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16. Abstract

This report presents the research undertaken within two areas of study of thin asphalt concrete (AC) overlays to rehabilitate continuously reinforced concrete pavements (CRCP). The first one is the development of a decision tree for the project selection of a rehabilitation of this kind. The second area is the testing of tack coats and AC mixtures for its use in AC overlays.

Within the first area, several decision criteria, based on the functional and structural characteristics of the existing pavement, were developed to evaluate the technical suitability of AC overlays on CRCP. These criteria are, namely, a profile criterion, based on the remaining life concept and on the formulation of a dynamic load factor; a condition survey criterion, which utilizes a pavement distress index and the rate of failures per mile per year; and a deflection criterion, which evaluates the hypothetical contribution of an AC overlay based upon deflection measurements performed on the existing structure.

The second part of the study investigated the interface shear strength of tack coats used for bonding AC and portland cement concrete pavement (PCCP), and the rutting resistance of asphalt mixtures for use as overlays on CRCP. A shear test was developed for the tack coats experiment, and it was found that mixes with finer gradations appear to improve the shear strengths of tack coat interfaces. Based on the results of the rutting resistance tests, it is recommended to use Superpave, CMHB, and Type C mixes for AC overlays on CRCP, and to avoid using siliceous river gravel aggregates in these mixes, because of its inferior performance.

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# ASPHALT CONCRETE OVERLAYS ON CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS: DECISION CRITERIA, TACK COAT EVALUATION, AND ASPHALT CONCRETE MIXTURE EVALUATION


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Research Report Number 0-4398-3

Research Project 0-4398

Develop Guidelines for Designing and Constructing Thin Asphalt Concrete Pavement Overlays on Continuous Reinforcement Concrete Pavements

conducted for the

**Texas Department of Transportation**

by the

**CENTER FOR TRANSPORTATION RESEARCH**

THE UNIVERSITY OF TEXAS AT AUSTIN

April 2004

Prepared in cooperation with the Texas Department of Transportation and the Federal Highway Administration.

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## PRODUCTS

This report contains products P3 and P4, according to the Deliverables Table of Project 0-4398. Product P3, Decision Tree for ACP Overlay, is presented throughout Chapter 2, and summarized in pages 17 through 19. Product P4, Recommendations for New Specifications, is presented in Chapter 8, on pages 127 through 130.

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## SUMMARY

This report presents the research undertaken within two areas of study of thin asphalt concrete (AC) overlays to rehabilitate continuously reinforced concrete pavements (CRCP). The first one is the development of a decision tree for the project selection of a rehabilitation of this kind. The second area was the testing of tack coats and AC mixtures for its use in AC overlays.

The decision tree consists of a series of steps and criteria arranged in a systematic way to help the pavement engineer determine whether this type of rehabilitation strategy is the optimal solution for the problem at hand. If an AC overlay is not the ideal solution, the application of the decision tree will indicate if other options are more suitable, such as a bonded concrete overlay (BCO) or an unbonded concrete overlay. Three decision criteria were developed, which are based on the functional and structural characteristics of the existing pavement. The profile criterion attends to the functional condition of the CRCP, and is deemed as a primary indicator of whether a pavement needs an overlay rehabilitation, while the other two criteria, the condition survey criterion and the deflections criterion, consider the structural properties of the pavement system, and indicate what type of overlay is more adequate.

The second part of the study investigated the interface shear strength of tack coats and the rutting resistance of asphalt mixtures for use as overlays on CRCP

The shear strength performance of tack coats utilized to bond AC and portland cement concrete pavement (PCCP) specimens was evaluated using a shear test developed as part of the study. Four influence factors were considered as part of the experiment including mix type, tack coat type, tack coat application rate, and Hamburg wheel tracking. Both the MMLS3 and the Hamburg wheel were used to test AC mixtures for rutting resistance.

# CHAPTER 1. INTRODUCTION

This report, the third one pertaining to Project 0-4398, presents a decision tree for the project selection for a thin AC overlay on CRCP. The work shown in this report represents the continuation of the investigation presented in the first and second reports of this series 4398-1, “Applicability of Asphalt Concrete Overlays on Continuously Reinforced Concrete Pavements,” and 4398-2, “Techniques and Procedures for Bonded Concrete Overlays.”

This introductory chapter covers the pertinent background leading to this report, as well as the objectives and scope of the project and this report.

## BACKGROUND

Thin-bonded asphalt concrete (AC) overlays placed on existing portland cement concrete pavements (PCCP) have demonstrated their value as a cost-effective means for restoring the riding quality and extending the service life of deteriorated pavements. The Texas Department of Transportation (TxDOT) has been using this technique for 40 years. Even though it is acknowledged that a thin AC overlay is not applicable in all situations, the decisions regarding the utilization of this kind of rehabilitation have been mostly based on experience, because of a lack of established procedures for its implementation. The need to develop criteria and procedures to ensure that a thin AC overlay is implemented under ideal conditions for its success prompted TxDOT to develop Project 0-4398 “Develop Guidelines for Designing and Constructing Thin Asphalt Concrete Pavement (ACP) Overlays on Continuous Reinforcement Concrete Pavement (CRCP).”

The primary benefits of applying a thin AC overlay rehabilitation on a continuously reinforced concrete pavement (CRCP) are:

a) restoration of the riding quality

b) reduction of dynamic impact loading
c) increase in CRCP service life by delaying its deterioration
d) reduction of moisture intrusion into the pavement structure, performing as a moisture barrier, thus, preserving the structural integrity of the subgrade
e) decrease in noise levels generated by traffic on a tined CRCP

The limitations of this type of rehabilitation are:

a) The thin AC overlay does not add structural capacity to the existing pavement, therefore, the CRCP has to be structurally sound
b) Unrepaired CRCP distresses may reflect through the AC overlay

The decision tree is a tool that facilitates the decision process when facing a rehabilitation problem, by providing a series of steps and criteria conducive to finding the best rehabilitation alternative given the pavement conditions. The decision tree presented includes the decision of whether to conduct a rehabilitation with an overlay, the decision whether to use an AC overlay or a PCCP overlay, and, if a PCCP overlay is chosen, the decision whether to use an unbonded concrete overlay or a bonded concrete overlay (BCO). The first report, 4398-1, entitled "Applicability of Asphalt Concrete Overlays on Continuously Reinforced Concrete Pavements," presented the conditions for ideal application of such rehabilitation, along with the results of the first stage of the investigation. The second report, 4398-2 [Trevino 2004], entitled "Techniques and Procedures for Bonded Concrete Overlays," establishes the appropriateness of a BCO as a pavement rehabilitation strategy, presenting guidelines for project selection, design, construction, and quality control and quality assurance (QC/QA) resulting from years of experience in numerous BCO projects.

As stated previously, this report is the continuation of Report 4398-1. For the reader's convenience, the contents of the first report are summarized in the following paragraphs.

The first part of the 4398-1 report included a literature review, followed by a discussion of the most common modes of failure experienced by AC overlays. The Texas experience with AC overlays on CRCP was first studied by means of interviews and surveys with the district pavement engineers, and by means of condition surveys conducted on a few pavement sections. The approach for outlining a decision tree was also introduced.

The literature review was conducted focusing on two aspects: the condition of the existing CRCP that warrant a successful implementation of an AC overlay, and the asphalt characteristics appropriate for such rehabilitation. The primary findings of the literature review are as follows:

- A thin AC overlay does not increase the structural capacity.
- Bonding between layers is key.
- Milling and tack coat are the most important factors regarding bond.
- Repetitive thin overlays are cost-effective.
- High moisture content of the mix increases the probability of debonding and stripping.
- Using proper materials is important to control rutting.

The most frequent mode of failure for an AC overlay is delamination. This, and other modes of failure such as stripping, rutting, reflective cracking, and slippage cracking were discussed.

A series of interviews were conducted by CTR personnel with the district pavement engineers to gather information on their experiences using this kind of overlays in their respective districts. This investigation was complemented with diagnostic field studies on selected sections where AC overlay treatments on CRCP have yielded both good and bad performances. These condition surveys covered selected CRCP sections in three different districts: Yoakum, Bryan, and Atlanta.

Finally, the first report concluded with the introduction of the decision tree, and a brief discussion of the three criteria that constitutes the foundation of the decision making process. As the project progressed, the research team modified and refined the decision tree to reflect the findings of the investigation. Therefore, the version of the decision tree that appears in the 4398-1 report is slightly different from its final version, which is presented in this report.

## PROJECT OBJECTIVE

The primary objective of this study is to maximize the performance of CRCP with a thin AC overlay. This objective leads to the following sub-objectives:

1. To evaluate the causes for the AC overlays premature failures and mitigate their occurrences.
2. To study the field performance of thin AC overlays on CRCP
3. To summarize the best practices for the utilization of thin AC overlays on CRCP
4. To provide recommendations to prevent the debonding phenomenon
5. To provide recommendations on tack coat performance testing and rutting resistance for thin AC overlays

## SCOPE

This project studies the applicability, design and performance of thin AC overlays placed on CRCP. The decision tree presented in this report was developed on the basis of more than 40 years of TxDOT's experience with this type of rehabilitation. The cases studied are limited to projects in the state of Texas, developed by TxDOT. The decision tree includes PCCP overlays as well, to present the entire range of overlay rehabilitation alternatives. The second report of this series focuses on BCOs.

## REPORT OBJECTIVES

This third report of Research Project 4398 presents the following objectives:

a) The decision tree for thin AC overlays on CRCP

b) The development of the criteria for project selection for an AC overlay

c) The results of the AC mix factorial investigation of tack coat performance and wheel tracking tests.

## REPORT ORGANIZATION

The report is organized as follows:

The introduction to the project's research statement, objectives and scope are presented in Chapter 1. The decision tree is presented in Chapter 2, featuring the conditions of the existing CRCP that are suitable for an AC overlay, as well as the asphalt characteristics for an AC overlay on CRCP. The decision criteria are introduced; the subsequent chapters are dedicated to discussing the development and application of each of the criteria.

Chapter 3 covers the profile criterion, which is analyzed as the decisive factor to ascertain the implementation of an overlay.

In Chapter 4, the condition survey criterion is presented, including its two components: the pavement distress index, and the rate of failures occurrence.

Chapter 5 presents the third criterion, the deflection criterion, which is the basis to the determination of the structural soundness of the existing CRCP.

Chapter 6 discusses the AC overlay design considerations, in particular tack coats interface shear strength and permanent deformation.

A discussion of the results is presented in Chapter 7, including the possible implementation issues.

Finally, Chapter 8 summarizes this report with conclusions and recommendations for future research.

# CHAPTER 2. DECISION TREE

When a pavement experiences failures and is in need of repair, it may need a major rehabilitation. The first step followed by pavement engineers is to select the best type of rehabilitation, according to the type of failures, the origin of the problem and the availability of resources to conduct such repairs. This is conducted during the project selection stage. This chapter presents the decision tree for thin bonded asphalt concrete (AC) overlays placed on existing portland cement concrete pavements (PCCP), more specifically, on continuously reinforced concrete pavements (CRCP). The decision tree is intended as a tool to facilitate the project selection stage, by providing a systematic series of steps to aid in the decision making process. These steps include decision criteria, which involve tests, to determine whether an AC overlay is suitable for a particular rehabilitation situation.

Because the scope of this project was broadened to include PCCP overlays, the decision tree was expanded as well to consider this type of rehabilitation. The main focus of the project, however, remained the rehabilitation with AC overlays on CRCP. The usage of BCOs was discussed extensively in Report 4398-2 [Trevino 2003a], in which a decision tree shows the steps involved in the selection of BCOs and unbonded concrete overlays. The subsequent chapters are dedicated to the discussion of the decision criteria for AC overlays on CRCP.

The first stage of a pavement rehabilitation is project selection, in which several options are evaluated to determine the most suitable alternative for the project characteristics. Subsequent stages are design, construction, and quality control and quality assurance (QC/QA). These stages are illustrated in Figure 2.1, where project selection has been highlighted, because the decision tree is a fundamental component of this stage.

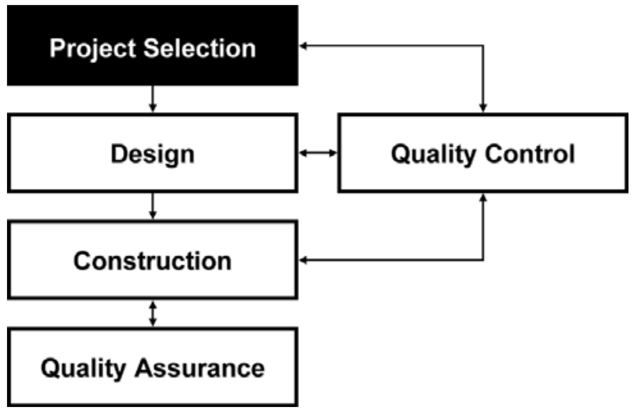



*Figure 2.1 Project selection as part of the rehabilitation process*

In the following sections, the different components of the decision tree are discussed. The trigger for the process is the need for rehabilitation, and then a rehabilitation decision is made, based on the evaluations conducted on the structure. The profile criterion is a key element in making that decision. Once it is determined that an overlay is necessary, the next step is determining what type of overlay will be utilized, based on the conditions of the existing pavement. To make this assessment, the decision criteria are evaluated.

**TRIGGER**

The need for rehabilitation in a pavement manifests itself as a decline of the pavement serviceability level.

The three major factors that influence the loss of serviceability of a pavement structure are traffic, time and environment. These factors interact to trigger the need for the pavement rehabilitation. Sometimes it may be a single element, however, most of the time it is their interaction that causes the need for rehabilitation. The effects of these factors can be categorized as loads, age and traffic increases.

### *Loads*

Every pavement is subjected to loads, which cause damage. Loads occur in a pavement even before it has opened to traffic, as a result of the environment and restraint inherent to the position of the pavement relative to other elements, such as the underlying substrate, adjacent structures and its own reinforcement. The environment causes contraction and expansion of the materials that compose the pavement; however, in most of the cases, they contract or expand at different rates because of their different thermal properties. The environment also makes the materials lose or gain moisture, which in turn causes changes in their composition and volume. These changes are known as environmental loads. Once the pavement opens to vehicle circulation, it is subjected to traffic loads. The effects of loads add up as the pavement ages. As a consequence of normal and excessive loads, cumulative traffic and environmental effects, pavements experience damage, which accumulated effects translate into failure.

Therefore, it is a fact that at some stage of its life, the pavement will show the effects of damage in the form of distresses. A certain level of distresses will be the criterion to determine that the pavement has reached a condition of failure.

### *Age*

Pavements are designed to last for a planned period of time, which is determined by the design life. Pavement structures are typically designed for periods ranging from 20 years to 40 years. Based upon traffic estimates for the requirements of the facility, the pavement thickness is determined. Thus, as the facility's service life comes to an end, it is expected that the amount of traffic loads imposed onto the structure will be similar to the number of load applications the pavement was originally designed to withstand. On the other hand, it is known that the properties of materials that constitute pavements change with time, changes that may be beneficial to performance; however, in most cases, the overall influence of age is detrimental to pavement serviceability.

### *Traffic Increases*

Oftentimes, the predicted amount of traffic during the design stage is surpassed well in advance of the end of the pavement design life. An obvious reason for this kind of discrepancies is the inherent difficulty of the traffic prediction task. Also, with growth in population and land development, the usage of the road in question may change from its original intended purpose to satisfy more ambitious transportation goals, becoming a more transited road, perhaps connecting to new highways or becoming part of an important corridor or network in a way that was impossible to predict at the time of design.

## DECISION TO REHABILITATE

When a pavement structure approaches the end of its intended service life, or experiences certain degree of deterioration, or it is anticipated that there will be an increase in traffic, rarely the solution to this problem is to tear it apart and to build a new facility. Rehabilitation of the pavement, in most cases, is the best choice of the available alternatives.

Pavement engineers will seek for ways to preserve the integrity of the roadway by means of rehabilitation before considering building a new structure, because rehabilitation means utilizing the existing structure to its fullest possible extent, therefore, making better use of the existing infrastructure and optimizing the use of the resources. In summary, it is the best economical solution, unless the structure is in an extremely deteriorated condition.

Since the success of the rehabilitation is dependent upon economical as well as technical considerations, at this point, the agency must decide whether to embark on a rehabilitation project based on the availability of funds for such an endeavor.

## TYPE OF REHABILITATION

The solution as to how to approach the rehabilitation is not unique. An AC overlay is just one of several rehabilitation alternatives, and it is only applicable under certain conditions. If the conditions are not met, the AC overlay may perform poorly and may not fulfill the purpose of its implementation. Thus, an AC overlay is an optimal solution only in certain cases. The decision tree provides the steps to evaluate all the available alternatives and select the one that will maximize performance.

### *Overlay versus Non-Overlay*

Once the availability of resources has been established, the next decision that the designer faces is whether to use an overlay or to use rehabilitation methods other than overlay. A feasible alternative is one that addresses the cause of the problem motivating the rehabilitation; therefore, the pavement condition must be investigated before making the decision. The reason for the rehabilitation need may be the structural or the functional condition of the pavement. Structural condition refers to whether or not the pavement is capable of supporting current and future traffic loads over the desired design period. The functional condition encompasses those pavement characteristics related to the way the road serves the user in terms of safety and comfort, such as skid resistance, roughness, appearance and hydroplaning.

The evaluation of the structural condition involves studying the distress patterns of the pavement, which will provide information on the impact of past traffic loadings. This is assessed by means of a visual condition survey. The visual inspection is normally conducted by personnel with training in distress type identification and with experience on their causative mechanisms. Photographic equipment and audio tape recorders can be advantageously utilized in recording and extracting the data. Historical information on patching, slab replacement and other repairs are other valuable sources for structural condition assessment. Finally, destructive and non-destructive testing (NDT) methods are extremely helpful in determining the structural integrity of the pavement. Among the NDT procedures, the most common is deflection testing. Destructive testing implies the extraction of samples from the pavement for their laboratory evaluation. The evaluation of the functional condition requires the measurement of roughness and skid resistance and an assessment of the present serviceability. The roughness measurement is conducted with an inertial profiler, profilograph or straight edge.

A key element to consider is that an overlay can provide structural improvements that are not achievable by non-overlay methods. Non-overlay methods can only correct functional deficiencies; hence, only structurally sound pavements are candidates for rehabilitation without overlay [AASHTO 1993].

There are numerous non-overlay methods available; their applicability depends on the condition attempted to remedy. Most of them can be used in conjunction with each other or with other techniques. In fact, some of these might be utilized as part of the pavement repairs prior to the placement of an AC overlay. For instance, if the CRCP just shows surface spalling, then minor partial-depth repairs may be needed, and if the CRCP exhibits punchouts, more expensive full-depth repairs may be necessary, prior to the placement of the overlay. A discussion of non-overlay methods is beyond the scope of this study.

***Type of Overlay***

Once an overlay has been decided over non-overlay methods, depending on the evaluation of the pavement condition, the next resolution involves the type of overlay to apply.

***PCC versus AC Overlays***

In general, overlays can be classified as AC or PCCP overlays. AC overlays are also known as flexible and PCCP are also referred as rigid overlays. Both types are applicable to CRCP.

The decision as whether to utilize an AC overlay or a PCCP overlay depends on the pavement condition as well as economic considerations. Some of the factors to take into account when deciding upon the overlay type are:

- Thin AC overlays are not able to remedy structural deficiencies
- AC overlays represent a smaller initial investment
- PCCP overlays, in general, will last longer and require less maintenance, but their initial cost is higher
- Considering life-cycle costs, PCCP overlays may be more cost-effective
- AC overlays may be placed as an interim rehabilitation procedure, anticipating the placement in the near future of an unbonded concrete overlay

Thus, conducting a life-cycle cost analysis is advisable in deciding between AC and PCCP overlays.

### *Bonded versus Unbonded PCC Overlays*

PCCP overlays over CRCP may be bonded or unbonded. The decision to use either type depends on the type of failures present in the existing pavement. An in-depth analysis of the choice between bonded and unbonded PCC overlays is presented in Report 4398-2 [Trevino 2004].

## DECISION CRITERIA

To enable the selection of the most suitable type of rehabilitation for each case, three criteria have been developed. These criteria analyze the current pavement condition, and depending on the results, indicate whether a thin AC overlay is adequate for the case in question. The decision criteria are the profile criterion, the condition survey criterion, and the deflection criterion.

### STRUCTURAL FAILURE VERSUS FUNCTIONAL FAILURE

The types of failure are related to the structural and functional conditions of the pavement. A CRCP structural failure occurs when a pavement reaches an established unacceptable level of distress, such as spalling or punchouts. Since the main characteristics of functionality in a pavement are safety and comfort for the user, a functional failure refers to that stage in which the pavement has become unsafe or uncomfortable. In terms of serviceability, using the present serviceability index (PSI), Figure 2.2 conceptually shows the typical applicability of thin AC overlays and BCOs relative to the serviceability stage of the pavement, where $P_0$ and $P_t$ are the initial and terminal serviceability, respectively. A thin AC overlay is applicable when the serviceability is still relatively high, whereas a BCO is more suitable when the serviceability has dropped to a lower level, but before the pavement reaches its terminal serviceability. Note that there is an overlap in the ranges of application of thin AC overlays and BCOs.

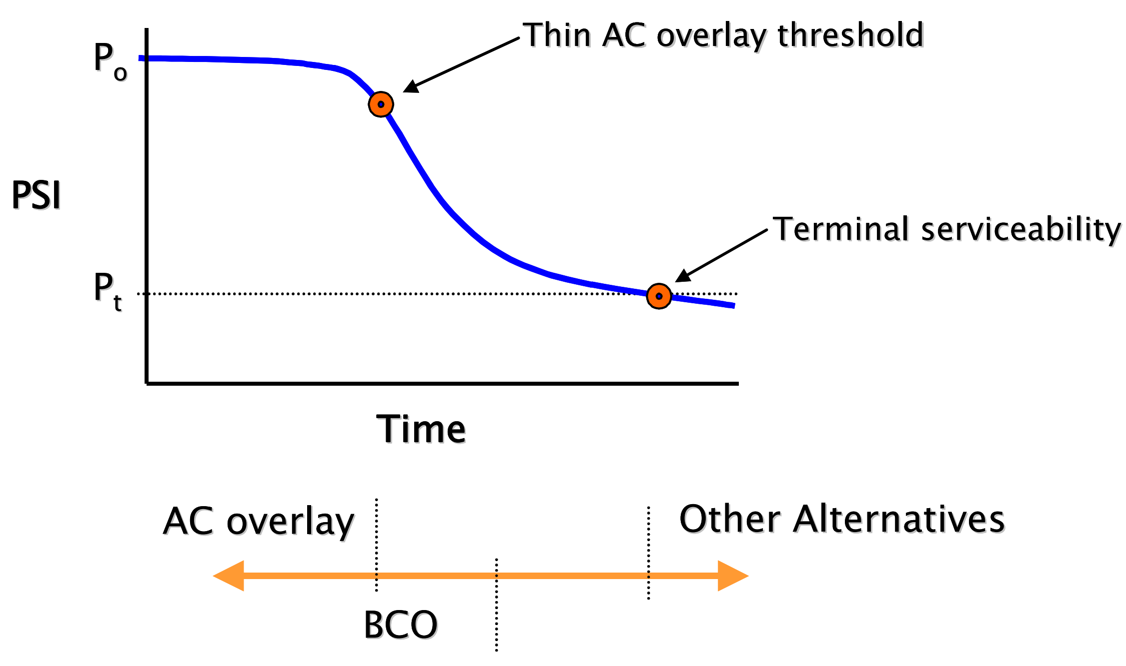



*Figure 2.2 Applicability of rehabilitation alternatives relative to PSI*

### PROFILE CRITERION

Riding quality is an indicator of the pavement roughness. Since pavement roughness is the key term in the PSI equation, the PSI, measured with a profilometer, may be used as a riding quality term. If the riding quality of the road surface is very good, (i.e., the PSI is close to 4.5), an overlay may not be necessary. On the other hand, a pavement with very poor riding quality (i.e., PSI equal or less than 2.5) in all likelihood has also experienced structural failure and therefore, it is not a good candidate for a BCO or an AC overlay rehabilitation. There are cases in which the pavement is still structurally sound even though its riding quality is low. This roughness problem may be due to the presence of swelling clays or the occurrence of differential settlements. In these cases, the application of an AC overlay is a good choice, as it will reduce the roughness, thereby restoring the riding quality. An AC overlay is ideal for such cases in which poor profile causes dynamic impact loading while the CRCP remains structurally sound.

In addition to PSI, the International Roughness Index (IRI) is used to establish this criterion. To assess the dynamic loading, a dynamic load factor is utilized. This factor is the ratio of the maximum dynamic load to the static load. The development of this criterion is explained in detail in Chapter 3.

### CONDITION SURVEY CRITERION

Once it has been decided that the pavement needs some type of rehabilitation, the assessment of the pavement condition is performed by means of a visual survey. The ideal observable and quantifiable behavioral characteristic is the appearance of failures. The data are collected by condition surveys involving the use of visual inspection to record the type and severity of distress.

The application of this criterion is based on two concepts, which stem from the analysis of the visual condition survey:

1. The pavement distress index (PDI)

2. The rate of failures occurrence with time

The PDI was developed in the 1980s at the Center for Transportation Research. In this study, the original equation was modified to incorporate the consideration of spalls. Prior to the 1980s, the occurrence of spalls was not a frequent incidence; therefore, spalls were not considered in the original equation. The original equation as well as the derivation of the new version is explained in detail in Chapter 4.

The second concept in the application of this criterion considers the rate of appearance of failures per unit distance over time. Failures in concrete pavement are defined as punchouts or patches —either concrete or asphalt.

A study developed by the CTR analyzed the history of failures of approximately 25 CRCP sections in Texas and found that whenever the annual failure rate for a particular pavement was below 3 failures per mile per year, it was economical to use a BCO, but when the rate surpassed 3, an unbonded overlay was the best decision [Taute 1981]. With this information, in this study the criterion was broadened to AC overlays, by establishing an annual rate of failures at which the placement of an AC overlay is advisable. The detailed development of this criterion is presented in Chapter 4.

## DEFLECTION CRITERION

The third criterion for the evaluation of the feasibility of an AC overlay on CRCP is the measurement and analysis of deflections, which constitute an invaluable tool in assessing the structural capacity of the pavement. Deflection measurements are normally made by means of several types of non-destructive testing devices, among which the most common in Texas is the Falling Weight Deflectometer (FWD). In the past, other frequently used devices were the Benkelman beam, Dynaflect and Road Rater, but nowadays most agencies use FWD.

The criterion developed in this study is based on stress calculations and deflection measurements taken at the cracks and at the midspan of pavement slabs. A stress ratio and a deflection ratio are computed. The stress ratio is calculated conceptually, from elastic layered theory and includes the evaluation of the stress with and without an overlay. The deflections ratio compares deflections at midspan to deflections at cracks, which is essentially an evaluation of the load transfer efficiency of the pavement. This criterion is presented in detail in Chapter 5 of this report.

## DECISION TREE

The decision tree for the project selection of an AC overlay on CRCP integrates the application of the three aforementioned criteria in a flowchart, which summarizes in a simplified way the methodology proposed for the project selection stage. The decision tree is presented in Figures 2.3a and 2.3b. A decision tree for AC overlay design is developed in Chapter 6.

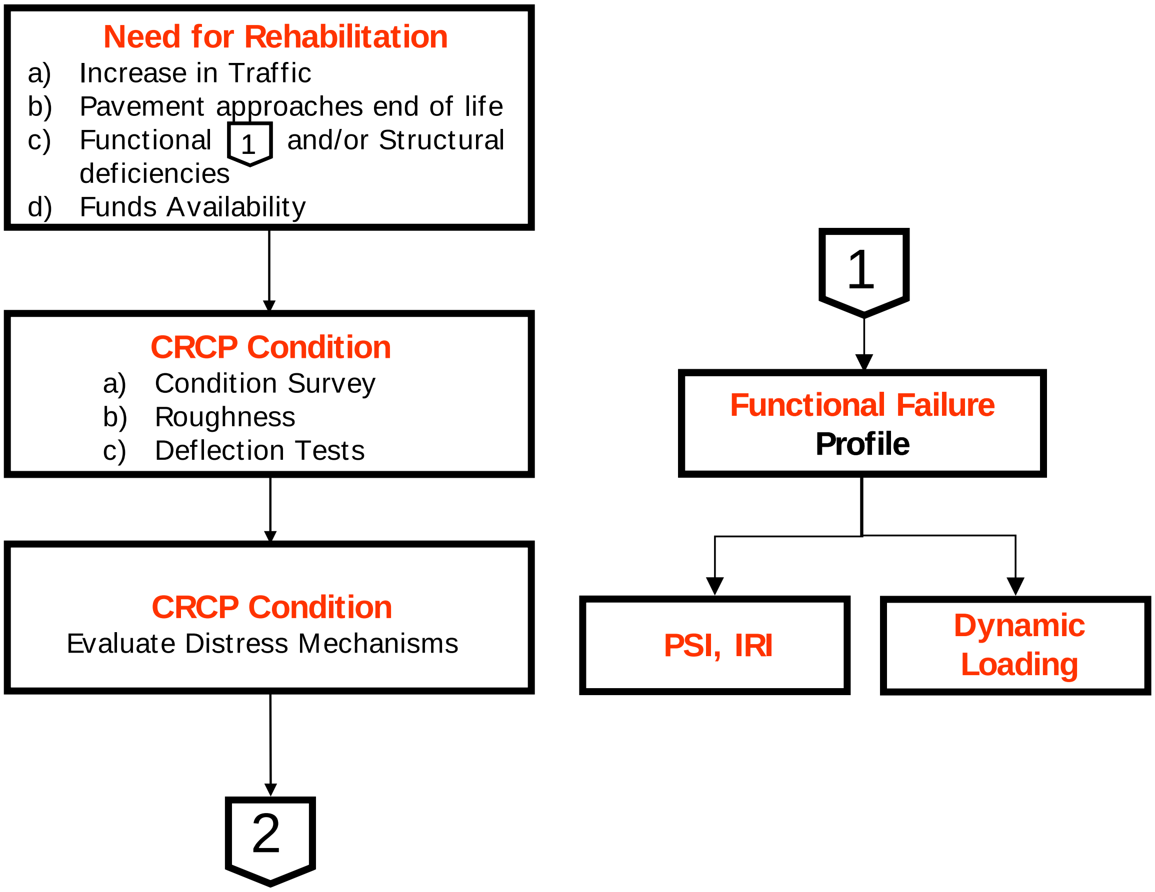



*Figure 2.3a Flowchart of the Decision Tree*

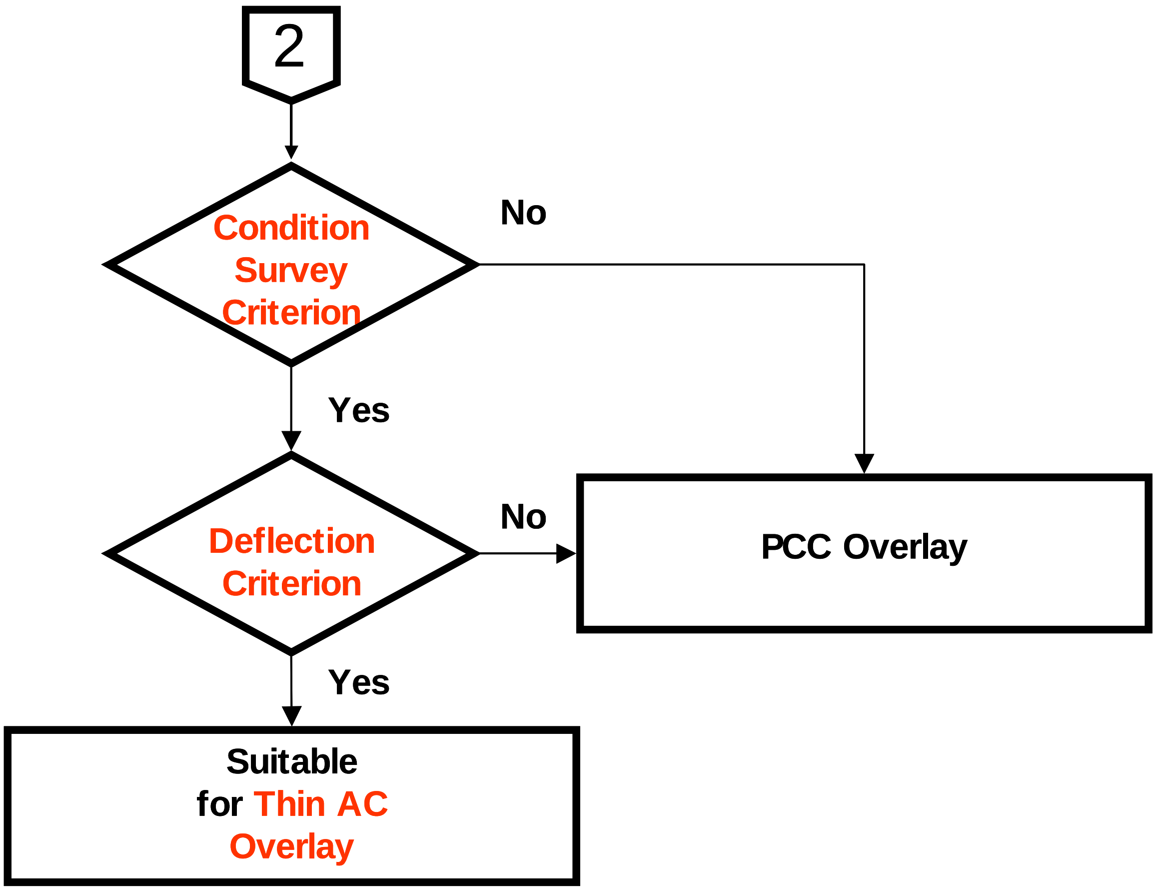



*Figure 2.3b Flowchart of the Decision Tree*

**SUMMARY**

The decision tree is a tool utilized during the project selection stage to evaluate the feasibility of a thin AC overlay on CRCP. The methodology involves a series of assessments and decisions that enable the designer to choose the best rehabilitation alternative. Three criteria have been developed that constitute the framework for the decision making process. The decision criteria are the profile criterion, the condition survey criterion, and the deflection criterion. The decision tree, illustrated in a flowchart, may appear as a rigid sequence of methodic comparisons, test and decisions. Nonetheless, there is room for improvisation, ingenuity and engineering judgement in every step of the process. In other words, the guidelines outlined in this chapter as part of the decision tree are not absolute; they engage subjective judgements based on experience as well as probabilities in the involved decisions, making the ultimate success of the Project Selection a stochastic event.

