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CIVIL ENGINEERING

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JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP-94/12
Final Report

CONCRETE PAVEMENT CRACK AND SEAT
PERFORMANCE AND OVERLAY DESIGN

Haiming Huang and Thomas D. White

PURDUE UNIVERSITY

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<p>16. Abstract The object of this study is to develop guidelines and criteria for design and construction of overlays on cracked and seated concrete pavements. The main factors which are investigated in this study include: number of the seating roller pass, weight of seating roller, thickness of overlay, fiber content and fiber layer location.</p> <p>Eleven test sections were constructed on I-65 and twenty two test sections were constructed on US-31. An extensive investigation was conducted involving both field and laboratory testing. Field investigations included pavement condition survey and non destructive testing (NDT) of the pavements using a FWD or Dynaflect. The laboratory investigation focused on physical property testing, complex modulus testing and fatigue testing.</p> <p>Three fiber extraction methods have been developed for this project. Preliminary test Results show good precision of these methods. These fiber extraction methods should be used in future projects.</p> <p>Deflection measurements can be used to determine the appropriate weight and drop height of the breaking head. The Guillotine breaker used in this project is effective in breaking PCC pavement.</p> <p>A 40 ton roller is recommended because the 50 ton roller appears to "overwork" the cracked concrete pavement. Roller passes have no significant effect on the maximum deflections. The optimum roller pass can not be determined exclusively based on the maximum deflections. One to three passes of a 40 ton pneumatic-tired roller is adequate when seating is needed to assure contact of cracked pavement pieces with the subgrade.</p> <p>Use of fiber and the fiber layer location do not have a significant effect on pavement strength in term of the maximum deflection. But use of fiber will delay the crack development. The least cracks were developed in the sections which have fiber in all three layers. It is recommended that fiber be used in all overlay layers.</p> <p>Overlay thickness has an important effect on the maximum deflections and the development of reflective cracks.</p> <p>General trends of crack development showed in the first two years. Cracked and seated sections have less cracks than non cracked and seated sections, with other condition being same. The general trends are not obvious after two years.</p> <p>Beams of 3x3 in. are recommended to be used for flexural fatigue tests in the future due to small variability. Addition of fiber increases the fatigue life of overlay mixture. Composite samples with fiber(4.5 lb.) have predicted fatigue life two times greater than those without fiber. Loading frequency and temperature have significant effects on dynamic modulus and phase angle of overlay mixture. The addition of fiber will change the viscoelastic properties of mixture.</p>			
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IMPLEMENTATION REPORT

CONCRETE PAVEMENT CRACK AND SEAT PERFORMANCE AND OVERLAY DESIGN

Research was conducted that involved construction, testing and evaluation of cracking and seating of concrete pavements with subsequent asphalt overlay. This process is one option for rehabilitation of concrete pavements. In the current study fiber in varying amounts was incorporated in the mixtures of one or more layers of the overlay. The goal of the research was to provide guidance and criteria for wider implementation of the crack and seat options.

Eleven test sections and twenty two test sections were constructed in Indiana on I-65 and on US-31, respectively. Field investigations included pavement condition surveys and non destructive testing (NDT) of the pavements using a FWD or Dynaflect. The laboratory investigation focused on physical property, complex modulus and fatigue testing.

A number of specific results were obtained from this study. Details of the research are provided in the final report. Key implementable results that will benefit INDOT in application of the crack and seat option for concrete pavement rehabilitation and for general use of fiber in asphalt layers are given below.

Due to lack of a standard method for fiber extraction, three fiber extraction methods (Method A, B, and C) were developed for this project. Preliminary tests were conducted to evaluate the precision of Method B and Method C for fiber extraction. Results show that over 95% of fiber can be extracted from paving mixtures using these tests. The precision of both methods is good. These fiber extraction methods should be used in quality control on future projects.

Fiber extraction results show that fiber contents in the test sections for this project were different from target fiber contents by a significant amount. The differences between actual fiber contents and target fiber contents affect the breadth of the study. The technique for adding fibers on these projects was to blow the fibers onto the conveyor lifting the mixture to the surge

bin. This technique was recommended by the fiber supplier. However, this technique was not satisfactory. Current techniques utilized by INDOT are:

Drum Plants - Fiber is blown into the drum using a pipe running parallel to the asphalt line with a diversion to add the fibers onto the mixture slightly above the drum flights.

Batch Plants - A - Fiber is blown into the pugmill

B - Fiber is added to the pugmill by hand.

Deflection measurements are recommended to determine the appropriate weight and drop height of the breaking head during the cracking process to ensure that the breaker is effective in breaking PCC pavement. The deflection tests should be tested before and after cracking and the results compared. It is important to determine cracking effectiveness because the energy required for cracking may vary from site to site.

As a result of this study, a 40 ton roller is recommended because the 50 ton roller appeared to "overwork" the cracked concrete pavement. Rolling should consist of one (1) to three (3) passes of a 40 ton pneumatic-tired roller to assure adequate contact of cracked pavement pieces with the supporting surface.

At the end of the two year survey period in the study, results indicated that fiber is beneficial in reducing the rate at which cracks develop when included in all layers. Lower amounts of cracking in cracked and sealed sections after one year was not sustained in all sections at the end of two years. It is recommended that five and eight year condition surveys be conducted to establish the longer term performance of these sections. Not only will the rate of cracking be better defined but other distresses (e.g. spalling, potholes) will likely develop in association with the cracking. Consequently, the additional surveys should be comprehensive distress surveys. The PAVER procedure was utilized in the original distress surveys.

Long term evaluation can consider three criteria:

1. Cracking

2. Deflection
3. Roughness (serviceability)

The amount of cracking can be examined by the rate at which it develops. Observed cracking can also be in terms of the maximum possible cracking for an uncracked and seated pavement. US 31 consisted of 10 inch jointed plain concrete with 12 foot by 20 foot slabs and I-65 consisted of 10 inch jointed reinforced concrete with 12 foot by 40 foot slabs. Realistically, the I-65 slabs could be expected to have cracked into 12 foot by 20 foot slabs. As a result, on both pavements the maximum transverse cracking would be 3168 feet per lane-mile. It is possible for crack and seated sections to exceed this amount of cracking.

An eight year evaluation period is recommended because after this period of time a conventional overlay can exhibit enough deterioration to become a candidate for rehabilitation. Consequently, either the fiber modified overlays will exhibit significant distress or comparison with the conventional overlays will provide a basis for evaluation. Ultimately, a cost-benefit analysis will provide criteria to justify the use or exclusion of fiber.

Flexural fatigue testing was used to compare the fatigue life of fiber modified mixture with conventional mixture. It was also found that tests of 3x3 in. beams have substantially less variability. Results of fatigue tests indicate that fiber increases the fatigue life of overlay mixtures. It follows that fiber is likely to improve the fatigue life of asphalt mixture in general.

Although the results of the complex modulus test for both I-65 and US-31 show inconsistencies for the significance of fiber content on both dynamic modulus and phase angle, the results do indicate that the addition of fiber will change the viscoelastic properties of mixture. Further study should be conducted to determine the viscoelastic properties of fiberized mixture.

CHAPTER 1

INTRODUCTION

1.1 Background

Significant mileage of the nation's highways was constructed with concrete in 50's, 60's and 70's. These concrete pavements have reached or exceeded their design service life and reconstruction or rehabilitation is necessary to restore rideability and/or structural strength. Like other states, Indiana has been maintaining an increasing number of miles of such deteriorating pavements. Pavement rehabilitation is attractive because of costs and being able to utilize existing pavement structure. Several rehabilitation options for a given pavement may be viable. However, performance relations are not available for some options and as a result the advantages and disadvantages are not clear.

Concrete pavement recycling and asphalt overlay are two widely used techniques for reconstruction and rehabilitation of concrete pavements [Mujahed & White, 1989]. Concrete pavement recycling can be done effectively at a central plant but results in significant traffic delays and high costs. Asphalt overlay of deteriorated concrete pavements is widely used but usually experiences extensive reflection cracks as a results of thermal cycling, repeated traffic stress and differential vertical movement of underlying slabs [Joseph & Haas, 1989; Carpenter & Darter, 1989; Thompson, 1989]. Preventing, or at least reducing, reflective cracks in asphalt overlays can prolong the overlay service life and reduce future maintenance costs. Various approaches have been tried to control reflective cracks. One recent option that has been tried with some success is "breaking or cracking and seating" of Portland cement concrete (PCC) pavements prior to asphalt overlay. A number of states, such as Illinois, Kentucky, Michigan and California have experience with this technique[Mujahed & White, 1989]. The Indiana Department of Transportation (INDOT) began experimental cracking and seating studies in the spring of 1984 with a series of test sections on I-74 from SR-39 to 2.0

miles west of Montgomery/Boone county line in Indiana [EL-Sheikh et al., 1989]. Monitoring results over a five year period suggests that this technique is effective in reducing the reflective cracks.

Addition of fibers to the asphalt overlay mixture has been used to further reduce reflective cracks. This addition of fibers is intended to increase overlay tensile strength and strain tolerance. In the test sections on I-74 the rate at which reflective cracks developed was reduced and pavement deflections were less for the same thickness of overlay without fiber.

1.2 Problem Statement

There are reports of beneficial application of the cracking and seating technique. However, more definitive information is required for its general implementation. There are questions that need to be answered related to optimum fragment size, and effect of breaking and seating methods.

The addition of fiber still needs further research, especially concerning the optimal fiber content and the effective location of the fiber layer. Therefore, it is the intention of this study to conduct field and laboratory investigations to evaluate cracking and seating performance and properties of fiber modified asphalt overlay in order to provide guidelines and criteria for further implementation.

1.3 Objectives of the Study

The object of this study is to develop guidelines and criteria for design and construction of overlays on cracked and seated concrete pavements. This would be accomplished by concentrating on the following areas:

1. Field evaluation of techniques and equipment for cracking and the subsequent seating process by means of deflection testing with Falling Weight Deflectometer (FWD).

2. A study of the performance of overlays on cracked and seated concrete pavements based on the data from new test sections as well as existing test sections.
3. Evaluation of the effect of adding fiber to the overlay asphalt mixture through laboratory testing, field distress surveys and deflection tests.

1.4 Study Outline

The literature review was conducted to provide a bases for experimental designs for the study and guidelines for construction of test sections. Field testing was conducted during and after construction of test sections. Cores and slabs were cut from test sections for laboratory testing. Analysis and evaluation were conducted on the results from field testing and laboratory testing.

The main components in this study are shown in Figure 1.1.

1.5 Report Organization

This study contains six chapters. Chapter 2 reviews the cracking and seating technique and its application in Indiana and in other States. It discusses the effects of fiber addition to overlay mixture. A comprehensive discussion is included of existing thickness design methods for overlay on cracked and seated concrete pavement. Chapter 3 contains a description of materials, test sections, equipment and procedures employed in this study. Chapter 4 presents an outline of the experimental designs conducted in this research. Chapter 5 contains test results, data analysis, and evaluation of test sections and asphalt mixtures. Chapter 6 presents major results of this study, recommendations for implementation and for further research.

1.6 Implementation

Implementation of the results of this study will provide a number of benefits to INDOT. Firstly, INDOT guidelines and specific criteria for cracking and seating techniques and overlay

design will be presented. Secondly, mechanical properties will be determined for fiber modified asphalt mixtures. Finally, structural characteristics of fiber modified asphalt overlays of cracked and seated concrete pavements will be evaluated relative to conventional asphalt overlays.

Figure 1.1 Study Flow Chart

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Asphalt overlay of Portland Cement Concrete (PCC) pavement is a frequently used rehabilitation alternative. A problem associated with this option is the reflective cracks which mirror the crack patterns that exist in the old concrete pavement. A number of techniques have been used to prevent or reduce the reflective cracks in asphalt overlay. One technique that is being used is cracking and seating the concrete pavement prior to overlaying.

A comprehensive survey conducted by the National Asphalt Pavement Association (NAPA) [Crawford, 1986] indicated that eighteen state highway agencies used the crack and seat technique in the 1985-1986 period. Three major steps are involved in this technique:

1. Breaking the old concrete pavement which is going to be overlaid.
2. Seating the broken pavement by rolling.
3. Applying an asphalt overlay.

The performance of overlays on cracked and seated pavement depends on the effectiveness of cracking and seating techniques as well as the properties of the asphalt overlay mixture. Adding fiber to asphalt overlay mixture can reduce the reflective cracks by improving the asphalt mix tolerance to strain.

Cracking and seating is a relatively new rehabilitation technique. The overlay thickness design for cracked and seated pavement presents a new challenge for the pavement designer. Although there have been attempts at improving overlay design procedures, there are a number of limitations with current design procedures. Development of a better overlay design procedure for cracked and seated concrete pavement is needed for better utilization of this promising technique.

Major steps involved in cracking and seating are discussed in this chapter, followed by discussion of effects of fiber in the asphalt mix. Finally, this chapter presents a comprehensive overview of overlay design methods related to cracking and seating

2.2 Cracking and Seating

2.2.1 Concrete Cracking Process

The intent of cracking the concrete pavement is to create concrete pieces that are small enough to reduce the slab thermal movement which contributes to reflective cracks. These pieces should be large enough to retain some structural capacity and there should be interlock of the pieces to effect significant load transfer. The common types of cracking equipment used are pile-driving hammer, whip hammer and guillotine hammer.

1. Pile-driving Hammer

A pile-driving hammer typically produces longitudinal and diagonal cracks. The hammer is usually mounted on a trailer which is tractor-drawn. Force or energy of impact with the pavement may be adjusted by changing the fuel input into the firing cylinder of the hammer. The actual breaking pattern depends on the energy applied to the pavement slab, condition and thickness of the existing concrete pavement, and quality of the subgrade. Experience in Kentucky [Sharpe et al., 1988] has shown that 18 to 24 in. fragments may be achieved when the pile-driving hammer makes three or four equally spaced passes across a lane with a 12 to 18 in. longitudinal interval between impact blows of the hammer. The fragment size can be varied, for example, at the same longitudinal interval, a fragment size of 30 to 36 in. may be achieved with two or three passes of the pile-driving hammer or a fragment size of 3 to 12 in. could be obtained by applying seven or eight equally spaced passes.

The shape of the head striking the pavement is another important factor affecting the breaking pattern with the pile-driving hammer. A plate type of head is preferred because it

minimizes penetration into the Portland cement concrete pavement. The most effective head is an 18 by 18 in. square rotated 45 degrees to the direction of travel. This shape produces diagonal breaking interconnected with longitudinal cracks.

The pile-driving hammer is capable of high production rates. The shortcomings with this equipment are the noise, dirt and perception of damage it creates to surrounding areas.

2. Whip Hammer

The whip hammer consists of an impact hammer attached to the end of a leaf-spring arm. The impact force is developed by the whipping action of the leaf-spring arm and hammer head. The equipment is mounted on the rear of a truck and is operated by one person. This machine is capable of doing lane widths of pavements as it travels in one path longitudinally along the road because the leaf-spring arm can be rotated to cover the width of a lane.

The impact energy of the whip hammer is transmitted to the pavement through a base plate in much the same manner as that with the pile-driving hammer. Typically, the plates are diamond, square or rectangular. Based on experience in Kentucky [Sharpe et al., 1988], an 18 to 24 in. breaking pattern can be achieved with one blow of the whip hammer per square foot of pavement surface area.

3. Guillotine Hammer.

The guillotine hammer has a 5 to 7 ton steel blade that is approximately 36 in. wide. The blade is dropped within vertical guides and the impact force can be varied by the drop height of the guillotine blade. Typically the drop height is 18 inches. The machine is self propelled and the breaking process can be monitored by the operator walking or riding the machine. The operator controls the longitudinal interval between impacts through the speed of the truck on which the guillotine hammer is mounted. Generally, this equipment induces transverse cracking which is favored by many agencies [Crawford, 1987].

2.2.2 Cracking Pattern and Fragment Size

Effective cracking results in through-slab cracks. The size of the fragments has a direct impact upon the design and long term performance of the overlay. Small fragments will most certainly reduce and possibly eliminate reflective cracking in the asphalt concrete overlay but use the least structural potential of the existing Portland concrete pavement. Conversely, large fragments may maximize the structural potential of the existing Portland concrete pavement but may be large enough to permit reflective cracking. Large fragments may also have more potential for rocking as a result of ineffective seating and therefore increase the potential for overlay cracking

The cracking pattern and fragment size for best performance remain controversial. Wisconsin and California prefer larger sizes, generally four feet to six feet square. Several other states, including North Dakota and Minnesota specify transverse cracking generally at intervals of three feet. Kentucky recommends that the pavement be broken to a nominal 24-inch size [Sharpe et al., 1988]. In Indiana, transverse cracks, spaced 18 to 24 in, were required for crack and seat project on I-74 in Montgomery/Boone county. So far, there are no definite optimum dimensions. However, it can be said in general that 1.5' to 3' is a range that experience seems to favor [Crawford, 1987].

