road was built in the Chippewa National Forest near Cass Lake, Minnesota. The road is in an area that is low on good gravel but has much local outwash sand. In the area of section A, the sand is very one-sized and has a low percentage passing the No. 200 sieve.

There were no well-documented design procedures for a sand-asphalt road section, so the elastic-layer theory was used to calculate the strains in the pavement section after moduli were determined in the laboratory. Existing fatigue relations developed by Dorman and Metcalf (7) for permanent deformation and for cracking by Monismith and others (15) were used to estimate the number of equivalent loads to failure. A traffic analysis was then used to relate the calculated traffic to the number of years to failure. The design curves thus set up were used to determine thicknesses for lives of two, five, and eight years. These resulted in thicknesses of 1.5, 3.5, and 5 in. It was decided to increase the thicknesses to 3.5, 5.0, and 8.0 in, which represented lives of 5, 8, and 15 years. Sections of each of these thicknesses stabilized with asphalt cement (200-300 penetration) and MC-800 cutback were constructed in duplicate along the 3.2 miles of section A of the Third River Road in September 1972. No seal coat was added to the sections. The surface was the stabilized sand. The properties of the stabilized materials and the soil are included in the paper.

The traffic started on the road in fall 1972 and continues today. The traffic in terms of equivalent 18 000-lb single-axle loads has developed about as predicted on the SWBL and not as fast in the NEBL. After 10 years of service, none of the sections of this portion of the test road has shown much deterioration. The condition is defined by PSR, rut depth, and cracking (pattern, transverse, and longitudinal). By using each of these parameters, the performance is very good after 10 years.

The strains and deformations in the sections have been monitored by using inductance coils. The horizontal tensile strain on the asphalt layer and the vertical compressive strain on the embankment were measured under moving vehicle loads. The tensile strain has been related to cracking of the pavement and the vertical strain to permanent deformation or rut depth. It was found that the measured strains were less than the calculated values when using the elastic-layered system. The field moduli may be higher or load application faster than was assumed for the laboratory analysis.

Static or permanent deformation has also been measured throughout the 10 years. The static movements vary throughout the year with the largest variations in the thin MC-stabilized sections. The most deformation occurs in the winter when the frost swells up the pavements. However, this phenomenon has not adversely affected the ridability of the sections.

It is apparent from the performance of the sections so far that the design thicknesses are on the conservative side for the traffic and materials at the Third River site. Because of the loaded timber trucks, there is a significant amount of traffic in terms of equivalent 18 000-lb single-axle loads. For a design period of 8-10 years with up to 28 000 equivalent loads, a 3.5-in-thick stabilized sand will perform well on an A-3 sand embankment. This sand may be stabilized with an asphalt cement or an MC-800. Similar materials did not perform well when stabilized with an asphalt emulsion. This may be because

1. The grade was lower, which resulted in a wetter subgrade;
2. The emulsion-stabilized sand was placed in

relatively thin lifts with no tack coat between them; or

3. The embankment soil was finer under these sections.

When the good performance of the asphalt cement and MC-800-stabilized sections is used for design, the following items must be considered:

1. The embankment was compacted to 95 percent of modified density according to the AASHTO T-180 test,
2. The stabilized sand was mixed in a hot-mix plant and placed with a paver,
3. There was good construction supervision and control, and
4. The cross section of the road includes a good ditch section for the most part, which results in good drainage of the subgrade.

These items will help any job perform above general expectations.

The Forest Service intends to monitor the performance of the Third River test sections by using the parameters discussed. The Forest Service is considering this type of construction for roads with similar traffic in areas of fine outwash or beach sand. There are many areas near Cass Lake where this procedure would have an initial cost less than a gravel road and would certainly require less maintenance and provide a better ride over a period of years. A design thickness of 3.5 in is appropriate if control is maintained as well as was done in 1972 at the Third River Road. However, for normal construction control or if in-place stabilization is used, 4.5-5.0 in is recommended. This thickness should be considered only if the CBR is 10 or higher, the R-value is 60 or higher, and there is less than 4 percent passing the No. 200 sieve. In others words, the natural material is a clean A-3 fine sand. It should be stabilized with a minimum of 6 percent AC (200-300 penetration) or 6.5 percent MC-800. More work should be done before asphalt emulsion stabilization is tried with this material. It is suggested that some 4-in emulsion-stabilized sections be tried either stabilized full depth in one pass or paved with two 2-in lifts with a tack coat between them. These procedures need to continue to be explored to help conserve aggregate resources in these aggregate-poor areas.

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