# CHAPTER 3.  PROFILE CRITERION

This chapter presents the first decision criterion for the selection of a thin asphalt concrete (AC) overlay on continuously reinforced concrete pavements (CRCP), the profile criterion.  This criterion is part of the decision tree, which consists of a series of systematic steps, evaluations and decisions to assist the pavement engineer in the project selection stage of a pavement rehabilitation project of this kind.  The decision tree concept was introduced in Chapter 2 of this report.  Then, in Chapter 4, the second major component of the decision tree is presented, the condition survey criterion, and Chapter 5 presents the third decision criterion, the deflection criterion.  In this chapter, the details of the development of the first assessment, the profile criterion will be presented.

The profile criterion, the first decision element in the decision tree, is a functional assessment, while the subsequent evaluations (condition survey and deflections) reflect the structural characteristics of the pavement.

## DYNAMIC WHEEL LOAD

If a pavement surface has roughness, the load imposed by moving vehicles will have variations in the magnitude due to the surface roughness.  These load variations are directly related to both the pavement life and users' perception.  The pavement stresses are dependent on the load magnitude, and the users' perception is dependent on the vehicle vibration.  Therefore, it is essential to use dynamic loading caused by surface roughness as an indicator to determine the need for resurfacing.

The effect of the surface roughness on the dynamic loading was examined first using the real surface profile data collected on IH-20.  This 3.4-mi. long CRCP section located in Harrison Co., near Marshall, was used extensively throughout Project 4398 as a source of valuable information for various analyses, such as the profile information.

This section has been overlaid with AC on several occasions. One of those overlays was removed in 2001. Shortly thereafter, a new AC overlay was placed in December of that year.

The surface profiles obtained from a typical section of the old AC overlay and the new AC overlay are shown in Figure 3.1. By simply looking at the profile data, it is very difficult to predict which one will induce higher dynamic loading. To predict the dynamic wheel loading, a computer program developed for the TxMLS project was used [Kim 1995]. Figure 3.2 shows the load variations on the front- and rear-axle tires when a truck moves over approximately 30 ft of the old and new AC overlays at a speed of 20 mph. It is apparent that the older pavement induces higher dynamic loading. On the front-axle tires, the highest dynamic load on the short section of the old overlay is about 19.3 kips, and that on the new surface is about 18.6 kips, which are 7.2 and 3.3 percent higher, respectively, than the static load of 18 kips (Figure 3.2(a)). On the rear-axle tires, as shown in Figure 3.2(b), the highest dynamic loads are 19.5 and 18.8 kips (8.3 and 4.4% over static load) on the old and new overlays, respectively. The new overlay reduces the increase in the maximum dynamic load about 50 percent in this short sample of the roadway. It is possible, therefore, to use the dynamic load to compare surface profiles and to determine the need of resurfacing.

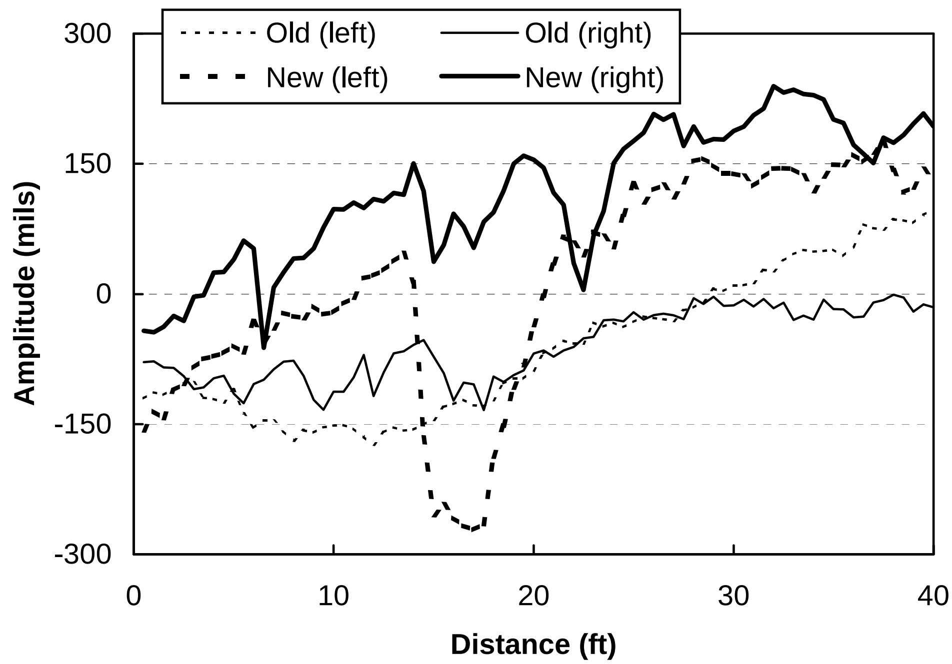



*Figure 3.1. Typical surface profile data collected on IH-20 on old and new AC overlays placed over a CRCP*

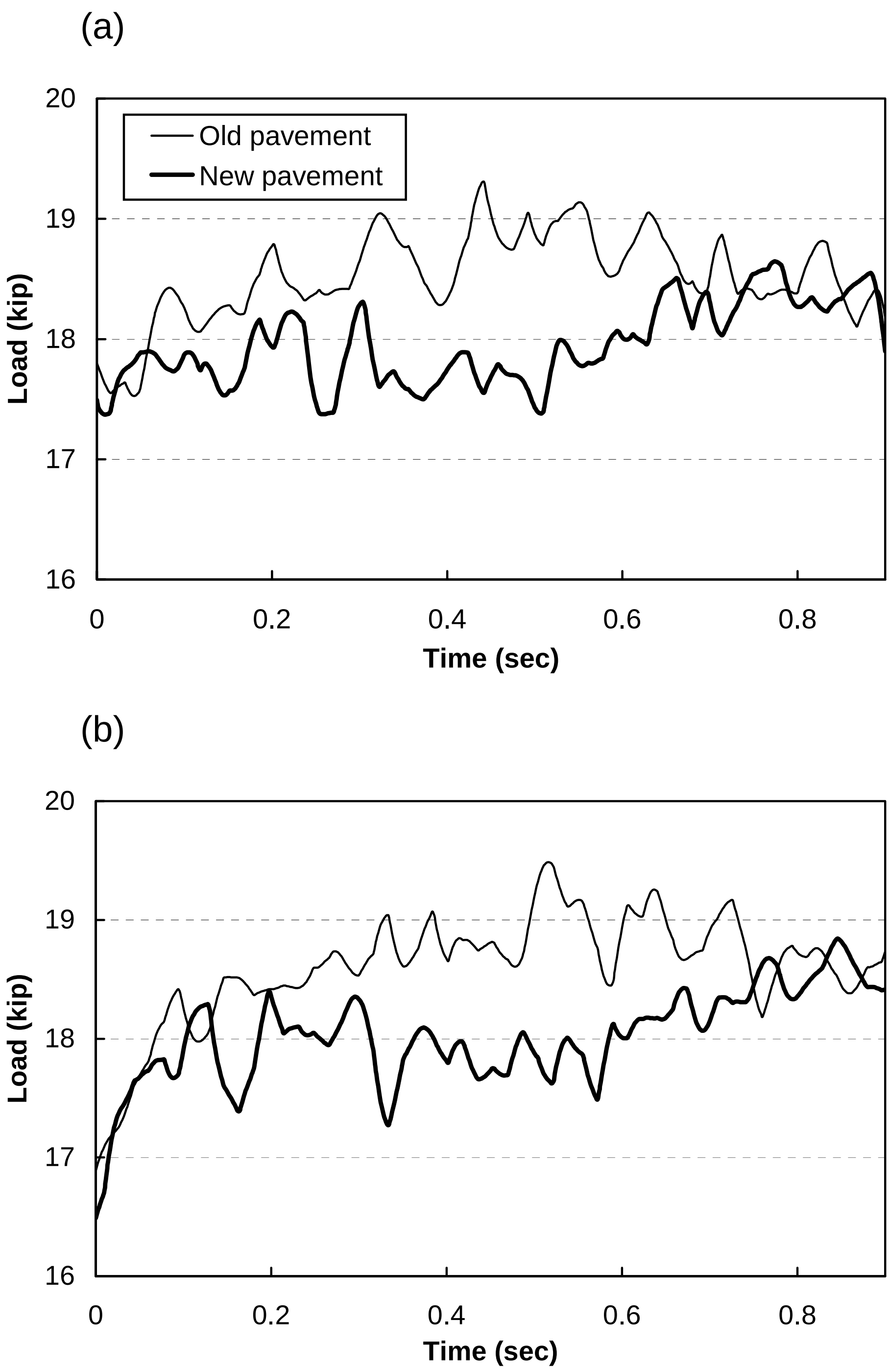
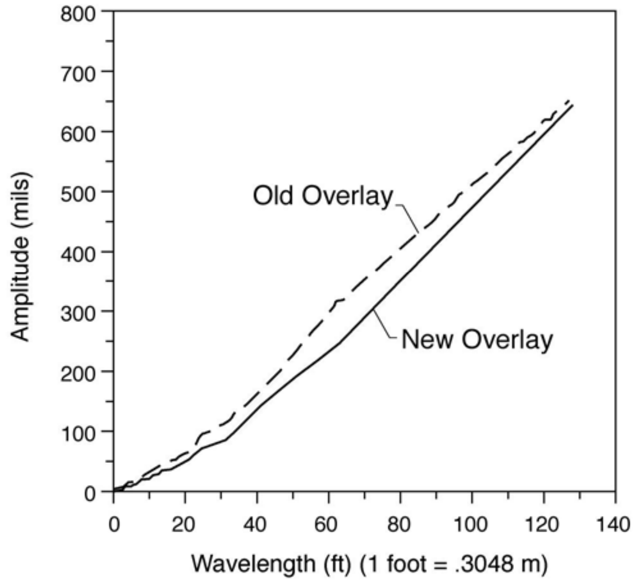



*Figure 3.2. Load time histories on (a) front- and (b) rear-axle tires with vehicle speed of 20 mph*

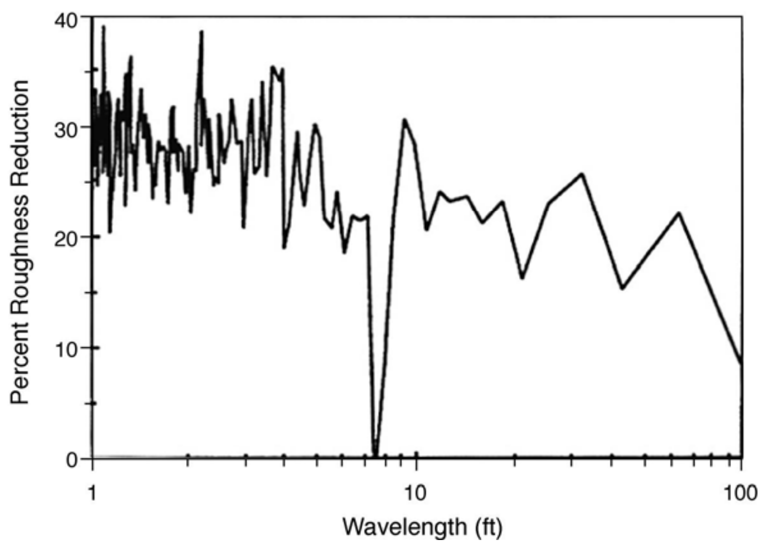
## PARAMETRIC STUDY

To investigate the effects of parameters, such as wavelength and amplitude of roughness and vehicle speed, on the moving dynamic load, the artificial profile data has been made and used for the dynamic analysis. Figure 3.3 shows the relationship between the roughness amplitude and wavelength obtained from the actual profile data collected on IH-30 in Bowie County [McCullough 1994]. It is shown in Figure 3.3(a) that the overlay reduces the roughness amplitude at a given wavelength. The maximum decrease in the roughness amplitude due to an overlay can be observed around the wavelength of 60 ft. Figure 3.3(b) presents the percent of roughness amplitude reduction. The percent reduction in the roughness amplitude is large with smaller wavelengths and tends to decrease as the wavelength increases.

(a) Absolute values




(b) Percent reduction




(a)

*Figure 3.3. Roughness amplitude vs. wavelength [McCullough 1994]; (a) absolute values, (b) percent reduction*

For the parametric study, the wavelengths of 4, 8, 20, and 40 ft, the roughness amplitudes of 200, 400, 600, and 800 mils, and the vehicle speeds of 20, 40, 60, and 80 mph have been considered. Many sets of the artificial profile data have been assembled by combining different roughness amplitudes and wavelengths. The artificial profiles used for the parametric study are shown in Figure 3.4.

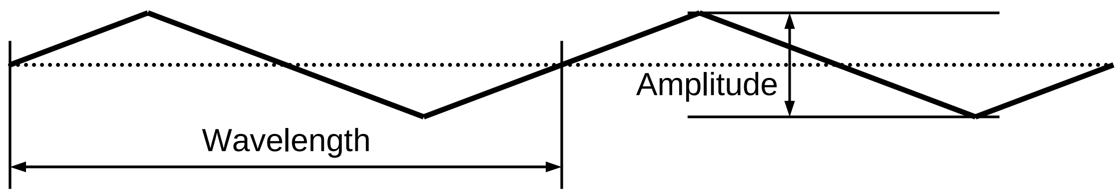



*Figure 3.4. Artificial profiles*

The time histories of the dynamic loads on the front- and rear-axle tires are investigated first when the vehicle moves on the artificial profile consisting of 4-ft wavelength and 200-mil roughness amplitude. When the vehicle speed is 20 mph, as shown in Figure 3.5(a), the dynamic loads on the front-axle tires show more fluctuations and the maximum dynamic load occurs on the rear-axle tires although the maximum dynamic loads on the front- and rear-axle tires are very close. When the vehicle speed is 60 mph (Figure 3.5(b)), the fluctuations of the dynamic loads on the front- and rear-axle tires are very similar, but the maximum dynamic load occurs on the front-axle tires. Therefore, the location where the maximum dynamic load occurs depends on the vehicle speed and profile data.

The effect of the roughness amplitude on the maximum dynamic load has been investigated and the results are shown in Figures 3.6 and 3.7. The maximum dynamic load increases as the roughness amplitude increases for a given wavelength and a vehicle speed. When the speed is 20 mph, as shown in Figure 3.6, the different wavelengths do not clearly affect the maximum dynamic load. For higher speeds, on the other hand, shorter wavelengths (4 and 8 ft in this case) of the profile yield higher maximum dynamic loads. In other words, the maximum dynamic load is not affected by the vehicle speed when the profile has a larger wavelength, as shown in Figures 3.7(c) and (d).

The effect of the wavelength of the profile on the maximum dynamic load is shown in Figures 3.8 and 3.9. Except for the vehicle speed of 20 mph, higher dynamic loads can be observed when the wavelengths are 4 and 8 ft.

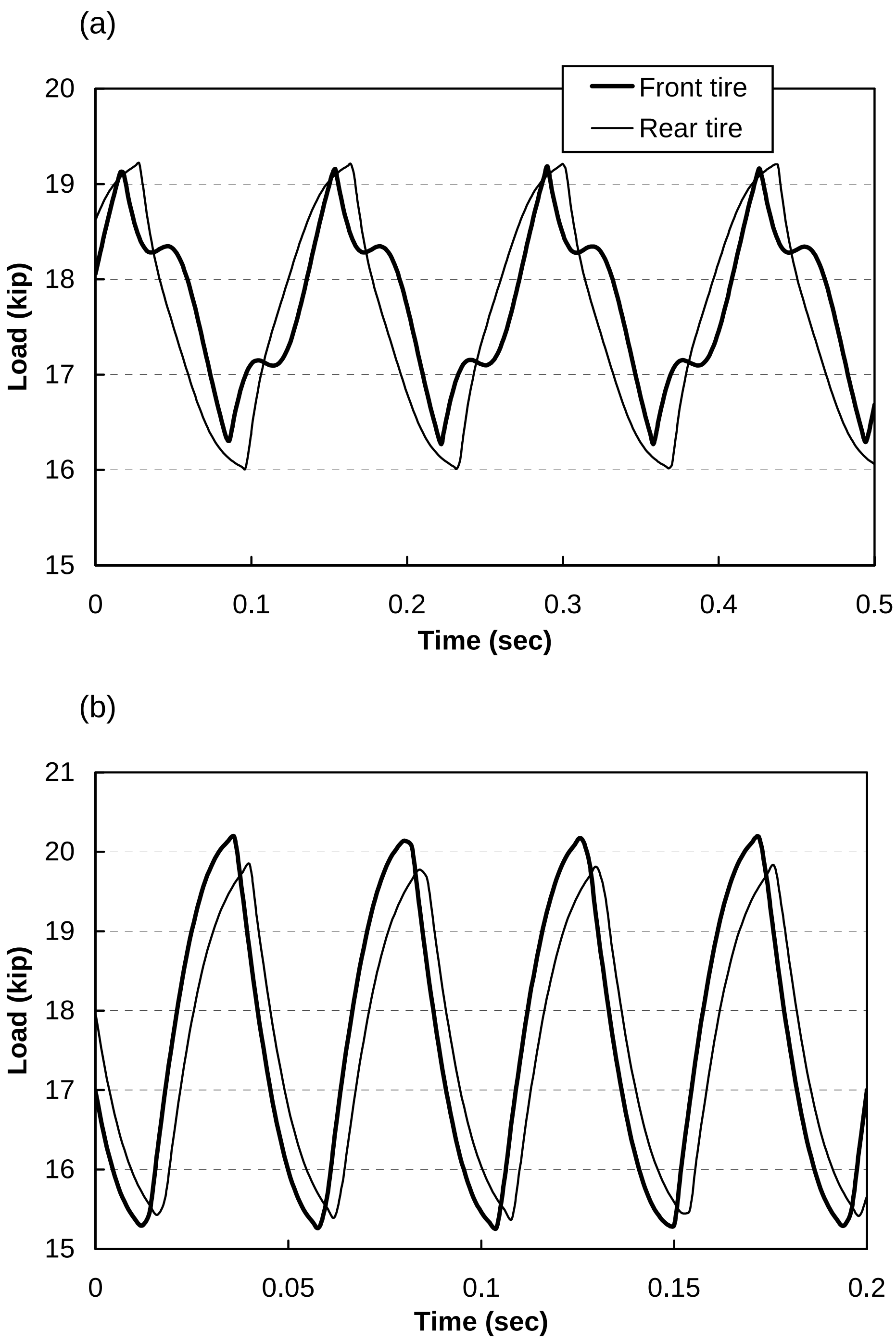



*Figure 3.5. Load time histories on front- and rear-axle tires when vehicle speed is (a) 20 and (b) 60 mph*

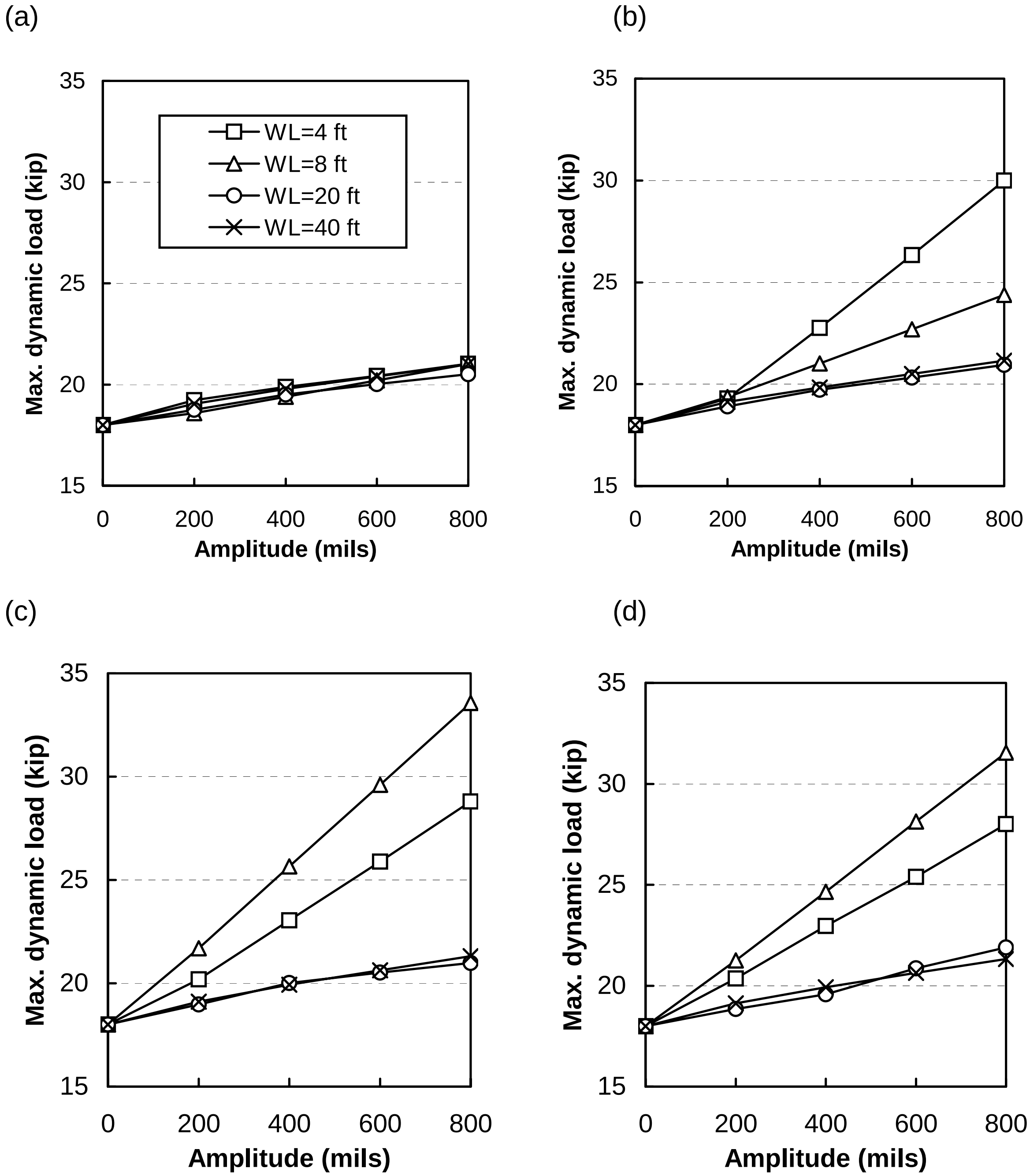
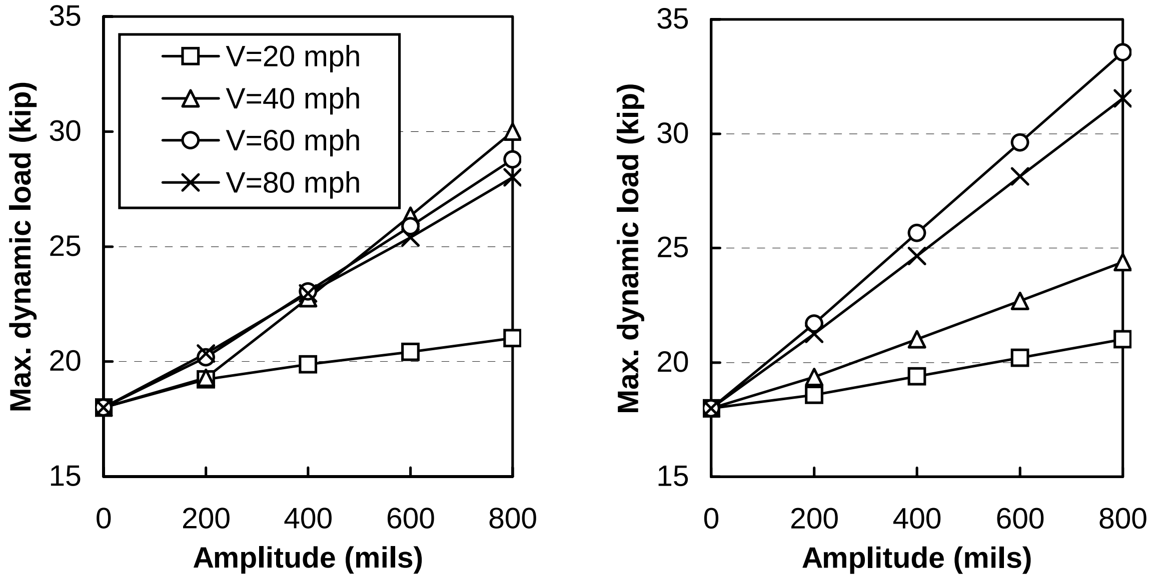



*Figure 3.6. Effect of roughness amplitude on dynamic loading for a vehicle speed of (a) 20, (b) 40, (c) 60, and (d) 80 mph*

(a) (b)




(c) (d)

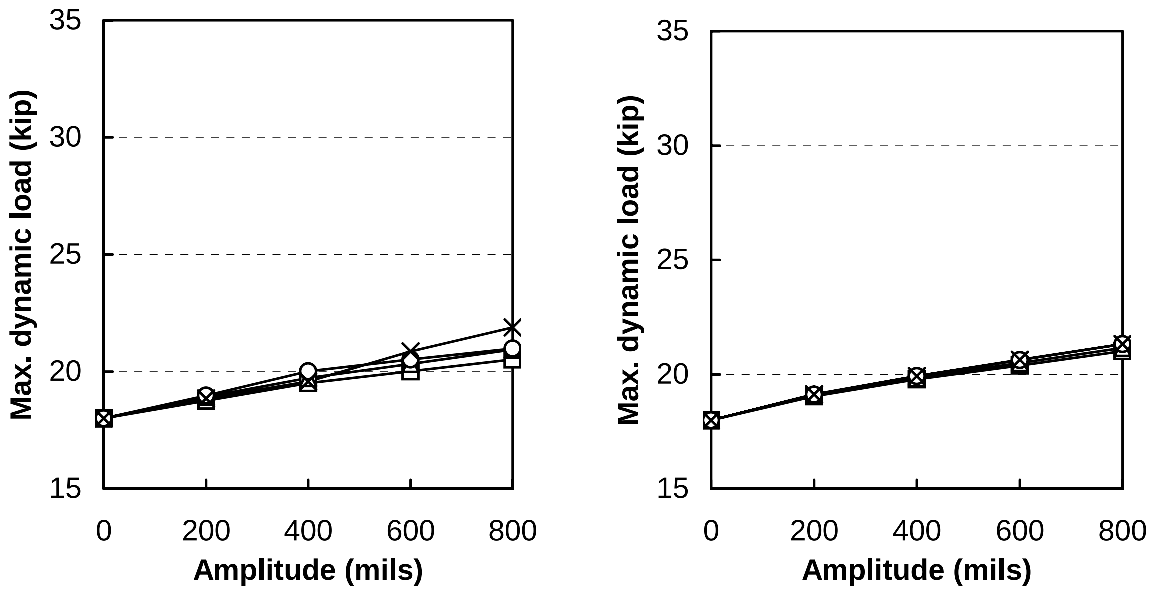
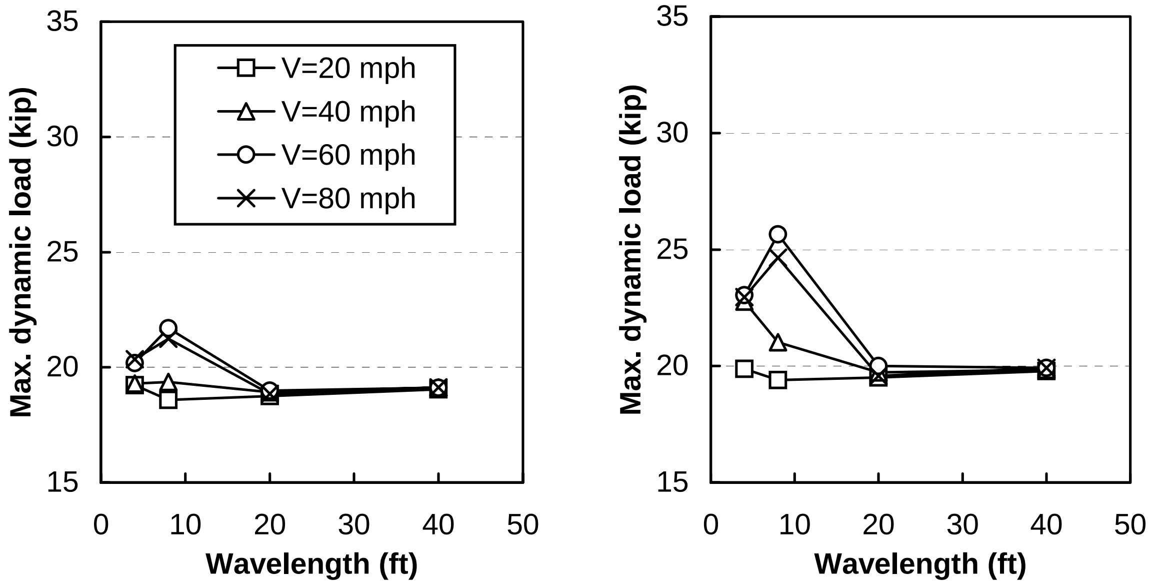
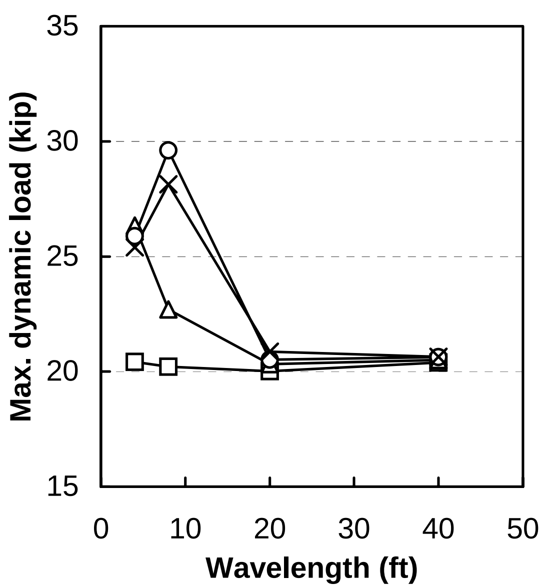