Research is continuing to determine the optimum size of cracked pieces. One approach could be to conduct deflection tests before and after breaking to evaluate the effects of cracking patterns and fragment sizes.

2.2.3 Seating Cracked Pavement

The cracked PCC slab is "seated" by rolling. Seating of the broken slabs after cracking is intended to re-establish support between the sub-base and the slab to assure a stable foundation for the asphalt overlay. With inadequate seating, individual fragments of pavement

tend to rock and cause reflection cracks in the asphalt overlay. Conversely, overrolling may destroy interlock between individual fragments.

A wide variety of roller equipment is used and includes:

1. Pneumatic-tired roller
2. Steel drum roller
3. Vibratory roller

In general, the most effective seating is accomplished with a heavy pneumatic-tired roller. Steel drum rollers and vibratory rollers tend to bridge over the cracked pieces, thus, they are not satisfactory for seating purposes [Crawford, 1987; Sharpe et al., 1988].

2.2.4 Effects of Weight and Pass of Roller

Seating requirements and characteristics depend on fragment size, quality of the existing pavement and subgrade [Sharpe et al., 1988]. The Seating process is affected by weight and number of roller passes. Deflection measurements obtained during rolling can be used to evaluate the effectiveness of seating. Deflection data from Kentucky [Sharpe et al., 1988] indicate the following general trends:

1. An increase in deflection after the initial roller pass.
2. A reduction or stabilization of deflection with additional roller passes, and
3. An increase in deflection with a large number of roller passes.

The increase in deflection after the initial roller pass is generally supported by observations from other states. These observations bring some concern about seating requirements. The higher deflections might indicate overrolling which destroy the interlock between individual fragments and reduce load transfer. Conversely, failure to achieve proper seating could allow the fragments to rock and promote reflective cracking.

Agencies have used different combinations of roller weight and number of passes. Experience tends to dictate use of heavy rollers and a minimum number of passes as opposed to a greater number of passes of lighter rollers. Very heavy rollers(50 tons or greater) may overload bridges and be less maneuverable. Lighter rollers may require more passes to achieve proper seating, but the added maneuverability permits more uniform coverage of the pavement.

Experience in Kentucky thus far has indicated that the most efficient seating may be accomplished by rolling with a pneumatic-tired roller weighing between 30 and 50 tons. Two passes with a 50 ton pneumatic tired roller have provided adequate seating. Adequate seating has also been obtained with five passes of a fully ballasted 35-ton multi-wheeled pneumatic-tired roller.

In California [Crawford, 1987], a vibratory sheepsfoot roller was used to seat a 1500 foot section of PCC pavement. The roller was operated with a dynamic force of 44,000 pounds at a frequency of 1700 vibrations per minute and a rolling speed of five mph. The rolling load was 44,000 pounds. Ten passes were applied to each side of a 12-foot test lane. The roller was 8 feet wide which resulted in double rolling in the middle four foot portion of the lane. Deflection measurements after seating showed that there was an increase of deflections in more than 50 percent of the measuring locations after rolling. Caltrans presently requires not less than five passes of an oscillating pneumatic-tired roller of not less than 15 tons or a vibratory sheepsfoot roller which exerts a dynamic centrifugal force of at least 20,000 lbs..

2.3 Effects of Fibers on the Asphalt Overlay Mixture

2.3.1 Introduction

The capacity of pavement layers to resist cracking is an important characteristic. Unfortunately, conventional asphalt pavement mixture shows much less tensile strength and strain capacity compared with compressive strength and strain capacity, often by an order of magnitude. As a result, tensile cracking occurs from loading, volume changes caused by wetting and drying, temperature and shrinkage, and differential movement.

Investigations into the reinforcement of bituminous mixture to improve its tensile properties have been conducted utilizing homogeneously dispersed fibers within the mixtures. The potential benefits obtained by reinforcing asphalt concrete pavements include [Bushing et al., 1970; Button & Epps, 1981]:

1. Increased tensile strength and resistance to cracking.
2. Coherence of the pavement even after cracking.
3. Increased fatigue resistance.
4. Increased shearing resistance due to lateral restraint provided.
5. Significant economic saving.

Based on a five year (1984-1989) evaluation of test sections on I-74, INDOT reported [EL-Sheikh et al., 1989] that the addition of fibers to the asphalt overlay of cracked and seated pavements reduced reflective cracking by 85% over that of a non cracked control section and 10% over cracked and seated pavements overlaid with asphalt without fibers.

2.3.2 Types of Fibers

Both natural fibers and synthetic fibers have been used to modify asphalt concrete. A brief description of various types of fibers that have been used is presented [Bushing et al., 1970; Button & Epps, 1981; Mills & Keller, 1982].

1. **Cotton.** Cotton fibers were incorporated into asphalt concrete test pavements in the mid-thirties[Bushing et al., 1970]. Although the tensile strength of cotton fibers is high,

they are degradable natural fibers and therefore can not provide long-term reinforcement.

2. **Short asbestos fibers.** Asbestos fibers were incorporated into mixtures in the United States during the 1950's and 1960's. It is another natural fiber with suitable properties. However, it is considered a health hazard by the Environmental Protection Agency(EPA) and is not used today.
3. **Nylon.** Nylon is a synthetic material. It is relatively expensive and does not show any technical advantages over other fibers in asphalt mixture. It has not been used since 1977.
4. **fiberglass.** Fiberglass is a synthetic material and it is not popular because fiberglass strands cut themselves at intersections within the mixture.
5. **Polyester.** Polyester is a polymerized product of crude oil in fiber form.
6. **Polypropylene.** Polypropylene is also derived from petroleum and produced in a fiber form.

Polyester and polypropylene are the two major types of fibers which are used in asphalt mixtures. Accordingly, most of the test results that have been reported have been conducted on asphalt mixtures with these two types of fibers. Consequently, the following discussions focus on these two types of fibers.

2.3.3 Physical characteristics and requirements

Size, length and concentration of fibers are the major factors considered in the design of fiber modified asphalt mixtures. Fiber size is measured by micron or denier. Denier is the unit of weight commonly used for textile fibers and is defined as the weight in grams of 9000 meters of the fiber. Freeman et al (1989) conducted research on the effects of changing the denier, length and percentage of polyester fiber type in asphalt mixture. They found that a 30

denier fiber had less tendency to "bundle" when mixed with heated aggregate, compared with a 15 denier fiber. Also 1/4-in length fibers were easier to mix with the dry aggregate than 1/2-in length fibers. The 30 denier fiber mixtures had a lower air void content than the 15 denier fiber mixtures. An optimum fiber percentage of 0.35% (based on total weight of mix) was recommended based on wet indirect tensile strength.

Mills et al (1982) found that fiber "balls" or "lumps" appeared when polyester fibers longer than 1/2 inch were introduced into the pugmill simultaneously with a Delaware type "C" wearing course. A 0.4 inch long fiber was satisfactorily mixed. The 1/2 in. fiber was effectively mixed with a Delaware type "B" base course. Based on information available in the literature, lengths of fiber used in current practice range from 0.02 in. to 0.75 in. (0.5~19.1 mm) and sizes range from 12 to 182 microns (1.5~15 denier). Optimum fiber contents are in the range of 0.3~0.35% of total mix weight.

2.3.4 Additional cost in construction

The added cost of including fiber in asphalt mixtures is between \$5 and \$10 per ton [Ling, 1986 ; Mills & Keller, 1982]:

1. The current cost of polyester fiber is about \$1.05 per pound and the cost for polypropylene is 1.25 per pound.
2. Additional asphalt cement in the range of 0.2 ~ 0.3 percent is added to the mix.
3. Pugmill mixing time is increased by about 30 seconds.
4. Labor cost at the plant is increased for introducing the fibers into the mix.

2.3.5 Methods to introduce fibers in mixtures

Performance is improved by uniform dispersion of fibers in the asphalt mixture. Currently, there are several alternatives for introducing the fibers.

1. The fibers may be preblended with the asphalt in special equipment. Subsequently the blended asphalt and fiber is metered into the asphalt plant;
2. In a batch plant, the fibers can be introduced into the mixer with the heated aggregate and dry mixed for 30 seconds before introduction of the asphalt;
3. In a drum plant, the fibers may be introduced by use of the reclaimed material feed system at approximately mid point of the drum or;
4. The fibers can be blown into the bottom of the conveying system that lifts the mixture up to the surge bin. Blending is assumed to occur during the lifting, unloading, transporting and laydown operations.

2.3.6 Laboratory tests and mechanical properties of fiber mixture

Laboratory testing of fiber-modified asphalt concrete specimens, as well as of conventional asphalt concrete specimens, has been undertaken by various agencies to investigate the potential benefits of fibers in asphalt concrete. The major tests and results are briefly described below.

1. Marshall and Hveem stability tests

The Marshall test was developed in the late 1930's and early 1940's and is widely used in the United States for purposes of asphalt mixture design. Marshall stability and flow values are measures of the material stability and plasticity flow [Button & Epps, 1981].

The Hveem stability test was developed by the California Division of Highways in the late 1930's and is currently used by approximately 15 state departments of transportation for asphalt concrete mixture design. Similar to the Marshall test, the Hveem stability value of asphalt concrete is a measure of the material's ability to resist plastic flow [Button & Epps, 1981].

Button and Epps (1981) drew conclusions about the applicability of these test procedures for design of asphalt mixtures with fiber.

- (1) Marshall or Hveem test procedures appear to be acceptable for use in selecting optimum asphalt for fiber modified paving mixtures. Therefore, standard mixture design test procedures could be used without any modifications.
- (2) Addition of fibers will decrease Marshall stability of asphalt concrete but not below acceptable levels if appropriate quantities of fibers are used.
- (3) Use of fibers may either increase or decrease the Hveem stability depending on the size (length and diameter) and amount of the fibers. It appears that proper utilization of fibers in asphalt paving mixtures can significantly improve Hveem stability.
- (4) Inclusion of fibers increases the amount of asphalt cement required to achieve maximum stability. Bulk specific gravity of the Marshall compacted specimens decreases and air void content increases in proportion to the quantity of fibers added.

2. Direct tension and indirect tensile tests

Based on the study by Button and Epps (1981), the results of uniaxial direct tension tests showed no consistent relationships between fiber diameter or length and tensile strength or strain at failure at any rate of displacement. The tensile strength of mixtures decreased when the fiber content was increased above 0.2 percent. Tensile strain at failure increased with increased fiber content.

Indirect tensile tests conducted by Freeman et al (1989) showed that the dry tensile strengths of fiber mixtures were 3 to 15 percent lower than the conventional mixture. The strengths decreased as fiber content increased. This fiber-induced weakness may be a result of the fiber strands having a tendency to remain together as bundles, which could introduce "weak spots"

3. Resilient modulus

Results from resilient modulus tests by Button and Epps (1981) showed that resilient modulus of fiber modified mixture increased significantly. Another encouraging finding was the increase in resilient modulus for fiber mixtures at higher temperatures.

4. Fatigue test

Fatigue tests are useful in evaluating asphalt mixtures when subjected to repetitive loading such as from traffic. Results from the beam flexural fatigue test [Resource International Inc., 1989] shows improved fatigue life of fiber mix when compared with conventional mixtures.

A cyclic, in plane displacement fatigue testing machine or "overlay tester" developed at Texas A&M University was used to test fiber reinforced asphalt samples. The equipment simulates thermally induced reflection cracking in the field. Based on test results using this equipment, fiber reinforced asphalt concrete was found to be more resistant to simulated thermal strains than conventional asphalt concrete mixtures.

2.4 Overlay Design

2.4.1 Introduction

Asphalt overlays of PCC pavements usually develop reflection cracks as a result of thermally induced stress, repeated traffic stress and differential vertical movement of underlying slabs [Joseph & Haas, 1989; Carpenter & Darter, 1989]. Cracking and seating has been used for more than 25 years as a method to minimize reflection cracks in asphalt overlays [Ahlich, 1989]. There has been increasing use of cracking and seating as a result of improved equipment and the satisfactory results derived from this method. However, a uniformly recognized standard for construction and general guidelines for overlay design are not available so far because of the lack of comprehensive evaluation and the number of variables which are felt to be significant to the performance of a cracked and seated pavement. There is

a need to develop such guideline for better use of this technique. The review will discuss behavior of cracked and seated pavements and thickness design methods of overlay on cracked and seated concrete pavement.

2.4.2 Impact of Cracking and Seating on PCC Pavement and Foundation.

Cracking and seating of PCC pavement will change the performance of the pavement significantly. Before cracking and seating, the pavement behaves as a rigid slab. Flexural stresses in slabs are high and vertical stresses and strains in the subgrade are quite low. Surface deflections are small so the deflection basin is shallow and broad. However, after the pavement has been cracked and seated, the features and behavior of such pavements are different in several aspects:

1. The slab size is reduced, typically from 10-12 feet by 20-40 feet to 3 by 3 feet.
2. There is no moment capacity at the cracks.
3. Shear transfer between fragments is dependent on the interlock between fragments.
4. Flexural stress in the segment decreases due to the reduction in size.
5. Subgrade stresses will increase significantly due to reduced stiffness of the cracked slabs.
6. Surface deflection will increase.

In conclusion, cracking and seating will cause a significant change in the structural response and behavior of the PCC slab and subgrade. Table 2.1 lists the ratios of deflections after cracking to those before cracking [Sharpe, 1988].

From Table 2.1, we can conclude that with a segment size of about 24", deflections after cracking are about 3 times the deflection before cracking. Segments of smaller sizes have higher deflections which means less load distribution.

Table 2.1 Ratios of Deflections After Cracking to Those Before Cracking

Segment Size	Ratio of deflections
6" - 12"	9.93
18" - 24"	3.01
30" - 36"	2.54

Note: sensor is 5.25" away from load center.

The combined effective modulus of subgrade and cracked concrete were predicted using back-calculation techniques for several pavement sections in Illinois. These results are shown in Table 2.2 [Schutzbach, 1989]. The average "effective" modulus for these above pavements is 546,000 psi as compared with a normal concrete modulus of 4,000,000 psi for an uncracked slab. The average "effective" moduli calculated for cracked pavements are much higher than typical base course moduli which range from 30,000 psi to 100,000 psi [Ahlrich, 1989].

Table 2.2 Cracked and Seated Concrete Modulus Values

Road Name	Deflections(mils)	Moduli (kpsi)
Lincoln Trail	7.8	1080
Lincoln Trail	8.4	770
Illinois 101	14.2	420
Illinois 101	10.1	600
Rockton Road	14.0	150
Rockton Road	13.7	240
Rockton Road	12.3	300
U.S. 6	14.7	680
U.S. 6	12.2	670

Average: 546

Changes of vertical stresses on the top of the subgrade are listed in Table 2.3 [Ahlrich, 1989]. These data are from test sections at the Aberdeen Proving Ground, Maryland. The predicted "effective" modulus and standard 18 kip wheel load were used in the calculations.

As can be seen , the stress on the top of the subgrade after cracking increased by 86 percent , from 2.0 psi to 3.77 psi. Stresses were decreased by application of an overlay. Based

on the discussion above, there is a significant change in behavior of PCC slabs after cracking and seating. A cracked slab behaves more like a semi rigid or flexible base than a rigid slab. Those facts have to be considered in thickness design of asphalt overlays on cracked and seated concrete pavements.

Table 2.3 Changes of Vertical Stresses on Top of the Subgrade

Case	Maximum Vertical Stress(psi)
Before cracking	2.02
After cracking	3.77
After 4 in. overlay	2.97
After 6 in. overlay	2.33

2.4.3 Current Design Methods

As discussed in the previous section, cracking and seating of concrete pavements creates a complex pavement structure. Rational thickness design of overlays of cracked and seated PCC pavement will come from a better understanding of the many factors affecting the response of such pavements. The major factors that are considered in overlay thickness design are as follows:

1. **Characteristics of underlying base materials and subgrade soil properties.** These values can be obtained by running CBR or plate loading tests in the field or running laboratory CBR tests on bulk field samples.
2. **Traffic characteristics.**
3. **Characteristics of cracked and seated concrete slabs.** Structural characteristics of cracked and seated concrete can be estimated by considering the following factors: (a) type and condition of existing PCC pavement, (b) pattern and procedure of cracking and seating, and (c) size of segments. These characteristics may be estimated from other

cracked and seated sections which have similar conditions and have been evaluated after cracking.

