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PERFORMANCE OF RUBBLIZED PAVEMENT SECTIONS IN ALABAMA

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PERFORMANCE OF RUBBLIZED PAVEMENT SECTIONS IN ALABAMA

Final Report

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ABSTRACT

The Alabama Department of Transportation (ALDOT) is faced with the challenges of producing a safe and cost effective rehabilitation strategy for an aging Portland Cement Concrete pavement (PCCP) highway system. The need for this strategy is increasing as the PCCP infrastructure ages. The purpose of this project was to evaluate the different concerns of rehabilitating the PCCP.

A main concern when rehabilitating aging PCCP systems is the elimination of reflective cracking. Over the last 10 years, ALDOT has rubblized a number of PCCP, overlaid pavements with Hot Mix Asphalt (HMA), and periodically surveyed their condition. In the past, these surveyed distress data have not been organized to accurately quantify the performance of the rubblized pavements. This report presents the results of organizing these data and assessing the performance of the respective sections.

Recognizing the importance of evaluating the performance of the rubblized pavement system, to determine if the rubblization process was a viable rehabilitation technique, would eventually improve the final product. With this serving as the ultimate objective, data were collected from 9 rubblized sections in Alabama and a process was established to efficiently and accurately process the data.

With the processed data, a database was constructed that contained all of the surveyed distress data that pertained to the rubblized pavement sections. From this database, the rubblized pavement sections were graphically and statistically analyzed. This analysis included comparing the rubblized sections to industry standards and setting up and computing a multiple linear regression for the collected data.

These analyzed data led to the following conclusions. The rubblized pavement sections have performed very well and have proven to be an efficient means of rehabilitating PCCP. There were, however, certain patterns that were recognized in the analysis. Namely, the continuously reinforced concrete pavements exhibited the worst performance and caution should be exercised in the future before rubblizing sections of this type.

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CHAPTER 1 - INTRODUCTION

BACKGROUND

There are approximately 500,000 lane miles of Portland Cement Concrete (PCC) pavements in the United States that are nearing the end or have surpassed their original design life and are in need of major rehabilitation (Bemanian, 1997). This presents engineers responsible for the maintenance of these pavement systems with the problem of choosing the most efficient means of maintaining a high quality, safe, and economical infrastructure for the American public. There are many different methods available for properly maintaining a pavement system. The focus of this research was to monitor and assess the performance of the PCC pavements in Alabama that have been rehabilitated with the rubblization process. This process will be discussed fully in Chapter 2.

When dealing with an aging PCC pavement system, a main concern is the faulting of the individual concrete slabs, defined as the difference in elevation across a joint. Some of the common causes of faulting are settlement due to a soft foundation, pumping or eroding of material from under the slab, and/or curling of the slab edges due to temperature and moisture changes (Shahin, 1994). Once the agency responsible for the pavement has deemed that the PCC pavement is in need of rehabilitation because the measured distress levels have reached an established threshold, or the system has reached a poor serviceability level, it is imperative that the pavement be rehabilitated.

In the past, one option was to simply overlay the existing pavement with hot mix asphalt (HMA) pavement. Over time, cracks along the faulted PCC pavement propagated through the overlaid asphalt revealing themselves on the pavement surface. These cracks that have appeared in the HMA overlays are referred to as reflective cracks. To eliminate the presence of reflective cracking, a number of different strategies have been employed that involve reducing the slab-action. Some examples of these strategies are crack and seat, break and seat, and rubblization. These three rehabilitation strategies are techniques that use the fracture slab approach that will be discussed in further detail in Chapter 2.

There are three main objectives of the crack and seat, break and seat, and rubblization processes. The first is to break the PCC in a manner so that the movement at a crack or joint is not sufficient enough to cause the HMA to crack. The second is to ensure that the subgrade is not damaged in any way. If the fractured PCC is forced into the subgrade this could create a weak spot in the pavement system that could accelerate the formation of distresses. The third is to make certain that the PCC pavement is broken into uniform particle sizes and that the breakage is evenly distributed (Kirk, 2001). Reducing the original size of the concrete slab will eliminate the slab action or the movement, which promotes reflective cracking. These three methods are discussed in further detail in the contents of this report.

The Alabama Department of Transportation (ALDOT) has completed nine rubblized sections in an eleven year period that are located on the interstate system. The rubblized sections were constructed at different times, with different construction equipment, subbase thicknesses, HMAC overlays thickness and having varying amounts of traffic applied. ALDOT has implemented the rubblization process as a rehabilitation technique and collected the distress information over the years. However, an organized performance evaluation of the rubblized sections has not been carried out. With this valuable information readily available, it is imperative that an evaluation take place to obtain a measure of the rubblized pavement performance.

OBJECTIVES

The first objective of this project was to determine the performance of HMA mixtures placed over rubblized PCC pavements in Alabama. The second was to establish a procedure for monitoring the performance of the rubblized PCC sections in Alabama for future use. The third was to evaluate, based on historical data, any factors that contributed to the performance of the rubblized sections in Alabama.

SCOPE OF WORK

ALDOT interstate projects that have been rehabilitated using the rubblization process were the focus of this investigation. Three automated distress surveys, conducted on nine rubblized sections, provided by ALDOT and one distress survey was made available through the use of the Highway Research Center's Automatic Road Analyzer (ARAN van) provided the data in which statistical tools were used to evaluate the performance of the rubblized sections in Alabama.

REPORT ORGANIZATION

This report is divided into six chapters in which the objectives are systematically accomplished. Following this chapter, the literature review covers the reasons for rehabilitation and includes the distresses that cause PCC pavements to become unserviceable. The review discusses one of the performance issues for the rehabilitation process, reflective cracking. Also investigated are the causes of the various pavement distresses, mitigation strategies, and a description of the rubblization process. The rubblized sections that were evaluated for this project are described in Chapter 3. The research and methodology portion of the report (Chapter 4) describes the procedures to meet the objectives specified above. The Results and Discussion portion (Chapter 5) includes the outcome from the procedure that was outlined in the Research Methodology chapter. The Conclusion and Recommendations (Chapter 6) discusses the findings and how they affect the rubblization process and performance in Alabama.

CHAPTER 2 - LITERATURE REVIEW

PAVEMENT REHABILITATION AND REFLECTION CRACKING

A major design concern when one considers the rehabilitation of a PCC pavement is the current state of the section being rehabilitated. There are a wide variety of distresses that reduce the quality of PCC and provide the impetus for rehabilitation. For example, transverse cracking, corner cracking, crack spalling, longitudinal cracking, joint deterioration and faulting are all common PCC distresses that over time reduce the serviceability of the pavement.