*Figure 3.7. Effect of roughness amplitude on dynamic loading for a wavelength of (a) 4, (b) 8, (c) 20, and (d) 40 ft*

(a) (b)




(c)




(d)

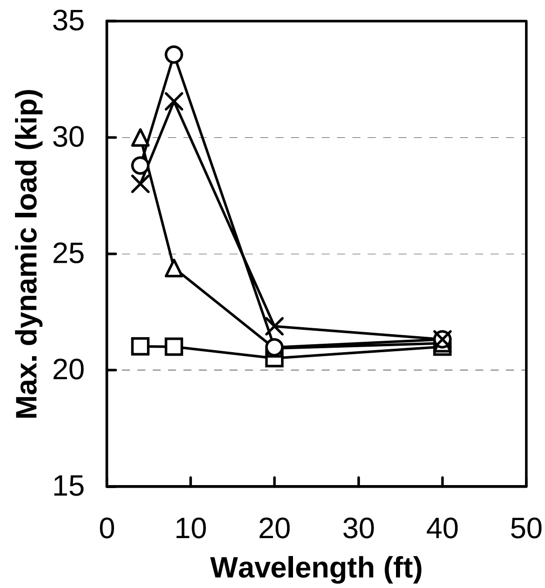



*Figure 3.8. Effect of roughness wavelength on dynamic loading for a roughness amplitude of (a) 200, (b) 400, (c) 600, and (d) 800 mils*

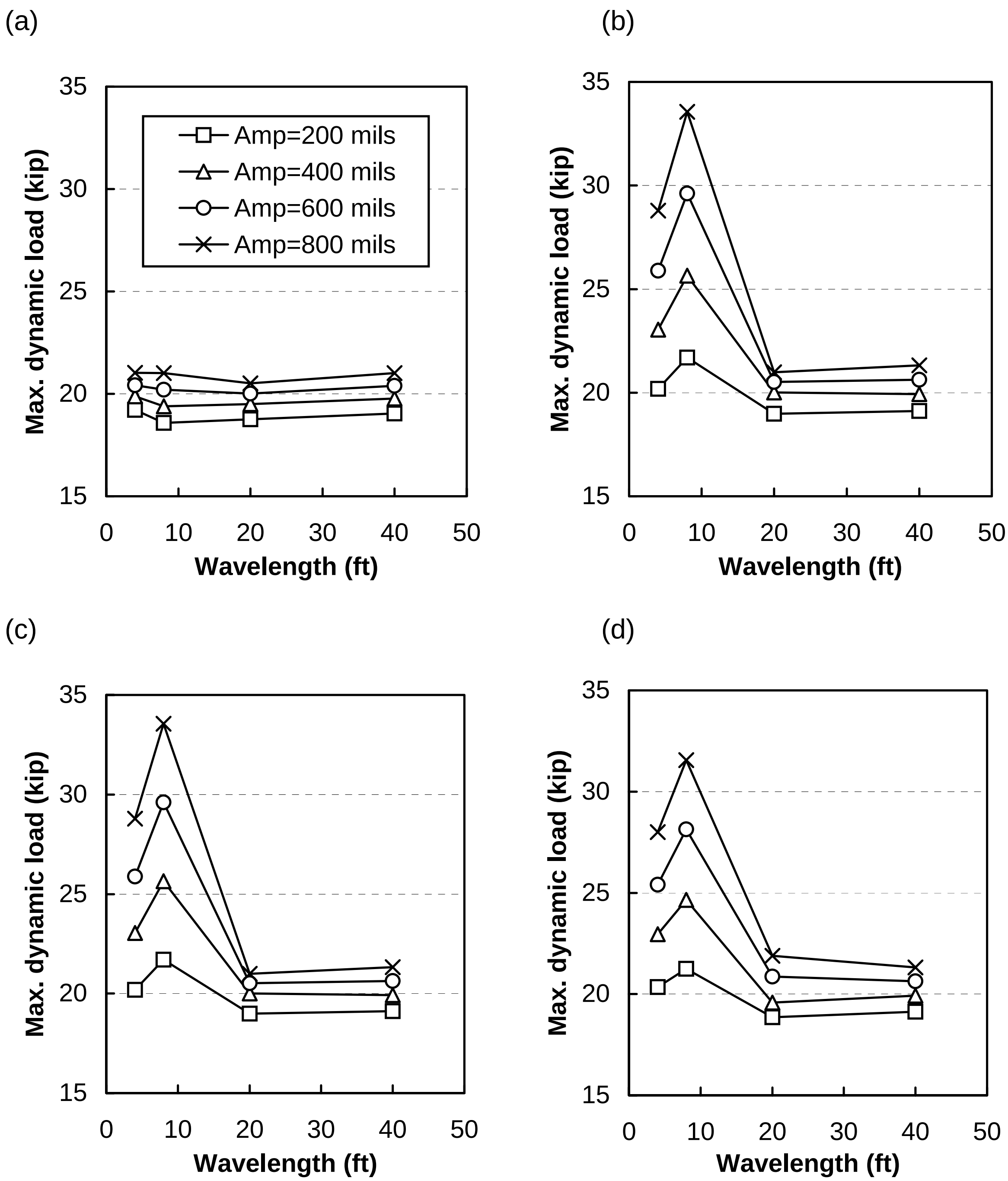



*Figure 3.9. Effect of roughness wavelength on dynamic loading for a vehicle speed of (a) 20, (b) 40, (c) 60, and (d) 80 mph*

The effect of the vehicle speed on the maximum dynamic load is shown in Figures 3.10 and 3.11. As the vehicle speed increases, the maximum dynamic load tends to increase initially and then becomes almost constant when the speed is higher than a certain level that depends on the wavelength and amplitude of the profile.

From this study, it has been found that the dynamic load can be significantly larger than the static load when the wavelength of the profile is short (smaller than about 15 ft), and the vehicle speed is high (higher than about 30 mph). The roughness amplitude has an almost linearly proportional relationship to the maximum dynamic load, and the slope of the linear relationship depends on the vehicle speed and the roughness wavelength.

Since significantly large dynamic loads are observed when the vehicle speed is 60 mph, as shown in Figures 3.10 and 3.11, the expressions to estimate the maximum dynamic load are developed in this study. When the vehicle speed is 60 mph and the wavelength of the profile is less than 20 ft, the dynamic load factor, which is a ratio of the dynamic load to the static load, can be obtained by

$$LF = 1 + a\left(\frac{-0.0002\,(\lambda)^2 + 0.0036\,(\lambda) + 0.0015}{18}\right) \tag{3.1}$$

where

$a$=amplitude (mils)

$\lambda$=wavelength (ft)

$\lambda$<20 ft

When the vehicle speed is 60 mph, and the wavelength of the profile is larger than 20 ft, the dynamic load factor can be calculated by:

$$LF = 1 + a\left(\frac{2 \times 10^{-5}\ \lambda + 0.0038}{18}\right) \tag{3.2}$$

where

$a$ and $\lambda$ are as defined previously

$\lambda$>20 ft

Thus, the relationship between dynamic and static loads can be expressed as:

$$P_D = (LF)P_S \tag{3.3}$$

where

$P_D$=dynamic load

$P_S$=static load

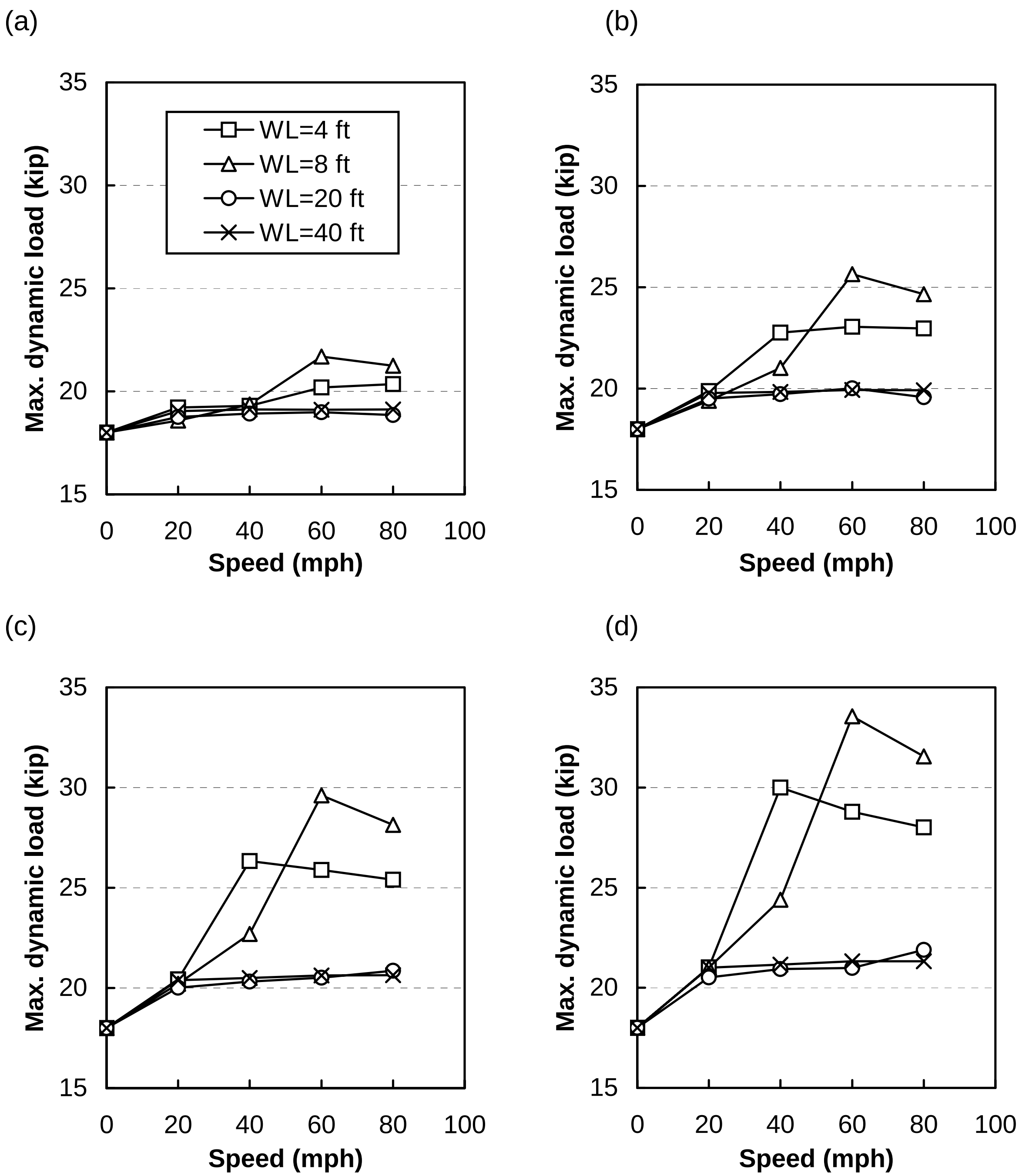



*Figure 3.10. Effect of vehicle speed on dynamic loading for a roughness amplitude of (a) 200, (b) 400, (c) 600, and (d) 800 mils*

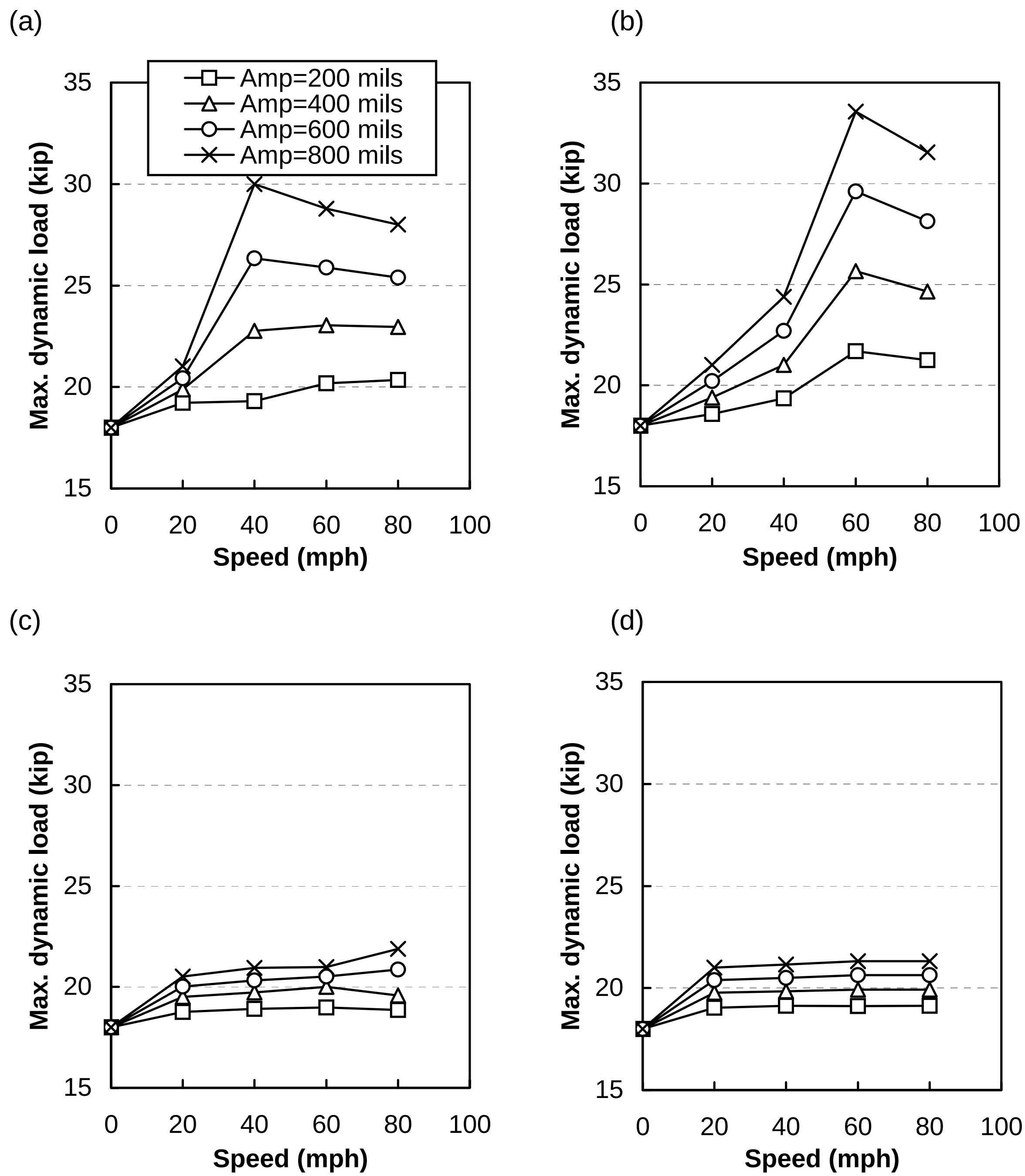



*Figure 3.11. Effect of vehicle speed on dynamic loading for a roughness wavelength of (a) 4, (b) 8, (c) 20, and (d) 40 ft*

## REMAINING LIFE

As investigated in the previous sections, the loads imposed by the moving vehicles have variations in the load magnitude because of the surface roughness of the pavement. Since the maximum magnitude of the dynamic load is normally larger than the static load, the pavement life will be reduced from the design life that is obtained based on the static load. With time and traffic, pavement roughness increases, and this is reflected in a lower PSI. The dynamic load pattern is site specific, thus it will vary on different roadways with a PSI of 2.0. For instance, Figure 3.12 shows typical pavements with different PSI values, where the load on the perfectly smooth pavement surface is the same as the static load, but the maximum load becomes higher as the present serviceability index (PSI) value decreases. The maximum dynamic load for the PSI 2.0 pavement on this example is about 40% larger than the static load.

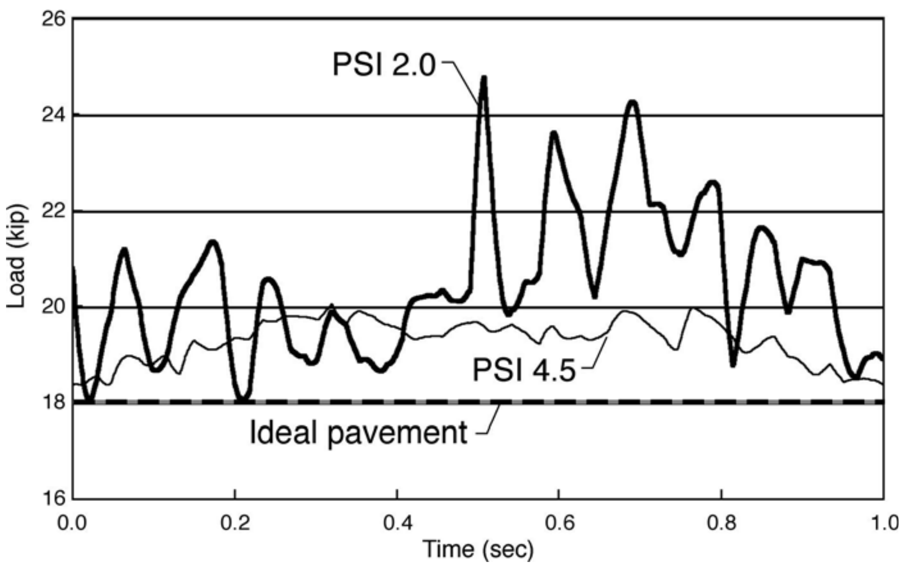



*Figure 3.12. Load time histories on front-axle tires for different surface profiles*

When the dynamic load is considered, the phase between the front- and rear-axle loads can affect the pavement responses. As shown in Figure 3.13, the general shapes of the load time histories on the front- and rear-axle tires are similar, but there is a very clear phase between them. In other words, when the front-axle load increases, the rear-axle load decreases, and vice versa. Generally, for rigid pavement systems, the phase effect can increase the pavement stresses and decrease the pavement deflections [Kim 2001, Kim 2002, and Kim 2003].

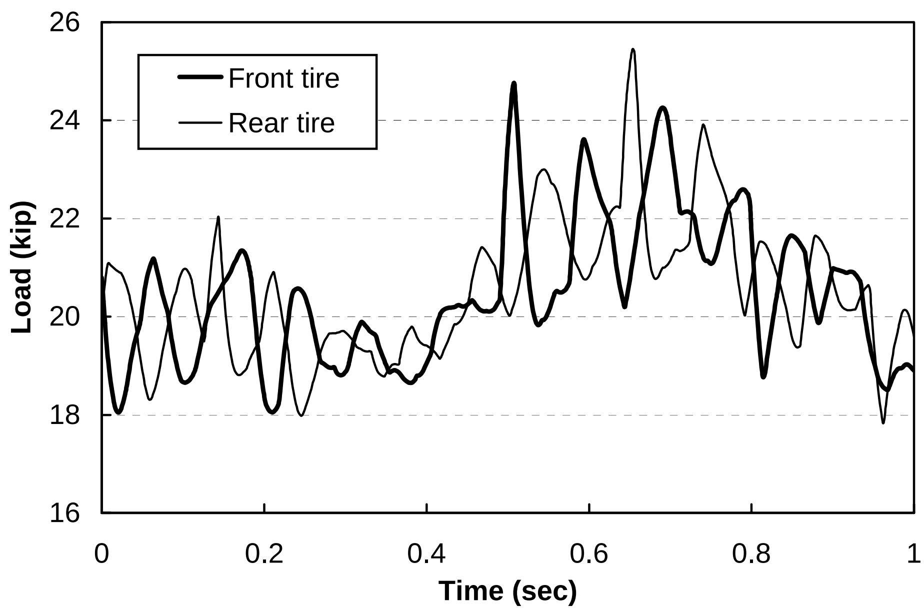



*Figure 3.13. Load time histories on front- and rear-axle tires on PSI 2.0 pavement*

The responses of the pavement systems subjected to moving dynamic loads can be obtained using several different methods. In this study, the transformed field domain analysis has been used based on the Fourier transforms in the time, space, and moving space [Kim 2001, Kim 2002, Kim 2003, Kim 1997, and Kim 1998]. The time histories of the pavement stresses under the rear-axle tires are shown in Figure 3.14. The shapes of the stress time histories are very similar to those of the load time histories. The maximum stress increases as the PSI value decreases. The maximum stress on the PSI 2.0 pavement is about 40% larger than the stress on the perfectly smooth pavement, which is very close to the increment amount in the dynamic load as mentioned previously.

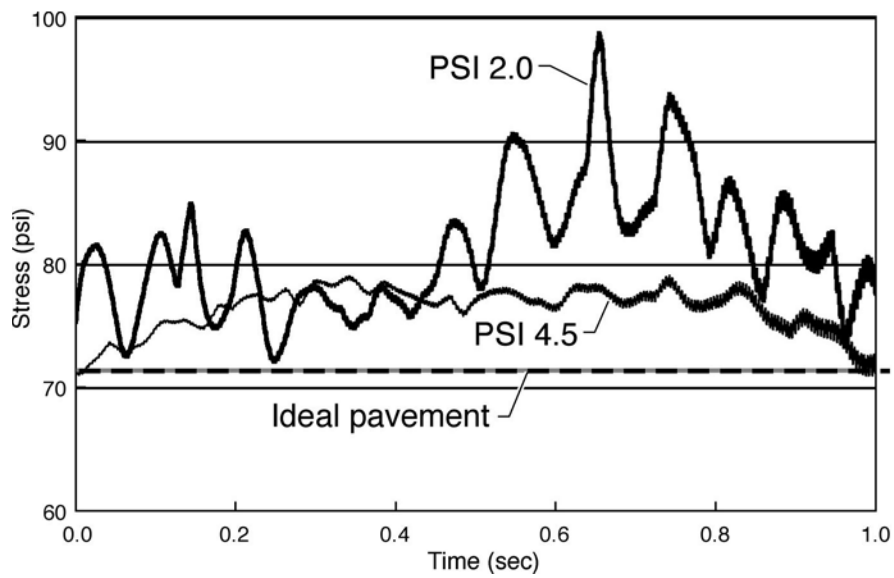



*Figure 3.14. Stress time history under rear-axle tires*

Once the pavement stresses are obtained, the pavement remaining life can be calculated by means of the AASHTO load equivalency factor, which may be approximated by the fourth power law as follows:

$$LEF_i = \left( \frac{Stress\ \ due\ \ to\ \ axle\ \ load\ \ i}{Stress\ \ due\ \ to\ \ 18\ \ kip\ \ axle\ \ load} \right)^4 \tag{3.4}$$

where $LEF_i$ is the load equivalency factor of an axle load $i$. If the stress time histories shown in Figure 3.14 are considered, the load equivalency factors can be obtained as shown in Figure 3.15.

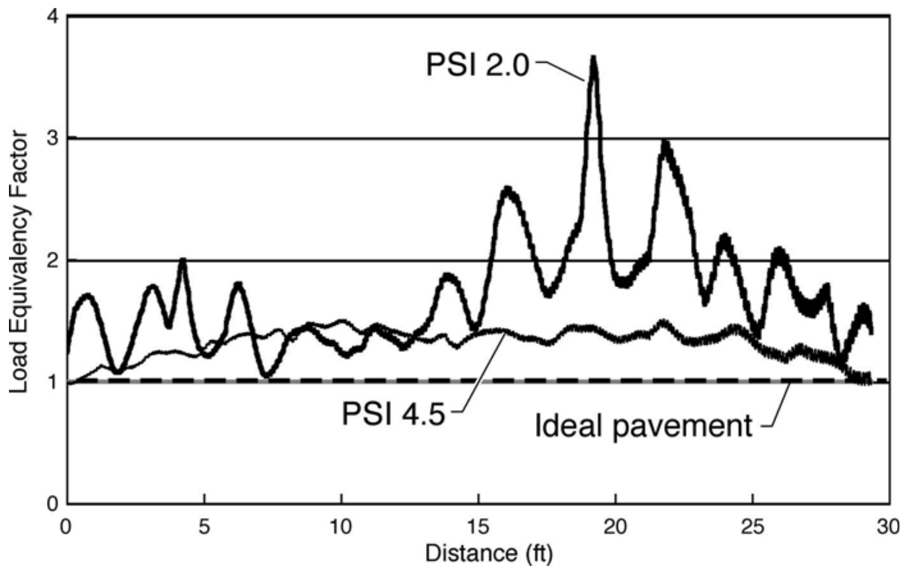



*Figure 3.15. Load equivalency factor*

As the graphs indicate, the equations apply to a point in the pavement and not to the entire section. The remaining life of the pavement is inversely proportional to the load equivalency factor and can be defined by

$$\textit{Pavement Remaining Life} = 1 / \textit{LEF} \qquad (3.5)$$

Figure 3.16 shows the remaining life of the pavement corresponding to the load equivalency factor shown in Figure 3.15. The remaining life of the PSI 2.0 pavement near the distance of 20 ft is about 25% of the design pavement life as shown in Figure 3.16. It is noted that the dynamic load factor can be substituted for the stress ratio in the load equivalency factor because the ratio of the stress is very close to the ratio between the dynamic load and the static load as indicated in Figures 3.12 and 3.14.

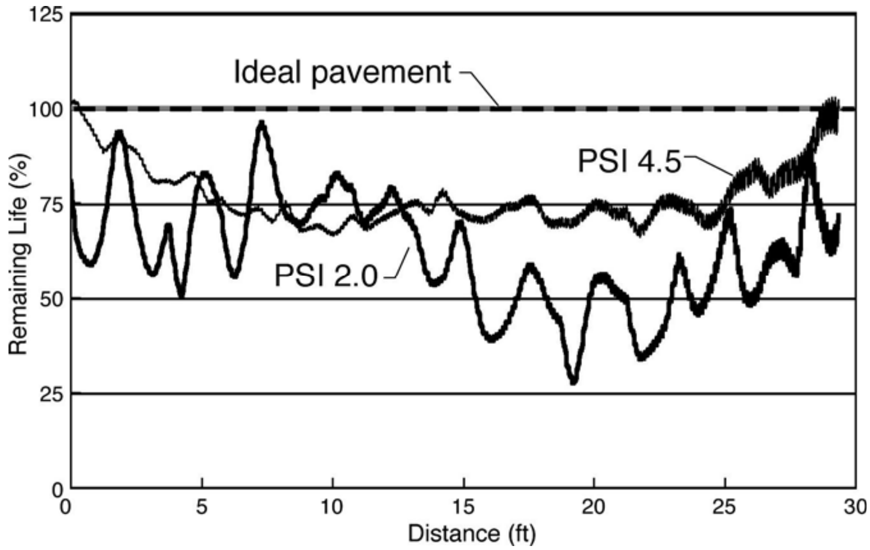
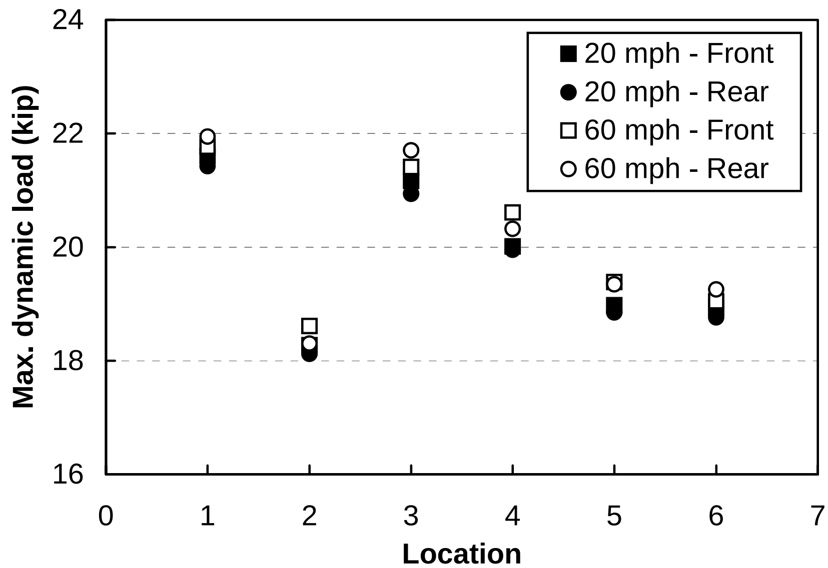



*Figure 3.16. Remaining life*

**CRITERION**

To develop a profile criterion for determination of pavement resurfacing, the actual profile data collected on IH-20 was used. Thus, it should be acknowledged that only data from one project could be utilized for the development of this criterion. It is recommended that, for future research, data from more projects are considered for the calibration and verification of this criterion. The data utilized in this case includes the profile of the pavement with the old AC overlay, and the profile of the pavement just after the new overlay was placed. The profiles of the randomly selected pavement sections were used. Each pavement section has the length of 100 ft. The effect of the vehicle speed on the dynamic load is shown in Figure 3.17. In the figure, the x-axis represents each of 100-ft long pavement sections. For instance, Location 1 is a 100-ft long randomly selected pavement section, Location 2 is another 100-ft long pavement section, and so forth. At a given speed, the maximum dynamic load can occur either on the front- or rear-axle tires as shown in the figure. At a given pavement profile, the maximum dynamic load is slightly larger with a high speed.

(a)




(b)

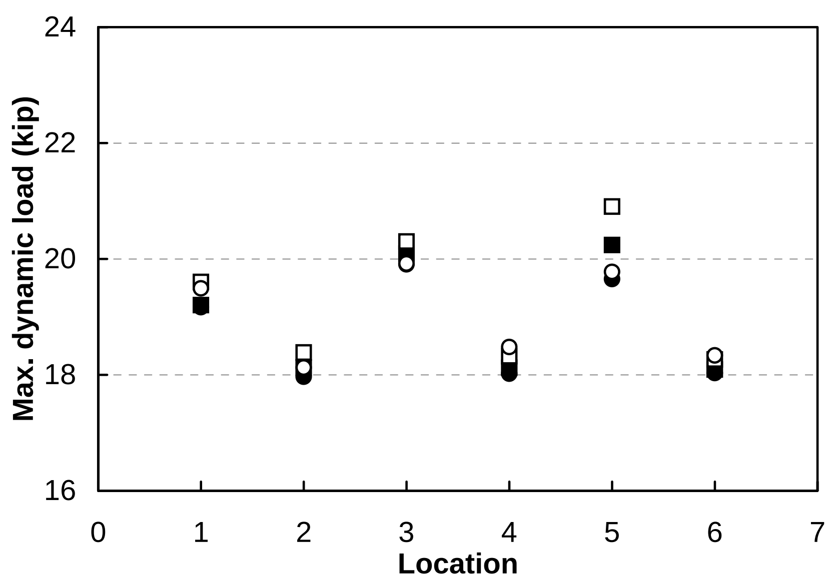



*Figure 3.17. Dynamic loads on the pavements on IH-20 (a) before and (b) after new overlays*

Figure 3.18 compares the dynamic loads on the old and new pavements. The maximum dynamic load on the old pavement is generally higher, but at Location 5 in the figure, the profile of the new overlaid pavement generates higher maximum dynamic loads. This implies that the dynamic load is normally higher on the old pavement, but the localized variation can exist. However, if the average values of the maximum dynamic load on the old and new pavements are compared, it is very clear that the old pavement induces higher dynamic loads, as shown in the figure. When the vehicle speed is 20 mph, the new pavement has an average maximum dynamic load of about 5% higher than the static load, and the old pavement has that of about 10% higher than the static load. When the speed is 60 mph, as shown in Figure 3.18(b), the new and old pavements have average maximum dynamic loads of about 6 and 12% higher than the static load, respectively.