Non destructive testing (NDT) equipment such as the falling weight deflectometer (FWD), Dynaflect and Benkelman beam are being used widely to evaluate the structural capacity of pavements. These devices give deflection measurements which can be used as input to back-calculate the moduli of existing pavements. Also load transfer capacity among the PCC segments can be measured by using certain sensor configurations. NDT data after AC overlay placement with other factors such as preconstruction condition of PCC slabs and cracking and seating procedures will aid in the evaluation of the structural effectiveness of cracked and seated PCC layer.

Several design methods have been proposed for AC overlay of cracked and seated pavement. Some of the major design methods are discussed below.

1. AASHTO METHODS

As mentioned earlier, a cracked PCC pavement tends to act as a semi-rigid or flexible system. The AASHTO overlay design method assesses the cracked and seated PCC slab on the basis of an equivalent structural capacity. The overlay thickness can be determined by using the flexible pavement design approach which uses structural number (SN) as a measure of the pavement structural capacity. The required overlay thickness is determined by the following equation:

$$SN_{ol} = a_{ol} \times D_{ol} = SN_f - SN_{eff} \dots\dots\dots(2.1)$$

where :

SN_{ol} : required overlay structural number.

a_{ol} : structural coefficient for the AC overlay

D_{ol} : required overlay thickness, inches

SN_f : structural number required to carry future design traffic.

SN_{eff} : effective structural number of existing layers after fracturing.

Structural coefficient for the AC overlay (a_{oi}) can be determined by Figure 2.1.

Required structural number for future traffic (SN_f) is determined by Figure 2.2. The required inputs are effective design subgrade resilient modulus, design PSI loss, overlay design reliability, overall standard deviation and predicted future 18 kip ESALs.

Effective structural number (SN_{eff}) of the existing fractured pavement is determined by component analysis using the structural number equation:

$$SN_{eff} = a_2 D_2 + a_3 D_3 m_3 \dots\dots\dots(2.2)$$

where D_2, D_3 : thicknesses of fractured slab and base layers

a_2, a_3 : corresponding structural layer coefficients

m_3 : drainage coefficient for granular subbase

Limited guidance is available for the selection of layer coefficients for cracked and seated concrete slab. Each agency should adopt its own set of layer coefficient values for fractured slabs. Some suggested layer coefficients are listed in Table 2.4 (Darter et al., 1991)

Table 2.4 Suggested Layer Coefficient in AASHTO

Slab Type	Slab Condition	Coefficient
JPCP	Pieces one to three feet	0.25 to 0.35
JRCP	Pieces greater than one foot with ruptured reinforcement	0.25 to 0.35

The specific steps in determining the overlay thickness for cracked and seated PCC pavements in the AASHTO method are as follows:

- (1) Review existing pavement design and construction.

- (2) Conduct traffic analysis to predict future 18-kip ESALs in the design lane over the design period.
- (3) Evaluate the existing pavement by condition survey, deflection testing or coring and material testing.
- (4) Determine required structural number for future traffic.
- (5) Determine effective structural number of the existing fractured slab pavement.
- (6) Compute the thickness of AC overlay as follows:

$$D_{ol} = \frac{(SN_f - SN_{eff})}{a_{ol}}$$

Figure 2.1. Dense-graded AC Layer Coefficient (AASHTO, 1986)

Figure 2.2. AASHTO Nomograph for SN Determination (AASHTO, 1986)

2. METHOD BASED ON FATIGUE CRITERIA

After a PCC slab is cracked the modulus is decreased significantly. A typical overlaid cracked and seated pavement will behave like a semi-rigid or flexible system. Thus bending strains develop at the bottom of the AC layer and fatigue cracking will develop. As a result, fatigue criteria can be used as the bases of an overlay design method for cracked and seated pavement .

Thompson (1989) developed and applied an overlay design procedure for a cracked and seated pavement on Illinois Route 99. The inputs for the design procedure were the asphalt concrete modulus, subgrade resilient modulus, "equivalent" modulus of the cracked and seated concrete slabs and thickness of each layer. ILLI-PAVE was then used to calculate the bending strain at the bottom of the asphalt concrete overlay for various thicknesses. Fatigue curves for typical asphalt concrete mixtures were used to estimate the number of repetitions to failure for a given bending strain.

The steps in the AC overlay design procedure based on fatigue criteria are:

- (1) Back-calculate the subgrade modulus, E_{sg} , using FWD tests on the existing PCC pavements.
- (2) Estimate AC overlay modulus , E_{AC} , from Figure 2.3 and Figure 2.4
- (3) Estimate the modulus of the cracked PCC slab, E_{cs} , based on FWD data from similar PCC pavements which have been cracked and seated and overlayed with asphalt mixture.
- (4) Calculate design overlay strains by a calibrated equation.
- (5) Develop fatigue curve for AC overlay mixture. Figure 2.5 is a typical fatigue response for Illinois DOT class I AC mixture.

- (6) Determine the thickness by comparing the estimated fatigue life with the design traffic ESALs.

Figure 2.3 Design Pavement AC Mixture Temperature in Illinois (Thompson, 1989)

Figure 2.4 Design Pavement Mixture Modulus(Thompson, 1989).

Figure 2.5 Typical Fatigue Response [Thompson, 1989]

3. OTHER METHODS

Most states or agencies design cracked and seated overlay thickness using the structural number approach. A layer coefficient value of the cracked slabs is assigned based on experience and engineering judgment. Layer coefficient values "a" assigned by different organizations are listed in Table 2.5.

Table 2.5 The Different Layer Coefficient Used in Practice

Agency or State	Coefficient	Notes
NAPA	0.25-0.45	0.25 for 1 ft crack spacing 0.45 for 3 ft crack spacing
Pennsylvania	0.2	
Minnesota	0.28	If AC is assigned an "a" value of 0.4
Wisconsin	0.2-0.25	Equivalent to a thickness of crushed stone or gravel
Kentucky	0.13	Equivalent to crushed stone base

The Asphalt Institute Overlay Manual proposes an effective thickness procedure. The cracked concrete layer is converted to an equivalent thickness of asphalt concrete by a conversion factor. A full depth AC pavement thickness is determined using the Asphalt Institute standard procedure.

CALTRANS uses a standard thickness of 4.2 in. for overlay over cracked and seated PCC. This thickness includes a 1.2" AC leveling course and a 3" AC surface course. To further mitigate crack reflection, a fabric interlayer is placed between the two courses.

CHAPTER 3

MATERIALS, EQUIPMENT AND TEST PROCEDURES

3.1 Introduction

As part of the study of overlay of cracked and seated PCC pavements, a series of test sections were constructed on two highways in Indiana. Eleven test sections were constructed in the southbound lanes on I-65 near Crown Point, Indiana and twenty two test sections were constructed in the north bound lanes on US-31 near Plymouth, Indiana. Each test section is 500 feet long and was constructed in the Summer of 1990. Locations of the test sections are shown in Figure 3.1. The PCC pavement on I-65 was a jointed 10" reinforced concrete with 12'x40' slab, and 12'x20', jointed 10" plain concrete on US-31.

The overlays consist of bituminous base, binder and surface course mixtures. Overlay included conventional asphalt mixture as well as fiber reinforced asphalt mixture. As part of research, fibers were incorporated in varying amounts and combinations of these asphalt layers. Also, the overlay thicknesses were varied. Table 3.1 and Table 3.2 show the layouts of test sections on both US-31 and I-65.

Deflection testing was conducted with FWD and Dynaflect to evaluate the pavement structure. FWD testing was conducted before and after cracking, after each pass of the seating roller and at various periods of time after overlay. Pavement condition surveys have been conducted since construction. Also cores and slabs were taken from test sections for laboratory tests.

3.2 Materials

The mixes used for both I-65 and US-31 were Indiana #5 base, #8 binder and #11 surface. Specification gradations for these mixes are shown in Table 3.3.

Figure 3.1 Locations of Test Sections on I-65 and US-31

Table 3.1 Combinations of Test Sections on US-31

Section No.	Station	Crack & Seat	Overlay Thickness(In.)	Fiber**				Remarks
				#/Ton	Base	Binder	Surface	
1 (180601)	1849+00 to 1854+00	No	No	No	No	No	No	Routine Maintenance
2 (180602)	1605+00 to 1610+00	No	No	No	No	No	No	Maximum Repairs
3 (180605)	1612+00 to 1617+50	No	No	No	No	No	No	Maximum Repairs
4 (180603)	1622+00 to 1627+00	No	4.0	7.5	Yes	Yes	Yes	Minimum Repairs
5 (180606)	1630+00 to 1635+00	No	4.0	No	No	No	No	Maximum Repairs
6 (180604)	1637+00 to 1642+00	No	4.0	No	No	No	No	Saw Seal Joints in Overlay
7 (180611)	1644+00 to 1649+00	No	4.0	No	No	No	No	Minimum Repairs
8 (180610)	1651+00 to 1656+00	No	5.5	7.5	Yes	Yes	Yes	Minimum Repairs
9 (180622)	1658+00 to 1663+00	No	5.5	5.0	Yes	Yes	Yes	Minimum Repairs
10 (180609)	1665+00 to 1670+00	No	5.5	No	No	No	No	Minimum Repairs
11 (180608)	1674+00 to 1679+00	Yes	8.0	No	No	No	No	
12 (180612)	1682+00 to 1687+00	Yes	10.0	No	No	No	No	
13 (180613)	1693+00 to 1698+00	Yes	5.5	7.5	Yes	Yes	Yes	
14 (180614)	1701+00 to 1706+00	Yes	5.5	7.5	Yes	Yes	No	
15 (180615)	1712+00 to 1717+00	Yes	5.5	7.5	Yes	No	No	
16 (180616)	1722+00 to 1727+00	Yes	5.5	5.0	Yes	No	No	
17 (180617)	1732+00 to 1737+00	Yes	5.5	5.0	Yes	Yes	No	
18 (180618)	1742+00 to 1747+00	Yes	5.5	5.0	Yes	Yes	Yes	
19 (180619)	1751+00 to 1756+00	Yes	4.0	7.5	Yes	Yes	Yes	
20 (180620)	1758+00 to 1763+00	Yes	4.0	7.5	Yes	Yes	No	
21 (180621)	1765+00 to 1770+00	Yes	4.0	7.5	Yes	No	No	
22 (180607)	1773+00 to 1778+00	Yes	4.0	No	No	No	No	

* Section numbers in parenthesis are the SHRP project number.

**These are target fiber contents. As discussed below, extracted fiber contents varied substantially from these target values.

Table 3.2 Combinations of Test Sections on I-65

Section No.	Station	Crack & Seat	Overlay Thickness(In.)	Fiber*				Remarks
				#/Ton	Base	Binder	Surface	
1	1102+50 to 1107+50	No	5.5	7.5	Yes	Yes	Yes	Normal Repairs
2	1092+50 to 1097+50	No	5.5	5.0	Yes	Yes	Yes	Normal Repairs
3	1082+50 to 1087+50	No	5.5	No	No	No	No	Normal Repairs
4	927+50 to 932+50	Yes	8.0	No	No	No	No	
5	917+50 to 922+50	Yes	10.0	No	No	No	No	
6	907+50 to 912+50	Yes	5.5	7.5	Yes	No	No	
7	897+50 to 902+50	Yes	5.5	7.5	Yes	Yes	No	
8	887+50 to 892+50	Yes	5.5	7.5	Yes	Yes	Yes	
9	877+50 to 882+50	Yes	5.5	5.0	Yes	Yes	Yes	
10	867+50 to 872+50	Yes	5.5	5.0	Yes	Yes	No	
11	857+50 to 862+50	Yes	5.5	5.0	Yes	No	No	

*These are target fiber contents. As discussed below, extracted fiber contents varied substantially from these target values.

Table 3.3 Specification Gradations

Sieve Size	% Pass		
	#5 Base	#8 Binder	#11 Surface
1½ in.	100		
1 in.	80-99	100	
¾ in.	67-90	80-98	
1/2 in.	42-74	56-80	100
3/8 in.	33-60	43-68	85-98
No.4	30±5	35±5	62±5
No.8	12-34	14-40	31-62
No.16	7-28	8-32	17-50
No.30	4-22	5-24	8-37
No.50	1-16	2-16	3-25
No.100	0-10	0-10	0-14
No.200	0-3	0-3	0-3

Asphalt

AC-20 asphalt from Amoco Oil Co. was used on I-65 and AC-20 asphalt from Laketon Refining was used on US-31.

Fiber

Fiber Pave 3010, Polypropylene fibers from Hercules, Inc., was used in various combinations of the overlay layers (base, binder and surface courses). The properties of the fiber are listed in Table 3.4.

Table 3.4 Properties of Polypropylene Fibers

Denier	4±1
Length	10±2 mm
Crimps	none
Tensile Strength	40,000 psi,min.
Specific Gravity	0.91±0.04
Melting Point	295°F,min.
Asphalt Affinity	≥ than 7.0 lb.
Moisture Absorption	less than 0.1%
Alkali Resistance	99% strength retained
Acid Resistance	99% strength retained

Aggregate

Two types of coarse aggregate used on US-31 were a gravel from J&M Gravel Co., Etna Green, Indiana and a slag from Levy Co., Gary, Indiana. Coarse aggregate used on I-65 was a slag from Levy Co., Gary, Indiana. Fine aggregate used on US-31 was a #24 natural sand from J&M Gravel Co., Etna Green, Indiana. Fine aggregates used on I-65 were manufactured limestone screening from Rensselaer Stone Inc., Pleasant Ridge, Indiana and #24 natural sand from Morroco Sand&Gravel, Rensselaer, Indiana. The percentages of aggregate in the mixtures are listed in Table 3.5.

3.3 Equipment

A guillotine hammer was used to break the concrete pavement for both I-65 and US-31. The weight of the guillotine hammer was seven tons and it was dropped from a height of 32 inches. Crack spacing achieved with this hammer weight and drop height was 22 to 24 inches. Figure 3.2 shows this equipment working on I-65. A heavy pneumatic roller was used to seat the cracked concrete pavement (weights were 40 and 50 tons on I-65 and 40 tons on US-31). Figure 3.3 shows this equipment. Non destructive deflection tests were made using Dynatest 8000 and Dynaflect shown in Figure 3.4 and Figure 3.5 respectively.

Table 3.5 Percentages of Aggregate in the Mixtures

	I-65		US-31	
#5 Base	#5 Slag	72%	#5 Gravel	
	Limestone Screenings	10%	#5 Slag	
	#24 Natural Sand	18%	#11 Slag	
			#24 Natural Sand	
#8 Binder	#8 Slag	65%	#8 Slag	55%
	Limestone Screenings	17%	#11 Slag	10%
	#24 Natural Sand	18%	#12 Gravel	10%
			#24 Natural Sand	25%
#11 Surface	#11 Slag	40%	#11 Dolomite	27%
	Limestone Screenings	25%	#11 Slag	27%
	#24 Natural Sand	35%	#24 Natural Sand	46%

Figure 3.2 Guillotine Hammer

Figure 3.3 Pneumatic Roller

Figure 3.4 Dynatest 8000

Figure 3.5 Dynaflect

3.4 Field Investigations

Field investigations were conducted that included pavement condition surveys and non destructive testing (NDT) using a FWD or Dynaflect. Pavement condition surveys have been conducted each year since overlay construction to monitor the pavement condition and evaluate the performance of each test section. Deflection measurements were made on the concrete pavement surface before cracking, after seating, and on the overlay surface of all test sections. Subsequently, deflections were measured once a year in all test sections. Deflection measurements were also made on selected sections during seating to evaluate seating techniques.

3.4.1 Deflection measurement

Deflection measurements were made with both the Dynatest 8000 FWD and the Dynaflect at five different periods, namely before cracking, during seating , after seating, shortly after overlay, and one and two years after overlay. All deflections were taken in the outer wheel path of the driving lane, that is about 2.5 feet from outer edge of the pavement. The patterns which were followed in making the NDT deflection measurements are discussed below. Deflections were corrected for temperature of the asphalt overlay at the time of testing.