When rehabilitating PCC pavements, a common practice is to place an HMA overlay on top of the existing pavement structure (Freeman, 2002). However, a problem that has presented itself when utilizing an overlay is the development of reflective cracking through the depth of the newly placed overlay. This problem, known as joint reflective cracking, is a distress unique only to materials that have been laid over PCC pavements (Shahin, 1994). This type cracking is illustrated in Figure 2.1. The main concern with reflective cracking in HMA overlays is that reflective cracks accelerate the deterioration of the pavement structure. This deterioration is caused by water and incompressible solids filling the cracks that over time degrade the HMA (Yoder and Witczak, 1975).

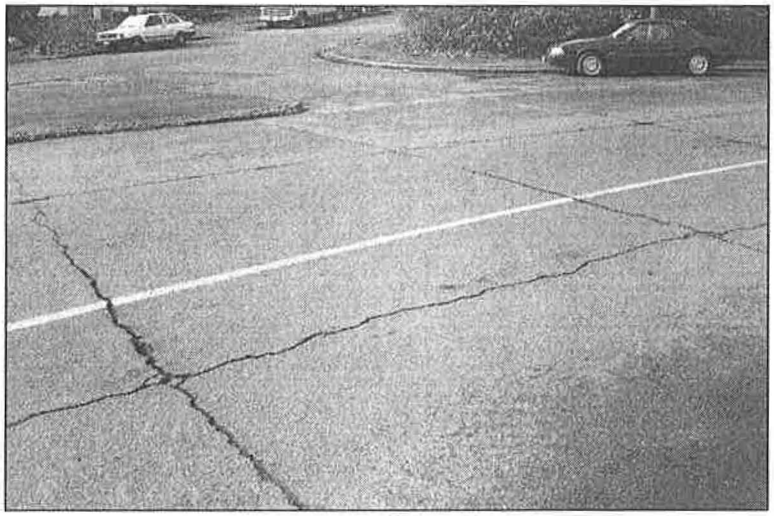

Figure 2.1 Reflective Cracking.

It is interesting to note that there are multiple views of the causes of reflective cracks. For example, Witczak and Rada (1992) stated that reflective cracks are mainly caused by thermal, moisture and/or traffic loading induced movement of the PCC pavement beneath the HMA overlays. The expansion and contraction of the slab causes horizontal movements that produce strains in the HMA overlays that exceed its tensile strength. Differential vertical movement can also be produced by traffic loads on the PCC pavement at either the existing joints and/or at any working cracks that are present in the slab. This localized movement over the joints and working cracks is more than the HMA overlay can support, resulting in the development of reflective cracks. McGhee (1975) also supports the idea that reflective cracking is a function of traffic.

Shahin (1994) contradicts the above theory stating that traffic loading does not cause reflective cracking. Instead, he concludes that the reflective cracks are a result of thermal and moisture induced movements of the PCC slab only. Shahin states that traffic loading does contribute to the deterioration of the pavement, but only after the thermal and moisture movements have created the reflective cracks.

Jayawickrama and Lytton (1987) stated that formation of reflective cracking is caused by shearing, bending, and thermal stresses. These three stresses are a function of traffic loading, thermally induced expansions and/or contractions or a combination of both. Figure 2.2 illustrates the theory behind the mechanisms that produce reflective cracks.

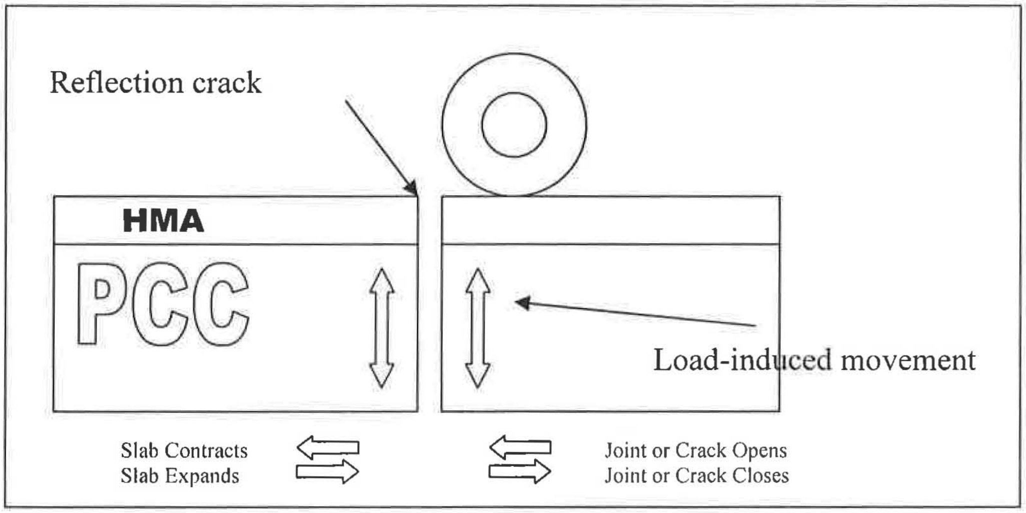

Figure 2.2 Reflective Cracking Description(After Fitts, 2001).

It is important to keep in mind that even though the reasons for the movement of the PCC are still in debate, it is common understanding that it is the movement that causes the reflective cracks to occur in the HMA overlay. The movement of the PCC slab causes excessively high strain levels to develop at the bottom of the asphalt layer, above the joints and cracks, which leads to upward crack propagation (Freeman, 2002). Jayawickrama and Lytton (1987) have done extensive research dealing with reflective cracking. Their research was based on the principles of fracture mechanics, which helped determine that PCC pavement movement causes reflective cracking in HMA overlays.

The presence of reflective cracks throughout the HMA overlay will decrease the quality of ride throughout the entire project. By rubblizing an existing PCC pavement, the PCC loses the ability to act as one unit and is then replaced by a crushed PCC slab that resembles a puzzle-like base. This base has proven to be 1.5 to 3 times stronger than a high quality, dense-graded crushed stone base (Thompson, 1997; Kirk, 2001). According to research conducted by Thompson (1997), this base is most effectively obtained with a resonance breaker. This transformation process aims to eliminate the reflective cracking and increase the ride quality by totally changing the behavior of the layer directly beneath the HMA overlay.

REFLECTIVE CRACKING MITIGATION STRATEGIES

Over the years, agencies responsible for the rehabilitation of PCC pavements have tried many materials, processes, and construction methods to eliminate or reduce reflective cracking in HMA overlays. These methods have included, but are not limited to, stress absorbing membrane interlayers (SAMIs), break and seat, crack and seat, and rubblization. The last three are commonly referred to as fractured slab approaches. Each of these will be addressed below.