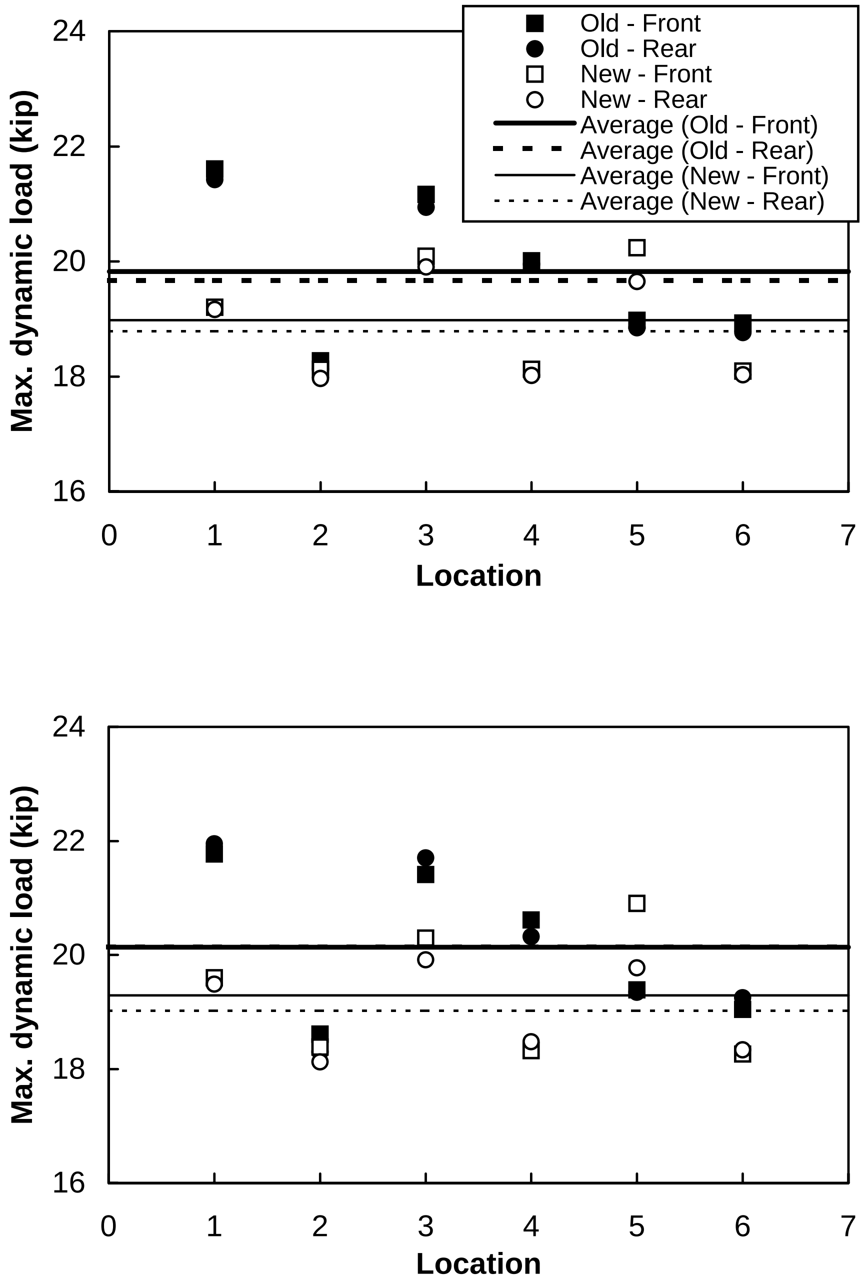
As explained previously, the remaining life of the pavement is related to the ratio of the pavement stresses when subjected to static and dynamic loads, and the stresses can be assumed to be proportional to the load. Therefore, the ratio between the dynamic and static loads, which is called a dynamic load factor, can be used as a criterion to determine the need of pavement resurfacing. As investigated with the actual profile data, the pavement before the new overlay was placed showed more than about 10% higher load magnitude due to the surface roughness. Based on this actual field data, it is recommended that the pavement need resurfacing if the dynamic load factor is larger than 10%. In other words, if the remaining life of the pavement is less than 68%, which is the inverse of the fourth power of the load ratio of 1.1 (10% higher than the static load), resurfacing is needed. The profile criterion for the pavement overlay can be summarized as

Pavement resurfacing is needed when:

(1) dynamic load factor > 1.1, or

(2) remaining life of pavement < 68% of design life

(a)




(b)

*Figure 3.18. Comparison of dynamic loads on the pavements on IH-20 for a vehicle speed of (a) 20 and (b) 60 mph*

# CHAPTER 4. CONDITION SURVEY CRITERION

This chapter presents the second decision criterion for the selection of a thin asphalt concrete (AC) overlay on continuously reinforced concrete pavements (CRCP), the condition survey criterion. This criterion is part of the decision tree, which serves as a framework for the project selection stage for a pavement rehabilitation project of this kind. The decision tree was introduced in Chapter 2 of this report. Chapter 3 presented the first one of the criteria developed for the selection of an AC overlay, the profile criterion. This chapter will illustrate in detail the concepts utilized for the development of this decision criterion. Chapter 5 will present the third decision element for this process, the deflection criterion.

As explained in the previous chapters, the first decision to be made in this process is whether to rehabilitate a pavement section, and then, whether an overlay is suitable. For that purpose, the profile criterion is the first pavement feature that is analyzed. Once it has been decided that the section is a candidate for overlay rehabilitation, it has to be determined which overlay type is the most appropriate. It is at this stage that the condition survey criterion is evaluated. Therefore, the first assessment to decide upon an overlay involves serviceability, which is associated with the profile criterion. If an overlay is deemed fitting, the ensuing evaluation is performed from a structural standpoint.

## FAILURES

The way this methodology is presented, it suggests that the first evaluation is functional, based on a serviceability criterion. That serviceability criterion is the profile criterion. Nonetheless, the evaluation should not be based solely upon the previous criterion. In fact, many authors recommend evaluating the pavement condition more from a structural standpoint rather than using serviceability criteria, as expressed in the following excerpt from [Barenberg 1981]:

> “Evaluating the true condition of the existing pavement is one of the most critical factors in selecting the best overlay option. This evaluation should reflect how the existing pavement will affect the behavior and performance of the overlaid pavement. Such an evaluation should be based on structural or behavioral considerations rather than serviceability considerations.”

In the development of this methodology it was considered to utilize the serviceability evaluation as a preliminary indicator, and from then, it is recommended to proceed to the structural evaluation for a more thorough analysis. Attending this concern, for a structural assessment, the ideal observable and quantifiable behavioral characteristic is the appearance of failures. The data are collected by condition surveys involving the use of visual inspection to record the type and severity of distress.

This criterion was developed based on two different approaches, both of which result from the analysis of visual condition surveys. These two concepts are based on the appearances of failures, but one of them is oriented toward distinguishing the types of failures, assigning various weighing factors to the failure types to establish a rehabilitation criterion, whereas the other one considers the time (pavement age) at the appearances of the failures. Hence, given that the approaches are fundamentally different, the application of this criterion has to be twofold. The approaches are:

1. The pavement distress index (PDI)
2. The rate of failures per mile per year

**PDI**

The PDI was originally developed in the 1980s at the Center for Transportation Research [Chou 1988]. The PDI is an indicator that considers several types of failures to assess whether to rehabilitate. The concept states that a pavement in perfect conditions with no distresses starts out with a PDI of 1; as the pavement deteriorates, the appearance of failures has a subtractive effect on that index. The result of the usage of the facility can be illustrated in terms of traffic or time, which make the PDI drop. The normal decline in PDI is illustrated in Figure 4.1, in which, while the pavement is still new, the number of failures is expected to be minimal, keeping the PDI fairly stable and close to a value of 1. As the pavement ages, the appearance of failures is more frequent and the deterioration rate increases, making the decline of the PDI curve more pronounced. Eventually, the PDI will plummet to a point in which the number of failures demands rehabilitation. Normally, the threshold value for a major rehabilitation is a PDI equal to zero.

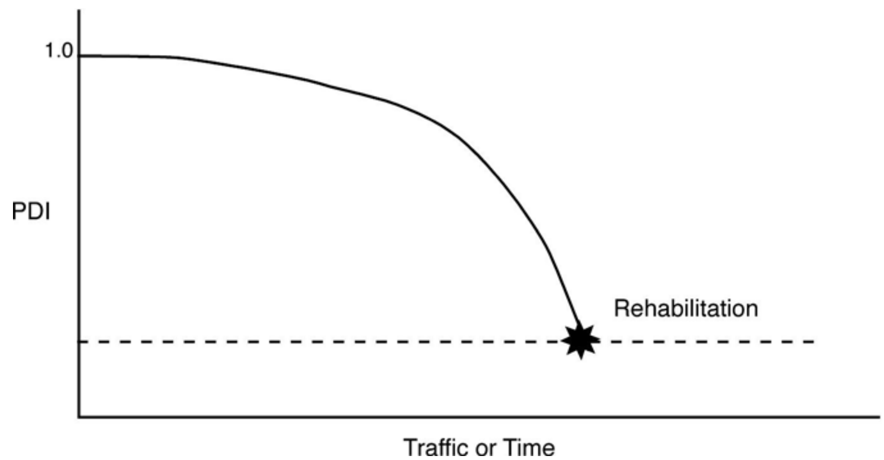



*Figure 4.1 PDI concept*

### *PDI as a Discriminant Score*

The PDI, when utilized in the way established in the previous statement, i.e., as a threshold value to determine whether a rehabilitation is needed, is called a discriminant variable. A discriminant analysis, which makes use of a discriminant variable, is a statistical procedure to classify data into groups by maximizing the differences between group means. For instance, in this case, a discriminant analysis is used to classify pavement sections into two groups, those that need rehabilitation and those that do not, with the discriminant variable being the PDI. Hence, discriminant variables measure characteristics in which the groups are expected to differ.

If the PDI can be used as a discriminant variable, the sections in need of rehabilitation would be expected to have a different PDI from that of sections that do not need to be rehabilitated. In practice, a more convenient way to state this comparison from a condition survey standpoint would be to contrast the PDI of sections that have been overlaid, using the survey information prior to the overlay construction, versus the PDI of sections that have not been overlaid. This statement assumes that the pavement condition reaches its lowest PDI just prior to the overlay placement time [Chou 1988].

It is expected that the individual PDI scores will be distributed normally around their means, i.e., their frequency distributions will be similar to the curve for a normal distribution, and since the means are expected to be different, their plot would look as illustrated in Figure 4.2. While in some sections the distinction between PDI values may be clear, there may be some others with borderline values. These values constitute the “zone of conflict” in Figure 4.2. This shaded area corresponds to sections in which the decision whether to rehabilitate is not clear.

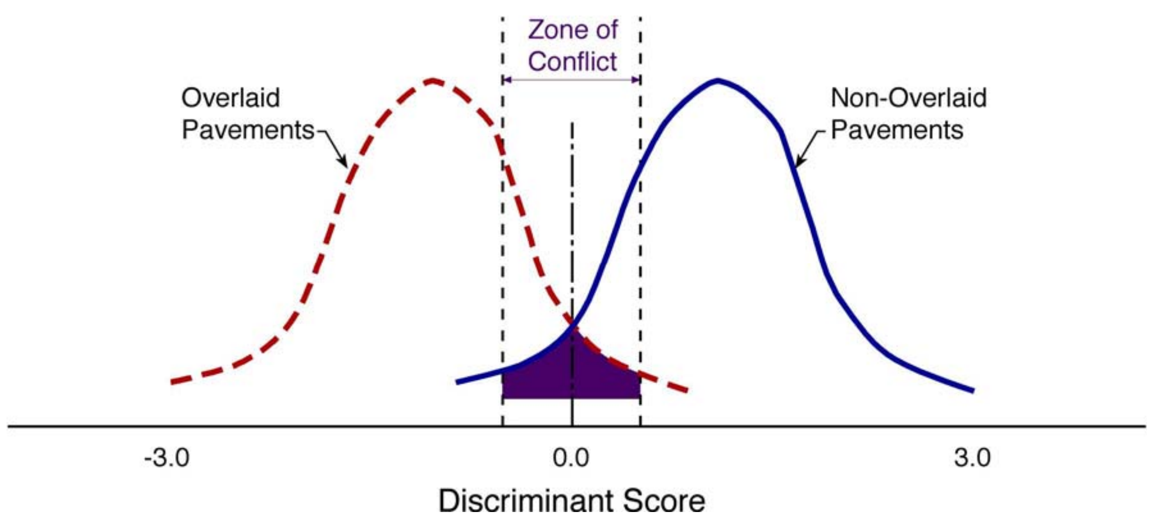



*Figure 4.2 PDI as a discriminant score for rehabilitation decision*

### *PDI Equation*

The graphic illustration of the idea of PDI showed in Figure 4.1 can be translated into an equation of the following form, in which various types of distresses can be considered, with each type having a weight in the deduction from the unit, and each can have as many degrees of severity as needed. A factor is added to take into account the extent of each class.

$$PDI = 1 - \sum_{i=1}^{n} \sum_{j=1}^{m} D_i \times S_{ij} \times E_{ij}$$

where

$D_i$ = deducted points for the $i^{th}$ type distress,

$S_{ij}$ = weight of the $j^{th}$ severity class of the $i^{th}$ type of distress,

$E_{ij}$ = extent of the $j^{th}$ severity class of the $i^{th}$ type distress

$n$ = number of distress types,

$m$ = number of severity classes

This equation shows the concept of the PDI. With it, any agency can develop an equation based on the types of distresses that occur in their pavements. The procedure would be to gather enough historic information from condition surveys and determine their relative weights to come up with the coefficients, and to calibrate in such a way that the PDI values obtained with its application can be used as a decision criterion to determine whether a section needs an overlay. In the following section, the original equation, derived from condition survey data is introduced, along with a new version of it, developed in this study.

### *Original PDI Equation and Spalling*

In this study, the original equation, developed two decades ago, was modified to suit the current pavement conditions across the state. Failures in concrete pavement are defined as punchouts and patches, either concrete or asphalt patches. The original equation included all these.  In the 1980s, the occurrence of spalls was not a frequent incidence and therefore, spalls were not considered in the original equation.  However, spalls are now a frequent distress found in concrete pavements.

The original equation, based on condition surveys conducted prior to 1984, is as follows:

$$PDI = 1.0 - 0.0071(MPUNT) - 0.3978\,(SPUNT) - 0.4165\,(PATCH) \qquad (4.1)$$

where

MPUNT= ln (minor punchouts per mile +1)

SPUNT= ln (severe punchouts per mile +1)

PATCH=ln (patches per mile+1)

A punchout is formed when closely spaced transverse cracks are connected by longitudinal cracks to form a block.  In a minor punchout, there are no signs of apparent movement of the block, the cracks are narrow, and few signs of spalling are present. Conversely, a severe punchout occurs when the block moves under traffic, the cracks are wide, and there are signs of pumping around the edges of the block.

The coefficients in the Equation 4.1 were the result of the analysis of ten years worth of condition survey information gathered across the state, between 1974 and 1984, including both rural and urban districts.  The data reduction procedure and analysis performed to develop the coefficients for this formula is presented in detail in [Chou 1988].

The effect of spalling on the PDI curve is illustrated in Figure 4.3, where conceptual representations of the original equation’s curve (dotted line) along with the accelerated drop rate that the spalls inclusion in the equation would cause are depicted.

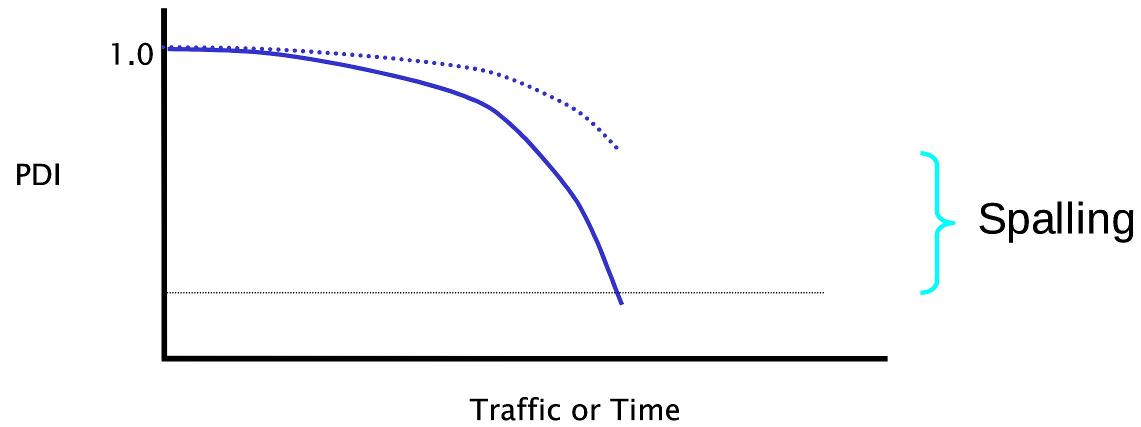



*Figure 4.3 Inclusion of spalls in the PDI equation*

To consider the effect of spalls in the equation, a new term became necessary, along with its corresponding coefficient. With this new term, the PDI equation is follows.

$$PDI = 1.0 - 0.0071(MPUNT) - 0.3978\,(SPUNT) - 0.4165\,(PATCH) - \mathrm{K}(SPALL) \tag{4.2}$$

where

SPALL= ln (spalls per mile +1)

K = coefficient for spalling

It should be mentioned that this modification to the original PDI equation was done only for the purpose of evaluating the relative influence of an additional element not considered in the initial development, and therefore, this supplementary term does not constitute a statistically valid addition.

***Computation of Spalling Coefficient***

The value of K, representing the relative weight of the effect of spalling in the PDI value, had to be found from condition surveys.

For that purpose, the condition survey information from the test sections on State Highway 6 (SH6) in Houston were utilized. The coefficient found from these data was verified by applying the new equation to different CRCP sections with known spalling problems.

The SH6 test sections, located in west Houston, were used throughout this project for data collection and analysis purposes. The reason these sections were selected for the computation of the spalling coefficient is because these sections have been surveyed continuously since their construction, being the subject of numerous pavement studies. Their history is well documented; therefore, it was an ideal case study. The other factor is that some of them have a spalling problem. The location of these sections is shown in Figures 4.4 and 4.5.

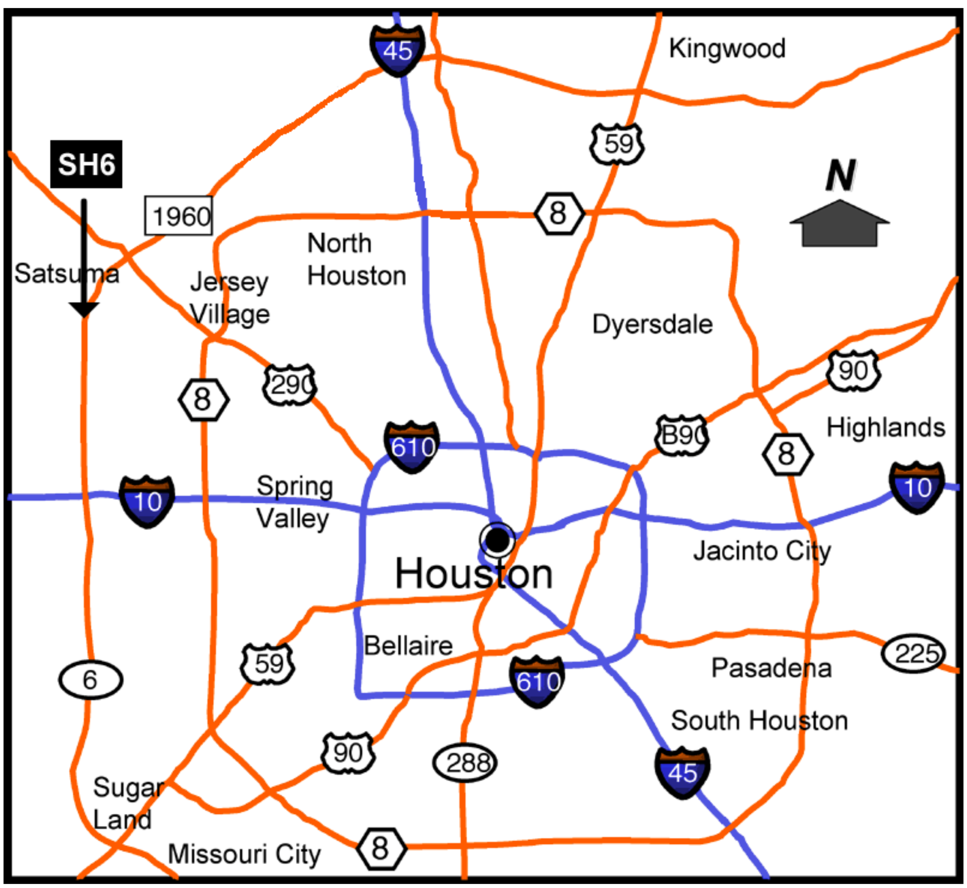



*Figure 4.4 Location of SH6 test sections in Houston*

Test sections on SH 6 were constructed both during winter time and summer time, using two types of coarse aggregates, namely, limestone (LS) and siliceous river gravel (SRG) and various percentages of reinforcement steel. The summer sections, located near Huffmeister Rd., just south of US 290, were constructed in June of 1989 whereas the winter sections, located near Patterson Rd, north of IH-10, were constructed in January 1990. Specifically, the data used to develop the spalling coefficient came from condition surveys conducted on the winter sections constructed with SRG, which as a result of their extensive spalling had to be overlaid with a thin AC (novachip).

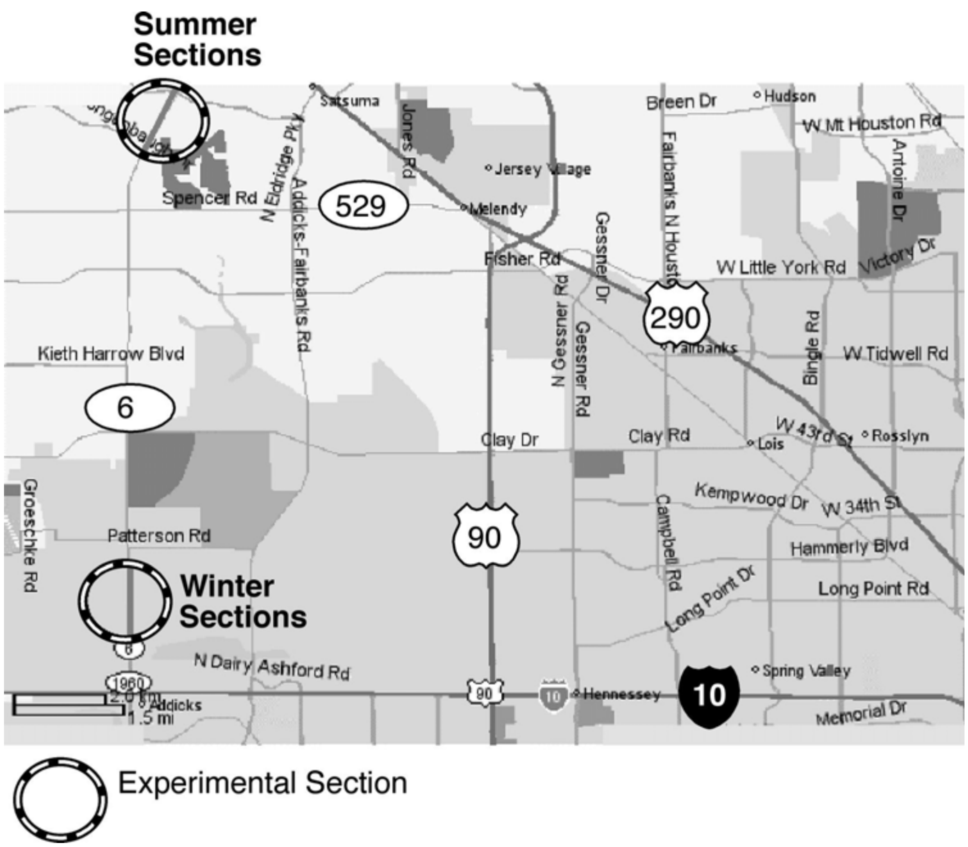



*Figure 4.5 Detail of experimental sections on SH6*

The last condition survey before the sections were overlaid was conducted on April 10, 2002. The results of this survey were used in the computations of the spalling coefficient in the new PDI equation. Figure 4.6 illustrates the extension of the spalling problem in the winter sections constructed with SRG, showing transverse cracks, spalls and percentage of spalled cracks. Each of the subsections (designated as A through D) is 230 ft long; therefore, the total length of these sections is 920 ft.

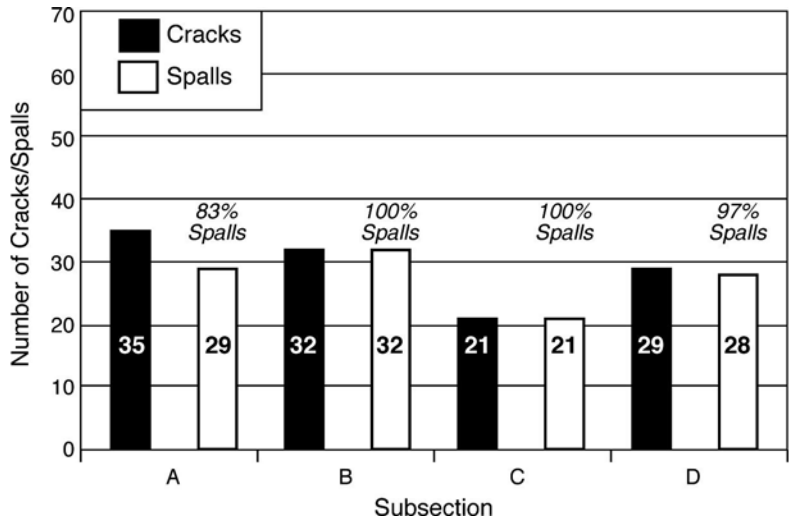



*Figure 4.6 Crack spalling on SH6 winter sections (SRG)*

The total number of spalls in the four subsections found in the 2002 survey was 111. This number of spalls in 920 ft results in 637 spalls per mile. Since these sections are currently overlaid, their PDI for the computations that follow was assumed to be –0.5 (prior to the overlay construction). As mentioned before, a PDI of zero or less is an indication that the section is in need of an overlay. The value of –0.5 was selected considering that the spalling was extensive as it can be verified in Fig. 4.6, with a high percentage of the transverse cracks showing some spalling, and hence, the rehabilitation was overdue.

With an overall PDI of –0.5, the value of the spalling coefficient, K, is equal to 0.2323. With this coefficient, the individual PDI values were calculated for each of the subsections, and these are shown in Table 4.1.

*Table 4.1 PDI Values for SH6 Winter SRG Subsections*

| | Spalls | Length (ft) | Spalls/mi | PDI |
|---|---|---|---|---|
| Section A | 29 | 230 | 665.7 | -0.51 |
| Section B | 32 | 230 | 734.6 | -0.53 |
| Section C | 22 | 230 | 505.0 | -0.45 |
| Section D | 28 | 230 | 642.8 | -0.50 |

To verify the new PDI equation, it was applied to a number of different sections that the research team had investigated with knowledge of the presence of some spalling problems.

The first group of sections used to verify the PDI equation includes four test sections in the Houston area, in which the SH6 sections happen to be included as well. However, these condition surveys were conducted in 1998. In addition to SH6 summer and winter sections, which at the time started to show signs of the spalling problem in the SRG segments, the sections surveyed in this group include Beltway 8 and IH-45. Besides spalls on the winter SRG sections of SH6 and on another section, the only other type of distress identified in those surveys were patches.

The number of distresses that were counted in those surveys as well as the calculated PDI with the new equation are presented in Table 4.2. In this table, the sections that have a negative PDI have been overlaid, and those with higher values have not been overlaid, which shows that the calculated PDI is a good indicator of the need for an overlay.

*Table 4.2 Condition Surveys from 1998 and PDI for Houston Sections*

| Test Section | | Patches | Spalls | PDI |
|---|---|---|---|---|
| SH 6 | Summer SRG | 3 | 0 | **-0.21** |
| SH 6 | Summer LS | 0 | 0 | **1.00** |
| SH 6 | Winter LS | 0 | 0 | **1.00** |
| SH 6 | Winter SRG | 7 | 75 | **-1.96** |
| BW8 | Winter LS | 0 | 0 | **1.00** |
| BW8 | Winter SRG | 0 | 4 | **0.26** |
| IH-45 | Winter LS | 1 | 0 | **0.21** |
| IH-45 | Winter SRG | 0 | 0 | **1.00** |

The next case that was utilized to evaluate the new PDI equation also corresponds to a CRCP in the Houston area with a well-documented spalling problem. This section was used in this evaluation because it was considered an ideal case where the pavement was in good structural condition, had no patches or punchouts, but it had a significant spalling problem which required the placement of an overlay to rehabilitate it. The pavement in question is located on Beltway 8, in north Houston, between Greenspoint Dr., just east of IH-45, and Aldine Westfield, near the Houston Intercontinental Airport. This section was studied by the CTR in 1995 and 1996 because of its notorious spalling problem. The study is documented in [Trevino 1996]. The conclusion of the study at the time was that the section was structurally sound, and that the spalling had been caused by excessive evaporation rate at the time of the CRCP construction in 1984. To remedy the situation, a BCO was recommended; as the structural properties of the CRCP were adequate, a 2-in thick BCO was designed. The following information, shown in Table 4.3, includes the condition survey information from 1995, prior to the placement of the BCO, as well as the calculated PDI values for the subsections of this project. The results of the PDI calculations yield appropriate figures, as the values are all close to zero, except for the subsection without distresses. These values are considered very consistent and representative of the CRCP conditions at the time, which warranted the placement of a new BCO. Had the evaluation been conducted using the original PDI equation, all the values would have been 1.0 due to the absence of other types of distresses, and the results would certainly not have been representative of the pavement conditions.

*Table 4.3 Beltway 8 Condition Survey and PDI*

| Section | Length (ft) | Spalls | Spalls/mi | PDI |
|---|---|---|---|---|
| 1 | 700 | 0 | 0 | 1.00 |
| 2 | 2100 | 13 | 32.18 | 0.19 |
| 3 | 1600 | 34 | 112.63 | -0.10 |
| 4 | 700 | 4 | 32.18 | 0.19 |
| 5.1 | 1300 | 32 | 128.72 | -0.13 |
| 5.2 | 1300 | 36 | 144.81 | -0.16 |
| 5.3 | 1300 | 55 | 225.26 | -0.26 |
| 5.4 | 1200 | 51 | 225.26 | -0.26 |
| **Total** | **10200** | **225** | **116.60** | **-0.11** |

The application of the new PDI equation to the few cases presented in this study indicated very positive results, which confirms the adequacy of the spalling coefficient added to the PDI formula, making it a suitable decision criterion to determine whether a pavement needs an overlay.

## RATE OF FAILURES OCCURRENCE WITH TIME

In this section, the second component of the condition survey criterion is explained. The rate of occurrence of failures in a particular pavement is essentially a measurement of where that pavement is in relationship to its service life span. In other words, the rate of failures per mile per year is an indication of the pavement current stage of deterioration. As such, it can be used as an intrinsic indicator of the feasibility and the timeliness of different types of rehabilitation. The failure rate will signify what type of overlay is more conducive to address the current stage of structural decline of the pavement. The rationale sustaining this criterion is that any given CRCP at some point in its service life will, first, be an ideal candidate for an AC overlay. As time goes by and the deterioration rate increases, if no treatment were applied in the first instance, an AC overlay becomes no longer feasible, and the pavement turns into an ideal candidate for a BCO. In a similar fashion, further down in the life of the structure, at a more advanced stage of deterioration, had no rehabilitation been applied at the previous juncture, the pavement would become an ideal candidate for an unbonded concrete overlay. These fundamental concepts are substantiated in the structural and functional characteristics of each rehabilitation strategy as well in economic considerations.