1. Layout of deflection measurements

(a) **Before cracking.** Deflections were taken for all test sections on both sides of joints or transverse cracks and at slab midpoints throughout the test sections. The pattern of the deflection tests is shown in Figure 3.6.

(b) **During seating.** Deflections during seating were taken in selected test sections to evaluate the effect of weight and pass of the roller on seating the cracked concrete pavement. A pneumatic roller was used to seat the cracked concrete pavement on both I-65 and US-31.

The roller weights used in this study were 40 and 50 tons on I-65 and 40 tons on US-31. The deflection testing sequence is illustrated in Figure 3.7.

At several randomly selected sites, at least 4 positions one foot apart were marked before the pavement was cracked. Since crack spacing is typically two to three feet and the diameter of FWD loading plate is about 12", this pattern would provide continuous deflection measurements and span at least one crack. Deflections at these positions were taken at different stages of cracking and seating, namely before cracking, after cracking, and after one, three, five and seven passes of the roller, respectively.

Figure 3.6 Testing Pattern Before Cracking

Fig 3.7 Testing Pattern During Cracking and Seating

(c) **After seating.** Deflections were taken for all test sections after the cracked concrete pavements were seated by the optimum number of roller passes. Typical layout of measurements on each test section is illustrated in Figure 3.8.

Eight sites were chosen on each test section to fall adjacent to joints and at the middle of slabs alternatively. At each site four measurements were taken one foot apart. As a result, 32 deflection measurements were taken for each test section.

Figure 3.8 Testing Pattern After Seating

(d) **Deflections after overlay, one year and two year deflections.** Deflections were taken shortly after the pavement was overlaid in November, 1990. Also, one year and two year deflections were taken respectively in September, 1991 and August, 1992. The deflection testing pattern is shown in Figure 3.9.

Figure 3.9 Testing Pattern After Overlay

As before, eight sites were tested. The first site was randomly selected within 100 feet of the beginning of the test section. Each additional test site was spaced at 55 feet. Four deflections were taken at one site one foot apart. A total of 32 deflections were taken for each test section.

2. Correction of deflections

Bituminous pavement stiffness varies with temperature. Subgrade support varies seasonally. As a result, pavement response to NDT loading will be different depending on the pavement temperature and season. Gupta[1987] developed procedures to correct for temperature and seasonal variations in Indiana. These procedures were utilized to correct deflection in this project and are summarized below

Temperature correction for flexible overlay: The procedure for making temperature corrections involves:

- (a) Collecting pavement surface temperature during the NDT test.
- (b) Compute mean air temperature as the average of the highest and lowest air temperatures for each day for five previous days. This temperature information can be obtained from the National Oceanic and Atmospheric Administration.
- (c) Predict pavement temperature at three depths (i) 1-inch below the surface, (ii) mid-depth of the AC overlay, and (iii) bottom of the AC overlay by utilizing the regression curves developed by Southgate[1968]. Separate regression curves for each hour of the day at various depths of the pavement have been developed. Figure 3.10 shows one set of those curves for 16:00 hours.
- (d) Compute mean pavement temperature as average of the temperatures at the three depths.

- (e) Apply a dimensionless temperature adjustment factor to the measured deflection. A set of temperature adjustment curves from Appendix L of the 1986 AASHTO Guide is shown in Figure 3.11.

Temperature correction for rigid pavement: Stiffness of rigid concrete pavement does not vary significantly with temperature and therefore no temperature correction was made for rigid pavements in this research.

Climate correction: Regression equations developed by Gupta[1987] provide a correlation between spring and fall/summer deflections for flexible and rigid pavements. In this research, deflections were taken during summer or fall. Therefore no climate correction was made for the deflections.

3.4.2 Pavement Condition Survey

Pavement condition surveys have been conducted four times since overlay construction to monitor the performance of each section. The Paver [PAVER Field Manual] condition survey procedure was used to identify and quantify the distress. Only two types of distresses were observed in the first condition survey which was conducted September 27, 1991. One was edge cracking and the other was lane/shoulder drop-off which was a result of the shoulder level not being adjusted when the overlay was constructed. No transverse and longitudinal reflective cracks were observed.

The second, third and fourth surveys were conducted April 12, 1992, November 24, 1992 and May 13, 1993, respectively. The results from these three condition surveys indicate that the quantity and severity of edge cracking and lane/shoulder drop-off remained the same. Transverse and longitudinal reflective cracking were recorded in these three surveys. Light rutting in the range of 1/4-3/8 in. was observed in three test sections on I-65 during the fourth condition survey. No other distresses were observed.

Figure 3.10 Temperature Prediction Graph for Pavement Greater Than Two Inches Thick (Southgate, 1968)

Curve Identification

Base Material	Curve (Base Thickness)
Asphalt (Full Depth)	A (All Thicknesses)
Asphalt (Deep Strength)	B (4" of granular subbase)*
Portland Cement Concrete	G
Granular (Non Stabilized)	C(6"); D(12"); E(20"); F(25")
Cement Treated Base	
Sound	D(4"); E(8")
Cracked	C(4"); D(8")

(*) If more than 4" of granular material present use "Granular (Non Stabilized)" base material category

Figure 3.11 Deflection - Temperature Adjustment Factor(AASHTO, 1986).

3.5 Laboratory Tests

This section presents the laboratory test procedures and testing sequence in this study.

Ten 6" cores and four 20"x12" slabs were cut from each of four sections with different fiber

contents on both I-65 and US-31. The 6" cores were separated by layers, stacked and capped for complex modulus testing. The overlay slabs were cut into 3"x3"x15" or 1.5"x1.5"x15" beams for fatigue testing. The balance of the samples were used for physical property testing of the asphalt mixture in accordance with the following ASTM procedures and a fiber extraction procedure developed for this research:

1. Bulk Specific Gravity (ASTM D2726-89),
2. Maximum Specific Gravity (ASTM D2041-90),
3. Sieve Analysis (ASTM C136-84a).

A flow chart showing the general test sequence is given in Figure 3.12.

Figure 3.12 Flow Chart of Laboratory Testing

3.5.1 Fiber Extraction Test

Due to lack of a standard method for fiber extraction, three fiber extraction methods have been developed for this project.

Method A uses trichloroethylene as a wash solvent. The solvent carrying the fiber is passed through a filter. The fiber is collected in the filter which is then dried to constant mass in an oven at $100\pm 5^{\circ}\text{C}$. Subsequently the dried fiber (mixture of fiber and fine aggregates) is ashed. The difference of weight before and after ashing is the weight of fiber.

Method B extracts asphalt from the mixture using TEST METHOD A in ASTM D2172 (Quantitative Extraction of Bitumen from Bituminous Paving Mixture). The fiber-aggregate mixture from the extraction unit bowl is dried to constant mass. The fiber is separated from the aggregate by covering the fiber-aggregate mixture with water in a container. The water carrying the fiber is passed through a filter. The fiber is collected in the filter which is then dried to constant mass in an oven at $100\pm 5^{\circ}\text{C}$. Subsequently the dried mixture of fiber and fine aggregates is ashed and the weight of fiber is determined as in Method A.

Method C is a combination of ASTM D2172 (Quantitative Extraction of Bitumen From Bituminous Paving Mixture) and ASTM C136-84a (Sieve Analysis). Asphalt is extracted from the paving mixture using TEST METHOD A in ASTM D2172. The fiber-aggregate mixture from the extraction unit bowl is dried to a constant mass and a sieve analysis is conducted. The fibers hold together on top of the aggregate on each sieve and are collected. The ashing process and the determination of fiber weight are the same as those in Method A.

If only fiber content in the paving mixture is required, Test Method A may be used because it is simple. But more trichloroethylene is consumed in this method and the test is time-consuming. Test Method B can be used where asphalt content and fiber content are the primary goal of the extraction. If sieve analysis is required for the aggregate in the paving

mixture extraction test, Test Method C is recommended. Test Method C has the least extra work introduced by fiber content determination compared with the other two methods.

Preliminary tests were conducted to evaluate the precision of Method B and Method C for fiber extraction. Method A was not evaluated because asphalt content is not determined in this method. Two laboratory batches of hot-mixed bituminous mixture with known fiber content were prepared with proportions listed in Table 3.6. Each batch was split into two samples for extraction tests. The test results are listed Table 3.7.

Table 3.6 Sample Data of Pilot Tests

	Batch 1	Batch 2
Wt.of(aggr.+asp.)(g)	1265	1257
Wt.of fiber(g)	4.8	4.8
Fiber content(%)	0.379	0.382

Table 3.7 Results of Pilot Tests

Batch	1		2	
Sample	1	2	1	2
Test Method	B	B	C	C
Total Weight(g)	640.2	622.2	611.2	641.0
Wt. of fiber with the fine (g) (before burned)	6.73	6.74	3.39	2.81
Wt. of the fine (g) (after burned)	4.30	4.47	1.06	0.56
Wt. of fiber (g)	2.43	2.27	2.33	2.25
Fiber content(%)	0.381	0.366	0.383	0.352
Average of fiber contents (%)	0.374		0.368	
Percent of fiber extracted (%)	98.5		96.2	
Percent of the fine in fiber-fine mixture (%)	63.9	66.3	31.2	19.9

Results in Table 3.7 show that over 95% of fiber is extracted from paving mixture. The precision of both methods is good. However, Method C was selected for further use in this project since sieve analysis is incorporated.

The pilot tests also show that the amount of fine aggregate in fiber-fine mixture ranges from 19.9% to 66.3% by weight of the fiber-fine mixture. As a result ashing is an essential step to obtain the correct fiber content.

3.5.2 Flexural Fatigue Test

Flexural fatigue tests were conducted on beam specimens cut from slabs of the in-situ pavements. The slabs were obtained from test sections of both conventional and fiber modified asphalt mixtures. Testing equipment, sample preparation, and test procedures are described below.

1. Test Equipment

A Materials Testing Systems Corp.(MTS) electro hydraulic testing machine was used for applying repeated loads to beam specimens. Beam specimens were held in an apparatus built for repeated flexural tests. The flexural apparatus was placed in an environmental chamber capable of maintaining the temperature within 0.5°C during the testing period.

A Shaevitz type GCD-121-050 linear variable differential transducer (LVDT) was used to measure deflection at the beam's center. The tip of LVDT core was placed on a nut bonded to the center of the specimen. LABTECH NOTEBOOK software [LABTECH NOTEBOOK, 1991] was used to record the data automatically through a 12 bit DT2801-A A/D conversion board mounted in a personal computer. The repeated flexure apparatus in the environmental chamber and the MTS system is shown in Figure 3.13 and Figure 3.14, respectively.

Figure 3.13 Repeated Flexure Apparatus

Figure 3.14 MTS System

2. Beam Preparation

Slabs, 12"x20", were cut from the test sections on I-65 and US-31 in April, 1991 by McMahan-O'Connor Construction Company. All of the slabs were cut from the pavement in the outside wheel path. These samples were transported to the bituminous laboratory at Purdue University. In the laboratory these slabs were cut into two sizes of beams:

- (a) 1.5"x1.5"x15" beams which were used to test the fatigue properties for surface and binder layers of I-65.
- (b) 3"x3"x15" beams which were used to test the fatigue properties for composite layers from test sections on US-31.

A 3 in. composite beam consists of 1.5 in. surface and 1.5 in. binder or 1.5 in. binder and 1.5 in. base. Figure 3.15 shows a slab taken from the test section. Figure 3.16 shows the saw used to cut the slabs into beams. The two sizes of beams are shown in Figure 3.17.

Figure 3.15 Slab From the Test Section.

Figure 3.16 Laboratory Saw

Figure 3.17 Two Sizes of Beams for Fatigue Testing.

A nut was bonded with epoxy cement to the center of the specimen bottom after the beam was cut from the slabs. The core of the LVDT was seated in the nut during testing. This maintained the position of LVDT contact with the beam and reduced variability in the measurements.

3. Test Procedures

Before testing, widths and depths of beams were measured to the nearest 0.01 in. at the center and on both sides 2 in. from the center. The measurements were averaged for subsequent calculations. The loading clamps were adjusted vertically to the same level as the reaction clamps and the beams were clamped in the apparatus so that the centers of the two loading clamps were 2 in. from the beam center and 4 in. from the centers of two reaction clamps. Double layers of sheet teflon were placed between the beam and the loading clamps to reduce friction which would result in shearing stresses near the clamps.

All samples were tested in a controlled deformation mode at 77F using a 1 Hz haversine wave form with 0.1 second loading period and 0.9 second rest period. The beams were placed in the temperature controlled cabinet for at least 24 hours before testing in order to bring the beam to the testing temperature. Load was applied until the flexural stiffness modulus was 30% of the initial stiffness modulus measured at 200 load applications

3.5.3 Complex Modulus Test

Complex modulus test is used to evaluate linear viscoelastic properties of a paving mixture. The complex modulus test was conducted on 6 in. cores of the in-site pavements. Cores were obtained from test sections of both conventional and fiber modified asphalt mixtures. Testing equipment, sample preparation, and test procedures are described below.

1. Test Equipment

An MTS electro-hydraulic testing machine was used to apply a compressive dynamic haversine load to cylindrical specimens. The specimens were tested in a temperature-controlled environmental chamber which was capable of maintaining the temperature within 0.5 C during the testing period. Specimen deformations were measured with two Shaevitz type 050MHR LVDTs with two ATA-101 analog transducer amplifiers. Loads were measured with a 1000lb electronic load cell (MTS, model 661.20). Figure 3.18 shows the loading apparatus and deformation measuring system for complex modulus testing.

Figure 3.18 Complex Modulus Testing Apparatus

2. Core Preparation

Forty 6 in. cores each were taken from US-31 in April, 1991 and from I-65 in July, 1991. Each core was separated by layer in the laboratory using a diamond bladed saw. The same layers from different cores were stacked to a desired thickness for testing. Cement grout was used to bond the layers.

Three specimens were prepared for each combination of layer and fiber content. Each stacked specimen was capped on both ends with sulfur capping compound to provide surfaces that were smooth and parallel. Figure 3.19 shows the capping equipment and Figure 3.20 shows the samples prepared for complex modulus testing.

Figure 3.19 Capping Equipment

Figure 3.20 Capped Samples

3. Test Procedures

Before testing, diameters of the cores were measured at two locations at mid thickness. These measurements were averaged in subsequent calculations. The cores were placed in the temperature controlled cabinet for at least 24 hours before testing in order to bring the core to the testing temperature. Aluminum LVDT holders were attached to the side of the test specimens, centered about the middle of the core. Two LVDTs were mounted on opposite sides of the specimen. The specimen was centered on the lower load platen.

A small preload of 0.5-5 psi (preload depends on the loading frequency) was applied to the specimen without impact for about one minute and subsequently a haversine load varying from the preload (0.5-5.0 psi) to 35 psi was applied to the specimen for 30 seconds. Each specimen was tested at loading frequencies of 1, 4, and 8 Hz at temperatures of 20, 30, and 40 °C, starting at the lowest temperature and running the three frequencies from fastest to

slowest. The specimens were tested at each temperature and frequency condition twice. These replicate tests were conducted before the temperature was changed.

CHAPTER 4

DESIGN OF EXPERIMENT

4.1 Introduction

The performance of fiber modified asphalt overlay on cracked and seated concrete pavement depends on effectiveness of cracking and seating, effective fiber addition to mixture, thickness of overlay and other factors. It is important to understand the role that fiber plays to evaluate and predict the effects of these factors.

Experimental designs were developed to guide field and laboratory investigations. Six sets of experiments are presented that focus on the effects of cracking and seating techniques, overall performance of test sections, and effects on the mechanical properties from adding fiber to the mixtures.

This Chapter describes the independent variables and response variables on which the experimental design was based.

4.2 Design No.1: Fiber Content

Test sections were designed with four different fiber contents in the asphalt surface, binder and base. The design was developed based on the assumption that the target fiber contents would be reasonably achieved. However, these target values were not achieved during construction because the fiber introduction methods are not well developed in practice, which affects the breadth of the study and the results.

This design focuses on the actual fiber contents in each of four test sections. The independent variables are:

1. Test road (2 levels): I-65 and US-31;
2. Overlay layer (3 levels): base, binder and surface;

3. Target fiber content (4 levels): 0 lb., 5 lb., 6 lb. and 7.5 lb.. (note: fiber content is based on pounds per ton of asphalt mixture)

The response variable is the actual fiber content obtained from the fiber extraction test which has been developed for this project. The experimental design for fiber content is shown in Table 4.1.