Stress Absorbing Membrane Interlayers

The SAMI was originally developed by the Arizona Refining Company (Molenaar, 1986). It consists of a rubber bitumen tack coat that includes precoated chips with a diameter of approximately 1/3 to 1/2 of an inch (8.5 to 13 mm). The application of the SAMI separates the existing PCC pavement from the HMA overlay to eliminate the presence of reflective cracking. This is accomplished by transferring the stresses horizontality rather than vertically, which will allow the different layers to act independently of one another, and thereby reduce reflective cracking.

Fractured Slab Approaches

The fractured slab approach is driven by the theory of Witczak and Rada (1992). They state, "The probability of reflective cracking is proportional to the horizontal movement at the joints and cracks, which in turn is directly proportional to the spacing between the joints." The crack and seat, break and seat, and rubblization processes are various methods that fracture the existing PCC pavement into smaller pieces. These processes reduce or eliminate the effective length of the original slab while still allowing for the pavement system to adequately protect the subgrade. This permits the applied load to be distributed over a large area. The use of the fractured slab approach for rehabilitation purposes has continued to grow in acceptance (Witczak and Rada, 1992).

Crack and Seat

The goal of the crack and seat process is to develop closely spaced cracks through the depth of the concrete slab. This permits load transfer through aggregate interlock with minimum loss to the structural value of the concrete layer. To achieve this, the PCC pavement is typically broken into 45 to 60 cm (18 to 24 in) pieces using a variety of modified hammers. These hammers consist of various types including pile-driving hammers, falling weight hammers, and hydraulic or impact hammers. After the slab has been effectively broken into the appropriate lengths, the pavement is then seated using a

35- to 50-ton (32 to 45 metric ton) pneumatic-tired roller. The seating of the PCC is essential to eliminate excessive void space in the fractured material, ensure sound structural support, and to obtain a consistent platform for paving.

Break and Seat

The break and seat process is used on jointed reinforced concrete pavements (JRCP). The goal of this method is to fracture the concrete around the distributed steel or to completely debond all the reinforcing steel within the slab. The debonding of the reinforcing steel is essential in the break and seat process. If the concrete is effectively broken into smaller pieces and the reinforcing steel remains bonded to the concrete, the slab will still behave as one unit and consequently not reduce the effective length of the original slab. Like the crack and seat method, the break and seat process also requires compaction after fracture using a 35- to 50-ton (32 to 45 metric ton) pneumatic-tired roller.

Rubblization Process

Despite the efforts to eliminate reflective cracking in the HMA overlays, the crack/break and seat process have not proven to be an effective means of control (Freeman, 2002). Research has shown that the crack/break and seat process will just delay the occurrence of reflective cracking 3 to 5 years (Thompson, 1999; Thompson, 1997). It is for this reason that other methods of rehabilitation for PCC pavements have been explored. Rubblization is a relatively new process that is much more likely to reduce the effective slab size, which has proven to be effective in eliminating the reflective cracking.

Rubblization of PCC pavement is a rehabilitation technique that uses a form of the fractured slab process, which includes transforming the original concrete slab into an aggregate base. The objective is to break the PCC pavement without allowing the slab to expand excessively, damage the subgrade in any way, and still produce a rubblized PCC that is uniform in particle size and distribution (Kirk, 2001). The rubblization of a PCC pavement transforms existing concrete slabs and recycles it in place to form a high quality granular base as seen in Figure 2.3. This fractured slab technique, along with a properly designed edge drain system, has proven to be an effective means of rehabilitating PCC pavement (Ksaibati, 1998).

The new granular base is considered to be a high-strength base material because of the particle-shape properties that rubblized particles possess. These properties can be described as a puzzle (Kirk, 2001). The granular puzzle acts as a flexible system using aggregate interlock to transfer the applied loads to the subbase or subgrade.

Figure 2.3 Rubblized Pavement.

There are currently two methods available to rubblize a PCC pavement, the resonance frequency breaker and the multiple head breaker. The process of rubblization is most effectively accomplished with the resonance frequency breaker according to Witczak and Rada (1992). This process has been proven effective on all types of PCC pavements.

In 1986 John Brizzell, a New York Department of Transportation materials engineer, observed the resonance breaker and thought that it might eliminate reflective cracking. He recommended that the process be called rubblization (Kirk, 2001). The resonance frequency breaker uses a 15 cm (6 in) shoe located on the end of a pedestal that is connected to a beam with 5443 kg (12,000 lbs) counter weight located at the opposite end of the beam to break up the PCC pavement. This machine uses a low amplitude (<12.5 mm) high frequency (44 Hz) shoe to deliver energy to the slab that is near the harmonic frequency of PCC. Figure 2.4 shows an illustration of the shoe that is used to rubblize PCC. This results in an area of high tension at the top of the PCC and causes the slab to fracture on a shear plane approximately 35 degrees from the pavement surface (Fitts, 2001).

The resonant frequency breaker can effectively rubblize 1.0 to 1.7 lane-miles/day (1.6 to 2.7 lane kilometers/day). Kirk (2001) suggests, in a study that was performed nine years after the Witczak and Rada study, that rubblization with the resonance breaker may not be the best rehabilitation alterative if the existing subgrade condition is deteriorated significantly, has extensive clays, and/or has entrained water. This observation is important because it stated that even though the rubblization process was originally recommended for all types of subgrade conditions, research suggests that it may perform better under certain circumstances. These conditions are not favorable for rubblization with the resonance breaker because protecting the subgrade is a top priority. With the adverse conditions mentioned earlier, the rubblization process could damage the subgrade (i.e., by forcing fractured PCC pieces into the subgrade causing structural inconsistencies or stress focal points).

The multiple head breaker (MHB) is the newest of the rubblization tools for fracturing PCC pavement. The MHB was developed by Badger State Highway Equipment, Inc. who provided the first prototype that was available for the 1995 construction season (Thompson, 1997).

Figure 2.4 Resonance Frequency Breaker (Photo courtesy of Dr. Kevin Hall, University of Arkansas).

The MHB (Figure 2.5) operates by dropping a series of twelve to sixteen 102 to 123 lbs. (454 to 545 N) hammers on the PCC slab generating between 2,000 ft-lbs and 12,000 ft-lbs. (2,700 and 16,000 J). This crushes the pavement into a granular base and is capable of pulverizing pavements widths ranging from 2 ft to 12.5 ft (.61 to 4.0 m) (Fitts, 2001). The MHB allows a one-pass rubblization process and eliminates the subgrade stability issues. The one pass approach of the MHB eliminates the shoving or punch through of fractured PCC pieces which damages the subgrade. It can effectively rubblize 0.75 to 1.0 lane-mile/day (1.2 to 1.7 lane-kilometers/day) (Thompson, 1997). It is interesting to note that although the MHB has a one-pass capability, the rate of rubblization per day is less than the rate of the resonance frequency breaker.