The development of the rate of failures per mile per year as a decision criterion for overlaying is based on past research conducted at CTR. In this previous research, a criterion for BCOs and unbonded concrete overlays was established, based on the Texas experience. In Project 249 [Taute 1981], the results of condition surveys conducted on CRCPs during 1974 and 1978 were used to come up with a criterion for rigid pavement overlays. The 4398 project went a step further, taking that wealth of information and research as a building base for the development of a criterion for AC overlays.

The aforementioned study analyzed the history of failures of approximately 25 CRCP sections in Texas and found that whenever the annual failure rate for a particular pavement was below 3 failures per mile per year, it was economical to use a BCO, but when the rate surpassed 3, an unbonded overlay was the best decision. The study plotted charts similar to Figure 4.7 for the pavements investigated, containing the development of failures per mile with age for each section. The chart shown here is only conceptual, but the actual plots with the projects' data are documented in [Taute 1981]. Of course, every pavement has a different annual failure rate, and the shape of the curve varies from project to project, but the value of 3 failures per mile per year was found to be a breakpoint for selecting between bonded and unbonded overlays, the reason being that once this rate is reached, the cost of repairs was considered excessive for a BCO. As stated before, an unbonded overlay does not require extensive repair of the existing pavement.

To arrive at this conclusion, an economic analysis was performed. The distresses quantities were gathered from condition surveys on CRCPs in Texas, where defects included punchouts and patches. Average cost of repairs as well as user delay costs due to patching had to be estimated.

Originally, the breakpoint was defined in the study as the point when it is better to rehabilitate than to continue with the routine maintenance activities. This was designated as the point of economic failure: when the present value of maintenance costs and the corresponding user costs occurring over a period of time exceed the cost of the rehabilitation strategy that would last for the same length of time. In other words, the economic analysis entails comparing the present value of a rehabilitation strategy to the present value of continued maintenance. When the latter exceeds the former, the point of economic failure has been reached.

The point of economic failure can also be interpreted as the breakpoint between bonded and unbonded concrete overlays. It is this interpretation what is assumed in this discussion as the failure criterion, illustrated in Figure 4.7, to choose between both types of rehabilitation.

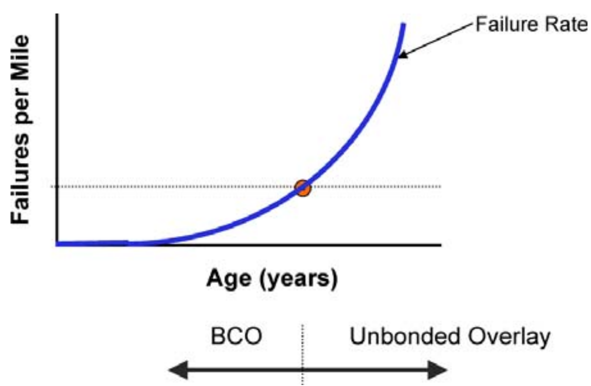



*Figure 4.7 Performance curve based on rate of failures per mile per year as criterion for bonded or unbonded overlays*

With this information, the next step was to determine a breakpoint between AC overlays and BCOs. Going back to the fundamental concept on which this criterion is based, explained at the beginning of this section, first, it is known that, for an AC overlay to be successful, it has to be applied to a pavement that is structurally sound, much like a BCO. Secondly, the thin AC overlay will not provide any structural enhancement to the CRCP, unlike the BCO. Thus, in general, the original CRCP has to be in better shape than it has to be for a BCO placement, therefore, the threshold value has to be less than the breakpoint between BCO and unbonded concrete overlay. In other words, the rate of failures per mile per year will occur earlier in the life of the CRCP for a thin AC overlay than for a BCO. The economical decision for an AC overlay will have to occur earlier in the CRCP life before the BCO becomes the best economic alternative.

Much like stating that unbonded concrete overlays, while not the best economical solution, can be applied early in the life of the pavement and still perform well, the same can be said for the case of a BCO that is applied too early when a thin AC overlay is the best economic alternative.

Knowing that the breakpoint value for an AC overlay happens earlier than 3 failures per mile per year, the research team looked for cases in which the sections in question had a well-documented condition survey history and that had been rehabilitated with thin AC overlays. The section of IH-30 in Bowie County, presented in the following section, represented an ideal case to fit the needs of this research endeavor. This project had been closely followed and studied for a number of years, as it has been the subject of various AC overlays. Another benefit of analyzing this project is that there is plenty of information from some sections of this project that has been stored and studied in a pavement database.

### *IH-30 in Bowie Co. (Project 1342)*

This 10-mile section was originally constructed in 1972. It consists of 8-in thick CRCP slab on a cement treated subbase. The coarse aggregate is SRG. In April of 1986, an AC overlay was placed to reduce the long wavelength roughness of the CRCP surface caused by swelling clay movements, which produced significant dynamic impact loadings of heavy trucks moving at high speeds, which in turn, increased the incidence of failures. With the new overlay, the smoother pavement experienced a reduction stresses equivalent to normal dynamic loadings.

By June of 1993, the AC overlay had shown signs of deterioration, so it was rotomilled and a new 2-in. thick AC overlay was placed. The CRCP condition was examined before milling, after milling, and after the new overlay was in place.

A research study on this section was conducted by CTR in 1993 and 1994, which is documented in [McCullough 1994].

In that report, the history of failures per mile is documented before and after the initial AC overlay was constructed. Figure 2.2 in that report presents the historic failures per mile for the section. At the time of the AC overlay placement, it corresponded to a figure of 1.8 failures per mile per year. Furthermore, Figures 2.3 through 2.6, from that reference, offered the opportunity to analyze the same data in more detail. Those graphs present the historic failures per mile for the subsections of the entire project (Sections I through IV) from which it is observed that the subsections had less than two failures per mile per year, except for one. The values are as follows (Table 4.4).

*Table 4.4 IH-30 Bowie Co. History of Failures Per Mile*

| Subsection | Failures per mile per year |
|---|---|
| I | 1.43 |
| II | 1.60 |
| III | 2.77 |
| IV | 1.40 |
| Overall | 1.80 |

***IH-30 in Bowie Co. (Rigid Pavement Database)***

CTR maintains a Rigid Pavement Database (RPDB), which contains historic information collected for about 25 years on concrete pavements in Texas. This database contains information about the characteristics of numerous pavement sections that represent the conditions existing throughout the entire Texas concrete pavement network. Many features are collected from each of the sections in these archives, like pavement type, construction dates, location, overlay history, coarse aggregates, and condition survey historic information. For the purpose of this study, it was considered ideal to analyze the historic information available in the RPDB for the IH-30 section in Bowie Co. The sections archived in the database do not correspond exactly to the same segment studied in Project 1342 mentioned in the previous paragraphs. Nevertheless, the RPDB sections are all within the project limits. The project limits are from milepost 188 to milepost 198; the RPDB sections lie between mileposts 194 and 198, all on the westbound direction.

According to the RPDB, condition surveys on these sections were conducted in 1974, 1978, 1980, 1982,1984, 1987, 1994, and 1996. The failure history collected during those surveys is shown in Figure 4.8.

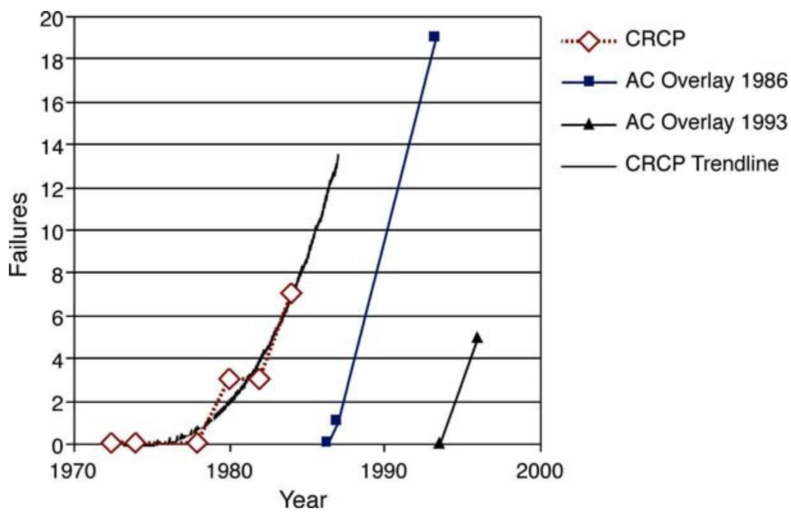



*Figure 4.8 IH-30 in Bowie Co. history of failures from RPDB*

On average, the rate of failures per mile per year for these RPDB sections is 2, which concurs with the findings of the overlay performance case study.

From these analyses of IH-30, it was concluded that a value of 2 failures per mile per year or less is an appropriate criterion to indicate that the construction of an AC overlay is a feasible solution. Thus, Figure 4.9 conceptually puts together this decision criterion with the analogous boundary value for BCOs and unbonded concrete overlays in a graphic way.

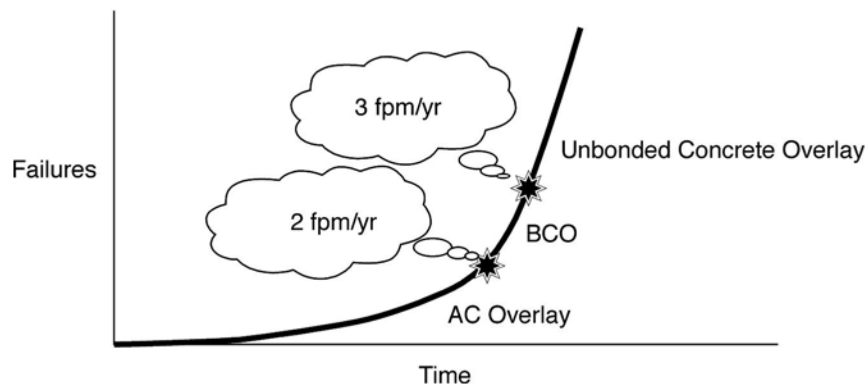



*Figure 4.9 Decision criteria for AC overlays, BCOs and unbonded concrete overlays based on failure rate*

**SUMMARY**

In this chapter, the development of a twofold decision criterion for the placement of thin AC overlays on CRCP based on condition surveys was presented. The two components of this criterion are the PDI and the rate of development of failures with time. Both approaches entail using data on the incidence of failures, but they analyze it in a different way. The rationale behind analyzing available information by two different approaches is to give the pavement engineer more elements to substantiate a better decision. Furthermore, this dual procedure offers the opportunity of utilizing whatever amount of pavement condition survey information is available, since it is recognized that in many cases it is impossible to have all the historic condition survey information.

The PDI is used as a discriminant score to determine when a section needs to be overlaid. The PDI considers various types of distresses, assigning them relative weights in an equation in which the computed index becomes the discriminant score. The original PDI equation, developed in the 1980s based on field data gathered across the state, was enhanced to include the incidence of spalling. A number of pavement sections were utilized to determine and calibrate a spalling coefficient to be included in the PDI equation.

The failure rate is intrinsically related to the timeliness of an overlay; it is utilized as an indicator of when a section needs rehabilitation and depending on the stage of deterioration, the failure rate will signify what type of overlay is more conducive to address the amount of distresses. The basic postulation that authenticates this criterion is that any given CRCP at some point in its service life will first become an ideal candidate for an AC overlay. As time goes by and the deterioration rate increases, if no treatment were applied in the first instance, then the pavement becomes an ideal candidate for a BCO. In a similar fashion, further down in the life of the pavement, at a more advanced stage of deterioration, had no rehabilitation been applied, the structure would become an ideal candidate for an unbonded concrete overlay.

If a CRCP approaches a rate of failure development of 2 failures per mile per year, an AC overlay is likely to remedy the situation and deliver good performance. If the rate is closer to 3 failures per mile per year, a BCO represents a better technical and economical strategy. If the deterioration rate has reached beyond 3 failures per mile per year, the section is already too damaged to be economically repairable by a BCO. The cost to fix those distresses prior to the BCO placement will make this too expensive of a rehabilitation option. In this case, the best strategy is an unbonded concrete overlay.

# CHAPTER 5. DEFLECTION CRITERION

This chapter presents the third decision criterion for the selection of a thin asphalt concrete (AC) overlay on continuously reinforced concrete pavements (CRCP), the deflection criterion. This criterion is part of the decision tree, which consists of a series of systematic steps, evaluations and decisions to assist the pavement engineer in the project selection stage of a pavement rehabilitation project of this kind. The decision tree was introduced in Chapter 2 of this report. Chapter 3 presented the first decision element developed for the selection of an AC overlay, the profile criterion. Then, in Chapter 4, the second major component of the decision tree was presented, the condition survey criterion. In this chapter, the details of the development of the third assessment, the deflection criterion, will be presented.

The recommended sequence of steps in the decision tree, as explained in the previous chapters, is to evaluate first the profile criterion, a functional assessment, followed by the condition survey criterion, and by the deflection criterion, both of which reflect the structural characteristics of the pavement.

## DEFLECTIONS

An invaluable tool in assessing the structural capacity of the pavement is the measurement of deflections. Deflection measurements are normally performed by means of several types of non-destructive testing devices, among which the most common is the Falling Weight Deflectometer (FWD). In the past, other frequently used devices were the Benkelman beam, Dynaflect and Road Rater, but nowadays most agencies use FWD.

The deflection criterion is based on two components, which are stress calculations and deflection measurements taken at the cracks and at the midspan of pavement slabs. These components are expressed as ratios, the deflection ratio and the stress ratio. The next sections explain these ratios, followed by how they are put together to become a decision element for AC overlays on CRCP.

## DEFLECTION RATIO

To compute the deflection ratio two types of deflection information should be collected. Measurements should be taken along continuous slabs of pavement, with Sensor Number 1 positioned at the midspan between two cracks; the second type of measurements should be conducted across transverse cracks. For this kind of measurement, it is recommended to arrange the FWD sensors with respect to the crack in the way illustrated in Figure 5.1, in which Sensor Number 1 is positioned next to the crack.

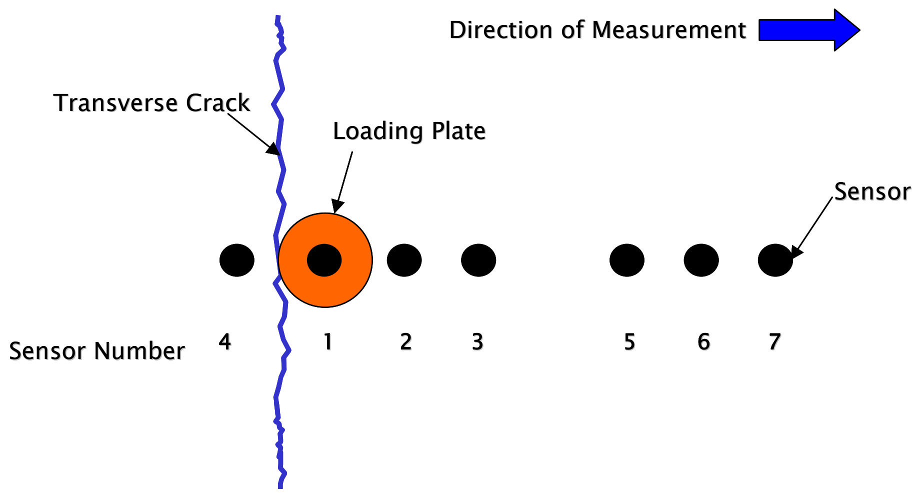



*Figure 5.1 FWD downside loading and sensor arrangement for deflection at crack measurement (plan view)*

The deflection ratio is expressed as follows:

$$\textbf{Deflection Ratio} = \frac{\textbf{Deflection at crack}}{\textbf{Deflection at midspan}}$$

Both measurements correspond to Sensor Number 1. Because of the effect of temperature on deflections, it is recommended to conduct the measurements in the morning, when the temperatures are lower, before crack openings might get tighter with slab thermal expansion. The deflection ratio is indeed a measurement of load transfer. It is inversely proportional to the structural capacity of the pavement. In an ideal case, both deflection measurements (at crack and at midspan) would be the same for a given location; this would mean that the load transfer at that spot is 100 percent, sign of good structural capacity. However, for a normal pavement, in reality, most likely the ratio will be greater than 1. It is unlikely that the deflection at crack is smaller than that at the midspan, in which case the ratio would be less than 1.

If the ratio is less or equal to 1, the pavement's structural capacity does not require rehabilitation. Some type of overlay rehabilitation may be required for deflection ratios greater than 1. The more the ratio deviates from a value of 1, the more of a structural remedy is required, because this ratio equates to the structural condition. Much like for the condition survey criterion, explained in the previous chapter, the three overlay types can be ranked by the structural benefits they achieve, and by their economical feasibility. First in the ranking will be the thin AC overlay (minimal structural benefit, very low cost), then the BCO (considerable structural improvement for an existing pavement that is not too deteriorated at a higher cost than a thin AC overlay) and finally the unbonded concrete overlay (for cases that need considerable structural improvement, and the most expensive of the three alternatives). The underlying principle for the application of this criterion is related to the stage of the pavement's service life. It is expected that as the pavement deteriorates, first, it will be enough with an AC overlay for a condition in which negligible or minimal structural improvement is demanded. These overlays are placed mostly for functional enhancements. If no overlay is applied at this instance, the pavement will continue to deteriorate, and at some point, the best economical solution will be a BCO. Finally, for the more deteriorated conditions, when the cost of pre-overlay repairs will be too high to consider a BCO, the unbonded concrete overlay becomes the best solution. Hence, it is expected that as the deflection ratio deviates from 1, a more expensive and a more radical structural improvement will be necessary.

## STRESS RATIO

In an analogous development, a stress ratio can be defined to ascertain the structural contribution of an overlay, if the stresses calculated with and without an overlay are compared. The stress ratio is defined as follows.

$$\textbf{Stress Ratio} = \frac{\textbf{Stress without overlay}}{\textbf{Stress with overlay}}$$

Based on the principle that a thin AC overlay placed on top of a CRCP will provide a minimal structural contribution, this ratio is expected to be close to 1 for a rehabilitation of this kind. Higher values that depart more from a ratio of 1 will indicate that a different kind of rehabilitation strategy is more appropriate, one that provides some structural contribution, such as a BCO or an unbonded concrete overlay. Since this evaluation is conducted during the project selection stage, the calculations should be performed from a theoretical standpoint. Thus, the stresses should be estimated mechanistically, and the proposed procedure is to compute them by means of elastic layer theory. A thickness for the AC overlay is assumed and the stresses with and without this layer are found. It is expected that a more structural type of rehabilitation is more necessary the further the ratio departs from 1.

A conceptual illustration of the deflection criterion, plotting both the stress ratio and the deflection ratio, representing the areas of adequacy of each of the three rehabilitation strategies mentioned is shown in Figure 5.2. Cases in which the ratios have values less than 1 do not require an overlay, at least from the structural standpoint.

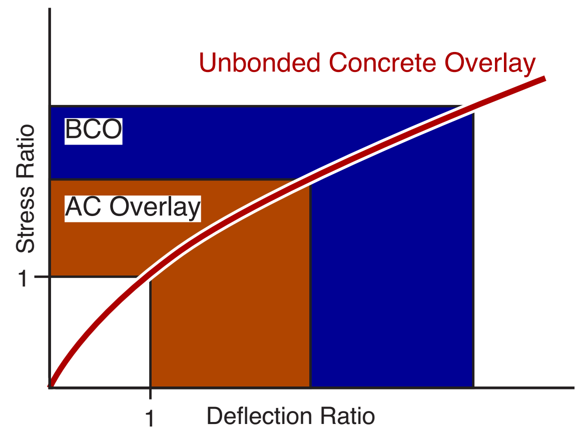



*Figure 5.2 Applicability of different types of overlay according to a deflection criterion*

To define the threshold values, in the following section an analogous criterion developed for BCOs and unbonded concrete overlays is introduced, which will serve as a background for the subsequent discussion, followed by a section presenting a case study.

***Deflection Criterion for BCO vs. Unbonded Concrete Overlay***

A previous research study conducted by CTR, Project 1205 [Van Metzinger 1991] investigated a criterion similar to this, to distinguish between the feasibility of BCOs and unbonded concrete overlays. The deflection ratio proposed in the 1205 study is defined in the same way as mentioned in the previous section, i.e., ratio of deflection at crack to deflection at midspan. This criterion also developed a stress ratio. However, the stress ratio is defined in a different way, since both the BCO and the unbonded concrete overlay are capable of providing substantial structural contributions. The ratio, as defined in Project 1205, compares the maximum tensile stresses at the bottom of the overlay to the maximum transverse stresses at the bottom of the existing pavement.

For the stress computations, low and high moduli of elasticity concrete were assumed, as well as three different thicknesses for the existing pavement, 8, 10 and 12 inches. An existing pavement stiffness of 4,500 ksi was utilized for the low-modulus concrete, and 6,000 ksi was used for the high-modulus concrete. This study concluded that a BCO is feasible when the deflection ratio is less than 1.7 for 8 and 10-in. pavements, and less than 1.85 for 12-in. thick concrete (Figure 4.4). Similarly, for a high-modulus concrete overlay, the placement of a BCO is advisable if the deflection ratio is less than 1.25 for 8 and 10-in. pavements and less than 1.40 for 12-in. thick pavement

Load transfer is reduced at the cracks, where the transverse stress becomes the critical stress. When the overlay is placed, among other benefits, it reinstates the load transfer capability of the structure. Nonetheless, if the stresses at the bottom of the overlay are still high, cracks will appear in the overlay, the structure will deteriorate and reach the original condition, and the original cracks will reflect in the overlay.

For the overlay rehabilitation to be cost-effective the stresses at the bottom of the overlay must be below the maximum transverse stress at the bottom of the existing pavement, otherwise the overlay will crack.

The deflection ratio for existing pavements was plotted versus the stress ratio. The graphs with the ratios of stresses and deflections are shown in Figures 5.3 and 5.4, for low-modulus and high-modulus, respectively. From these plots, it was concluded that when a low-modulus concrete overlay is used, a BCO is feasible when the deflection ratio is less than 1.7 for 8 and 10-in. pavements, and less than 1.85 for 12-in. thick concrete (Figure 5.3). Similarly, for a high-modulus concrete overlay, the placement of a BCO is advisable if the deflection ratio is less than 1.25 for 8 and 10-in. pavements and less than 1.40 for 12-in. thick pavement (Figure 5.4). These limits are found by the intersection of a stress ratio of 1 with the respective curves. Cases rendering values beyond these limits are better suited for unbonded concrete overlays.

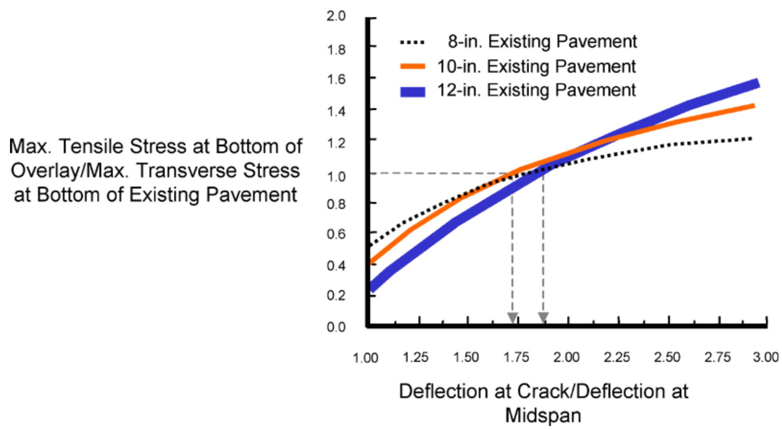



*Figure 5.3 Stress ratio versus deflection ratio for low-modulus overlay concrete as criterion for BCO selection*

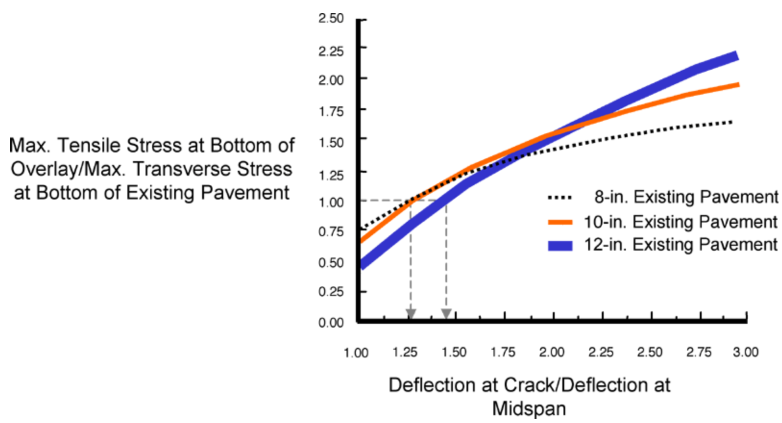



*Figure 5.4 Stress ratio versus deflection ratio for high-modulus overlay concrete as criterion for BCO selection*

These threshold values were taken into consideration in the development of the deflection criterion for thin AC overlays on CRCP, and additional threshold values were determined from the following case study.

***Case Study: IH-20 in Harrison Co.***

The 3.4-mi. long CRCP section, referred in Chapter 3, located on IH-20 in Harrison Co., near Marshall, was used extensively throughout Project 4398 as a source of valuable information for various analyses, especially deflection data. This section has been overlaid with AC on several occasions. One of those overlays was removed in 2001. Shortly thereafter, a new AC overlay was placed in December of that year. Deflection tests were performed at three stages during this period: before the original overlay was removed, while the CRCP was exposed after the overlay removal, and after the new overlay was in place.

The structure in this section consists of 8-in. thick CRCP, on top of 7 in. of cement-stabilized subbase, placed on 6 in. of cement-treated subgrade. The new AC overlay is 4-in. thick. The cross-sectional view of the structure is illustrated in Figure 5.5.

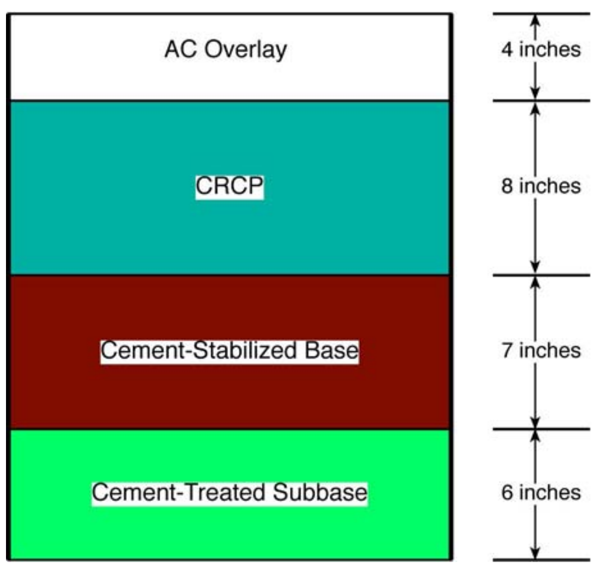



*Figure 5.5 Cross sectional view of the IH-20 structure in Harrison Co.*

Table 5.1 presents the stresses computed along with some of the layer properties assumed for those elastic-layer theory stress calculations. The elastic layer calculations were performed with the ELSYM 5 program [Hicks 1982], and the moduli of elasticity for the layers were backcalculated from FWD measurements.

*Table 5.1 Values for elastic layer theory stress calculation*

| Layer | Thickness (in.) | Modulus (ksi) | Poison's Ratio | Stresses (psi) | |
|---|---|---|---|---|---|
| | | | | With Overlay | Without Overlay |
| AC O/L | 4 | 200 | 0.35 | 21.5 | 27.46 |
| CRCP | 8 | 1000 | 0.22 | | |
| CTB | 7 | 500 | 0.30 | | |
| CTSB | 6 | 150 | 0.35 | | |
| Fill | 6 | 30 | 0.40 | | |
| Subgrade | Semi-infinite | 15 | 0.40 | | |

The calculation of ratios is summarized in Table 5.2. Deflections corresponding to the eastbound direction had to be disregarded due to some missing data in the deflection files.

*Table 5.2 Summary of stress and deflection ratios calculations*

| Stress Ratio | | |
|---|---|---|
| Stresses (psi) | | Ratio |
| Stress with overlay | 21.50 | 1.28 |
| Stress without overlay | 27.46 | |
| **Deflection Ratio** | | |
| Deflection (mils) | | Ratio |
| WB Outside Lane: | | 1.06 |
| Average Deflection at cracks | 3.08 | |
| Average Deflection at midspan | 2.90 | |
| WB Inside Lane: | | 1.15 |
| Average Deflection at cracks | 3.24 | |
| Average Deflection at midspan | 2.83 | |

The results from Table 5.2 have been plotted in Figure 5.6, using the value of 1 as a reference for both the stress ratio and the deflection ratio. Any departure from these reference values indicates a need for a more structural solution to the rehabilitation decision. In this case, the IH-20 segment analyzed presented deflection ratios for both the outside and inside lanes that were fairly close to 1, indicating that the load transfer in the CRCP signals a structurally sound pavement. The same can be said for the stress ratio of 1.28, which implies that the structural contribution of the overlay to the existing pavement will be very small.

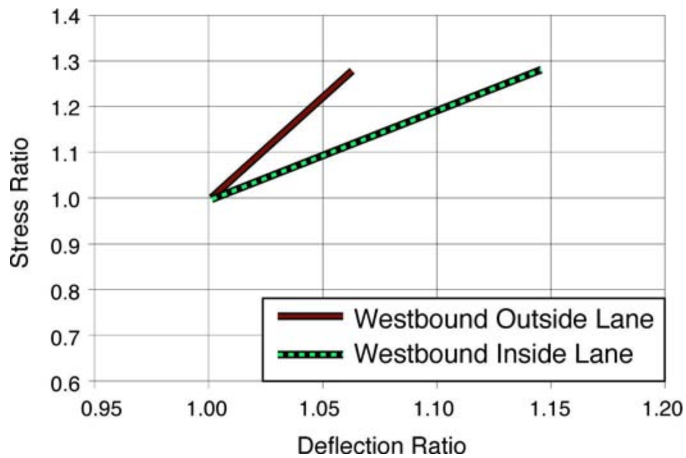



*Figure 5.6 Stress and deflection ratios for IH-20 section near Marshall*

Acknowledging that one case study is not a statistically significant basis to establish a general guideline, but taking into account the results from Project 1205, presented in the previous section, it was found that the results from the IH-20 section studied in this project correspond to what was expected for a thin AC overlay deflection criterion. The deflection criterion threshold values for an AC overlay, therefore, can be defined, with the amount of information available to this point, as a stress ratio between 1 and 1.3, and a deflection ratio between 1 and 1.2. Values greater than these will warrant the consideration of a BCO, or, if the threshold surpasses those established for BCOs, of an unbonded concrete overlay.

## SUMMARY

In this chapter, the development of a decision criterion, based on deflections, for the selection of a thin AC overlay on CRCP was presented. This criterion implies a structural evaluation of the existing pavement, but it also engages a theoretical assessment of a future overlay and its structural contribution, with the computation of stresses. The deflection criterion is integrated by two components, a deflection ratio, and a stress ratio. The deflection ratio requires measurement of deflections at both the midspan and at cracks on the CRCP. The ratio of deflections at cracks to deflections at midspan is an indicator of the structural integrity of the existing pavement. An elastic layer theory calculation of stresses will provide information for the stress ratio, which measures the hypothetical structural contribution of the overlay, by comparing the stresses without the overlay to the stresses with the overlay. The deflection criterion will indicate whether an AC overlay would be a good solution, if both the deflection ratio and the stress ratio are close enough to 1, or if a more structural remedy is necessary. In general, a considerable departure from a value of 1 for both ratios signifies that the pavement needs an overlay that can provide more structural benefits than an AC overlay.