4.3 Design No.2: Effects of Cracking and Seating Techniques

This design focused on the effects of cracking and seating techniques. The concrete slabs were cracked by a Guillotine hammer and seated by a heavy pneumatic-tired roller. The number of passes and the weight of the roller are the two major independent variables to be investigated. The independent variables are:

1. Construction stage (6 levels): before cracking (BC), after cracking (AC), After one pass (1P), after three passes (3P), after five passes (5P), and after seven passes (7P).
2. Weight of roller (2 levels): 40 and 50 tons.

Based on FWD tests, the response variables are the maximum deflection (D_{max}) in the center of the loading plate and Spreadability (SPR) of deflection basin which is defined by the following equation:

$$SPR\% = 100 \times \frac{\sum (D_i)}{n D_{max}} \dots \dots \dots (4.1)$$

where: D_i = deflection at sensor i

n = total number of sensors

During construction, one group of from 10 to 30 positions on a test section was randomly selected for deflection testing. Subsequently, another set of positions was selected and deflections were determined with another level of the roller weight.

Table 4.1 Design No.1 Fiber Content

Testing Road	Layer	Target Fiber Content (lb.)	Sample	
I-65	Surface	0	1	X
			2	X
		5	3	X
			4	X
		6	5	X
			6	X
		7.5	7	X
			8	X
	Binder	0	9	X
			10	X
		5	11	X
			12	X
		6	13	X
			14	X
		7.5	15	X
			16	X
	Base	0	17	X
			18	X
		5	19	X
			20	X
		6	21	X
			22	X
		7.5	23	X
			24	X
US-31	Surface	0	25	X
			26	X
		5	27	X
			28	X
		6	29	X
			30	X
		7.5	31	X
			32	X
	Binder	0	33	X
			34	X
		5	35	X
			36	X
		6	37	X
			38	X
		7.5	39	X
			40	X
	Base	0	41	X
			42	X
		5	43	X
			44	X
		6	45	X
			46	X
		7.5	47	X
			48	X

#: Pounds of fiber per ton of mixture.

The layout of this design is shown in Table 4.2.

Table 4.2 Design No.2 The Effects of Cracking and Seating Technique

Sample	Construction Stage					
	BC	AC	1P	3P	5P	7P
1	X	X	X	X	X	X
2	X	X	X	X	X	X
.	.	.	.	.	.	.
.	.	.	.	.	.	.
.	.	.	.	.	.	.
.	.	.	.	.	.	.
N-1	X	X	X	X	X	X
N	X	X	X	X	X	X

4.4 Design No.3: Overall Performance of Test Sections Based on Deflections

These designs focus on the effects of different treatment combinations on the performance of asphalt overlays. The independent variables are

1. Condition of concrete pavements before overlay (2 levels): Cracked and seated and non cracked and seated,
2. Thickness of overlay (4 levels): 4", 5.5", 8", and 10",
3. Actual fiber content (3 levels): 0, 5.2, and 6.1 lb. on I-65; 0, 4.5, and 4.8 lb. on US-31,
4. Location of layers with fibers (4 levels): No layer, only base layer, (base + binder) layers and (base + binder + surface) layers.

Response variables are the deflections before cracking, after seating and after overlay. There are 18 treatment combinations in test sections on US-31 and 11 treatment combinations on I-65 (one treatment combination in one test section). The layout of this design is shown in Table 4.3 and Table 4.4.

Table 4.3 Design No.3 Overall Performance of Test Sections on US-31

Actual Fiber Content (lb.)	Thickness	4"		5.5"		8"		10"	
	Cracked	Y	N	Y	N	Y	N	Y	N
	Location								
0		22	5,6,7		10	11		12	
4.5	BA	21		15					
	BA+BI	20		14					
	BA+BI+SU	19	4	13	8				
4.8	BA			16					
	BA+BI			17					
	BA+BI+SU			18	9				

Note: The numbers in the cell represent the section number.

Table 4.4 Design No.3 Overall Performance of Test Sections on I-65

Actual Fiber Content (lb.)	Thickness	5.5"		8"		10"	
	Cracked	Y	N	Y	N	Y	N
	Location						
0			3	4		5	
5.2	BA	6					
	BA+BI	7					
	BA+BI+SU	8	1				
6.1	BA	11					
	BA+BI	10					
	BA+BI+SU	9	2				

Note: The numbers in the cell represent the section number.

4.5 Design No.4: Overall Performance of Test Sections Based on Results of Condition Survey.

These designs focus on the effects of different treatment combinations on the performance of asphalt overlays based on results of condition surveys. The independent variables are the same as those in design No.3. The response variables are the total transverse crack length of each section. There are 18 treatment combinations in test sections on US-31

and 11 treatment combinations on I-65 (one treatment combination in one test section). The layout of this design is same as those shown in Table 4.3 and Table 4.4.

4.6 Design No.5: Fatigue Life of Fiber Modified Mixture

This design deals with the fatigue life of mixtures with and without fiber based on results from beam fatigue test. The independent variables are:

1. Fiber (2 levels): with fiber, without fiber
2. Overlay layer (2 levels): surface and binder for 1.5" beams, (surface+binder) and (binder+base) for 3" composite beams

Response variables are the cumulative number of loading cycles when a beam fails. Two sizes of beams were tested. One size is 1.5"x1.5"x15" which is comprised of the same material. Another size is 3"x3"x15" which is a composite beam, 1.5" surface plus 1.5" binder or 1.5" binder plus 1.5" base. The experimental design for tests of the 1.5" beams is shown in Table 4.5 and the experimental design for tests of the 3" beams is shown in Table 4.6.

Table 4.5 Design No.5 Fatigue Tests of 1.5" Beams.

		Layer	
		Surface	Binder
Fiber	With Fiber	X	X
	Without Fiber	X	X

Table 4.6 Design No.5 Fatigue Tests of 3" Beams.

		Layer	
		Surface+Binder	Binder+Base
Fiber	With Fiber	X	X
	Without Fiber	X	X

4.7 Design No.6: Complex Modulus Test

This design focuses on determining characteristics of overlay layers with different fiber contents based on results from laboratory complex modulus tests. The independent variables are:

1. Actual fiber content (4 levels): 0, 3.4, 5.2, and 6.1 lb. on I-65; 0, 3.5, 4.5, and 4.8 lb. on US-31,
2. Overlay layer (3 levels) : base, binder and surface,
3. Testing temperature (3 levels): 20°C, 30°C, 40°C
4. Loading frequency (3 levels): 1 Hz, 4 Hz, 8 Hz

Response variables are the dynamic modulus which is a measure of the material elasticity and the phase angle which is a measure of the viscous response of the material. The experimental design is shown in Table 4.7 and Table 4.8 for I-65 and US-31, respectively.

Table 4.7 Design No.6 Dynamic Modulus tests on I-65

Actual Fiber Content (lb.)	Layer	Temp.	20°C			30°C			40°C		
		Freq. Sample	1 Hz	4 Hz	8 Hz	1 Hz	4 Hz	8 Hz	1 Hz	4 Hz	8 Hz
0	Surface	1	X	X	.	.	.	.	.	.	X
		2	X	X	.	.	.	.	.	.	X
		3	X	X	.	.	.	.	.	.	X
	Binder	4	.	.	.	.	.	.	.	.	.
		5	.	.	.	.	.	.	.	.	.
		6	.	.	.	.	.	.	.	.	.
	Base	7	.	.	.	.	.	.	.	.	.
		8	.	.	.	.	.	.	.	.	.
		9	.	.	.	.	.	.	.	.	.
3.4	Surface	10	.	.	.	.	.	.	.	.	.
		11	.	.	.	.	.	.	.	.	.
		12	.	.	.	.	.	.	.	.	.
	Binder	13	.	.	.	.	.	.	.	.	.
		14	.	.	.	.	.	.	.	.	.
		15	.	.	.	.	.	.	.	.	.
	Base	16	.	.	.	.	.	.	.	.	.
		17	.	.	.	.	.	.	.	.	.
		18	.	.	.	.	.	.	.	.	.
5.2	Surface	19	.	.	.	.	.	.	.	.	.
		20	.	.	.	.	.	.	.	.	.
		21	.	.	.	.	.	.	.	.	.
	Binder	22	.	.	.	.	.	.	.	.	.
		23	.	.	.	.	.	.	.	.	.
		24	.	.	.	.	.	.	.	.	.
	Base	25	.	.	.	.	.	.	.	.	.
		26	.	.	.	.	.	.	.	.	.
		27	.	.	.	.	.	.	.	.	.
6.1	Surface	28	.	.	.	.	.	.	.	.	.
		29	.	.	.	.	.	.	.	.	.
		30	.	.	.	.	.	.	.	.	.
	Binder	31	.	.	.	.	.	.	.	.	.
		32	.	.	.	.	.	.	.	.	.
		33	.	.	.	.	.	.	.	.	.
	Base	34	X	X	.	.	.	.	.	.	X
		35	X	X	.	.	.	.	.	.	X
		36	X	X	.	.	.	.	.	.	X

Table 4.8 Design No.6 Dynamic Complex Modulus tests on US-31

Actual Fiber Content (lb.)	Layer	Temp.	20°C			30°C			40°C		
		Freq. Sample	1 Hz	4 Hz	8 Hz	1 Hz	4 Hz	8 Hz	1 Hz	4 Hz	8 Hz
0	Binder	1	X	X	.	.	.	.	.	.	X
		2	X	X	.	.	.	.	.	.	X
		3	X	X	.	.	.	.	.	.	X
	Base	4	.	.	.	.	.	.	.	.	.
		5	.	.	.	.	.	.	.	.	.
		6	.	.	.	.	.	.	.	.	.
3.5	Binder	7	.	.	.	.	.	.	.	.	.
		8	.	.	.	.	.	.	.	.	.
		9	.	.	.	.	.	.	.	.	.
	Base	10	.	.	.	.	.	.	.	.	.
		11	.	.	.	.	.	.	.	.	.
		12	.	.	.	.	.	.	.	.	.
4.5	Binder	13	.	.	.	.	.	.	.	.	.
		14	.	.	.	.	.	.	.	.	.
		15	.	.	.	.	.	.	.	.	.
	Base	16	.	.	.	.	.	.	.	.	.
		17	.	.	.	.	.	.	.	.	.
		18	.	.	.	.	.	.	.	.	.
4.8	Binder	19	.	.	.	.	.	.	.	.	.
		20	.	.	.	.	.	.	.	.	.
		21	.	.	.	.	.	.	.	.	.
	Base	22	X	X	.	.	.	.	.	.	X
		23	X	X	.	.	.	.	.	.	X
		24	X	X	.	.	.	.	.	.	X

CHAPTER 5

RESULTS AND ANALYSIS

This chapter presents the testing data collected from field and laboratory investigations which were guided by the six experimental designs described in chapter 4. The response variables were analyzed with the aid of Statistical Analysis Software (SAS) [SAS Institute, Inc. 1989]. An analysis of variance (ANOVA) was performed in order to determine the significance of the effects of certain factors and/or interactions of factors. A discussion is presented based on the testing data and the statistical analysis.

5.1 Fiber Extraction and Other Verification Tests

5.1.1 Fiber Extraction

1. Results

Extraction results of fiber contents are listed in Table 5.1.

2. Statistical Analysis

Statistical Analysis Software (SAS) was used to analyze the data. The following linear model was assumed in the analysis of fiber content.

$$Y_{ijkl} = \mu + C_i + L_j + CL_{ij} + R_k + CR_{ik} + LR_{jk} + CLR_{ijk} + S_{l(ijk)} + \varepsilon_{ijkl} \dots\dots\dots(5.1)$$

Where: Y_{ijkl} = response variable, actual fiber content

- μ = overall mean
- C_i = effect of target fiber content.
- L_j = effect of layer
- R_k = effect of test road
- $S_{l(ijk)}$ = within error of CLR combination
- ε_{ijkl} = experimental error

CL_{ij} , CR_{ik} , LR_{jk} , CLR_{ijk} = effect of interactions of main factors

$i = 1, 2, \dots, 4$ $j = 1, 2, 3$ $k = 1, 2$ $l = 1, 2, \dots, 48$

ANOVA was performed on the test results in Table 5.1. Results of ANOVA are summarized in Table 5.2. The main effect, target fiber content and test road, are significant. The main effect, layer, is not significant, which means that the fiber contents are not significantly different among the layers within a section designed with one target fiber content.

A Student-Newman-Keuls (SNK) test was used to evaluate the mean of actual fiber content for each combination of test roads and target fiber content. The mean fiber contents as well as SNK grouping are listed in Table 5.3.

Table 5.1 Extraction Results of Fiber Contents

Test Roads	Layer Types	Target Fiber Content (lb.)	Actual Fiber Content (lb.)		
			Sample 1	Sample 2	Avg.
I-65	Surface	0	0.0	0.0	0.0
		5	5.1	6.3	5.7
		6	3.1	3.6	3.4
		7.5	6.1	6.1	6.1
	Binder	0	0.0	0.0	0.0
		5	5.1	5.3	5.2
		6	3.0	2.9	3.0
		7.5	4.8	5.3	5.0
	Base	0	0.0	0.0	0.0
		5	7.6	7.2	7.4
		6	4.1	3.6	3.9
		7.5	4.5	4.6	4.5
US-31	Surface	0	0.0	0.0	0.0
		5	4.6	4.6	4.6
		6	4.3	4.1	4.2
		7.5	4.3	4.2	4.2
	Binder	0	0.0	0.0	0.0
		5	5.6	5.7	5.7
		6	3.4	3.4	3.4
		7.5	4.7	5.4	5.0
	Base	0	0.0	0.0	0.0
		5	4.0	4.1	4.1
		6	3.3	2.6	2.9
		7.5	4.2	4.2	4.2

Table 5.2 ANOVA Results for Fiber Contents

Source of Variation	df	SS	MS	F	Pr>F
C	3	213.55	71.184	930.99	0.0001
L	2	0.19	0.095	1.24	0.3080
L*C	6	2.74	0.456	5.97	0.0006
R	1	2.83	2.828	36.98	0.0001
R*C	3	4.04	1.348	17.63	0.0001
R*L	2	3.85	1.927	25.20	0.0001
R*L*C	6	7.36	1.226	16.04	0.0001
S(R*L*C)	24	1.84	0.076		

Note: S(R*L*C) is used as an error term for tests of hypotheses

Table 5.3 Mean Fiber Contents

Test Road	Target fiber Content(lb.)	Actual Fiber Content(lb.)	SNK Grouping	Percentage Different(%)
I-65	0.0	0.0	A	0.0
	5.0	6.1	B	+21.8
	6.0	3.4	C	-43.5
	7.5	5.2	D	-30.4
US-31	0.0	0.0	A	0.0
	5.0	4.8	B	-4.4
	6.0	3.5	C	-41.7
	7.5	4.5	D	-40.3

Fiber extraction results show that fiber contents are different from target fiber contents by a significant amount. Fiber contents in all fiberized sections except for the 5 lb. section on I-65 are lower than target fiber contents by a range of 4.4% to 43.5%. The 5 lb. section on I-65 has a higher fiber content than the target fiber content by 21.8%. The differences between

actual fiber contents and target fiber contents reflect the need for improved techniques to introduce fiber into the mixture during the mixing process.

SNK groupings show that actual fiber contents are significantly different from each other among the four sections within the test road. It should be noted that the difference of actual fiber contents between 5 lb. section and 7.5 lb. section is very small numerically even though actual fiber contents in these two sections are statistically significantly different.

5.1.2 Asphalt Extraction Test

The results of asphalt extraction tests are summarized in Table 5.4. Two values of asphalt contents, actual asphalt content and effective asphalt content are specified in Quality Assurance (Q/A) Test Data. Actual asphalt content is the percent of asphalt required by the job mix formula. The effective asphalt content is the percent of asphalt obtained from extraction. Comparing the test results with effective contents in Q/A Test Data (with tolerance of 0.5% from ASTM), it can be seen that asphalt contents are well controlled during the construction of test sections.

5.1.3 Bulk Specific Gravity, Maximum Specific Gravity and Air Voids

Table 5.5 summarizes the test results of bulk specific gravity (BSG), maximum specific gravity (MSG) and air voids of the mixtures. Comparing the actual air voids of mixture with target air voids in Q/A Test Data, it is concluded that mixtures are not compacted to the required air voids during construction.