Figure 2.5 Multi-Head Breaker (photo courtesy of Dr. Kevin Hall, University of Arkansas).

Performance of Rehabilitation Strategies

When evaluating the performance of a rubblized PCC pavement with an HMA overlay, one of the main concerns is the reduction or elimination of reflective cracking. A number of investigations have been conducted to determine performance of rubblized projects and are described below. The Asphalt Institute conducted a study that included forty-three rubblized pavement sections that ranged in age from one to thirteen years (Fitts, 2001). The study consisted of data collected from various sources which included construction companies that had completed rubblized projects and the National Asphalt Pavement Association (NAPA). The information compiled included geographic location, traffic classification, environmental information, age, and pavement condition rating (PCR). The condition of the rubblized projects was measured in accordance with the publication IS-169,"*A Pavement Rating System for Low-Volume Asphalt Roads*". The PCR rating that was applied, in accordance with IS-169, was used as a baseline to evaluate the performance of the rubblized sections. Supplemental information that was collected, where available, included the type of concrete, thickness, type of subbase, thickness of asphalt overlay, and the existence of drainage systems.

Based on the limited amount of information, firm conclusions were not obtained from this study. However, it is important to note that there were no signs of reflective cracking in any of the rubblized pavement overlays. There were some transverse cracks reported on some of the older sections in a wet-freeze environment, but the cracks did not correspond to the joint locations in the original PCC pavements. Because of this, the transverse cracks were not found to be reflective cracks. Overall, the information obtained in the study supports the contention that rubblization eliminates reflective cracking in HMA overlays (Fitts, 2001).

Another nationwide study that predates Fitts' research, performed by Witzack and Rada (1992), included a total of 454 field projects in 34 states. This study evaluated the performance of the HMA overlays that was placed over the various fractured slab techniques, which included rubblized, crack and seat and break and seat. Of the 454 total projects that were included in the scope of this investigation, 19 of the sections were rubblized, 250 were crack and seat, 150 were break and seat, and the remaining 35 sections did not have complete data available, therefore it was impossible for the classification to be divided into three categories; rubblization, crack and seat and break and seat.

The objectives of this study were to increase the fundamental knowledge of the design, construction, and performance models available for rubblized pavements. The data were collected from various state and federal organizations. These included Transportation Research Board (TRB), National Asphalt Pavement Association (NAPA), and the Asphalt Institute (AI). Additional information that was obtained from the test sections included a visual distress survey, made in accordance with the pavement condition index (PCI) highway methodology, and nondestructive deflection testing (NDT) using a falling weight deflectometer (FWD).

Extensive analyses were conducted on the information obtained to conclude that the rubblization process performed the best, followed by crack and seat and break and seat performing the worst. Table 2.1 shows some results of Witzack and Rada's research. Here, they illustrate the different PCI prediction equations for the three rehabilitation types. It is important to note the predictors and the coefficients of each of the equations. The main predictors that contribute to the deterioration of the pavement are its age, air temperature, precipitation and thickness of each section.

Table 2.1 PCI Predictive Equations.

Rehabilitation Type	PCI Prediction Model	R^2
Rubblize	$PCI = 173.77 - 0.878t + 0.389h_o - 0.744P - 0.719T$	0.753
N = 19	$PCI_{lt} = 217.28 - 0.615t + 0.310h_o - 1.14P - 1.13T$	0.665
Crack/Seat	$PCI = 127.02 - 3.03t + 2.4h_o - 0.34P - 0.44T + 0.09E_{SUB}$	0.755
N = 250	$PCI_{lt} = 116.8 - 1.42t + 0.658h_o - 0.13P - 0.25T + 0.001E_{SUB}$	0.511
Break/Seat	$PCI = 100.0 - 0.249t^2 - 0.0027(T*JS) + 0.072E_{SUB} + 0.465h_o$	0.393
N = 150	$PCI_{lt} = 100.0 + 5.61t - 0.77t^2 - 0.0032(T*JS) + 0.81h_o$	0.044

Note: t = time in years; h_o = HMA overlay thickness (inches); P = annual average precipitation (inches); T = annual average temperature range (F°); E_{SUB} = subgrade (subbase) modulus (ksi); JS = joint spacing (feet); PCI = pavement condition index; PCI_{lt} = pavement condition index of longitudinal/transverse cracking; N = number of pavement sections.

Freeman (2002), with the Virginia Transportation Research Council (VTRC), conducted a study evaluating concrete slab fracturing techniques. He investigated the effectiveness of fractured slab techniques in arresting or retarding reflective cracking through asphalt overlays placed over severely distressed PCC pavements. The study included five pavement sections rehabilitated over a period of eight years. Of the five pavements studied, two were originally jointed plain (unreinforced) concrete pavement and the other three were jointed reinforced concrete pavement.

It is important to note that there were control sections used in this project. In the control sections, the pavement was not fractured prior to the application of the HMA overlay and was located just beyond the bounds of the fractured pavement. Freeman used annual visual distress surveys and transverse crack density to compare the test sections. It was noted that although this means of comparison did not capture the severity of the transverse cracking, the frequency in which the transverse cracks appeared was represented accurately.

The study found that reinforced pavements that had been fractured, seated, and overlaid did not perform well. However, when the fracturing slab techniques were applied to JPCP, they were effective means of retarding the formation of reflective cracking (Freeman, 2002).

Mr. Gary Brunson, an ALDOT Materials Engineer, has experienced similar positive results with the rubblized pavement sections that are located in Alabama. When compared to other rehabilitation techniques employed in Alabama highway systems, Brunson reported excellent performance from the rubblized sections. He also reported that the performance life of the rubblized pavement ranges from eight to ten years (2001).

Because of Mr. Brunson's preliminary assessment, additional research into quantifying the performance of rubblized pavement was investigated as part of this research.

COST OF CONSTRUCTION

With research to support the feasibility of the fractured slab procedures, another very important concern that all agencies face when dealing with pavement rehabilitation is the cost of construction. The World Bank (1996) conducted research on the feasibility of pavement system maintenance. They found that it will cost four to five times more if maintenance is delayed until the pavement loses 80% of its original quality than it would if maintenance was done during the first 40% drop in pavement quality (Figure 2.6). It is also important to notice the trend in which the pavement tends to deteriorate; with this downward exponential deterioration rate, not only does the rehabilitation save monetary resources, but also increases the projected life of the pavement tremendously.