# CHAPTER 6. TACK COAT AND ASPHALT INVESTIGATION

This chapter presents the results of the second part of the thin-bonded asphalt concrete (AC) overlays placed on existing portland cement concrete pavements (PCCP) study, which deals with laboratory experiments on tack coats and AC mixes. The first part of the investigation, presented in the previous chapters, addressed the development of decision criteria for the selection of a project of this nature.

The chapter outlines the objectives and scope of the asphalt part of the study. The laboratory procedure and experiments undertaken to evaluate tack coat interface shear strength are then reported. This is followed by the results of the Model Mobile Load Simulator (MMLS3) and the Hamburg Wheel Tracking Device (HWTD) tests to identify rut resistant asphalt mixtures. The findings of the study were used to develop a methodology for evaluating the suitability of asphalt mixtures for overlays on continuously reinforced concrete pavement (CRCP).

Modes of failure investigated include permanent deformation and stripping using wheel tracking devices, i.e., the HWTD, and the MMLS3, and the interface shear strength performance of tack coats used to bond asphalt concrete to CRCP. The latter addresses to an extent debonding and slippage cracking failures. Reflection cracking failures are not addressed.

## ASPHALT STUDY OBJECTIVES AND SCOPE

The primary objective of the AC part of the study was the development of a methodology for evaluating the suitability of AC mixtures for use as overlays on CRCP. Resistance to permanent deformation and tack coat interface shear strength were identified as primary influence factors and were investigated separately.

To evaluate the interface shear strength of tack coats used to bond composite AC and CRCP, an objective of the study was to determine the influence of and interaction between three asphalt concrete mixture types (Type D –a smooth mix almost exclusively used for surface applications, such as overlays, with a maximum aggregate size of ½-in., PFC – Porous Friction Course, and CMHB – Coarse Matrix-High Binder), three tack coat types (SS-1, CSS-1h and AC-10), three tack coat application rates (0.04 gal/yd$^2$ (0.22 L/m$^2$), 0.08 gal/yd$^2$ (0.43 L/m$^2$) and 0.12 gal/yd$^2$ (0.65 L/m$^2$)) and Hamburg wheel tracking (0, 10000 and 20000 cycles) on tack coat shear strength towards the selection of appropriate tack coats and application rates to be used for specific AC overlay mixes on CRCP pavements in Texas. A four-factor, three-level partial factorial experiment was designed and analyzed using STATISTICA [StatSoft 1997]. Four replicates were tested at each factor combination of mix type, tack coat type, application rate and Hamburg cycles. Thus in total, the experiment required testing of 108 composite specimens. Factors not considered in the study, primarily because of time constraints, include temperature and aggregate type. The relevance of these is yet to be considered.

An objective established as part of the study was the development of a practical laboratory test procedure that could be implemented towards evaluation of the shear strengths of tack coat interfaces between AC and PCCP bonded specimens. Shear strengths of composite specimens were determined using direct shear testing to be described shortly. A further objective was to determine which of the shear strength parameters measured as part of the laboratory test procedure best defined the nature of the tack coat interface between the composite specimens tested and the interactions between the influence factors evaluated.

The objective of the wheel tracking tests was to evaluate the suitability of different mixture and aggregate types for use as overlay materials on CRCP. AC mixtures used as overlays on these structures are subjected to high compressive stresses. Suitable mixtures must therefore be able to resist these stresses.

## TACK COAT INTERFACE SHEAR STRENGTH

Reported research on the shear performance of tack coats has focused primarily on the interface characteristics between asphalt layers. A study [Uzan 1978] evaluated the direct shear resistance of a neat asphalt binder (Pen 60/70) tack coat. Direct shear tests at a constant shearing rate of 0.1 in./min (2.5 mm/min) were done at 77 °F (25 °C) and 131 °F (55 °C), and optimum tack coat application rates were identified to maximize shear resistance at these temperatures.

Another study [Mrawira 1999] reports shear testing at a constant rate of 0.04 in./min (1 mm/min) and 72 °F (22 C) to investigate the influence of an emulsion grade SS-1 tack coat between freshly paved asphalt layers. Contrary to expectations, it was found that non-tacked overlays exhibited slightly higher maximum shear strengths than tack-coated overlays.

The influence of asphalt tack coat materials on the shear strength of interfaces between asphalt layers was recently investigated and reported [Mohammad 2002]. In this study, the goal was to investigate the influence of four different emulsions and two PG grade binders used tack coats, five different tack coat application rates ranging from 0 gal/yd$^2$ to 0.2 gal/yd$^2$ (0.9 L/m$^2$), and test temperatures of 77 °F (25 °C) and 131 °F (55 °C). Simple shear tests using the Superpave shear tester (SST) were done by applying a shearing load at a constant rate of 50 lb/min (222.5 N/min). These results indicated that the CRS-2P emulsion evaluated was the best tack coat type and 0.02 gal/yd$^2$ (0.09 L/m$^2$) was the optimum application rate at which maximum interface shear strength was measured for both test temperatures.

This part of the 4398 study evaluated the interface shear strength of tack coats between asphalt concrete and PCCP specimens. Poor tack coat performance was identified as a critical factor responsible for premature failures of AC overlays over CRCP, typically resulting in debonding and delamination. A laboratory test procedure was developed to evaluate the shear strength of tack coats. It was implemented as part of a partial factorial experiment designed to investigate the influence of (1) AC mixture type, (2) tack coat type, (3) tack coat application rate, and (4) Hamburg wheel tracking on tack coat shear performance.

The reason for evaluating these factors is discussed briefly. The experimental procedure followed is then outlined. Materials and mixture designs, as well as specimen preparation procedures are then discussed. The results of Hamburg wheel tracking and direct shear tests done to evaluate tack coat performance are then reported. A statistical analysis of the test results was undertaken to determine significant factors and interactions influencing tack coat interface shear strength performance.

## TACK COAT PERFORMANCE INFLUENCING FACTORS

AC mixtures used for thin overlays on CRCP in Texas range from the traditional Texas Type C and D mixes to rut resistant mixes such as PFC and CMHB AC. The relative performance, appropriate tack coats and optimum application rates for different overlay mixture types is not always obvious. Given the differences in surface areas apparent between a coarser PFC mixture and a finer Type D mixture, differences in tack coat performance are likely to be expected and occur.

Emulsified asphalt binders have essentially replaced the cutback asphalt binders (such as RC 250) historically used in Texas. Emulsified asphalts such as SS-1, SS-1h and CSS-1h make up the vast majority of the asphalt binders used for tack coats. Neat asphalt binders such as AC 5, AC 10 and their performance grade equivalents are also used.

Residual application rates specified for emulsions range from 0.03 gal/sq. yd (0.16 L/m$^2$) to 0.07 gal/sq. yd (0.38 L/m$^2$). Tack coat application rates for specific mixes to be used as overlays on CRCP are generally left to the discretion of the engineer. Flexible Pavements of Ohio [Ohio 2001] has published guidelines suggesting typical application rates for different surfaces. They recommend a residual application rate of between 0.04 gal/sq. yd (0.22 L/m$^2$) and 0.06 gal/sq. yd (0.33 L/m$^2$) for tacking PCCPs.

The possibility of tack coat stripping or degradation under trafficking has not been investigated in depth. Evidence of tack coat stripping was found under Model Mobile Load Simulator (MMLS3) testing of AC-PCCP composite specimens performed as part of the study. Results of these tests are discussed later in the report. Hamburg wheel tracking of composite AC-PCCP specimens, the preparation of which is discussed later in the report, was included as part of the laboratory procedure developed to evaluate tack coat performance and the influence thereof was investigated as part of the designed experiment.

## EXPERIMENTAL DESIGN

Given the factors outlined it was proposed to undertake a partial factorial experiment to evaluate tack coat performance. The 4-factor 3-level experimental matrix developed for the study is shown in Table 6.1.

*Table 6.1 Design matrix for tack coat performance evaluation*

| **Factor/Level** | Low | Medium | **High** |
|---|---|---|---|
| Mix Type | Type D | CMHB | PFC |
| Tack Type | Emulsion | AC-10 | Emulsion |
| Application Rate | Low | Med | High |
| Hamburg Cycles | 0 | 10000 | 20000 |

The low, medium and high application rates correspond to residual tack coat rates of 0.04 gal/sq.yd (0.18 L/m$^2$), 0.08 gal/sq. yd (0.36 L/m$^2$) and 0.12 gal/sq.yd (0.54 L/m$^2$) respectively. Four replicates were produced at each of the factor combinations. A total of 108 specimens were tested.

## MATERIALS, MIXTURE DESIGN AND SPECIMEN PREPARATION

### *Materials*

The tack coat materials used in the study include two slow setting emulsions (SS-1 and CSS-1h) and a neat AC-10 asphalt binder, all supplied by Koch. Table 6.2 outlines some of the rheological properties of the tack coats used. The AC mixes evaluated as part of the study include a Type D, a CMHB and a PFC mixture. These were obtained from asphalt plants around Texas.

### *Mixture design*

The gradations of the AC mixes are shown in Table 6.3 together with other relevant mix design information. The PFC mix was manufactured with one percent lime and 0.4 percent fibers.

*Table 6.2 Tack coat properties*

| Property/Tack coat | SS-1 | CSS-1h | AC-10 |
|---|---|---|---|
| Viscosity, Saybolt Furol @ 77 °F, sec | 22 | 26 | - |
| Viscosity, 140 F, poises | - | - | 1010 |
| Viscosity, 275 F, poises | - | - | 2.1 |
| Residue by distillation, % by weight | 62 | 62 | - |
| Pen @ 77 °F, 100 g, 5 sec | 134* | 104* | 90 |
| Ductility @ 77 °F, 5 cm/min, cm | 124* | 70+* | 100+ |

* Tests on residue from distillation.

*Table 6.3 AC mix gradations and mix design information*

| Property/Tack coat | Type D | CMHB | PFC |
|---|---|---|---|
| Sieve size: English (Metric) | | | |
| 3/4 " (19 mm) | 100.0 | 100.0 | 100.0 |
| 1/2 " (12.5 mm) | 100.0 | 99.9 | 90.4 |
| 3/8 " (9.5 mm) | 91.8 | 62.5 | 56.5 |
| No. 4 (4.75 mm) | 63.0 | 32.8 | 10.6 |
| No. 8 (2.36 mm) | 37.0 | 20.3 | 7.4 |
| No. 40 (0.425 mm) | 20.0 | 11.1 | 4.7 |
| No. 80 (0.18 mm) | 8.4 | 8.2 | 4.0 |
| No. 200 (0.075 mm) | 3.8 | 6.3 | 2.9 |
| Asphalt binder | PG 64-22 | PG 64-22 | PG 76 -22 TR |
| Specific gravity of binder | 1.030 | 1.030 | 1.024 |
| Binder content, % by mass | 5.7 | 5.2 | 6.0 |
| Rice gravity | 2.432 | 2.392 | 2.442 |

***Specimen preparation***

AC collected from the asphalt plants was stored at room temperature before use. The AC was reheated to a temperature of 259 °F (126 °C), typically over a 4-hour period and gyratory compacted to a fixed height of 2 in. (50 mm) and a diameter of 6 in. (150 mm). The mass of material required to achieve this height was determined beforehand using a trial and error procedure to ensure that the Type D and CMHB specimens had voids in the mix (VIM) of 7 percent and the PFC had VIM of 20 percent. As part of this procedure the maximum theoretical or Rice's density of the mixes is determined. In addition, duplicate samples at four different masses were compacted and the bulk densities of the compacted specimens determined. Compacted specimens were allowed to cool overnight after which densities were measured. Only Type D and

CMHB specimens with VIM of 7 % ±1 % and PFC specimens with VIM of 20 % ±1 % were selected for testing purposes.

The asphalt specimens had to be sawn for the Hamburg test configuration in which two specimens are placed side-by-side as shown in Figure 6.1. The sawn specimens were tacked directly to concrete disks having a thickness of 1 in. (25 mm) also sawn as indicated. The concrete disks were obtained by sawing 6 in. (150 mm) diameter concrete cores. The concrete cores were sawn with a fine blade and at a slow rate to ensure a smooth uniform contact surface.


$r := 75 \cdot mm$

$\theta := 60 \cdot deg$

$$Area := \frac{-1}{2} \cdot r^2 \cdot (\theta - \sin(\theta)) + \pi r^2$$

*Figure 6.1 Specimen configuration for Hamburg testing and associated surface area calculations*

The mass of tack coat required to achieve the required application rates was determined based on the surface area to be coated, calculated as shown in Figure 6.1. In the case of the emulsions it was also necessary to account for the residual binder contents. The tack coats were spread evenly over one surface of a concrete disk and in the case of the emulsions, allowed to break before the asphalt concrete specimen was attached. The AC-10 asphalt binder was heated to a temperature of 275 °F (135 °C) before applying the tack coat to the concrete specimens, which were also heated to 275 °F (135 °C) to prevent stalling of the binder during tack coat application. Once tacked, the composite AC-PCCP specimens were loaded by applying a pressure of 100 psi (690 kPa) using a Texas Gyratory compactor and maintaining this pressure on the specimens for a period of 5 minutes to improve the bond between the AC and PCCP specimens. A rubber pad was placed beneath the concrete disks to prevent cracking of the disks during this loading period. The prepared specimens were then placed within an environmental chamber set to a temperature of 77 °F (25 C) for a period of at least 48 hours before HWTD and/or direct shear testing.

Clearly the specimen preparation procedure as outlined differs from tacking procedures used in the field. An alternative approach considered was the compaction of the asphalt mixes directly onto tacked concrete disks placed within the gyratory compactor molds. This would require the use of specially prepared concrete disks having diameters small enough to fit within the molds and to account for the inclined gyratory angle during compaction.

## HAMBURG WHEEL TRACKING AND SHEAR TESTING

Hamburg tests were done using TxDOT Test Method Tex-242-F but at a temperature of 122 °F (30 °C) to minimize rutting of the composite specimens. After Hamburg wheel tracking, the tested specimens were placed in the environmental chamber and left at a temperature of 77 °F (25 °C) for at least 24 hours before shear testing.

Shear tests were done using a Marshall press modified to allow shearing of the composite specimens along the asphalt-concrete interface as shown in Figure 6.2. The concrete disks were clamped and the load was applied vertically to the asphalt specimens shearing the tacked halves apart. These tests were done within the environmental chamber at a temperature of 77 °F (25 C). The specimens were sheared at a constant displacement rate of 2 in./min (50.8 mm/min), the standard Marshall speed. The data acquisition system included a National Instruments controller to which the load cell was connected. The load signal was sampled every 0.05 seconds using software timing and the Lab View data acquisition software. This setup allowed a continuous force-displacement response to be captured.

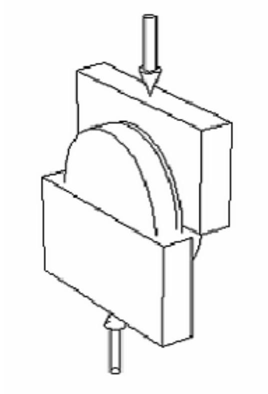


*Figure 6.2 Direct shear testing configuration*

***Test Results***

Shear stress ($\tau$) was computed based on the shearing load (P) and the tacked area (shown in Figure 6.1) as $\tau$=P/Area. Given that the tacked area is decreasing during testing as the composite halves shear apart, the shear stresses as reported are slightly on the conservative side. Shear stress versus displacement curves were plotted for each tested specimen. Typical shear stress response curves are shown in Figure 6.3.

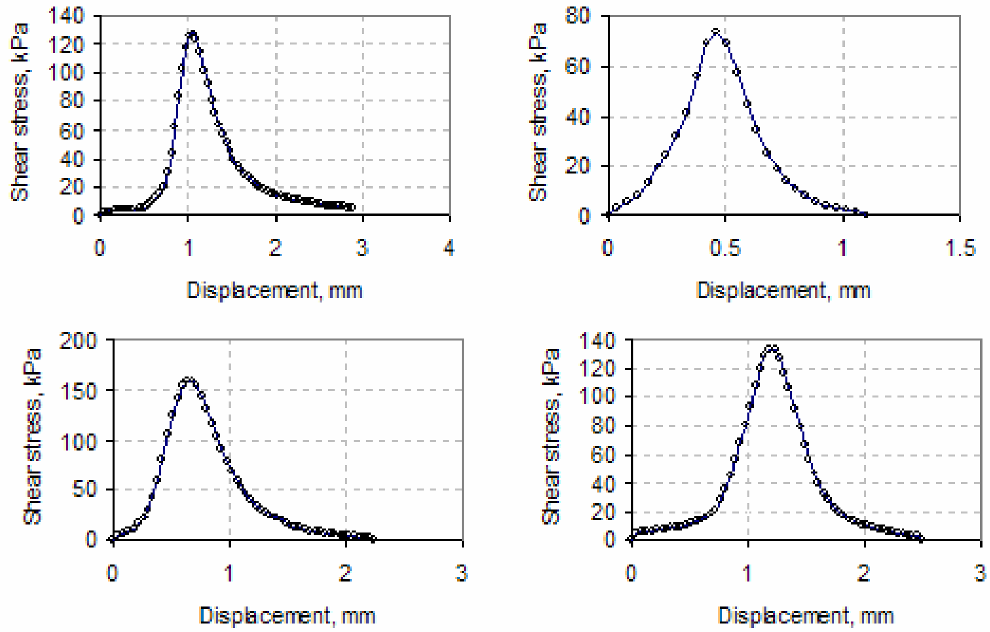



*Figure 6.3 Typical shear stress displacement curves*

The maximum shear stress (MAXS) was determined from the shear stress peak. In addition to this, a number of other parameters were determined from the response curves, including the displacement at maximum shear stress (MAXD), the area beneath the force displacement curves up until the displacement at maximum shear stress (A1), and the total area beneath the force displacement curves (A2). These parameters are defined as shown in Figure 6.4. The area beneath the stress-displacement curve provides an indication of the work required to break the interface, and is a measure of the residual shear resistance offered, an indicator of the tack coat interface toughness or tenacity.

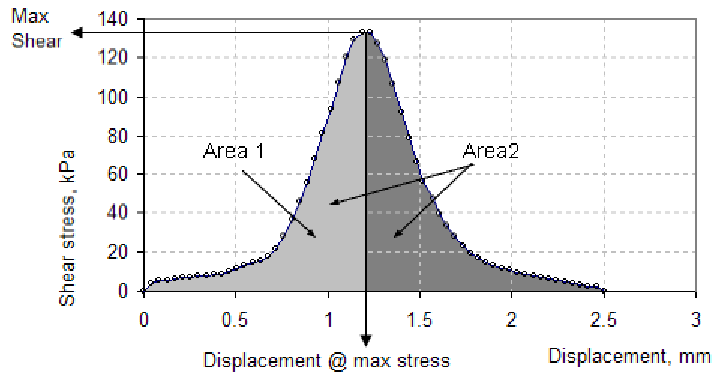



*Figure 6.4 Shear test parameters*

**STATISTICAL ANALYSES**

The results of the shear tests were used to investigate the influence of the experimental factors on the shear parameters outlined above. Results of statistical analyses on the response data are shown in Table 6.4. Analyses of variance (ANOVA) were done to identify significant factors and interactions for the different response variables at the 95 percent confidence level. The results of these analyzes are shown in Tables 6.5 through 6.8 for the different response variables evaluated. Also shown are the adjusted multiple regression correlation coefficients determined as part of the ANOVA. These indicate how closely the factors and interactions relate to the tabulated responses. The ANOVA results are summarized in Table 6.9, indicating significant factors and interactions. It can be seen from this table that none of the response variables evaluated identify a significant interaction between mix type and application rate. This may suggest that application rates for tack coats may be set without regard given to the type of asphalt mix being used as an overlay. It may, however, also be a perceived shortcoming of the procedure. The following is a summary of the determined shear responses:

***Maximum shear stress (MAXS)***

For maximum shear stress, based on the ANOVA (Table 6.5), each of the main factors has P-values below 0.05 and are statistically significant at the 95 percent confidence level, ranking in significance as:

1. Mix Type
2. Tack Type
3. Application Rate
4. Hamburg Cycles
5. Mix Type by Hamburg Cycles

The Type D specimens show higher maximum shear stress compared to the CMHB and PFC specimens. This may be related to the difference in aggregate structure of the mix gradations, i.e., with the Type D mixes, the surface finish is finer and less porous, which results in a higher contact area between the AC and PCCP. Tack type has a significantly influence on maximum shear stress, the AC-10 tack coat providing the highest shear stress, followed by the SS-1 emulsion. Higher tack coat application rates result in significantly higher tack coat shear strengths. Trafficking with the Hamburg wheel tracking device improved maximum shear stress after 10,000 cycles but a drop in shear stress is evident after the application of 20,000 cycles. This indicates that the shear strength of the tack coats may be adversely influenced by the effects of trafficking in the presence of water.

Shear stresses determined as part of the experiment compare favorably with those reported in the aforementioned paper by Mohammed et al. [Mohammad 2002] as part of their study to investigate the interface shear strength of tack coats. In this study, asphalt mixes were gyratory compacted upon a tacked lower AC layer within the compaction mold. The shear stress results from the present study suggest that the specimen preparation approach as adopted may be feasible to investigate interface shear strength.

***Maximum shear displacement (MAXD)***

Based on the ANOVA (Table 6.6), the following factors and interactions have P-values below 0.05 (at the 95 percent confidence level), and rank in significance as:

1. Hamburg Cycles
2. Mix Type by Tack Type
3. Tack Type
4. Mix Type by Hamburg Cycles
5. Application Rate by Hamburg Cycles
6. Mix Type
7. Application Rate
8. Tack Type by Application Rate

Compared to maximum shear stress, many more factors and interactions appear to be significant for displacement at maximum shear stress, although the adjusted correlation coefficient is lower suggesting that the displacement response does not correlate with the factors evaluated to the same degree as shear stress. In contrast to the others, this response variable is significant influenced by the interaction between mix type and tack type.

***Maximum shear area (A1)***

Based on the ANOVA (Table 6.7), significant factors for maximum shear area rank as follows:

1. Mix Type
2. Tack Type
3. Application Rate
4. Application Rate by Hamburg Cycles
5. Tack Type by Application Rate
6. Hamburg Cycles

7. Tack Type by Hamburg Cycles

More interactions are significant when evaluating the area beneath the stress-displacement curves compared to maximum shear stress alone. The area under the curve is a multiplicative effect of both maximum shear stress and displacement at maximum shear stress. The AC-10 tack coat and higher application rates result in greater maximum shear areas.

***Total shear area (A2)***

Based on the ANOVA (Table 6.8), significant factors for maximum shear area rank as follows:

1. Mix Type
2. Tack Type
3. Application Rate
4. Hamburg Cycles
5. Tack Type by Application Rate
6. Application Rate by Hamburg Cycles

Total shear area has the highest adjusted correlation coefficient and has more significant factors and interactions than maximum shear stress. The total shear area decreases with increasing Hamburg cycles for the mix types, tack coat types and application rates evaluated.

Based on the findings it may be concluded that each of the shear parameters evaluated have potential in identifying the significance of factors influencing the shear properties of tack coats.

*Table 6.4 Statistics of shear strength parameters determined*

| Mix | Tack | Rate | Ham. | | MAXS | | MAXD | | A1 | | A2 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Type | Type | gal/yd$^2$ | Cycles | N | MEAN | STD | MEAN | STD | MEAN | STD | MEAN | STD |
| TYPED | SS-1 | 0.04 | 0 | 4 | 119.9 | 14.7 | 0.73 | 0.14 | 29.6 | 7.3 | 108.0 | 11.1 |
| TYPED | SS-1 | 0.08 | 20000 | 4 | 191.3 | 35.2 | 0.76 | 0.41 | 54.9 | 39.4 | 158.0 | 51.5 |
| TYPED | SS-1 | 0.12 | 10000 | 4 | 152.7 | 50.9 | 0.58 | 0.15 | 33.2 | 15.5 | 160.1 | 39.3 |
| TYPED | CSS-1h | 0.04 | 10000 | 4 | 157.9 | 32.2 | 0.49 | 0.14 | 25.8 | 10.1 | 85.2 | 21.9 |
| TYPED | CSS-1h | 0.08 | 0 | 4 | 181.5 | 8.2 | 0.77 | 0.08 | 52.9 | 4.2 | 189.6 | 32.7 |
| TYPED | CSS-1h | 0.12 | 20000 | 4 | 223.8 | 32.6 | 0.69 | 0.23 | 55.1 | 14.8 | 187.2 | 52.5 |
| TYPED | AC-10 | 0.04 | 20000 | 4 | 199.2 | 10.4 | 0.74 | 0.04 | 50.7 | 8.6 | 135.2 | 11.6 |
| TYPED | AC-10 | 0.08 | 10000 | 4 | 198.3 | 34.5 | 0.58 | 0.13 | 44.4 | 15.3 | 176.3 | 28.9 |
| TYPED | AC-10 | 0.12 | 0 | 4 | 239.9 | 22.0 | 1.06 | 0.20 | 129.3 | 41.3 | 447.1 | 47.4 |
| PFC | SS-1 | 0.04 | 10000 | 4 | 41.7 | 28.5 | 0.42 | 0.06 | 6.8 | 4.1 | 18.4 | 10.8 |
| PFC | SS-1 | 0.08 | 0 | 4 | 31.3 | 10.0 | 0.66 | 0.18 | 9.8 | 4.6 | 29.5 | 12.0 |
| PFC | SS-1 | 0.12 | 20000 | 4 | 42.5 | 19.3 | 0.46 | 0.17 | 9.3 | 5.5 | 30.8 | 20.7 |
| PFC | CSS-1h | 0.04 | 20000 | 4 | 60.3 | 10.2 | 0.51 | 0.11 | 10.0 | 2.3 | 26.5 | 2.8 |
| PFC | CSS-1h | 0.08 | 10000 | 4 | 70.8 | 37.9 | 0.46 | 0.05 | 11.5 | 6.3 | 38.5 | 20.8 |
| PFC | CSS-1h | 0.12 | 0 | 4 | 51.3 | 11.3 | 1.01 | 0.21 | 19.8 | 5.6 | 65.0 | 16.9 |
| PFC | AC-10 | 0.04 | 0 | 4 | 22.6 | 11.3 | 0.61 | 0.14 | 7.5 | 4.6 | 18.6 | 11.2 |
| PFC | AC-10 | 0.08 | 20000 | 4 | 79.0 | 16.3 | 0.54 | 0.03 | 15.2 | 3.5 | 73.5 | 15.6 |
| PFC | AC-10 | 0.12 | 10000 | 4 | 100.1 | 45.4 | 0.61 | 0.14 | 28.8 | 19.6 | 134.1 | 69.5 |
| CMHB | SS-1 | 0.04 | 20000 | 4 | 54.1 | 32.7 | 0.48 | 0.11 | 10.8 | 6.1 | 27.9 | 17.0 |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CMHB | SS-1 | 0.08 | 10000 | 4 | 112.2 | 36.4 | 0.54 | 0.12 | 21.2 | 10.5 | 97.7 | 43.8 |
| CMHB | SS-1 | 0.12 | 0 | 4 | 126.1 | 24.6 | 0.76 | 0.20 | 29.6 | 3.7 | 190.7 | 19.4 |
| CMHB | CSS-1h | 0.04 | 0 | 4 | 103.0 | 11.9 | 0.79 | 0.14 | 18.5 | 4.0 | 70.1 | 6.5 |
| CMHB | CSS-1h | 0.08 | 20000 | 4 | 93.6 | 12.6 | 0.53 | 0.04 | 15.5 | 1.9 | 58.3 | 6.3 |
| CMHB | CSS-1h | 0.12 | 10000 | 4 | 220.6 | 15.8 | 0.64 | 0.06 | 49.2 | 6.0 | 201.7 | 12.9 |
| CMHB | AC-10 | 0.04 | 10000 | 4 | 171.1 | 21.9 | 0.68 | 0.09 | 47.7 | 12.6 | 141.7 | 27.2 |
| CMHB | AC-10 | 0.08 | 0 | 4 | 136.3 | 12.4 | 1.39 | 0.61 | 59.5 | 17.3 | 201.0 | 28.7 |
| | | | | 108 | 171.7 | 14.6 | 0.69 | 0.03 | 48.1 | 7.2 | 216.6 | 29.3 |

*Table 6.5 MAXS ANOVA results (Adj. $R^2$ = 0.782)*

| Factor | SS | df | MS | p |
|---|---|---|---|---|
| Mix Type | 304794.3 | 2 | 152397.1 | 0.0000 |
| Tack Type | 45666.4 | 2 | 22833.2 | 0.0000 |
| Application Rate | 35721.5 | 2 | 17860.7 | 0.0000 |
| Hamburg Cycles | 10136.7 | 2 | 5068.3 | 0.0089 |
| Mix Type by Tack Type | 606.3 | 1 | 606.3 | 0.4427 |
| Mix Type by Application Rate | 413.1 | 1 | 413.1 | 0.5261 |
| Mix Type by Hamburg Cycles | 7009.6 | 1 | 7009.6 | 0.0102 |
| Tack Type by Application Rate | 2353.6 | 1 | 2353.6 | 0.1322 |
| Tack Type by Hamburg Cycles | 102.5 | 1 | 102.5 | 0.7519 |
| Application Rate by Hamburg Cycles | 1303.0 | 1 | 1303.0 | 0.2613 |
| Error | 94872.3 | 93 | 1020.1 | |
| Total SS | 500582.3 | 107 | | |

*Table 6.6 MAXD ANOVA results (Adj. $R^2$ = 0.499)*

| Factor | SS | df | MS | p |
|---|---|---|---|---|
| Mix Type | 0.41 | 2 | 0.203 | 0.0047 |
| Tack Type | 0.53 | 2 | 0.264 | 0.0010 |
| Application Rate | 0.26 | 2 | 0.128 | 0.0313 |
| Hamburg Cycles | 2.02 | 2 | 1.009 | 0.0000 |
| Mix Type by Tack Type | 0.42 | 1 | 0.417 | 0.0009 |
| Mix Type by Application Rate | 0.01 | 1 | 0.005 | 0.7083 |
| Mix Type by Hamburg Cycles | 0.41 | 1 | 0.409 | 0.0010 |
| Tack Type by Application Rate | 0.16 | 1 | 0.163 | 0.0349 |
| Tack Type by Hamburg Cycles | 0.12 | 1 | 0.117 | 0.0735 |
| Application Rate by Hamburg Cycles | 0.39 | 1 | 0.388 | 0.0014 |
| Error | 3.31 | 93 | 0.036 | |
| Total SS | 7.61 | 107 | | |

*Table 6.7 A1 ANOVA results (Adj. $R^2$ = 0.630)*

| Factor | SS | df | MS | p |
|---|---|---|---|---|
| Mix Type | 28379.29 | 2 | 14189.65 | 0.0000 |
| Tack Type | 12422.63 | 2 | 6211.31 | 0.0000 |
| Application Rate | 8581.06 | 2 | 4290.53 | 0.0000 |
| Hamburg Cycles | 2261.35 | 2 | 1130.67 | 0.0271 |
| Mix Type by Tack Type | 354.93 | 1 | 354.93 | 0.2807 |
| Mix Type by Application Rate | 134.00 | 1 | 134.00 | 0.5066 |
| Mix Type by Hamburg Cycles | 172.37 | 1 | 172.37 | 0.4514 |
| Tack Type by Application Rate | 1958.11 | 1 | 1958.11 | 0.0124 |
| Tack Type by Hamburg Cycles | 1469.02 | 1 | 1469.02 | 0.0297 |
| Application Rate by Hamburg Cycles | 2541.67 | 1 | 2541.67 | 0.0046 |
| Error | 28031.80 | 93 | 301.42 | |
| Total SS | 87066.82 | 107 | | |

*Table 6.8 A2 ANOVA results (Adj. $R^2 = 0.786$)*

| Factor | SS | df | MS | p |
|---|---|---|---|---|
| Mix Type | 334421.2 | 2 | 167210.6 | 0.0000 |
| Tack Type | 136231.7 | 2 | 68115.9 | 0.0000 |
| Application Rate | 226633.4 | 2 | 113316.7 | 0.0000 |
| Hamburg Cycles | 37753.5 | 2 | 18876.7 | 0.0001 |
| Mix Type by Tack Type | 752.3 | 1 | 752.3 | 0.5351 |
| Mix Type by Application Rate | 443.2 | 1 | 443.2 | 0.6339 |
| Mix Type by Hamburg Cycles | 462.0 | 1 | 462.0 | 0.6268 |
| Tack Type by Application Rate | 21990.3 | 1 | 21990.3 | 0.0011 |
| Tack Type by Hamburg Cycles | 2963.5 | 1 | 2963.5 | 0.2197 |
| Application Rate by Hamburg Cycles | 20803.0 | 1 | 20803.0 | 0.0015 |
| Error | 180544.1 | 93 | 1941.3 | |
| Total SS | 970806.7 | 107 | | |

*Table 6.9 Summary of significant factors and interactions by shear response*

| Effect or interaction | MAXS | MAXD | A1 | A2 |
|---|---|---|---|---|
| Adj $R^2$ | 0.782 | 0.499 | 0.630 | 0.786 |
| Mix Type | X | X | X | X |
| Tack Type | X | X | X | X |
| Application Rate | X | X | X | X |
| Hamburg Cycles | X | X | X | X |
| Mix Type by Tack Type | | X | | |
| Mix Type by Application Rate | | | | |
| Mix Type by Hamburg Cycles | X | X | | |
| Tack Type by Application Rate | | | X | X |
| Tack Type by Hamburg Cycles | | X | X | |
| Application Rate by Hamburg Cycles | | X | X | X |

## MMLS3 AND HAMBURG WHEEL-TRACKING TESTS

A number of wheel tracking tests such as the Hamburg Wheel Tracking Device (HWTD) and the Asphalt Pavement Analyzer (APA) have been developed to evaluate the rutting performance of asphalt mixes in the laboratory. These tests are typically done at elevated temperatures on laboratory prepared specimens (briquettes) compacted to fixed voids levels. These wheel tracking tests provide a rapid evaluation of asphalt mixture properties, the results of which may be used in quality control assessments as part of product specifications. An example of the latter is the TxDOT initiative to include Hamburg wheel tracking specifications for quality control of some dense graded mixtures and stone matrix asphalt (SMA). Although the wheel tracking devices are primarily used to evaluate the rutting, moisture susceptibility and fatigue properties of asphalt, they are rarely used for performance investigations.