Figure 5.1 depicts the air voids with respect to the sections with different actual fiber contents. It can be observed that in general the conventional mixture has lower air voids than fiberized mixtures. Since the conventional mixture does not have significantly different asphalt content from fiberized mixtures, it appears that addition of fiber is the major cause for higher percent of air voids.

Table 5.4 Extraction Results of Asphalt Contents(%)

Test Road	Layer Type	Actual Fiber Content(lb.)	Test Result			Values in Q/A Test Data.	
			Sample 1	Sample 2	AVG.	Actual	Effective
I-65	Surface	0	6.0	6.1	6.0	6.5	6.2
		3.4	5.9	6.0	6.0	6.6	6.3
		5.2	5.8	6.0	5.9	6.6	6.4
		6.1	5.9	6.0	6.0	6.5	6.2
	Binder	0	5.5	5.8	5.6	5.7	5.4
		3.4	5.4	5.4	5.4	5.9	5.5
		5.2	5.5	5.7	5.6	6.0	5.6
		6.1	5.6	5.5	5.6	5.9	5.6
	Base	0	4.4	4.0	4.2	4.9	4.6
		3.4	4.4	4.4	4.4	4.8	4.5
		5.2	4.5	4.3	4.4	5.0	4.7
		6.1	4.5	4.9	4.7	4.8	4.5
US-31	Surface	0	5.9	5.9	5.9	6.1	5.8
		3.5	5.6	5.5	5.6	6.1	5.8
		4.5	5.9	5.8	5.8	6.1	5.8
		4.8	5.9	5.7	5.8	6.1	5.8
	Binder	0	4.9	5.2	5.1	4.9	4.7
		3.5	4.5	4.4	4.5	5.2	4.9
		4.5	4.6	4.4	4.5	5.2	4.9
		4.8	5.1	5.0	5.1	5.2	4.9
	Base	0	4.7	4.5	4.6	5.3	5.0
		3.5	4.9	4.7	4.8	5.3	5.0
		4.5	4.7	5.1	4.9	5.3	5.0
		4.8	4.6	4.9	4.8	5.3	5.0

Table 5.5 Results of BSG, MSG and Air Voids

Test Road	Layer Type	Actual Fiber Content(lb.)	Test Result			Voids in Q/A Test Data
			BSG	MSG	Voids	
I-65	Surface	0	2.299	2.389	3.8	6.0
		3.4	2.225	2.369	6.1	6.0
		5.2	2.224	2.398	7.2	6.0
		6.1	2.186	2.345	6.8	6.0
	Binder	0	2.281	2.322	1.8	5.9
		3.4	2.226	2.332	4.6	6.2
		5.2	2.176	2.326	6.4	5.9
		6.1	2.183	2.354	7.3	6.0
	Base	0	2.184	2.323	6.0	6.0
		3.4	2.107	2.331	9.6	5.9
		5.2	2.108	2.330	9.6	6.0
		6.1	2.061	2.304	10.5	5.8
US-31	Surface	0	2.230	2.426	8.1	6.0
		3.5	2.163	2.427	10.9	6.0
		4.5	2.207	2.394	7.8	6.0
		4.8	2.195	2.396	8.4	6.0
	Binder	0	2.188	2.360	7.3	6.0
		3.5	2.124	2.348	9.5	6.0
		4.5	2.094	2.305	9.1	6.0
		4.8	2.126	2.318	8.3	6.0
	Base	0	2.210	2.356	6.2	6.0
		3.5	2.168	2.336	7.2	6.0
		4.5	2.149	2.313	7.1	6.0
		4.8	2.155	2.319	7.1	6.0

Figure 5.1 Actual Air Voids

5.1.4 Sieve Analysis

Sieve analysis results are summarized in Table 5.6. Figure 5.2 through Figure 5.4 show the gradation curves of mixtures as well as the gradation curve envelopes in the specification. It is noted that the gradations satisfy the specification, except for the binder on I-65.

Table 5.6 Sieve Analysis Results

Layer Type	Sieve Size	I-65				US-31				% Passing in Spec.
		0 lb.	3.4 lb.	5.2 lb.	6.1 lb.	0 lb.	3.5 lb.	4.5 lb.	4.8 lb.	
S U R F A C E	1/2"	100	100	100	100	100	100	100	100	100
	3/8"	91	91	92	92	97	93	92	93	85-98
	#4	61	61	60	61	59	57	56	57	62±5
	#8	44	45	43	46	47	45	45	46	31-62
	#16	34	36	34	37	37	36	37	37	17-50
	#30	26	26	25	28	26	25	25	26	8-37
	#50	14	12	14	14	10	10	10	10	3-25
	#100	5	4	5	4	3	4	3	3	0-14
	#200	3.2	1.7	2.4	1.4	1.9	1.9	1.4	1.3	0-3
B I N D E R	1"	100	100	100	100	100	100	100	100	100
	3/4"	98	97	95	94	93	93	92	99	80-98
	1/2"	82	81	80	80	78	80	77	82	56-80
	3/8"	64	61	62	61	63	62	59	66	43-68
	#4	46	43	44	45	39	40	39	43	35±5
	#8	37	34	35	36	28	29	30	31	14-40
	#16	31	29	30	29	23	24	24	25	8-32
	#30	25	23	24	22	17	17	17	18	5-24
	#50	14	13	14	13	8	8	8	9	2-16
	#100	5	5	5	5	4	4	4	4	0-10
	#200	2.8	2.5	2.9	2.4	2.3	2.2	2.5	2.5	0-3
B A S E	1.5"	100	100	100	100	100	100	100	100	100
	1"	83	92	95	91	94	98	100	98	80-99
	3/4"	77	80	83	86	85	84	86	86	67-90
	1/2"	61	69	72	78	64	62	68	66	42-74
	3/8"	48	55	56	67	50	45	51	48	33-60
	#4	33	30	32	39	32	29	33	31	30±5
	#8	27	23	24	29	27	25	27	26	12-34
	#16	23	19	21	24	23	21	22	22	7-28
	#30	19	16	17	18	16	16	16	16	4-22
	#50	10	10	10	10	8	8	9	8	1-16
	#100	4	4	4	4	4	4	4	4	0-10
	#200	2.2	2.6	2.5	2.3	2.3	2.4	2.6	2.3	0-3

Figure 5.2 Gradation Curves of Surface Mixtures

Figure 5.3 Gradation Curves of Binder Mixtures

Figure 5.4 Gradation Curves of Base Mixtures

5.2 Effects of Cracking and Seating

1. Results

Deflection measurements were made on each test road using both the falling weight deflectometer and Dynaflect devices. The deflections at various stages are summarized in Table 5.7.

2. Statistical Analysis

The following linear model was assumed in analysis:

$$Y_{ij} = \mu + T_i + S_j + \varepsilon_{k(ij)} \dots \dots \dots (5.2)$$

where:

Y_{ij} = response variable, maximum deflection or spreadability

μ = overall mean

T_i = effect of cracking and seating

S_j = effect of deflection test position

$\varepsilon_k(ij)$ = experimental error

ANOVA was performed on the test results of the deflections using SAS. The $Pr>F$ values from ANOVA results are listed in Table 5.8.

Based on $Pr>F$ values, cracking and seating has significant effect on maximum deflection and spreadability. The main effect of deflection test position is significant in Section A, Section B, Section C, and not significant in Section D at the 95% confidence level, which means that the variability of the maximum deflection and spreadability from position to position is significant within each of the first three Sections and is not significant in Section D.

Table 5.7 Summary of Deflection Measurements

Section		A		B		C		D	
Location		I-65		US-31		I-65		I-65	
Equipment		FWD		FWD		Dynalect		Dynalect	
Weight of Roller		40		40		50		40	
Stage		D_{max} (mils)	SPR(%)	D_{max} (mils)	SPR(%)	D_{max} (mils)	SPR(%)	D_{max} (mils)	SPR(%)
	BC	0.525	83.4	0.384	79.5	0.421	86.6	0.485	83.0
	AC	0.830	76.2	0.753	79.7	0.856	82.0	0.384	79.5
	P1	0.904	73.3	0.733	80.2				
	P3	0.844	75.1	0.752	80.1	0.875	82.6	0.389	75.4
	P5	0.867	75.2	0.718	81.8	0.892	82.7	0.388	76.1
	P7	0.851	74.8	0.758	81.3	0.905	82.6		

Note : (1) D_{max} = maximum deflection in the center of the loading plate

(2)SPR = spreadability of deflection basin

(3)BC = before cracking

AC = after cracking

P1 = after one pass

P3 = after three passes

P5 = after five passes

P7 = after seven passes

Table 5.8 Pr>F values

Source of Variation	Section A		Section B		Section C		Section D	
	D_{max}	SPR	D_{max}	SPR	D_{max}	SPR	D_{max}	SPR
T	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
S	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.1010	0.9980

The Student-Newman-Keuls(SNK) test was used to evaluate the means of maximum deflection or spreadability for each stage of cracking and seating at a confidence level of 95 %. The SNK grouping for maximum deflection and for spreadability are shown in Tables 5.9 and 5.10 respectively.

Figure 5.5 and Figure 5.6 show plots of mean maximum deflection and mean spreadability during cracking and seating. It can be observed that the deflections after cracking increase dramatically in all Sections except Section D, and spreadability after cracking is significantly smaller than that before cracking and seating, which means that the cracking technique used in this project is effective. The reason for the decrease in maximum deflection in Section D is not clear.

Based on SNK Grouping, maximum deflection varies with the number of roller passes. However, there is no indication as to which is the optimum number of passes to obtain a stable maximum deflection. For a roller weight of 40 tons, there is no significant difference between the deflections after cracking and after seating. The roller pass effect is not significant. Maximum deflections increase as roller pass increases in case of 50 ton roller used. This suggests that the concrete slabs lost their interlocking strength with each pass of 50 ton roller and the heavier roller might be detrimental.

The following recommendations are made regarding to cracking and seating techniques:

1. The significant change in maximum deflections before and after cracking indicates that the Guillotine breaker is effective in breaking PCC pavement into smaller pieces. The

deflection measurements should be made at the beginning of cracking to determine the appropriate weight and drop height of breaking head for the existing concrete pavement.

2. The 50 ton roller is not recommended because it might "overwork" the cracked concrete pavement.
3. Roller passes have no significant effect on the maximum deflections. The optimum roller pass can not be determined exclusively based on the maximum deflections. When seating is needed to assure contact of cracked pavement pieces with the subgrade, one to three passes of a 40 ton pneumatic-tired roller is adequate.

Table 5.9 SNK Grouping for Maximum Deflections

Section A			Section B		
SNK Grouping	Mean (mil)	T (Treatment)	SNK Grouping	Mean (mil)	T (Treatment)
A	0.904	P1	A	0.758	P7
A			A		
B A	0.867	P5	A	0.753	AC
B A			A		
B A	0.851	P7	A	0.752	P3
B A			A		
B A	0.844	P3	B A	0.733	P1
B			B		
B	0.830	AC	B	0.718	P5
C	0.525	BC	C	0.384	BC
Section C			Section D		
SNK Grouping	Mean (mil)	T (Treatment)	SNK Grouping	Mean (mil)	T (Treatment)
A	0.905	P7	A	0.485	BC
A					
B A	0.892	P5	B	0.389	P3
B			B		
B C	0.875	P3	B	0.388	P5
C			B		
C	0.856	AC	B	0.384	AC
D	0.421	BC			

Note: Means with the same grouping letter are not significantly different.

Table 5.10 SNK Grouping for Spreadability

Section A			Section B		
SNK Grouping	Mean (%)	T (Treatment)	SNK Grouping	Mean (%)	T (Treatment)
A	83.4	BC	A	81.8	P5
B	76.2	AC	A	81.3	P7
B			B	80.2	P1
B	75.2	P5	B		
B			B	80.1	P3
B	75.0	P3	B		
B			B	79.7	AC
B	74.8	P7	B		
C	73.3	P1	B	79.5	BC
Section C			Section D		
SNK Grouping	Mean (%)	T (Treatment)	SNK Grouping	Mean (%)	T (Treatment)
A	86.6	BC	A	83.0	BC
B	82.7	P5	B	79.5	AC
B			C	76.1	P5
B	82.6	P7	C		
B			C	75.4	P3
B	82.6	P3			
B					
B	82.0	AC			

Note: Means with the same grouping letter are not significantly different.

Figure 5.5 Variations of Mean Maximum Deflections During Cracking and Seating

Figure 5.6 Variations of Mean Spreadability During Cracking and Seating

5.3 Evaluation of Test Sections Based on Overall Deflections

1. Results

The mean deflections of each tested section on I-65 and US-31 are listed in Table 5.11 and Table 5.12.

2. Statistical Analysis

The following linear model was assumed in analysis:

$$Y_{ij} = \mu + T_i + S_j + T*S_{ij} + \mathcal{E}k(ij) \dots \dots \dots (5.3)$$

where:

Y_{ij} = response variable, maximum deflection

μ = overall mean

T_i = effect of test sections

S_j = effect of construction stages

$T*S_{ij}$ = effect of main factor interactions

$\mathcal{E}k(ij)$ = experimental error

An ANOVA was performed on the mean deflections. The Pr>F values from ANOVA are listed in Table 5.13 and Table 5.14

Based on Pr>F values, all main and interaction effects are significant for maximum deflections. This suggests that there are significant changes in the maximum deflections at different stages of construction. Also the maximum deflections vary from section to section.

The Student-Newman-Keuls(SNK) test was used to separate the mean of maximum deflections by construction stage. These results are shown in Table 5.15.

Table 5.11 Mean Deflections of the test sections on I-65

Section	BC (mils)	AC (mils)	OL0 (mils)	OL1 (mils)	OL2 (mils)
1	0.974	0.869	0.309	0.325	0.313
2	1.122	0.638	0.288	0.332	0.295
3	1.661	0.782	0.329	0.369	0.324
4	0.965		0.272	0.310	0.257
5	0.985		0.274	0.305	0.276
6	1.533		0.510	0.452	0.446
7	1.020		0.397	0.346	0.337
8	1.222		0.432	0.392	0.379
9	1.281	0.941	0.503	0.427	0.428
10	0.938	0.862	0.494	0.551	0.511
11	0.898		0.578	0.507	0.482

Note: BC = before cracking
AC = after cracking
OL0 = after overlay
OL1 = one year overlay
OL2 = two year overlay

Table 5.12 Mean Deflections of the test sections on US-31

Section	BC (mils)	AC (mils)	OL0 (mils)	OL1 (mils)	OL2 (mils)
1	0.695				
2	0.480				
3	0.527				
4	0.455		0.273	0.286	0.287
5	0.564		0.308	0.318	0.336
6	0.527		0.346	0.336	0.353
7	0.553		0.307	0.338	0.349
8	0.504		0.290	0.310	0.348
9	0.409		0.243	0.254	0.307
10	0.557		0.247	0.302	0.355
11	0.511	0.661	0.266	0.283	0.362
12	0.507	0.605	0.258	0.237	0.279
13	0.475	0.614	0.414	0.350	0.368
14	0.477	0.513	0.327	0.299	0.321
15	0.490	0.542	0.338	0.313	0.339
16	0.428	0.497	0.336	0.334	0.354
17	0.680		0.345	0.338	0.352
18	0.465	0.572	0.389	0.374	0.376
19	0.524	0.571	0.446	0.454	0.484
20	0.641	0.492	0.425	0.384	0.399
21	0.464	0.480	0.371	0.350	0.375
22	0.500	0.596	0.458	0.393	0.437

Note: same as those in Table 5.11

Table 5.13 ANOVA Results for Deflections on I-65 at Different Stages

Source of variation	df	SS	MS	F	Pr>F
T	10	12.82	1.28	7.09	0.0001
S	4	188.83	47.21	260.99	0.0001
T*S	34	23.25	0.68	3.78	0.0001

Table 5.14 ANOVA Results for Deflections on US-31 at Different Stages

Source of variation	df	SS	MS	F	Pr>F
T	21	6.54	0.31	26.15	0.0001
S	4	30.13	7.53	633.07	0.0001
T*S	67	9.36	0.14	11.74	0.0001

Table 5.15 SNK Grouping of Maximum Deflections at Different Stages of Construction

I-65			US-31		
SNK Grouping	Mean D_{max}	Construction stage	SNK Grouping	Mean D_{max}	Construction stage
A	1.151	BC	A	0.558	AC
B	0.819	AC	B	0.520	BC
C	0.399	OL0	C	0.357	OL2
C	0.393	OL1	C	0.352	OL1
C	0.363	OL2	D	0.337	OL0

Figure 5.7 and Figure 5.8 depict the mean deflections of the test sections on I-65 and US-31 at different periods. It can be concluded that overlays reduce the deflections dramatically on both I-65 and US-31. Based on the SNK Grouping, the maximum deflections of two year's overlay is not significantly different from those of new overlay on I-65. The maximum deflections after one and two years are significantly different from those of new

overlay on US-31. However, the maximum deflections after two years are only 5.9% higher than those of new overlay.