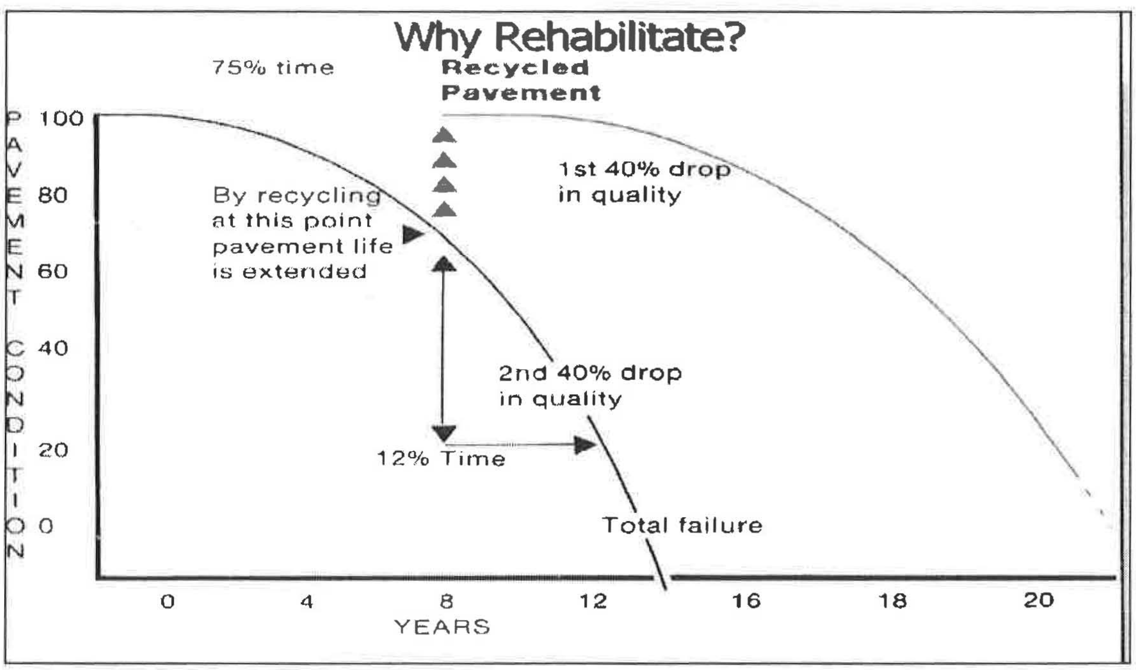

Figure 2.6 Benefits of Pavement Rehabilitation (Asphalt Recycling and Reclaiming Association, 1996).

The Nevada Department of Transportation (NDOT) conducted a cost analysis survey in 1997. They reported that the cost of Break/Crack and Seat was $341,000/mile, the cost of rubblization was $561,000/mile, and the cost of total reconstruction was $2 million/mile. NDOT went further to state that even though the initial cost of the Break/Crack and Seat was cheaper than the rubblization over a 35-year analysis period, the cost of both methods would eventually turn out to be approximately the same.

Analyses were also conducted from information provided by ALDOT. It revealed that the unit price of rubblization for the state of Alabama averaged a price of $1.99 per square yard. The prices referenced for Nevada DOT are for total construction cost, while ALDOT's unit price is based only on the cost of the actual rubblization of the PCC pavement. The cost information is presented in both manners to serve as a comparison to the total construction cost and the unit pricing.

DESIGN SPECIFICATIONS

After reviewing the costs of incorporating the fractured slab techniques, it is essential that the product is consistent and reliable. To this end, ALDOT and other agencies have formulated design and construction specifications based on their engineering field experience with fractured slab techniques.

The Asphalt Institute has established a seven-part process for rubblizing PCC pavement that is outlined below (Fitts, 2001):

- Remove any existing overlay.
- Install an edge drainage system, preferably two weeks before fracturing the concrete.
- Sawcut the full thickness of the PCC pavement, along the longitudinal joint, if the adjacent pavement is to remain intact.
- Rubblize the PCC pavement.

- Cut and remove exposed reinforcement.
- Roll fractured PCC.
- Place HMA.

The thickness of the overlay is determined by using the "effective thickness" approach that is detailed in 1993 AASHTO *Guide for Design of Pavement Structures*. The AI recommends a minimum overlay thickness of 5 in (125 mm) over the fractured concrete. The typical overlay thickness used on Interstate highways ranges from 8 to 12 in (200 to 300 mm). This value depends on the amount of traffic and the existing site conditions (Fitts, 2001).

ALDOT's specification for rubblized concrete pavement can be found in section 459 of the Alabama Standard Specification, 2001 Edition. It should first be noted that it is very important to choose the proper equipment to rubblize the PCC pavement. Equipment should be chosen to achieve its specific duty without displacing any rubblized pavement into the base or subgrade. The contractor is responsible for monitoring the progress of the rubblization equipment throughout the duration of the project.

The first step in the rubblization process of a PCC pavement, according to ALDOT, is for the contractor to set up a "test strip". A "test strip" is located within the section being rubblized and will be no longer than 1,000 feet in one traveled lane. The "test strip" will allow the contractor to make the appropriate adjustments to the rubblization equipment to compensate for the strength of the subgrade. It will also help determine the suitable coverage of each roller in order to achieve the required seating for the project. When the contractor chooses a location for the "test strip", it is good idea to consider an area which has both a portion of cut material and a fill material to gain a more accurate representation of how the entire project will react to the rubblization process (ALDOT, 2001).

Located within the "test strip", an area of 3 by 3 feet (1m by 1m), at intervals determined by the engineer, the rubblized pavement shall be totally removed. These 9 square feet ($3m^2$) will act as a gauge to see if the minimum particle sizes are being met and also aid the engineer in determining the most efficient process for completing the entire project. Once this process has been established, it will be used for the remainder of the project unless the project conditions change (ALDOT, 2001).

Before the HMA can be placed, the proper seating of the rubblized material must be achieved. The compaction is obtained using no less than one pass of a vibratory steel wheel roller having a nominal gross weight of at least 20 tons (ALDOT, 2001). Figure 2.7 demonstrates the use of the steel roller to seat the rubblized pavement. In the next step of the compaction process, the steel wheel roller will be followed by at least one pass of a self-propelled pneumatic roller. The pneumatic roller should have a gross weight of at least 25 tons. This roller is to be followed by two additional passes of the steel wheel roller. When rubblizing reinforced concrete, the reinforcing steel is to be left in place, unless it is exposed at the surface. In the event the reinforcing steel is exposed during the rubblization or compaction process, it is to be cut below the surface and removed from the job site (ALDOT, 2001).