The HWTD was originally developed in Hamburg, Germany, where it is used as a specification requirement to evaluate rutting and stripping of asphalt mixes. The device was introduced to the USA in the 1990s and has been modified from its original design to test cylindrical specimens. Specimens are tested submerged in water heated to temperatures up to 122 °F (50 C) by loading with a steel wheel having a diameter of 7.9 in. (200 mm) and a width of 1.9 in. (47 mm). Additional loading is applied to the wheel such that the total load applied to the test specimens is in the order of 158 lbf (0.7 kN). This results in very high contact pressures on test specimens. HWTD tests are conducted at a rate of 52 load applications per minute. This equates to a speed of about 0.8 ft/s (0.25 m/s). The loaded wheel oscillates back and forth over the tested specimens. One forward and backward motion comprises two loading cycles.

HWTD specimens are laboratory compacted to fixed voids levels, typically 7 percent and a height of 2.5 in. (63 mm). Specimens are cut to allow two specimens to be aligned alongside each other such that the trimmed edges are in contact. This ensures that the loaded wheel is always in contact with the asphalt concrete specimen as it rolls back and forth over its surface during testing. The widths of the cut specimen leading edges in contact are 2.4 in. (60 mm), thus the ratio of the width of the leading edges to the width of the HWTD wheel is 60/47=1.3. This ratio reflects on the degree of possible edge effects and specimen confinement.

In the TxDOT HWTD procedure [TxDOT 2002], testing of specimens is continued until a total of 20,000 cycles are applied or until rutting of the tested specimens exceeds 0.5 in. (13 mm). A specification requirement adopted is that rutting be less than 0.5 in. (13 mm) after the application of 20,000 at 122 °F (50 °C). Rut depth is measured continuously during trafficking. Various parameters are used in the interpretation of HWTD test results, i.e., rut depth, creep slope, stripping slope and stripping inflection point. The creep slope is defined as the inverse of the deformation rate within the linear range of the deformation curve after densification and prior to stripping (if stripping occurs). The stripping slope is the inverse of the deformation rate within the linear region of the deformation curve after stripping occurs. The creep slope relates primarily to rutting from plastic flow (shear failure) and the stripping slope indicates accumulation of rutting due to moisture damage. The stripping inflection point is the number of wheel passes corresponding to the intersection of creep slope and stripping slope.

An excellent correlation between the HWTD and pavements with known field performance is reported in [Aschenbrener 1995]. This paper mentions that the HWTD is sensitive to asphalt cement stiffness, the quality of aggregate, length of short-term aging, asphalt cement source, liquid and hydrated lime anti-stripping agent and compaction temperature. Izzo and Tahmoressi [Izzo 1998] investigated the repeatability of the device and concluded that it yielded repeatable results for mixtures produced with different aggregates and with test specimens fabricated by different compaction devices.

The MMLS3 consists of four recirculating axles, each with a single 11.8 in. (300 mm) diameter wheel that applies uni-directional loading. The tires may be inflated up to pressures of 116 psi (800 kPa). Axle loads varying between 470 lbf (2.1 kN) and 650 lbf (2.9 kN) are possible. The axle loads are automatically kept constant at a predetermined level by a patented suspension system. Nominal wheel speed is 8.2 ft/s (2.5 m/s), applying up to 7200 loads per hour. The speed of trafficking can be varied. For the MMLS3 tests described in the case studies that follow the wheel loads applied were kept constant at 600 lbf (2.7 kN) and tire pressures at 100 psi (690 kPa).

Figure 6.6 shows a photo of the new MMLS3 test configuration. Aluminum moulds or clamps are used to confine the specimens during testing. These are shaped to ensure load transfer between the tested specimens. The mould configuration may be placed within a water bath for heated wet testing, ensuring that the tops of the specimens are submerged approximately 0.1 in. (3 mm) beneath the water level during testing. The MMLS3 is aligned above the mould configuration such that trafficking occurs along the center of the aligned specimens. The specimens are compacted to a height of 4 in. (100 mm), in the case of laboratory briquettes, or cut to this height if field cores are tested. Specimen thicknesses may be varied between 1.5 in. (35 mm) and 4 in. (100mm) in the latest test beds. The leading edges of the trimmed specimens are 4 in. (100 mm) to allow an unobtrusive passageway for the 3.2 in. (80 mm) wide MMLS3 inflated tire along the specimens. Thus, the ratio of the width of the specimen leading edges to that of the tire width is 100/80=1.3, the same as that for the HWTD.

***Materials and mix design***

Asphalt mixtures and aggregate types evaluated as part of the study are shown in Table 6.10. The siliceous gravel was sourced from Hanson, Prescott, the sandstone from Meridian, Sawyer, and the quartzite from Martin Marietta, Jones. A PG 76-22 binder (Wright Asphalt of Houston, Texas) was used for all the mixes. The binder contents of the different mixes are shown in Table 6.10. All the mixes contained 1 percent lime.

*Table 6.10 Binder contents of mix and aggregate types tested*

| Mix/Material | Siliceous Gravel | Quartzite | Sandstone |
|---|---|---|---|
| Superpave | 5.0 % | 5.1 % | 5.1 % |
| CMHB-C | 4.7 % | 4.8 % | 4.8 % |
| Type C | 4.4 % | 4.6 % | 4.5 % |

Aggregate gradations for the Superpave, CMHB and Type C mixes are shown in Table 6.11 through Table 6.13. The tack coat used for the MMLS3 specimens was an SS-1 slow setting emulsion. An application rate of 0.04 gal/sq. yd (0.18 L/m$^2$) was applied.

*Table 6.11 Aggregate gradations for Superpave mixes*

| Sieve Size, mm | Siliceous Gravel | Sandstone | Quartzite |
|---|---|---|---|
| 19 | 100 | 100 | 100 |
| 12.5 | 92 | 92.1 | 93.7 |
| 9.5 | 84.8 | 79.4 | 81.7 |
| 4.75 | 52.4 | 49 | 45.5 |
| 2.36 | 30.9 | 29.2 | 31.4 |
| 1.18 | 20.4 | 22.4 | 21 |
| 0.6 | 13.9 | 18.9 | 17.7 |
| 0.3 | 8.8 | 14.9 | 11.8 |
| 0.15 | 4.5 | 10.2 | 8.2 |
| 0.075 | 3.2 | 6.5 | 5.6 |

*Table 6.12 Aggregate gradations for CMHB-C mixes*

| Sieve Size | Siliceous Gravel | Quartzite | Sandstone |
|---|---|---|---|
| 7/8" | 100 | 100 | 100 |
| 5/8" | 99.7 | 99.6 | 100 |
| 3/8" | 64.5 | 65.6 | 65.4 |
| #4 | 34.3 | 34.2 | 38 |
| #10 | 21.8 | 24 | 24 |
| #40 | 16.2 | 14.5 | 16.4 |
| #80 | 9.8 | 9.1 | 10.9 |
| #200 | 6.4 | 5.9 | 6.4 |

*Table 6.13  Aggregate gradations for Type C mixes*

| Sieve Size | Siliceous Gravel | Quartzite | Sandstone |
|---|---|---|---|
| 7/8" | 100 | 100 | 100 |
| 5/8" | 100 | 99.8 | 99.8 |
| 3/8" | 75.8 | 79.1 | 80.7 |
| #4 | 49.2 | 51.4 | 46.2 |
| #10 | 31.5 | 34 | 30.9 |
| #40 | 18.2 | 17.9 | 15.6 |
| #80 | 11.7 | 10 | 9.6 |
| #200 | 5.8 | 5.3 | 5.8 |

***Specimen preparation***

Asphalt specimens were plant-prepared but laboratory compacted.  This required reheating of the mixes.  The specimens, having diameters of 6 in. (150 mm) and thicknesses of 2 in. (50 mm), were prepared by gyratory compaction to a density of 93 percent (%Gmm), i.e., 7 percent voids in the mix (VIM).  The asphalt concrete specimens had to be sawn for the MMLS test configuration in which specimens are placed side-by-side (see Figure 6.6) and confined within aluminum moulds.  The sawn specimens were tacked to concrete disks having a height of 2 in. (50 mm) also sawn as indicated.  The concrete disks were obtained by sawing 6 in. (150 mm) diameter PCCP cores.  The surface finish of the sawn concrete cores was relatively smooth.

Nine specimens were prepared for the MMLS3 tests.  Of these, only six were used for the tests (dummy specimens were placed on the ends of the setup shown in Figure 6.6).  Two of the remaining three specimens were used for the Hamburg wheel-tracking tests.  Since these specimens had already been prepared for the MMLS3 tests, it was necessary to use gypsum to correct the height and shape of the specimens for the Hamburg moulds as shown in Figure 6.7.

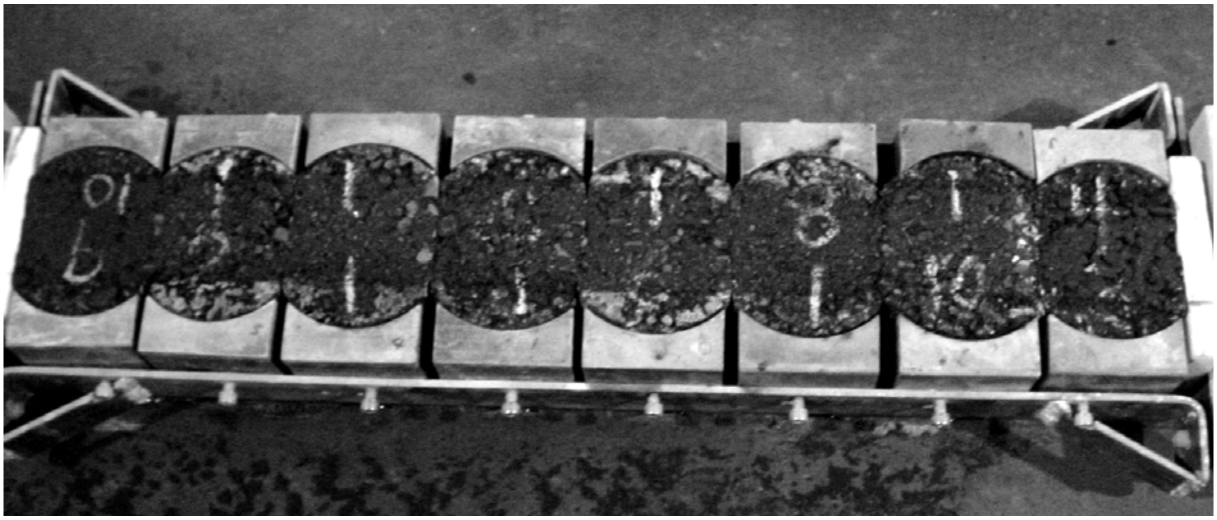


*Figure 6.6  MMLS3 test configuration*

*Figure 6.7  Gypsum repaired Hamburg specimen*

### *MMLS3 Tests and Results*

MMLS3 tests were done wet and heated at a temperature of 122 °F (50 °C). The testing configuration shown in Figure 6.6 was used for the first time. Specimens are placed in-line within a water bath. The following MMLS3 test conditions were applied:

- Wheel load: 607 lbf (2.7 kN)
- Tire pressure: 100 psi (690 kPa)
- Load rate: 6000 axles/hr
- Temperature: 122 °F (50 °C)

A total of 120,000 axles were applied. Four transverse profilometer measurements were taken intermittently during trafficking, i.e., after the application of 0k, 30k, 60k and 120k axles. Table 6.14 summarizes the MMLS3 rutting results. It shows the maximum average rutting measured on each of the mixes tested after the application of 120,000 MMLS3 axles. Figure 6.8 shows the average cumulative rutting curves for each of the mixes tested. The results are discussed later in this chapter.

*Table 6.14  Average MMLS3 rutting [mm] measured intermittently during trafficking*

| Mix Type | Aggregate | Axles, Thousands | | | |
|---|---|---|---|---|---|
| | | 0 | 30 | 60 | 120 |
| Superpave | Gravel | 0 | 1.6 | 1.9 | 2.3 |
| Superpave | Quartzite | 0 | 1.1 | 1.4 | 1.7 |
| Superpave | Sandstone | 0 | 1.0 | 1.0 | 1.2 |
| CMHB | Gravel | 0 | 3.4 | 3.8 | 3.9 |
| CMHB | Quartzite | 0 | 1.0 | 1.3 | 1.5 |
| CMHB | Sandstone | 0 | 1.2 | 1.4 | 1.7 |
| Type C | Gravel | 0 | 2.8 | 3.2 | 3.4 |
| Type C | Quartzite | 0 | 1.1 | 1.3 | 1.6 |
| Type C | Sandstone | 0 | 0.8 | 1.3 | 1.4 |

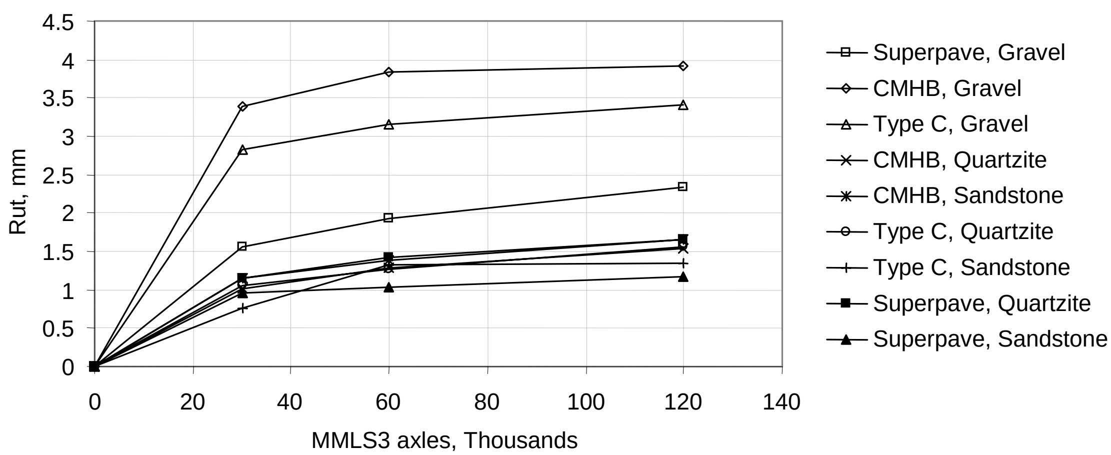



*Figure 6.8  Average cumulative MMLS3 rutting*

As part of the experiment, it was planned to do simple shear tests to investigate the performance of the tack coats before and after MMLS3 testing.  It was found, however, that the tack coats were completely stripped from all the MMLS3 specimens after testing.

### *Hamburg Wheel-tracking Tests and Results*

Hamburg wheel tracking tests were done as outlined in test method Tex-242-F at a temperature of 122 °F (50 °C). A total of 20,000 cycles were applied. None of the mixes failed the TxDOT requirements for mixes containing PG 76-22 binder, i.e., rutting was less than 0.5 in. (12.5 mm) after 20,000 cycles. Table 6.15 summarizes the Hamburg wheel tracking test results. It shows the cumulative rutting measured for each of the mixes tested after selected cycle intervals. Figure 6.9 shows the Hamburg test cumulative rutting curves for each of the mixes tested.

## DISCUSSION OF RESULTS

### *MMLS3 tests*

Immediately apparent from Figure 6.9 is the poorer performance of the mixes with siliceous gravel aggregates. Figures 6.10 through 6.12 summarize the MMLS3 performance of the mixes grouped by mix type. Figures 6.13 through 6.15 summarize the MMLS3 performance grouped by aggregate type. At this stage, there are no established criteria to judge the performance of asphalt mixes tested using the MMLS configuration shown in Figure 6.1. Based on the results, however, it may be concluded that each of the mix and aggregate type combinations generally performed similarly and that the performance of the mixes was adequate, i.e., none of the mixes exhibited significant shear failure.

*Table 6.15 Cumulative Hamburg rutting [mm] for mixes tested*

| Type | | Cycles, Thousands | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Mix | Aggregate | 0 | 0.1 | 0.2 | 0.4 | 0.8 | 1.6 | 3.2 | 6.4 | 12.8 | 20 |
| Superpave | Gravel | 0 | 0.9 | 1.1 | 1.4 | 2.1 | 2.4 | 3.4 | 4.6 | 6.1 | 8.1 |
| Superpave | Quartzite | 0 | 1.3 | 1.8 | 1.8 | 2.7 | 3.3 | 3.7 | 3.9 | 4.1 | 4.7 |
| Superpave | Sandstone | 0 | 0.5 | 0.8 | 1.1 | 1.6 | 2.2 | 2.8 | 3.4 | 4.0 | 4.3 |
| CMHB | Gravel | 0 | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | 0.7 | 1.0 | 1.2 | 1.4 |
| CMHB | Quartzite | 0 | 0.5 | 0.6 | 0.8 | 1.0 | 1.1 | 1.4 | 1.8 | 3.0 | 3.8 |
| CMHB | Sandstone | 0 | 0.5 | 0.6 | 0.8 | 1.0 | 1.2 | 1.4 | 1.7 | 2.0 | 2.2 |
| Type C | Gravel | 0 | 1.1 | 1.3 | 1.6 | 1.7 | 2.1 | 2.4 | 2.8 | 3.2 | 3.3 |
| Type C | Quartzite | 0 | 1.2 | 1.6 | 1.9 | 2.3 | 2.5 | 2.6 | 3.0 | 3.4 | 3.9 |
| Type C | Sandstone | 0 | 0.4 | 0.5 | 0.6 | 0.8 | 0.9 | 1.1 | 1.3 | 1.5 | 1.7 |

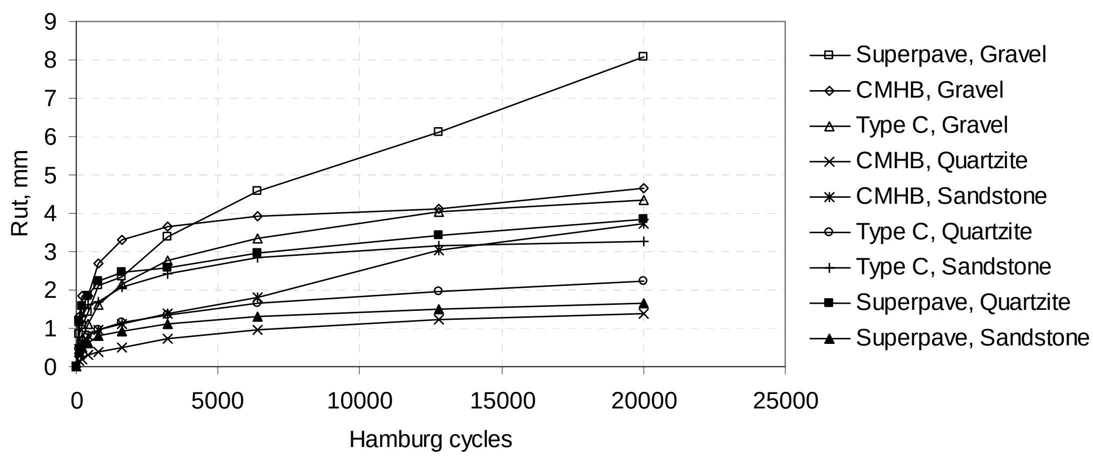



*Figure 6.9 Hamburg cumulative rutting*

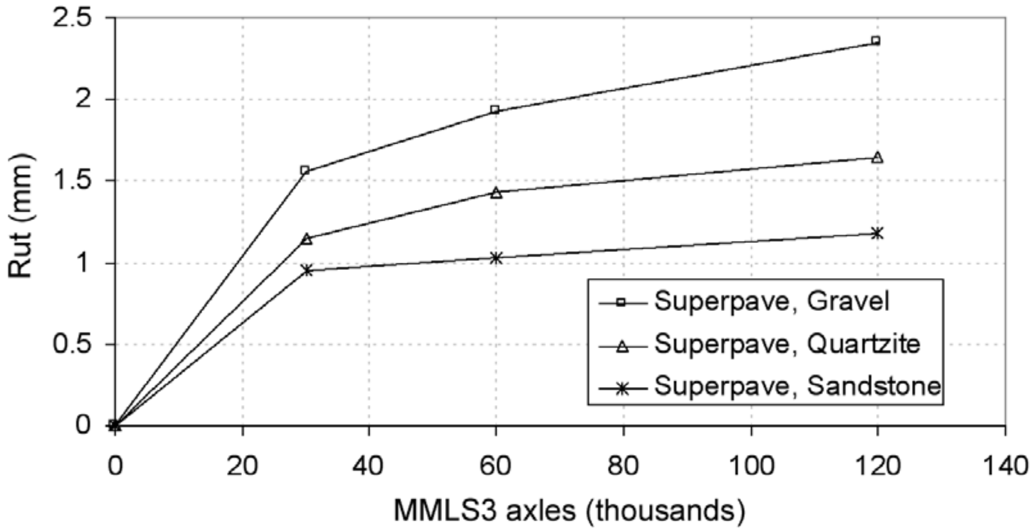



*Figure 6.10  MMLS3 performances of Superpave mixes*

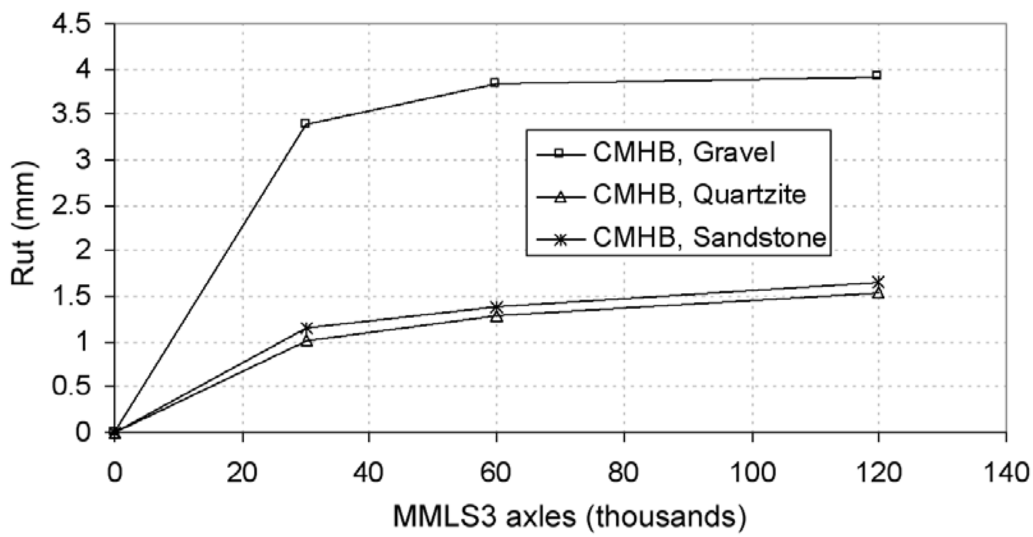



*Figure 6.11  MMLS3 performances of CMHB mixes*

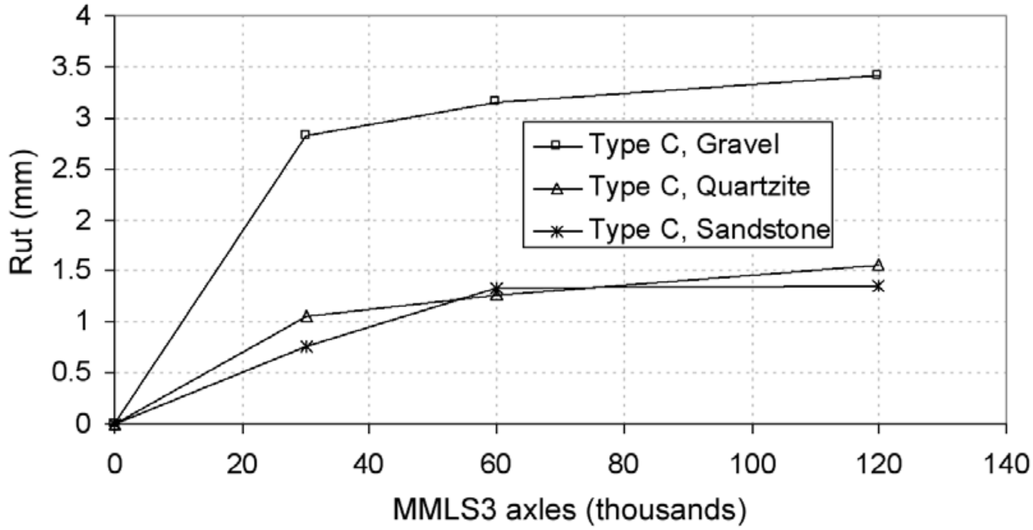



*Figure 6.12  MMLS3 performances of Type C mixes*

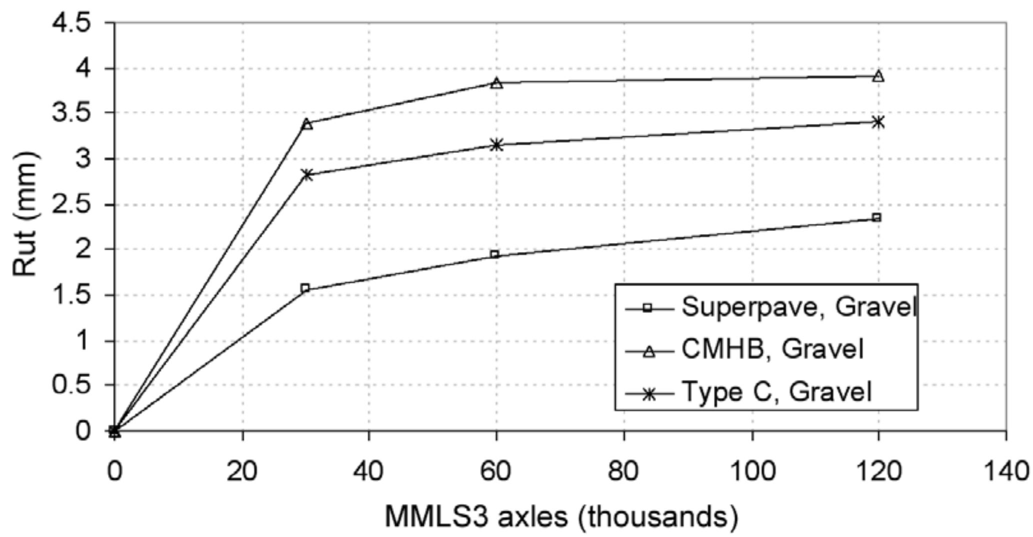



*Figure 6.13  MMLS3 performances of gravel mixes*

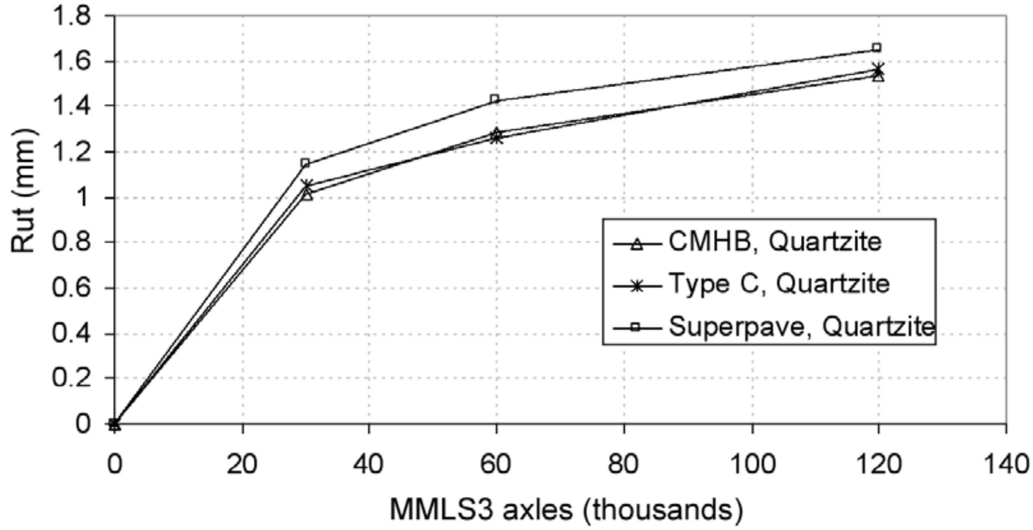



*Figure 6.14  MMLS3 performances of quartzite mixes*

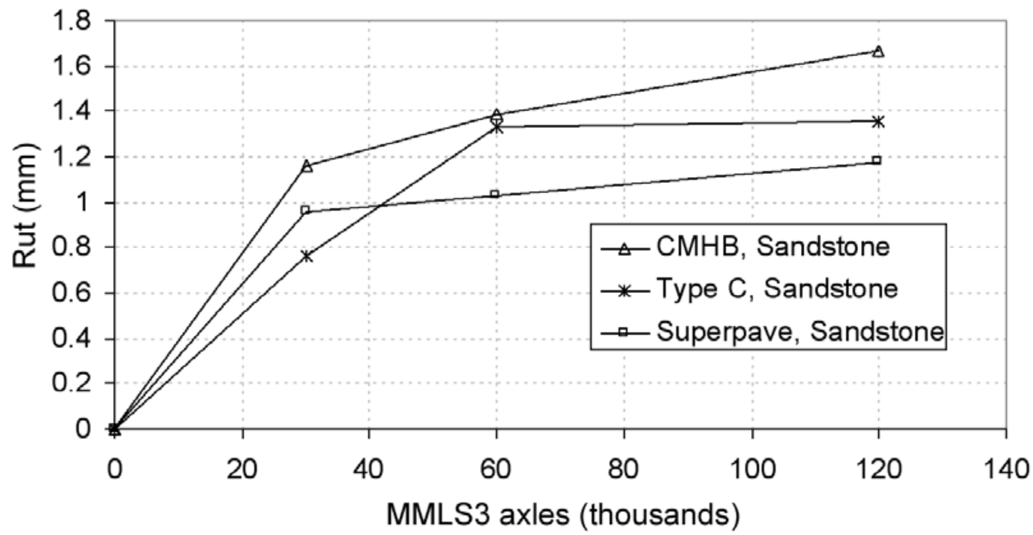



*Figure 6.15  MMLS3 performances of sandstone mixes*

The poorer performance of the mixes with gravel aggregates was expected given that these aggregates are more rounded and less angular than the quartzite and sandstone aggregates.  Rounded aggregates do not develop shear resistance to the extent of more angular type aggregates. This is particularly the case with stone skeleton mixes such as CMHB that relay on stone interaction for stability.