Figure 5.9 and Figure 5.10 show the deflections after overlay and percent decrease of deflection after overlay for cracked and seated sections on I-65. It is noted that the two thickest overlay sections (Section 4 and Section 5) have the smallest deflections after overlay and the highest percent decrease of deflections, compared with the rest of cracked and seated sections. There is no general trend of maximum deflection in section 6 to section 11 which have different fiber contents and fiber locations.

Figure 5.11 and Figure 5.12 show the deflections after overlay and percent decrease of deflection after overlay for cracked and seated sections on US-31. Based on these two figures, it is again illustrated that the two thickest overlays (Section 11 and section 12) have the smallest deflections after overlay and the highest percent decrease of deflections. It also appears that the fiber content and fiber location do not have significant effects on maximum deflections.

3. Summary of results

- (1) The overlay decreases the maximum deflections of cracked and seated test sections significantly, by 35.6% to 72.2% on I-65 and 13.6% to 59.8% on US-31. The sections with thicker overlay have higher percentages of deflection decrease.
- (2) The use of fiber and the fiber layer location do not have a significant effect on pavement strength in term of the maximum deflections.
- (3) There has been no significant strength decrease during the initial two year period.

Figure 5.7 Mean Deflections of I-65 Test Sections

Figure 5.8 Mean Deflections of US-31 Test Sections

Figure 5.9 Deflections on I-65(After Overlay)

Figure 5.10 Percent Decrease of Deflections on I-65 (Before Cracking vs. After Overlay)

Figure 5.11 Deflections on US-31(After Overlay)

Figure 5.12 Percent Decrease of Deflections on US-31 (Before Cracking vs. After Overlay)

5.4 Evaluation of Test Sections Based on Pavement Condition Survey

The lengths of transverse cracks for each test section were obtained from pavement condition surveys. These are summarized in Table 5.16 and Table 5.17.

Data from these two Tables are plotted in Figures 5.13 and 5.14, respectively. In general 8 in. or 10 in. overlays of cracked and seated sections (section 4 and 5 on I-65 and section 11 and 12 on US-31) have the least amount of cracking. As expected, the 4 in. overlays of cracked and seated sections (sections 19 through 22 on US-31) have the greatest amount of cracking. Obviously, overlay thickness has an important effect on the development of reflective cracks. For analysis of the effect of fiber layer location and cracking and seating technique on development of reflective cracks, four sets of data from Table 5.16 and Table 5.17 are plotted in Figure 5.15 through Figure 5.18.

Figure 5.15 illustrates the effects of cracking and seating on crack length which was surveyed in November, 1992. It is clear that all cracked and seated sections but one have less cracks than non cracked and seated sections, with other condition being the same. The 4 in. non fiberized overlay sections on non cracked and seated sections have the most reflective cracks. Also, development of reflective cracks in the 4 in. sections is more sensitive to cracking and seating technique.

Figure 5.16 shows the effects of fiber layer location on crack development which was surveyed in November, 1992. It can be observed that the least cracks were developed in the sections which have fiber in all three layers. The sections with fiber in both base and binder have less cracks than those with fiber only in the base. In general, the 4 in. sections have more cracking than the 5.5" sections. Compared with the 5.5" sections, the development of reflective cracks in the 4 in. sections is more sensitive to the fiber layer location.

Figure 5.17 and Figure 5.18 shows similar crack development data which was collected in May, 1993. The general trends are not as obvious as those shown in Figure 5.15 and Figure

5.16. This suggests that fiber as well as cracking and seating technique are significant in delaying reflective cracking development in first two year. It is not as effective after two years.

Table 5.16 The Length of Transverse Cracks on I-65

Test Section	Crack Length(ft.)		
	April, 92	Nov., 92	May, 93
1	48	84	84
2	8	8	8
3	36	60	72
4	0	12	36
5	10	10	15
6	24	24	108
7	12	12	48
8	0	0	12
9	0	0	0
10	39	48	144
11	48	48	120

Table 5.17 The Length of Transverse Cracks on US-31

Test Section	Crack Length(ft.)		
	April, 92	Nov., 92	May, 93
4	108	132	156
5	108	132	300
6	0	0	0
7	276	312	576
8	72	72	72
9	0	0	0
10	24	48	156
11	0	0	0
12	0	0	12
13	24	48	180
14	60	60	84
15	72	108	276
16	36	36	132
17	0	48	132
18	0	12	168
19	36	36	228
20	144	156	432
21	156	264	432
22	48	48	228

Figure 5.13 Length of Transverse Cracks on I-65

Figure 5.14 Length of Transverse Cracks on US-31

Figure 5.15 Effect of Cracking and Seating on Crack Length(Surveyed in November, 1992)

Figure 5.16 Effect of Fiber Location on Crack Length(Surveyed in November, 1992)

Figure 5.17 Effect of Cracking and Seating on Crack Length(Surveyed in May, 1993)

- Notes
- 1 -- Only in the base
 - 2 -- both in the base and binder
 - 3 -- in all three layer(base, binder, and surface)

Figure 5.18 Effect of Fiber Location on Crack Length(Surveyed in May, 1993)

5.5 Flexural Fatigue Test

1. Results

Results of fatigue tests are summarized in Tables 5.18 and 5.19. In these tables fatigue life is defined as the loading cycles when the beam stiffness reaches 50% of the initial stiffness [$N_f(50\%)$]. Similarly, $N_f(30\%)$ is the number of loading cycles when the beam stiffness reaches the 30% of the initial stiffness.

Table 5.18 Fatigue Test Results of Big Beams(3x3 in.)

Sample No.	Strain (10^{-3})	N_f (50%)	N_f (30%)	Sample No.	Strain (10^{-3})	N_f (50%)	N_f (30%)
Composite Layer(Surface+Binder)				Composite Layer(Binder+Base)			
Without Fiber				Without Fiber			
1	0.433	61014	104200	1	0.336	32420	88155
2	0.515	26845	32500	2	0.428	20370	40732
3	0.661	11714	19880	3	0.581	5850	11095
4	0.772	8400	9878	4	0.663	5908	8885
5	1.016	4703	7450	5	0.866	1537	2050
6	1.331	1035	1615	6	1.021	1361	2210
7	1.909	265	484	7	1.216	480	605
				8	1.332	395	585
With Fiber(4.5#)				With Fiber(4.5#)			
1	0.432	83040	144210	1	0.390	33760	73595
2	0.514	56807	104400	2	0.447	22480	122200
3	0.662	26835	48235	3	0.562		11980
4	0.780	9170	13450	4	0.705	4220	9239
5	0.848	4386	5660	5	1.016	1775	4122
6	1.010	4510	7960	6	1.363	760	1100
7	1.368	1550	2615				
8	1.908	506	770				

Table 5.19 Fatigue Test Results of Small Beams(1.5x1.5 in.)

Sample No.	Strain (10^{-3})	N_f (50%)	N_f (30%)	Sample No.	Strain (10^{-3})	N_f (50%)	N_f (30%)
Surface Layer				Binder Layer			
Without Fiber				Without Fiber			
1	0.448	120610		1	0.319	104200	172200
2	0.462	117600		2	0.438	35420	
3	0.525	67500	82500	3	0.477	26660	
4	0.601	24810		4	0.524	7200	7900
5	0.643	26600		5	0.604	2515	4275
6	0.742	9460	11380	6	0.661	4520	6220
7	0.806	6495	7665	7	0.703	2960	4200
8	0.917	4409	4894	8	0.769	850	1270
9	1.110	2460	3000	9	0.812	440	920
10	1.274	1580		10	0.930	2262	2680
11	1.280	972	1240	11	1.020	1180	1360
12	1.606	424	496	12	1.234	700	960
13	1.857	380					
14	2.282	246	326				
With Fiber(5.2#)				With Fiber(5.2#)			
1	0.373	234200	384200	1	0.330	152200	226200
2	0.467	34072	58200	2	0.451	14300	20300
3	0.588	28865	52845	3	0.462	17400	22600
4	0.622	8425	10615	4	0.580	12380	15040
5	0.739	18270	32270	5	0.677	6650	8450
6	0.876	5477	6717	6	0.788	2260	3240
7	0.968	3700	5080	7	0.929	250	380
8	1.251	610	890	8	1.222	1172	1560
9	2.074	102	166	9	1.277	370	640
				10	1.709	124	260

2. Statistical Analysis

The following model was assumed for analysis of effects of fiber on fatigue life

$$Y_{ij} = \mu + F_i + S_j + \varepsilon_{ij} \dots \dots \dots (5.4)$$

Where:

Y_{ij} = response variable, fatigue life in log scale

μ = overall mean

F_i = effect of fiber

S_j = effect of strain level in log scale

ε_{ij} = experimental error

A similar model was used for analysis of effect of layer on fatigue life, which is

$$Y_{ij} = \mu + L_i + S_j + \varepsilon_{ij} \dots \dots \dots (5.5)$$

Where: L_i = effect of layers

Other factors are the same as in equation 5.4.

The results of an ANOVA for Nf(50%) and Nf(30%) are presented in Table 5.20. A type I error $\alpha = 0.05$ is used for test of significance.

ANOVA results for fatigue test on large beam(3x3 in.) show that fiber has a significant effect on fatigue life of mixtures. ANOVA results for fatigue test on small beam(1.5x1.5 in.) show no significant fiber effects on fatigue life of mixtures. For both large beams and small beams, layer mixture has a significant effect on fatigue life. As further analysis of test results, data from Table 5.18 and Table 5.19 are plotted in Figure 5.19 through Figure 5.26. It can be seen that the variability of results of the flexural fatigue test is increased when small beams are used. Due to this large variability, a large number of small beams would have to be tested in order to detect a significant difference between mixtures with and without fiber. The number of samples to be tested would result in considerable increase of testing time. It is recommended that 3x3 in. beams be used for flexural fatigue tests in the future.

Results of fatigue tests indicate a linear relation between $\text{Log } \epsilon$ and $\text{log } N_f$. The least square regression equations were found and the regression lines are plotted in Figures 5.19 through 5.26 along with data points. For fatigue life N_f (30%), Figure 5.21 shows that the fatigue life of the surface layer without fiber is about 4 times that of binder layer without fiber. Figure 5.22 shows that the fatigue life of the surface layer with 5.2 lb. fiber is about 3 times that of binder layer with the same fiber content. For composite beams (surface+binder or binder+base), Figure 5.23 and Figure 5.24 indicate that composite samples with 4.5 lb. fiber have predicted fatigue life two times greater than those without fiber. Figure 5.25 and Figure 5.26 shows that surface+binder beams have a predicted fatigue life of over two times that of binder+base beams both in non fiberized sections and in 4.5 lb. sections.

Table 5.20 ANOVA Results for Fatigue Test

Size	Material	Comparison Group	Source of Variation			
			Fiber		Layer	
			Nf(50%)	Nf(30%)	Nf(50%)	Nf(30%)
Big Beams	Surface +Binder	without fiber vs. with fiber	S	S		
	Binder+Base	without fiber vs. with fiber	S	S		
		Surface+Binder vs. Binder+Base			S	S
Small Beams	Surface	without fiber vs. with fiber	S	NS		
	Binder	without fiber vs. with fiber	NS	NS		
		Surface vs. Binder			S	S

Notes: S = significant at $(1-\alpha)$

NS = not significant at $(1-\alpha)$

Figure 5.19 Fatigue Results of Surface Layer

Figure 5.20 Fatigue Results of Binder Layer

Figure 5.21 Comparison of Fatigue Results Between Surface and Binder (No Fiber)

Figure 5.22 Comparison of Fatigue Results Between Surface and Binder (5.2# Fiber)

Figure 5.23 Fatigue Results of Composite Layer(Surface+Binder)

Figure 5.24 Fatigue Results of Composite Layer(Binder+Base)

Figure 5.25 Comparison of Fatigue Results
Between Surface+Binder and Binder+Base (No Fiber)

Figure 5.26 Comparison of Fatigue Results
Between Surface+Binder and Binder+Base (4.5# Fiber)

5.6 Complex Modulus Test

1. Results

The results of complex modulus test are summarized in Tables 5.21 and 5.22.

Table 5.21 Complex Modulus Testing Data from I-65

Layer	Fiber Content (lb.)	Load Freq. (Hz)	Test Temperature					
			20°C		30°C		40°C	
			E (10 ⁶ psi)	Φ	E (10 ⁶ psi)	Φ	E (10 ⁶ psi)	Φ
Surface	0	1	0.622	24.1	0.336	27.4	0.195	24.7
		4	0.854	22.2	0.492	26.5	0.274	27.6
		8	0.967	22.1	0.575	27.6	0.328	29.2
	3.4	1	0.486	23.2	0.268	26.8	0.154	24.2
		4	0.642	20.0	0.394	25.3	0.227	28.1
		8	0.699	20.1	0.461	24.4	0.271	31.6
	5.2	1	0.373	20.9	0.242	25.0	0.139	24.0
		4	0.484	18.5	0.343	23.4	0.196	27.5
		8	0.539	20.5	0.395	23.2	0.229	29.4
	6.1	1	0.482	24.9	0.307	30.4	0.186	25.7
		4	0.657	21.9	0.466	29.1	0.282	31.0
		8	0.689	21.7	0.552	28.6	0.347	33.4
Binder	0	1	0.963	21.2	0.591	19.2	0.397	17.2
		4	1.268	19.8	0.794	20.8	0.514	21.6
		8	1.459	18.8	0.911	23.4	0.593	24.6
	3.4	1	0.691	21.0	0.364	25.9	0.222	21.3
		4	0.897	19.0	0.530	25.3	0.308	25.9
		8	1.010	19.6	0.624	25.3	0.371	28.5
	5.2	1	0.541	22.5	0.314	26.4	0.160	22.4
		4	0.716	20.2	0.460	25.9	0.228	27.3
		8	0.802	20.5	0.564	24.4	0.278	30.1
	6.1	1	0.594	22.9	0.373	24.3	0.217	17.1
		4	0.811	21.0	0.533	24.7	0.279	22.9
		8	0.863	21.4	0.634	26.9	0.331	25.9
Base	0	1	0.739	22.1	0.466	25.0	0.231	23.2
		4	0.984	19.4	0.670	24.6	0.336	27.2
		8	1.089	21.1	0.793	24.6	0.420	29.5
	3.4	1	0.551	23.7	0.296	26.8	0.151	21.3
		4	0.752	22.0	0.440	27.1	0.210	27.4
		8	0.835	22.1	0.540	28.3	0.261	32.8
	5.2	1	0.492	21.2	0.276	23.8	0.168	19.3
		4	0.638	19.4	0.393	24.6	0.221	23.9
		8	0.702	20.1	0.464	26.1	0.264	27.2
	6.1	1	0.548	21.5	0.301	24.1	0.182	18.7
		4	0.732	20.2	0.432	25.0	0.243	25.1
		8	0.814	20.1	0.522	25.5	0.296	28.4

Table 5.22 Results of Complex Modulus Test from US-31

Layer	Fiber Content (lb.)	Load Freq. (Hz)	Test Temperature					
			20°C		30°C		40°C	
			E (10 ⁶ psi)	Φ	E (10 ⁶ psi)	Φ	E (10 ⁶ psi)	Φ
Binder	0	1	0.608	22.3	0.413	24.7	0.193	22.1
		4	0.800	21.0	0.592	25.3	0.274	26.8
		8	0.903	21.6	0.697	25.3	0.340	29.8
	3.5	1	0.466	21.6	0.362	22.5	0.191	21.1
		4	0.607	19.9	0.486	21.9	0.263	25.1
		8	0.656	20.2	0.546	22.6	0.314	27.6
	4.5	1	0.521	20.9	0.317	23.3	0.193	19.4
		4	0.679	19.5	0.444	23.2	0.258	23.9
		8	0.751	20.1	0.516	23.9	0.311	26.5
	4.8	1	0.563	21.1	0.389	22.9	0.209	21.4
		4	0.745	19.6	0.542	22.2	0.288	25.1
		8	0.833	19.7	0.616	21.4	0.347	27.1
Base	0	1	0.562	24.7	0.277	27.6	0.174	18.0
		4	0.808	23.4	0.422	29.0	0.232	25.1
		8	0.952	23.6	0.529	29.8	0.284	29.5
	3.5	1	0.597	21.1	0.381	23.6	0.216	19.1
		4	0.779	20.2	0.534	24.1	0.289	23.4
		8	0.877	20.9	0.631	24.5	0.345	26.5
	4.5	1	0.458	21.5	0.340	22.4	0.215	18.5
		4	0.600	21.3	0.469	22.9	0.282	22.7
		8	0.666	21.1	0.540	23.7	0.332	25.1
	4.8	1	0.659	21.5	0.339	23.6	0.226	16.0
		4	0.891	21.4	0.488	25.3	0.289	21.6
		8	1.001	22.0	0.587	27.0	0.356	23.5

2. Statistical Analysis

The following linear model was assumed in statistical analysis.

$$Y_{ijklm} = \mu + C_i + L_j + CLF_{ij} + S_{k(ij)} + T_l + CT_{il} + LT_{jl} + CLT_{ijl} + ST_{kl(ij)} + F_m + CF_{im} + LF_{jm} + CLF_{ijm} + SF_{km(ij)} + TF_{lm} + CTF_{ilm} + LTF_{jlm} + CLTF_{ijlm} + STF_{klm(ij)} + \varepsilon_{n(ijklm)} \quad \dots(5.6)$$

Where: Y_{ijklm} = response variable, dynamic modulus E or phase angle Φ

μ = overall mean

C_i = effect of fiber contents.