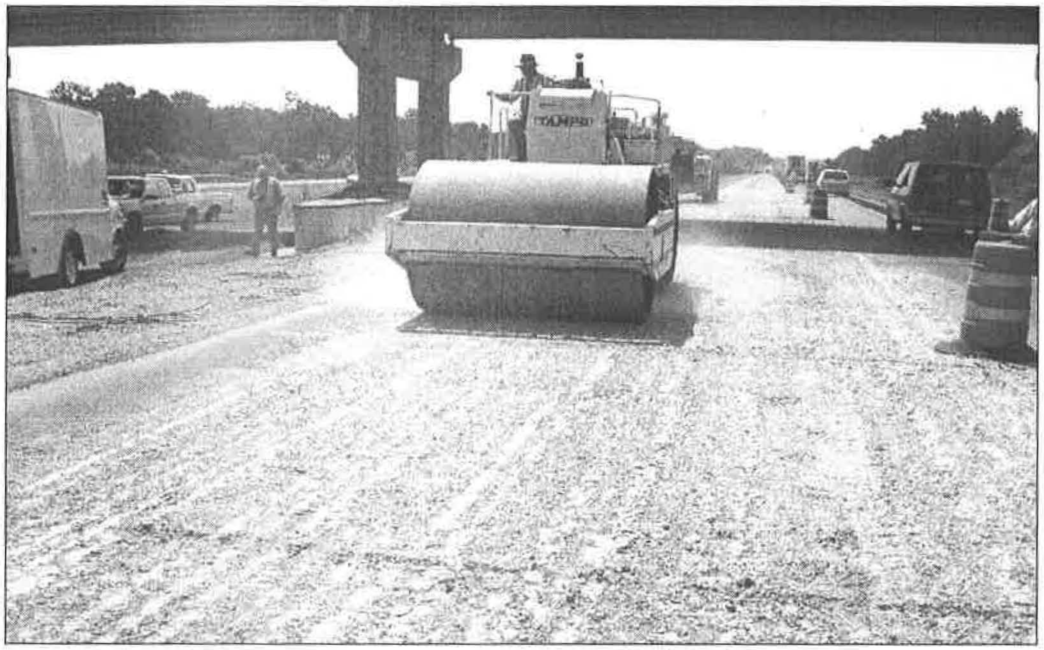

Figure 2.7 Seating the Pavement (photo by Kevin D. Hall, University of Arkansas).

When dealing with traffic, the specification states that traffic is not allowed on the rubblized pavement before the initial asphalt overlay is in place. It also states that under no circumstances should the time between the completion of rubblization phase and the first asphalt overlay be more than 48 hours.

There are a few differences between the AI and ALDOT specifications for rubblizing PCC pavement. One major difference is the requirement of edge drains. The AI requires installation of the drains two weeks prior to rubblization, however, ALDOT does not require them, but tends to incorporate them into the rubblized projects. The edge drain requirement is significant because of the time delay that the AI requires between the installation of the drains and rubblization of the PCC pavement. This two-week delay in construction can prove to be inconvenient and expensive. The construction cost increase would be caused by the relocating of personal and equipment to and from the job site to accommodate for this time-delayed interruption.

A second difference is that ALDOT requires and closely monitors the test strip. The advantage of using a test strip is to calibrate the rubblization equipment to the existing site conditions. The 3 by 3 foot pit to physically observe the performance of the rubblized equipment confirms or denies the required particle sizes that are obtained at the bottom of the PCC pavement layer.

Although there are some differences between the AI and ALDOT specifications for rubblizing PCC pavement, the goal of each method remains the same. There are no major differences with the equipment and theories used in each specification. The variation lies in the process in which the rubblization is completed. For this reason, either of the above specifications can be used to rehabilitate PCC pavement, while protecting the existing subgrade.

DESIGN PROCEDURES

After the PCC pavement has been successfully rubblized and properly seated, the next task is to construct the HMA overlay. An overlay thickness must be specified before construction can begin. In Alabama, the AASHTO method is used when determining the thickness of the overlay. Alabama engineers assign a structural coefficient of 0.23 per inch of material rubblized and use 8 years for the design period in the design process of an overlay. Using this design information, ALDOT has had thicknesses that ranged from 5.4 inches on the low traffic volume roads to 13 inches on interstate routes (Brunson, 2001).

SUMMARY

This review has provided an understanding of pavement rehabilitation and reflective cracking. There are multiple views on the direct causes of reflective cracking, but all agree that they are the result of PCC pavement movement. The review states that these reflective cracks can be eliminated or reduced by several different means, such as SAMI, break/crack and seat, and rubblization techniques. All of these techniques are still utilized today; however, the most widely used technique is through the previously stated fractured slab procedures. The next chapter will focus on the rubblized sections in Alabama that were the basis of this investigation.

CHAPTER 3 - TEST SECTIONS

INTRODUCTION

Eleven years ago, ALDOT opened their first rubblization project to traffic. At that time, rubblization had only been in practice for approximately five years as a rehabilitation technique (Kirk, 2001). Because this process was still in the early stages of development, the full advantages or disadvantages were largely unknown. Over the years, more research has been done and to date ALDOT has proposed seventeen rubblization projects throughout the state. Of these seventeen original sections, only nine to date (August, 2002) were available to evaluate the performance and two are near completion. There were eight sections unavailable for observation for one or more of the following reasons:

1. The initial scope of the rehabilitation changed.
2. There was not enough data to obtain an accurate representation of the performance of the pavement.
3. The rubblization project has not been completed.

The age of the available nine-test sections range from the youngest at 2.5 years (section 9) opened to traffic, to the oldest, at 11 years (section 4) opened to traffic.

The next two sections will discuss the ALDOT rubblized projects in detail. Geographic and environmental information are examined, as well as the maintenance and rehabilitation efforts.

GENERAL PROJECT INFORMATION

The rubblized pavement sections are all located on interstate highway systems with relatively high traffic volumes. The rubblized sections are exposed to an average annual temperature of 65°F with approximately 54 inches of rainfall annually. As for the geographic location of the nine pavement sections, five are located in the north central portion of the state, two are located in the west central portion of the state, and the remaining two sections are positioned in the southern portion of the state (Figure 3.1).

These test sections have a variety of base and sub-base materials along with lift thicknesses. The thickness of each pavement layer was obtained from the original construction plans. The materials used in the rubblized sections included an improved roadbed, lime-stabilized base, cement treated base, and 6-inch aggregate subbase. These materials were not used in every section.

The PCC pavements that were rubblized had an average thickness of 9.3 inches before rubblization, with a minimum thickness of 8 inches and a maximum thickness of 10 inches. The HMA overlay thickness was obtained by converting spread rates from the rehabilitation constructions plans to a total lift thickness. Each test section is comprised of different materials. Figures 3.2 and 3.3 graphically depict the type and amount of thickness of each material present.