The exceptional performance of the mixes in the new MMLS3 setup may be exaggerated given the high degree of confinement provided by the aluminum moulds and side support.

## HAMBURG TESTS

As was apparent from the results of the MMLS3 tests, the mixes with siliceous gravel aggregates also exhibited poorer performance in terms of Hamburg wheel-tracking rutting. Figures 6.16 through 6.18 summarize the Hamburg wheel-tracking performance of the mixes grouped by mix type. Figures 6.19 through 6.21 summarize the Hamburg performance grouped by aggregate type. The Hamburg rutting of all the mixes evaluated was considerably less than 0.5 in. (12.5 mm) indicating acceptable performance all-round.

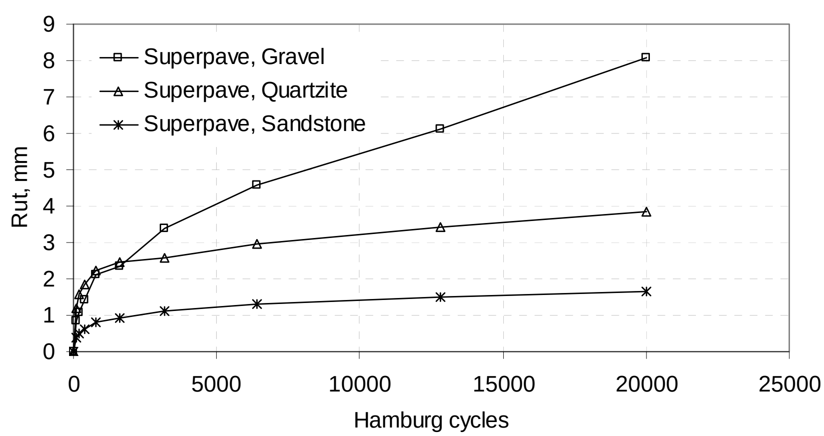



*Figure 6.16 Hamburg performance of Superpave mixes*

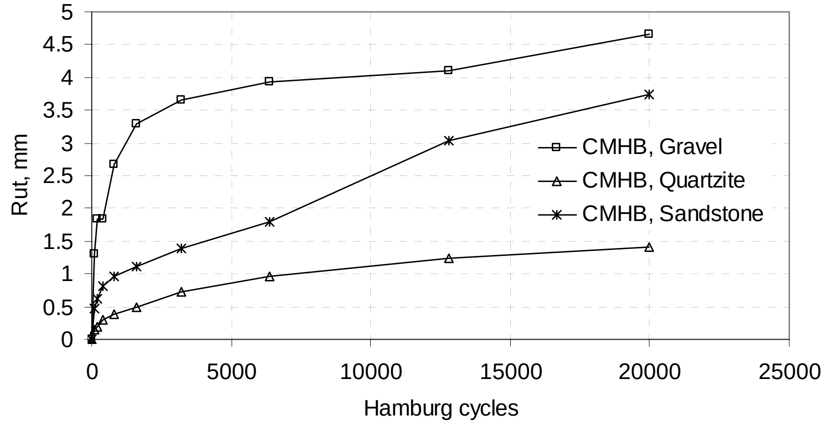



*Figure 6.17  Hamburg performances of CMHB mixes*

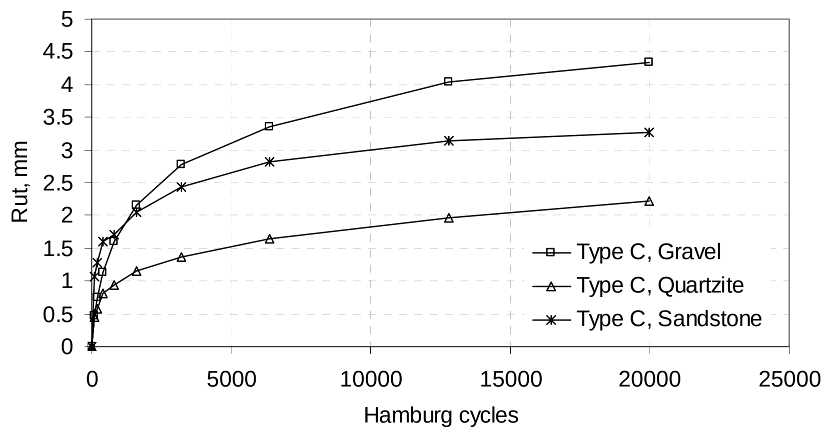



*Figure 6.18  Hamburg performances of Type C mixes*

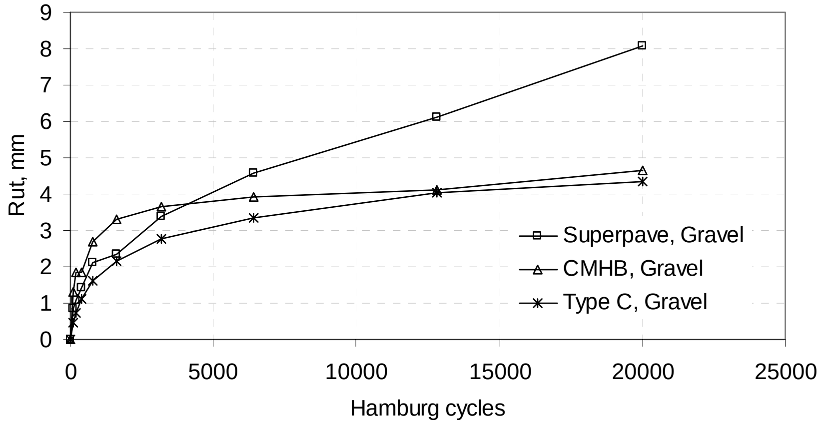



*Figure 6.19  Hamburg performances of gravel mixes*

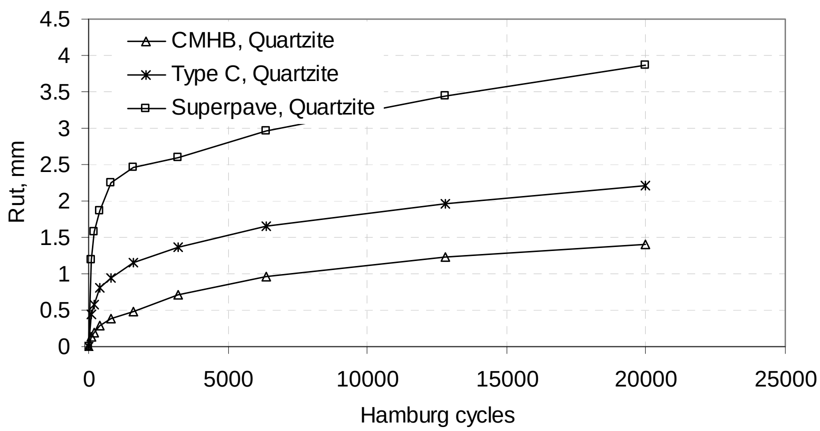



*Figure 6.20  Hamburg performances of quartzite mixes*

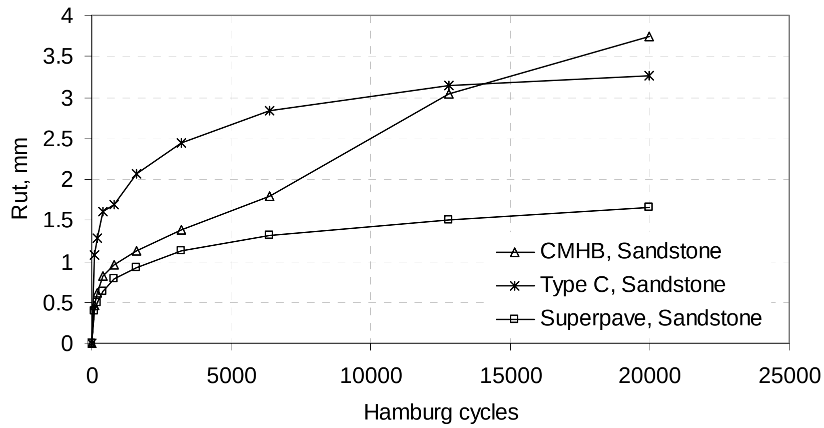



*Figure 6.21 Hamburg performances of sandstone mixes*

## SUMMARY

The objective of this part of the study was the development of a methodology for the selection of suitable AC mixtures for use as overlays on CRCP. The shear strength performance of tack coats serving to bond AC and PCCP specimens was investigated using a shear test developed as part of the study. The apparatus applies a shear load to the interface of composite specimens at a constant displacement rate. Shear tests were done at a temperature of 77 °F (25 °C). Four influence factors were investigated as part of the experiment including mix type, tack coat type, tack coat application rate and Hamburg wheel tracking. Tack coat performance influence factors were investigated at three levels. Hamburg tests were done at a temperature of 122 °F (30 °C). Four composite specimens were tested at each of the factor combinations.

As part of the experiment, gyratory compacted asphalt specimens are tacked onto concrete disks. A major benefit of this approach is that tack coat related performance results may be obtained from laboratory prepared specimens. Compared with published tack coat strength studies, results from the present study indicate that the approach as adopted may be feasible to investigate the interface shear strength of tack coats between AC and PCCP. It is recommended, however, that the option of compacting asphalt mixes directly on top of tacked concrete disks be explored further.

Statistical analyzes of the shear test results indicated that the factors that significantly influence tack coat performance include mix type, tack type, tack coat application rate and Hamburg wheel tracking. Mix types with finer and denser gradations appear to enhance the shear strengths of tack coat interfaces. The AC-10 tack coat provided more shear strength compared to the emulsion tack coats. Tack coat performance was better at the higher application rates applied. It was found that Hamburg trafficking improved the shear strength response after 10,000 cycles but that a decrease in shear strength was evident with the application of 20,000 cycles suggesting that tack coats may be vulnerable to the influence of trafficking in the presence of water. Certain interactions between the main effects were found to be significant and these differed depending on which shear parameter was evaluated. None of the parameters evaluated was able to identify a significant influence for the interaction between mix type and application rate. Overall it appears that the total shear area is the best parameter for investigating the significance of influence factors and corresponding interactions.

It is recommended that the experiment be expanded to investigate the influence of temperature and aggregate type. To better investigate the influence of moisture and the potential of debonding it is recommended that Hamburg tests at higher than 20,000 cycles be done. To relate laboratory and field performance, field cores from CRCP overlaid pavements should be shear tested.

Wheel tracking tests such as the MMLS3 and HWTD were found to be effective for evaluating the resistance to permanent deformation of asphalt mixtures for use as overlays on CRCP. In the study, both the MMLS3 and Hamburg tests highlighted the poorer relative performance of mixes with siliceous gravel aggregates. Overall, each of the mixes evaluated performed adequately. Stripping of the tack coats was evident for all mixes tested with the MMLS3.

Based on the results of the wheel-tracking tests it is recommended that the Superpave, CMHB and Type C mixes be considered for use as overlays on CRCP pavements. Siliceous gravel aggregates should preferably not be used with these mixes. The use of stiff binders (PG 76-22) and the addition of 1 percent lime to further stiffen the mixes and provide resistance to moisture susceptibility are recommended.

Based on the findings of the report the following methodology (outlined in Figure 6.22) is recommended for selection of AC for use as overlays on CRCP:

*Table 6.16: Methodology for AC mix and tack coat selection for AC overlays on CRCP*

| Step | Action |
|---|---|
| 1a | Select candidates for design:<br>• Mix types (preferably dense graded)<br>• Application rates (min 0.04 gal/yd2)<br>• Tack coat types (neat or emulsion based) |
| 1b | Select candidates selected based on:<br>• Cost<br>• Availability<br>Specification requirements |
| 2 | Undertake a rutting investigation by wheel tracking using the HWTD in accordance with TxDOT test method Tex-242-F. |
| 3a | Evaluate tack coat interface shear strength performance using the equipment as procedure as outlined in this report. |
| 3b | Investigate moisture sensitivity and influence of trafficking on tack coat performance using HWTD. |
| 4 | Select appropriate design to maximize shear stress resistance (> 50 kPa strength) and total shear area (> 100 kPa. mm tenacity). |

The criteria limits in Step 4, i.e., > 50 kPa strength, and > 100 kPa. mm tenacity, represent the 80 percentile values for maximum shear stress resistance and total shear area respectively as determined from the statistical analyses of the direct shear test data.

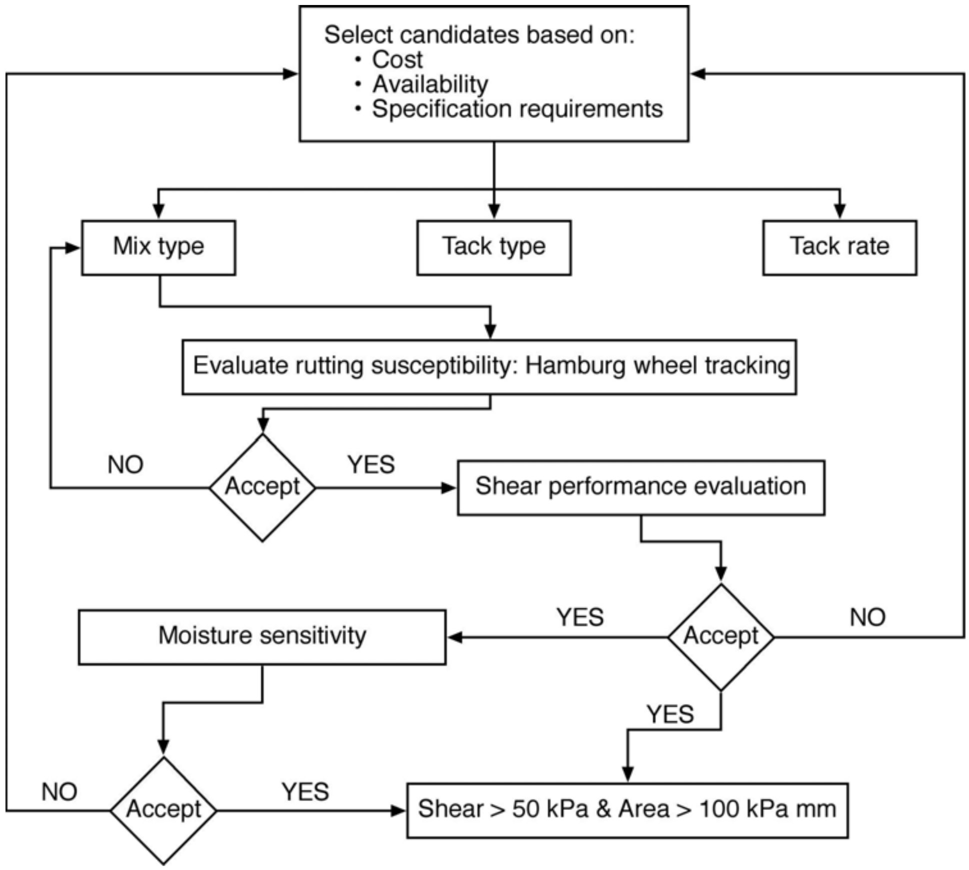



*Figure 6.22. Decision tree for AC mix and tack coat selection*

# CHAPTER 7. DISCUSSION OF RESULTS

The research undertaken in this project can be broadly categorized into two aspects, the development of criteria to decide whether to use thin asphalt concrete (AC) overlays on existing portland cement concrete pavements (PCCP), and the testing of tack coats and AC mixtures for use in such overlays. The first one of those categories was dedicated to ascertain the conditions that warrant the application of thin-bonded AC overlays on existing PCCP, while the second focused on the interface shear strength testing of tack coats and rutting resistance evaluation of AC mixtures. In this chapter, a summary of the results obtained from this research will be presented and discussed.

The preceding chapters of this report, the third one pertaining to Project 0-4398, specifically, Chapters 2 through 5, present a decision tree for the project selection for a thin AC overlay on CRCP, a tool that facilitates the decision process when facing a rehabilitation problem, by providing a series of steps and criteria conducive to finding the best rehabilitation alternative given the pavement conditions. The decision tree presented includes the decision of whether to conduct a rehabilitation with an overlay, the decision whether to use an AC overlay or a PCCP overlay, and, if a PCCP overlay is chosen, the decision whether to use an unbonded concrete overlay or a bonded concrete overlay (BCO). The results of the decision criteria are discussed below, followed by a discussion on the asphalt testing results.

## DECISION CRITERIA

A foremost objective of this research was the development of criteria for project selection of thin AC overlays on CRCP. To help the pavement engineer with the decision on whether to use an AC overlay as a rehabilitation solution, three criteria were developed, which were put together in a diagram, the decision tree.

Several tests on actual projects, along with abundant historic information from previous studies and other similar experiences from other projects were put together and analyzed to come up with these three decision criteria.

The profile criterion was deemed as the decisive factor to ascertain the implementation of an overlay. The rationale behind this statement is that, regardless of the type of overlay, if a pavement exhibits profile problems, it is a candidate for overlay rehabilitation. The type of overlay will be determined subsequently by the structural conditions of the pavement, which will be evaluated by the condition survey criterion and the deflection criterion, both of which evaluate the structural characteristics of the roadway.

The remaining life concept and the formulation of a dynamic load factor were the principles on which the profile criterion was developed. It has been demonstrated that when the pavement is subjected to dynamic loads, there may be a reduction in remaining life. The procedure to assess whether the structure may present such a reduction due to dynamic loading is by computing a dynamic load factor. The remaining life of the pavement is related to the ratio of the pavement stresses when subjected to static and dynamic loads. The stresses in the pavement are assumed proportional to the load. Therefore, the ratio between the dynamic and static loads, which is called a dynamic load factor, can be used as a criterion to determine the need of pavement resurfacing. As investigated with actual profile data, the pavement structure, before the new overlay was placed, showed more than about 10% higher load magnitudes due to the surface roughness. Based on this actual field data, it was recommended that the pavement be overlaid if the dynamic load factor is larger than 10%. In other words, if the remaining life of the pavement is less than 68%, which is the inverse of the fourth power of the load ratio of 1.1 (10% higher than the static load), resurfacing is needed.

As mentioned before, the other two criteria provide a structural evaluation of the existing pavement; therefore, these criteria will assess what kind of overlay is best for the structural conditions of the CRCP. The criterion that follows is the condition survey criterion.

The primary structural evaluation of a pavement normally comes from the condition survey, a fundamental step in any rehabilitation project. The detection of failures and distresses by visual means will give an immediate indication of the structural soundness of the pavement. Its quantification in this criterion is performed by two different approaches, namely, the pavement distress index (PDI) and the rate of failures per mile per year. The PDI is an index of pavement deterioration, and it can be used as a score to determine whether a section must be rehabilitated. The PDI considers various types of distresses, assigning them relative weights in an equation in which the computed index becomes the discriminant score. The original PDI equation, developed in the 1980s based on field data gathered across the state, was enhanced to include the incidence of spalling. A number of pavement sections were utilized to determine and calibrate a spalling coefficient to be included in the PDI equation.

The PDI equation developed for this purpose is as follows:

$$PDI = 1.0 - 0.0071(MPUNT) - 0.3978\,(SPUNT) - 0.4165\,(PATCH) - 0.2323\,(SPALL)$$

where

MPUNT= ln (minor punchouts per mile +1)

SPUNT= ln (severe punchouts per mile +1)

PATCH=ln (patches per mile+1)

SPALL= ln (spalls per mile +1)

A PDI value of zero or less indicates that the section needs an overlay. The PDI approach, much like the profile criterion, is not capable of determining what type of overlay is required by the pavement structure. The second approach, however, can establish what type of overlay is more appropriate for the case in question.

The rate of occurrence of failures, the second approach of the condition survey criterion, is essentially a measurement of where the pavement is in relation to its service life span. As such, it can be used as an intrinsic indicator of the feasibility and the timeliness of not only an AC overlay, but of different types of rehabilitation, namely, a BCO and an unbonded concrete overlay. The failure rate, computed from historic condition survey information, will signify what type of overlay is more conducive to address the current stage of structural decline of the pavement. The basic assumption behind this criterion is that any given CRCP at some point in its service life will first become an ideal candidate for an AC overlay. As time and traffic go by and the deterioration rate increases, if no treatment were applied in the first instance, then the pavement becomes an ideal candidate for a BCO. In a similar fashion, further down in the life of the pavement, at a more advanced stage of deterioration, had no rehabilitation been applied, the structure would become an ideal candidate for an unbonded concrete overlay. The criterion establishes two threshold values of failures per mile per year that divide those three stages. The threshold values are 2 and 3 failures per mile per year, respectively, and are applied in the following manner. If a CRCP approaches a rate of failure development of 2 failures per mile per year, an AC overlay is likely to remedy the situation and deliver good performance. However, if the rate approaches 3 failures per mile per year, a BCO represents a better technical and economical strategy. If the deterioration rate has reached beyond 3 failures per mile per year, the best solution is an unbonded concrete overlay; in this case, the section is already too damaged to be repaired by a BCO in an economic way. The cost to fix those distresses prior to the BCO placement will make this too expensive of a rehabilitation option, making it suitable for an unbonded concrete overlay.

The third decision criterion is the deflection criterion. The measurement of deflections is a basic structural evaluation for an existing pavement. This criterion, very much like the aforementioned rate of failures occurrence, establishes a boundary for the applicability of AC overlays with respect to other types of overlays. The idea is that if the deflection evaluation renders a structurally sound pavement it may be successfully rehabilitated by a thin AC overlay. To perform this assessment, a theoretical analysis estimates how much a hypothetical overlay would have to contribute to the overall structural integrity of the pavement, by means of a calculation of stresses. Therefore, this criterion encompasses a structural evaluation of the existing pavement, by means of the deflection measurements, in conjunction with a theoretical assessment of a future overlay and its structural contribution, with the computation of stresses.

The deflection criterion is integrated by two components, a deflection ratio, and a stress ratio. The deflection ratio requires measurement of deflections at both the midspan and at cracks on the CRCP. The ratio of deflections at cracks to deflections at midspan is an indicator of the structural integrity of the existing pavement. An elastic layer theory calculation of stresses will provide information for the stress ratio, which measures the hypothetical structural contribution of the overlay, by comparing the stresses without the overlay to the stresses with the overlay. The deflection criterion will indicate whether an AC overlay would be a good solution, if both the deflection ratio and the stress ratio are close enough to 1, or if a more structural remedy is necessary. In general, a considerable departure from a value of 1 for both ratios implies that the pavement needs an overlay that can provide more structural benefits than an AC overlay.

It is expected that the implementation of these criteria as the steps in the decision tree will ease the process of determining the adequacy of a thin AC overlay on CRCP by making it a more systematic approach.

## TACK COAT AND AC MIXTURE EVALUATION

The focus of this part of the project was on optimizing the interface shear strength of tack coats and evaluating the rutting resistance of asphalt mixtures for use as overlays on CRCP.

The shear strength performance of tack coats utilized to bond AC and PCCP specimens was investigated using a shear test developed as part of the study. Four influence factors were investigated as part of the experiment including mix type, tack coat type, tack coat application rate, and Hamburg wheel tracking. Tack coat performance influence factors were investigated at three levels.

Statistical analyses of the shear test results indicated that the factors that significantly influence tack coat performance include mix type, tack type, tack coat application rate, and Hamburg wheel tracking. Mix types with finer gradations appear to enhance the shear strengths of tack coat interfaces. Overall, it appears that the procedure as developed is feasible to investigate the interface shear strength performance of tack coats, and that the total shear area as defined is the best parameter for investigating the significance of influence factors and corresponding interactions.

Based on the findings of the research, a methodology for evaluating suitability of AC for overlays on CRCP is recommended. Interim criteria in terms of maximum shear strength and total area beneath the shear strength- displacement curves (determined from direct shear testing) are recommended to evaluate the performance of tack coats. It is recommended that these criteria be evaluated in terms of actual shear stresses prevalent between AC and PCCP structures using layer theory. It is also recommended that the direct shear strength experiment be expanded to investigate the influence of temperature and aggregate type. Furthermore, to relate laboratory and field performance, field cores from CRCP overlaid pavements should be shear tested.

Both the MMLS3 and Hamburg wheel tracking tests highlighted the poorer relative performance of mixes with siliceous gravel aggregates. Based on the results of these tests, it is recommended that the Superpave, CMHB, and Type C or D mixes be considered for use as overlays on CRCP pavements. Siliceous gravel aggregates should preferably not be used with these mixes. The use of stiff binders (PG 76-22) and the addition of one percent lime to further stiffen the mixes and provide resistance to moisture susceptibility are recommended.

# CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

This chapter presents concluding remarks and recommendations on this research.

The main objective of this report was to present the research conducted in developing a decision tree for choosing whether to use thin asphalt concrete (AC) overlays on existing portland cement concrete pavements (PCCP). This project selection tool is integrated by a series of steps, among which, the decision criteria establish parameters to determine whether a thin AC overlay is a suitable rehabilitation option for a continuously reinforced concrete pavement (CRCP). A significant part of this report is dedicated to the development of those criteria. Another goal of this report was to produce tests on tack coats and AC mixtures to evaluate them and make recommendations on their usage in AC overlay projects on CRCP. Thus, the contents of this report can be classified under those two major features.

The decision tree presented includes the decision of whether to conduct a rehabilitation with an overlay, the decision whether to use an AC overlay or a PCCP overlay, and, if a PCCP overlay is chosen, the decision whether to use an unbonded concrete overlay or a bonded concrete overlay (BCO). The results of the decision criteria are discussed below, followed by a discussion on the asphalt testing results.

The decision to use an AC overlay on CRCP should be based on a variety of decision factors. Thus, all criteria should be analyzed for the case in question. These factors are invariably affected by both technical and economical considerations. From the technical standpoint, some considerations regarding this type of rehabilitation are:

- Debonding of the overlay is a frequent source of failure for this rehabilitation. The procedure to address debonding problems is by making sure all major distresses on the CRCP are repaired prior to the overlay placement, removing all AC patches, debris and dust. Surface preparation is key to the success of the overlay. Tack coats can be used to improve the bond between substrate and CRCP, once the surface is clean and ready. More on the tack coats investigation will be mentioned below.

- A thin AC overlay on CRCP is not capable of providing much relief in terms of structural improvement to the existing pavement system. Therefore, if the structural capacity of the existing pavement has become a concern, as shown by analyzing the condition survey and deflection criteria, other options should be considered.
- A case in which a pavement has a rough profile but still has good structural integrity is ideal for this kind of rehabilitation, as these overlays can smooth a rough surface, enhancing the riding quality and preventing any further damage caused by dynamic impact loading.
- This type of overlay may be considered a protective layer for the pavement structure. Besides improving the riding quality and reducing tire-pavement noise, this layer protects the substrate from damage caused by other elements such as the environment (e.g., moisture, temperature, debris, etc.), so that the CRCP underneath can last longer. Therefore, this type of rehabilitation can extend the service life of the structure.

On the economic side, a foremost advantage of this type of rehabilitation is its low cost. Indeed, in general, the initial cost of a thin AC overlay is lower than that of other rehabilitation strategies such as a BCO, an unbonded concrete overlay, a thick AC overlay, or a full-depth reconstruction. Nevertheless, there are two fundamental aspects that affect the economic decision, which should not be overlooked here, namely, life cycle cost, and the applicability of the overlay.

Regarding life cycle cost, the economic impact of the overlay should be analyzed not only taking into account the amount of the initial investment, but the costs throughout the expected life of the rehabilitation. An AC overlay may be the least expensive of strategies, but it may not last as long as other solutions. Thus, the cost analysis has to be evaluated for the long run.

The applicability of the overlay is intrinsically linked to the concept of timeliness. The timeliness of an overlay is an idea that is considered time and again in the evaluation of the decision criteria. It is associated to the current condition of the existing pavement. For instance, if the condition of the CRCP is just slightly damaged, while it is still structurally sound, an AC overlay may be ideal. However, if the deterioration rate continues after some time, without the pavement receiving any rehabilitation, the condition may require a different strategy. An AC overlay may still be applicable, but it may not be the best economic decision. In general, there is a sequence for the ideal timeliness of the different pavement rehabilitation strategies in regard to the current stage of deterioration of the existing pavement. This sequence indicates that the first rehabilitation strategy to become optimal in the life of the pavement is a thin AC overlay; this is applicable when the structural damage is minimal and there are only profile deficiencies. As the pavement ages and becomes more damaged, a BCO becomes a better choice. Further down in the life of the pavement, with more traffic, distress and damage to the structure, a BCO will not be enough to repair it, and then, an unbonded concrete overlay or a thick AC overlay will become the ideal choices. When damage is widespread and overlays cannot remedy the situation, a full-depth reconstruction would be the only option left to consider. It is important to notice that the sequence goes progressively, from minimal to higher structural improvement. The same can be said about the initial cost of the rehabilitation in the sequence, it goes from low to high.

The decision criteria presented herein offer a very thorough and systematic way of analyzing the condition of the existing pavement. However, in some cases the solution may be too complex. In these cases, the decision may not be an obvious choice, because some of the criteria may indicate this type of overlay is appropriate and at the same time some other criteria may suggest otherwise. It should be kept in mind that the criteria, especially in cases like these, are not absolute. The decision, after all criteria have been analyzed should come down to engineering judgement.

In regard to the AC mixture and tack coats experiment, the overall goal was to investigate the interface shear strength of tack coats and evaluating the rutting resistance of asphalt mixtures for use as overlays on CRCP. The tests indicate that AC mixes with siliceous gravel aggregates delivered the poorer performances. Considering the results of these tests, it is recommended that the Superpave, CMHB, and Type C or D mixes be considered for use as overlays on CRCP pavements. Siliceous gravel aggregates should preferably not be used with these mixes. About tack coats, the use of stiff binders (PG 76-22) and the addition of one percent lime to further stiffen the mixes and provide resistance to moisture susceptibility are recommended.

It appears that the developed procedure is appropriate to investigate the interface shear strength performance of tack coats, and that the total shear area as defined is the best parameter for investigating the significance of influence factors and corresponding interactions between the four factors investigated, mix type, tack coat type, tack coat application rate, and Hamburg wheel tracking.

For future research, in terms of the decision criteria developed, it is advisable to apply the criteria to other existing projects in which it is known that AC overlays have been successfully applied, for the purpose of further calibration of the threshold values. The researchers have applied the values to every project for which the CRCP condition information was available, with positive results in all cases, but it is acknowledged that a shortcoming of some of those values may be that they were obtained from a limited number of cases.

In the case of tack coats, it is recommended that the criteria for maximum shear strength and total area beneath the shear strength displacement curves be evaluated in terms of actual shear stresses prevalent between AC and PCCP structures using layer theory. It is also recommended that the direct shear strength experiment be expanded to investigate the influence of other variables, such as temperature and aggregate type. To further relate laboratory and field performance, it is also recommended to test field cores from CRCP overlaid pavements for shear strength.

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