L_j = effect of different layers

$S_{k(ij)}$ = effect of testing Samples

T_l = effect of temperatures

F_m = effect of frequencies

$\mathcal{E}_{n(ijklm)}$ = experimental error

Others = effect of interaction of main factors

An ANOVA was performed on the test results using SAS computer software. Results of the ANOVA are summarized in Tables 5.23 through 5.26. A type I error, $\alpha = 0.05$, was used for test of significance. ANOVA results for the complex modulus test on I-65 show that all main factors including fiber content, layer, temperature, and frequency have significant effects on dynamic modulus. Although fiber content had no significant effect on the phase angle, other main factors did have a significant effect.

ANOVA results for the complex modulus test on US-31 show that the main factor, fiber content, has no significant effect on the dynamic modulus but does have significant effect on phase angle. Layer does not have significant effect on either dynamic modulus or phase angle. Temperature, and frequency both have significant effect on both dynamic modulus and phase angle.

The ANOVA results for both I-65 and US-31 show inconsistencies for the significance of fiber content on both dynamic modulus and phase angle. The reason is not clear. But the results do indicate that the addition of fiber will change the viscoelastic properties of mixture.

The Student-Newman-Keuls test was used to separate the means of dynamic modulus and phase angle by actual fiber content. The mean values are listed in Table 5.27. Table 5.27 shows that dynamic modulus of fiberized mixtures on I-65 is lower than that of mixtures without fiber and there is no significant difference of the phase angle among the mixtures with and without fiber. SNK Grouping also shows that there is no difference of dynamic modulus among the mixtures with different fiber content. The phase angle of the mixtures on US-31 is smaller when fiber is used in the mixture. The dynamic modulus remains the same after the addition of fiber. The SNK Grouping also shows that there is no difference of phase angle among the mixtures with different fiber contents.

Table 5.23 ANOVA Results for Dynamic Modulus on I-65

Source of variation	df	SS (10 ¹⁰)	MS (10 ¹⁰)	F	Pr>F
C	3	318.66	106.222	13.51	0.0001
L	2	155.51	77.756	9.89	0.0007
LC	6	47.04	7.840	1.00	0.4496
S(CL)	24	188.64	7.860		
T	2	1252.13	626.065	429.36	0.0001
CT	6	50.31	8.385	5.75	0.0001
LT	4	27.21	6.803	4.67	0.0029
CLT	12	6.70	0.558	0.38	0.9632
ST(CL)	48	69.99	1.458		
F	2	265.62	132.810	957.25	0.0001
CF	6	10.34	1.723	12.42	0.0001
LF	4	3.58	0.895	6.45	0.0003
CLF	12	1.05	0.087	0.63	0.8030
SF(CL)	48	6.65	0.138		
TF	4	24.10	6.025	121.99	0.0001
CTF	12	2.29	0.191	3.88	0.0001
LTF	8	1.14	0.142	2.9	0.0062
CLTF	24	1.32	0.055	1.11	0.3444
STF(CL)	96	4.74	0.049		

Table 5.24 ANOVA Results for Phase Angle on I-65

Source of variation	df	SS (10 ¹⁰)	MS (10 ¹⁰)	F	Pr>F
C	3	92.00	30.669	1.30	0.2984
L	2	332.48	166.244	7.03	0.0040
LC	6	447.71	74.618	3.15	0.0200
S(CL)	24	567.72	23.655		
T	2	1416.71	708.356	86.79	0.0001
CT	6	71.17	11.862	1.45	0.2144
LT	4	105.39	26.349	3.23	0.0200
CLT	12	143.65	11.970	1.47	0.1702
ST(CL)	48	391.75	8.161		
F	2	228.25	114.127	51.56	0.0001
CF	6	5.70	0.951	0.43	0.8555
LF	4	40.91	10.228	4.62	0.0031
CLF	12	24.08	2.007	0.91	0.5471
SF(CL)	48	106.25	2.213		
TF	4	934.91	233.728	192.28	0.0001
CTF	12	35.94	2.995	2.46	0.0075
LTF	8	9.63	1.204	0.99	0.4480
CLTF	24	39.99	1.666	1.37	0.1430
STF(CL)	96	116.69	1.215		

Table 5.25 ANOVA Results for Dynamic Modulus on US-31

Source of variation	df	SS (10^{10})	MS (10^{10})	F	Pr>F
C	3	20.83	6.945	0.90	0.4608
L	1	0.66	0.662	0.09	0.7728
LC	3	16.15	5.384	0.7	0.5651
S(CL)	16	122.89	7.680		
T	2	697.83	348.916	207.89	0.0001
CT	6	21.27	3.545	2.11	0.0792
LT	2	7.67	3.838	2.29	0.1179
CLT	6	15.01	2.502	1.49	0.2127
ST(CL)	32	53.70	1.678		
F	2	163.71	81.858	821.70	0.0001
CF	6	2.56	0.428	4.30	0.0028
LF	2	0.39	0.199	2.01	0.1510
CLF	6	0.47	0.079	0.80	0.5804
SF(CL)	32	3.18	0.099		
TF	4	15.31	3.829	129.70	0.0001
CTF	12	1.33	0.111	3.78	0.0002
LTF	4	0.75	0.188	6.39	0.0002
CLTF	12	0.72	0.060	2.05	0.0336
STF(CL)	64	1.88	0.029		

Table 5.26 ANOVA Results for Phase Angle on US-31

Source of variation	df	SS (10^{10})	MS (10^{10})	F	Pr>F
C	3	279.47	93.159	10.81	0.0004
L	1	6.73	6.731	0.78	0.3899
LC	3	17.03	5.678	0.66	0.5891
S(CL)	16	137.87	8.617		
T	2	358.69	179.348	70.30	0.0001
CT	6	38.04	6.341	2.49	0.0435
LT	2	186.65	93.329	36.58	0.0001
CLT	6	63.63	10.605	4.16	0.0034
ST(CL)	32	81.63	2.551		
F	2	240.79	120.397	198.81	0.0001
CF	6	8.71	1.453	2.40	0.0500
LF	2	19.08	9.544	15.76	0.0001
CLF	6	9.13	1.522	2.51	0.0417
SF(CL)	32	19.37	0.605		
TF	4	472.16	118.041	222.03	0.0001
CTF	12	13.29	1.108	2.08	0.0305
LTF	4	2.21	0.554	1.04	0.3926
CLTF	12	11.52	0.960	1.81	0.0659
STF(CL)	64	34.02	0.531		

Table 5.27 Mean of Dynamic Modulus and Phase Angle

	Actual Fiber Content (lb.)	Dynamic Modulus (10 ⁵ psi)		Phase Angle	
		SNK Grouping	Means	SNK Grouping	Means
I-65	0.0	A	6.615	A	23.5
	3.4	B	4.687	A	24.7
		B		A	
	5.2	B	3.934	A	23.6
		B		A	
	6.1	B	4.694	A	24.5
US-31	0.0	A	5.033	A	25.0
		A			
	3.5	A	4.746	B	22.5
		A		B	
	4.5	A	4.384	B	22.2
		A		B	
	4.8	A	5.203	B	22.3

Since the sections on US-31 with target fiber content 5 lb. and 7.5 lb. have very close actual fiber contents, 4.8 lb. and 4.5 lb. respectively, these sections can be considered as having an average fiber content of 4.6 lb.. The SNK test was conducted on the regrouped data. The results are listed in Table 5.28. Table 5.28 shows that same conclusions can be derived as those from Table 5.27.

Table 5.29 and 5.30 summarize the effects of temperature and frequency on overall means of dynamic modulus and phase angle. It is clear that there is a significant decrease in dynamic modulus when temperature increases. There is a significant increase of phase angle both for I-65 and US-31 when temperature changes from 20°C to 30°C, but there is no significant change of phase angle for I-65 when temperature changes from 30°C to 40°C. For US-31 mean values of phase angle are not much different numerically when temperature changes from 30°C to 40°C, but SNK grouping shows a significant decrease. Higher loading frequencies result in both higher dynamic modulus and higher phase angle.

Figures 5.27 through 5.31 show the effects of fiber content, loading frequency and temperature on dynamic modulus and phase angle.

Table 5.28 Mean of Dynamic Modulus and Phase Angle of Regrouped Data on US-31

	Actual Fiber Content (lb.)	Dynamic Modulus (10^5 psi)		Phase Angle	
		SNK Grouping	Means	SNK Grouping	Means
US-31	0.0	A	5.033	A	25.0
	3.5	A	4.746	B	22.5
		A			
		A			
	4.6	A	4.794	B	22.3

Table 5.29 Effect of Temperature on Overall Means of Dynamic Modulus and Phase Angle

	Temperature (°C)	Dynamic Modulus (10^5 psi)		Phase Angle	
		SNK Grouping	Means	SNK Grouping	Means
I-65	20	A	7.500	A	21.1
	30	B	4.754	B	25.4
	40	C	2.698	B	25.7
US-31	20	A	7.076	A	21.2
	30	B	4.774	B	24.3
	40	C	2.675	C	23.5

Table 5.30 Effect of Frequency on Overall Means of Dynamic Modulus and Phase Angle

	Frequency (Hz)	Dynamic Modulus (10^5 psi)		Phase Angle	
		SNK Grouping	Means	SNK Grouping	Means
I-65	1	A	3.784	A	23.2
	4	B	5.194	B	23.9
	8	C	5.971	C	25.2
US-31	1	A	3.695	A	21.7
	4	B	5.025	B	23.1
	8	C	5.804	C	24.3

Figure 5.27 Effects of Loading Frequency and Fiber Content on Dynamic Modulus(I-65)

Figure 5.28 Effects of Loading Frequency and Fiber Content on Dynamic Modulus(US-31)

Figure 5.29 Effects of Loading Frequency and Temperature on Dynamic Modulus

Figure 5.29 Effects of Loading Frequency and Temperature on Dynamic Modulus(Continued)

Figure 5.30 Effects of Loading Frequency and Fiber Content on Phase Angle(I-65)

Figure 5.31 Effects of Loading Frequency and Fiber Content on Phase Angle(US-31)

CHAPTER 6

SUMMARY AND CONCLUSIONS

Cracking and seating technique with the addition of fibers to the overlay mixture is used successfully in some states to control reflective cracking. There are many factors affecting the performance of overlays on cracked and seated concrete pavements. The main factors which are investigated in this study include: number of the seating roller pass, weight of seating roller, thickness of overlay, fiber content and fiber layer location.

Eleven test sections were constructed in the southbound lanes on I-65 near Crown Point, Indiana and twenty two test sections were constructed in the northbound lanes on US-31 near Plymouth, Indiana. An extensive investigation was conducted involving both field and laboratory testing. Field investigations included pavement condition survey and non destructive testing (NDT) of the pavements using a FWD or Dynaflect. The laboratory investigation focused on physical property testing, complex modulus testing and fatigue testing. Major conclusions as well as recommendations from this study are summarized as follows:

1. Due to lack of a standard method for fiber extraction, three fiber extraction methods (Method A, B, and C) have been developed for this project. Preliminary tests were conducted to evaluate the precision of Method B and Method C for fiber extraction. Results show that over 95% of fiber can be extracted from paving mixtures using these tests. The precision of both methods is good. These fiber extraction methods should be used in future projects.
2. Fiber extraction results show that fiber contents in test sections were different from target fiber contents by a significant amount. The differences between actual fiber contents and target fiber contents affect the breadth of the study and reflect the need for improved techniques of introducing fiber into the mixture.

3. The significant difference in maximum deflections before and after cracking indicates that the Guillotine breaker is effective in breaking PCC pavement. Deflection measurements can be used to determine the appropriate weight and drop height of the breaking head.
4. A 40 ton roller is recommended because the 50 ton roller appears to "overwork" the cracked concrete pavement.
5. Roller passes have no significant effect on the maximum deflections. The optimum roller pass can not be determined exclusively based on the maximum deflections. When seating is needed to assure contact of cracked pavement pieces with the subgrade, one to three passes of a 40 ton pneumatic-tired roller is adequate.
6. Overlay decreases the maximum deflections of test sections significantly. Thicker overlays decrease the deflection by a greater amount.
7. Use of fiber and the fiber layer location do not have a significant effect on pavement strength in term of the maximum deflection.
8. Overlay thickness has an important effect on the development of reflective cracks. The 4 in. overlay of cracked and seated sections had the greatest amount of cracking and the 8 to 10 inch overlay the least. Although the 8 to 10 inch overlay indicates some benefit in retarding reflective cracking, an alternative overlay option such as rubblizing with a thinner overlay may be more cost effective.
9. In the first two years, cracked and seated sections have less cracks than non cracked and seated sections, with other condition being same. The 4 in. non fiberized overlay sections on non cracked and seated sections have the most reflective cracks. Also, development of reflective cracks in 4 in. sections is more sensitive to cracking and seating technique.

10. In the first two years, the least cracks were developed in the sections which have fiber in all three layers. The sections with fiber in both base and binder have less cracks than those with fiber only in the base. Compared with the 5.5" sections, the development of reflective cracks in 4 in. sections is more sensitive to the fiber layer location. It is recommended that fiber be used in all overlay layers.
11. After two years, the general trends of crack development in the overlays are not obvious. This suggests that fiber as well as cracking and seating technique are significant in delaying reflective cracking development in the first two years. It is not as effective after two years.
12. Compared with 3x3 in. beams, 1.5x1.5 in. beams have larger variability of flexural fatigue test results. Due to this large variability, a large number of small beams would have to be tested in order to detect a significant difference between mixtures with and without fiber. The number of samples to be tested would result in considerable increase of testing time. It is recommended that 3x3 in. beams be used for flexural fatigue tests in the future.
13. Results of fatigue tests indicate that fiber increases the fatigue life of overlay mixture. Composite samples with fiber(actual fiber content 4.5 lb.) have predicted fatigue life two times greater than those without fiber.
14. Loading frequency and temperature have significant effects on dynamic modulus and phase angle of overlay mixture with and without fiber. The complex modulus test on I-65 show that fiber has significant effect on dynamic modulus and no significant effect on the phase angle. Dynamic modulus of mixture on I-65 is decreased when fiber is used in the mixture. The complex modulus test on US-31 show that fiber has no significant effect on the dynamic modulus but has significant effect on phase angle. The phase angle of the mixtures on US-31 is smaller when fiber is used in the mixture. Although the results for both I-65 and US-31 show inconsistencies for the significance of fiber

content on both dynamic modulus and phase angle, the results do indicate that the addition of fiber will change the viscoelastic properties of mixture. A further study should be conducted to determine the viscoelastic properties of fiberized mixture.

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