The types of rubblized concrete that were evaluated in this project were jointed plain concrete pavement (JPCP) and continuous reinforced concrete pavement (CRCP). Sections 3 and 4 were the only CRCP that were evaluated in this project; the remaining sections were constructed as JPCP.

There were two methods that were used to fracture the existing concrete. The multi-head breaker (MHB) was used in section one and the resonant head breaker (RHB) was used in the remaining eight sections. These two methods were discussed in detail in the previous chapter.

MAINTENANCE AND REHABILITATION

The rubblized sections have had very little maintenance performed since the date each section was opened to traffic. Only sections three and four have had any structural maintenance. This included a 3-inch (76mm) overlay for section 3 in July of 1999 and a 4-inch (102mm) overlay for section 4 in April of 2001. The only other maintenance reported was a 1-inch (25mm) wearing surface on section 7 in December of 1996; this maintenance occurred shortly after the section was rubblized and overlaid.

To summarize the maintenance and rehabilitation of the ALDOT sections, Table 3.1 and 3.2 are shown to depict the basic data from each section.

Figure 3.1 Map of Rubblized Sections.

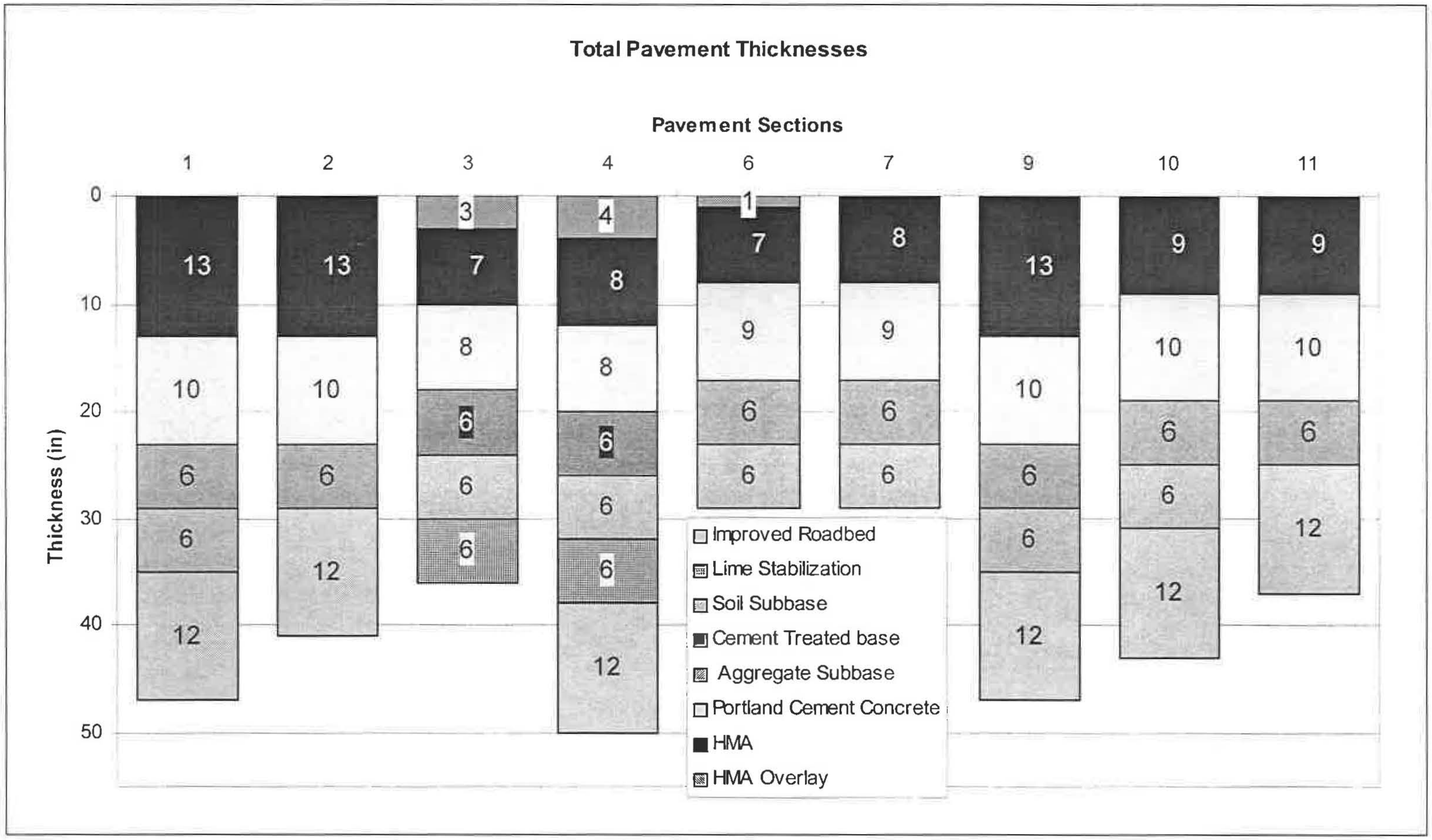

Figure 3.2 Total Pavement Thicknesses of Rubblized Sections.

Table 3.1 Section Information.

Section #	County	Division	Interstate	Section Markers (mi)	Total Length (mi)	Pavement Type	Unit Price Rubblizing ($/yd^2)	Date Accepted
1	Talladega	4	20	164.62 - 168.73	4.11	JPCP	2.00	NA
2	Talladega	4	20	168.73 - 174.11	5.38	JPCP	2.35	6/17/93
3	Sumter	8	59	17.43 - 21.5	4.07	CRCP	2.43	9/9/92
4	Sumter	8	59	21.5 - 26.89	5.39	CRCP	2.20	9/9/91
6	Butler	6	65	125.15 - 132.72	7.57	JPCP	1.86	6/23/97
7	Butler	6	65	132.72 - 139.9	7.18	JPCP	1.85	6/4/96
9	Jefferson & Blount	3	65	280.4 - 285.4	5.00	JPCP	1.69	10/3/01
10	Blount	3	65	285.4 - 291.2	5.80	JPCP	1.65	11/16/98
11	Cullman & Morgan	1	65	316.47 - 319.14	2.67	JPCP	1.92	7/10/96

Table 3.2 Section Information, Continued.

Section #	Date Opened to Traffic	ESAL's to Date	HMA Thickness, (in)	PCC Thickness, (in)	Original Joint Spacing (ft)	Were Edge Drains Used	Method of Rubblization
1	Aug-98	1.13E+07	13	10	unk	Yes	Multihead
2	Jun-93	1.99E+07	13	10	unk	unk	Resonance
3	NA	1.44E+07	7	8	NA	Yes	Resonance
4	NA	1.49E+07	8	8	NA	Yes	Resonance
6	Jun-97	6.40E+06	7	9	20	No, Granular System In place	Resonance
7	Jun-96	7.06E+06	7	9	20	No, Granular System In place	Resonance
9	NA	6.65E+06	13	10	20	Yes	Resonance
10	NA	8.09E+06	9	10	20	Yes	Resonance
11	Aug-95	1.22E+07	9	10	20	Yes	Resonance

Unk = Unknown; NA = Not Available

Given the details about each rubblized section above, the next chapter will focus on the research methodology and method of analysis regarding the data collected from the rubblized sections. It will describe the methods of data processing, comparison of the rubblization sections, how performance was evaluated and the statistical tools employed to assess the sections.

CHAPTER 4 - RESEARCH METHODOLOGY

BACKGROUND AND INTRODUCTION

ALDOT has contracted with Roadware to collect pavement distress information on Alabama's highway system to monitor its performance. As of 2002, the collection process is still evolving. Today, the state contract with Roadware is to inventory the pavement distresses every two years. This information is gathered and quantified using an electronic automated process provided by Roadware. The information is then reported to the state for further analysis to support the pavement management system.

For the purposes of this project, ALDOT personnel provided all of the information in Alabama's pavement management system (PMS) through the 2002 survey. As a result of the rubblization process being a relatively new procedure, there was limited number of surveys available for research.

To obtain the most up-to-date information, the Auburn University Highway Research Center's Automatic Road Analyzer (ARAN) van was employed to survey all of the rubblized pavement sections in the state, collecting international roughness index (IRI) and rut measurements during the summer of 2002. Both vehicles that the state and Highway Research Center employ are Roadware products and both utilize an automated electronic collection process to gather the distress information.

The major difference between the two collection devices is the number of types of distress that the state's vehicle has the capability of collecting. The DOT Roadware vehicle is capable of collecting a wide variety of distresses and road conditions. These are listed in Table 4.1. The Auburn University Highway Research Center vehicle uses the same technology, but is not equipped to collect all of the distresses that the state's vehicle collects. The information available with the Highway Research Center's van is the International Roughness Index (IRI) and the rut depth.

The first objective when this large amount of data was received was to inventory each database to find out what information was available for analysis. Then a method was chosen that could be applied to all of the test sections in an accurate and efficient manner. This chapter will explain the basic procedure used to effectively compare the performance of the rubblized pavement sections in Alabama. The topics discussed include the available data, the data processing, comparison of rubblized sections, and statistical analysis.

AVAILABLE DATA

Roadware provides the state with a wide variety of information dealing with the geographic location of the pavement section and also distresses that are present. Some of the key pieces of information that are included are the location of the surveyed pavement, the pavement surface type, and the surface distresses that are present, including quantity and severity. From the data provided by Roadware, the distresses that were examined included transverse cracking, IRI, rut depth, alligator cracking and longitudinal cracking. These distresses were chosen because of their engineering significance in relation the reflective cracking.

The distresses mentioned above were chosen for a variety of reasons. Rubblization was implemented as a rehabilitation technique to eliminate the development of reflective cracking by transforming the concrete slab into an aggregate base

(Thompson, 1999). Therefore, the distress data that are of primary importance are transverse cracking, because this is a measure of the effectiveness of the rubblization process. Another very important distress gathered from the Roadware data was International Roughness Index, or IRI, which is a measure of ride quality. Ride quality, in turn, is how the traveling public generally rates a pavement's performance.

A third distress investigated was the rut depth. It was examined for safety and structural reasons. Pavements that have a rutting failure have a greater potential to hold surface water. This would increase the chances of the vehicle hydroplaning and losing control. It was also examined because significant amounts of rutting may indicate major structural failure of the pavement.

The fourth distress was alligator cracking. Alligator cracking is a form of cracking caused by a fatigue failure in the pavement. This is monitored to evaluate the ability of a rubblized pavement section to withstand traffic loading (Shahin, 1994).

The last distress mentioned was the amount of longitudinal cracking. Longitudinal cracking was examined to investigate another form of pavement cracking that could be indicative of structural problems.

By evaluating each of the above-mentioned distresses, it was possible to gain a clearer picture of the performance of the rubblized sections. After the information provided by ALDOT had been assessed and the crucial parameters were identified, a process for dealing with the information was established. An accurate description of the performance was obtained after comparison of the above-mentioned performance indicators of each section to other known and accepted parameters within the pavement industry.

When dealing with the information from the pavement management system database, it was essential that all the information was evaluated in such a way that the pavement sections were compared using the same scale and parameters. For example, there were variables both in the construction of each section and in the time each of the pavement sections were rubblized that had to be taken into consideration. This was to ensure that the rubblized sections were compared on a relatively equal basis. The procedures that were applied to each of the rubblized sections data are discussed in further detail in the following sections.

DATA PROCESSING

Before any assessment of the performance of the rubblized sections could be made, the pavement distress data had to be extracted from ALDOT's PMS databases. It was very important to establish a procedure that could efficiently be applied to all the PMS databases while accurately representing the performance of the pavement. When establishing a procedure to present the distress data there were two main issues that had to be addressed:

- Comparison of the Distress Surveys
- Data Analysis and Presentation

The first issue was comparing the various distress surveys that were available to identify any possible differences in format or types of data available. The second issue was to address how to accurately obtain a description of the pavement performance that could be easily and effectively applied to each rubblized pavement section in Alabama. By successfully addressing the above two concerns, this would allow for a procedure to be established that will serve as a method for obtaining the current condition and tracking the future performance of Alabama's rubblized pavement sections.

Comparison of the Distress Surveys

To begin the evaluation of the available databases, the 2001 PMS database was evaluated to gain an initial estimate of the performance of the rubblized sections. This evaluation helped to finalize a procedure to work with the remaining databases.

The first step was to extract the specific distress data enumerated above. This extraction was performed with queries that searched and removed only the data that represented rubblized pavement sections. After the distress data were extracted, it was organized in a database that would house all of the performance information on Alabama's rubblized pavement sections. The distresses were then graphed in a variety of ways. This gave an objective view of the performance of the rubblized pavement.

The above process accomplished the goal of determining the performance of the rubblized sections as it related to the 2001 distress survey, now the remaining databases had to be examined to verify that the process could be applied. As the 1996 data were reviewed, it was found to have substantially less data points than the 2001 data. To compare the varying amount of individual data points reported for the same sections for different years, refer to Figure 4.1 for year 2001 and Figure 4.2 for 1996. Figures 4.1 and 4.2 represent the amount of data collected for each of the distresses. For this reason, only the IRI distress is presented here. Because of this variation in the amount of data collected over the years, the analysis options were limited, and other options of comparing the performance of the rubblized sections had to be explored.

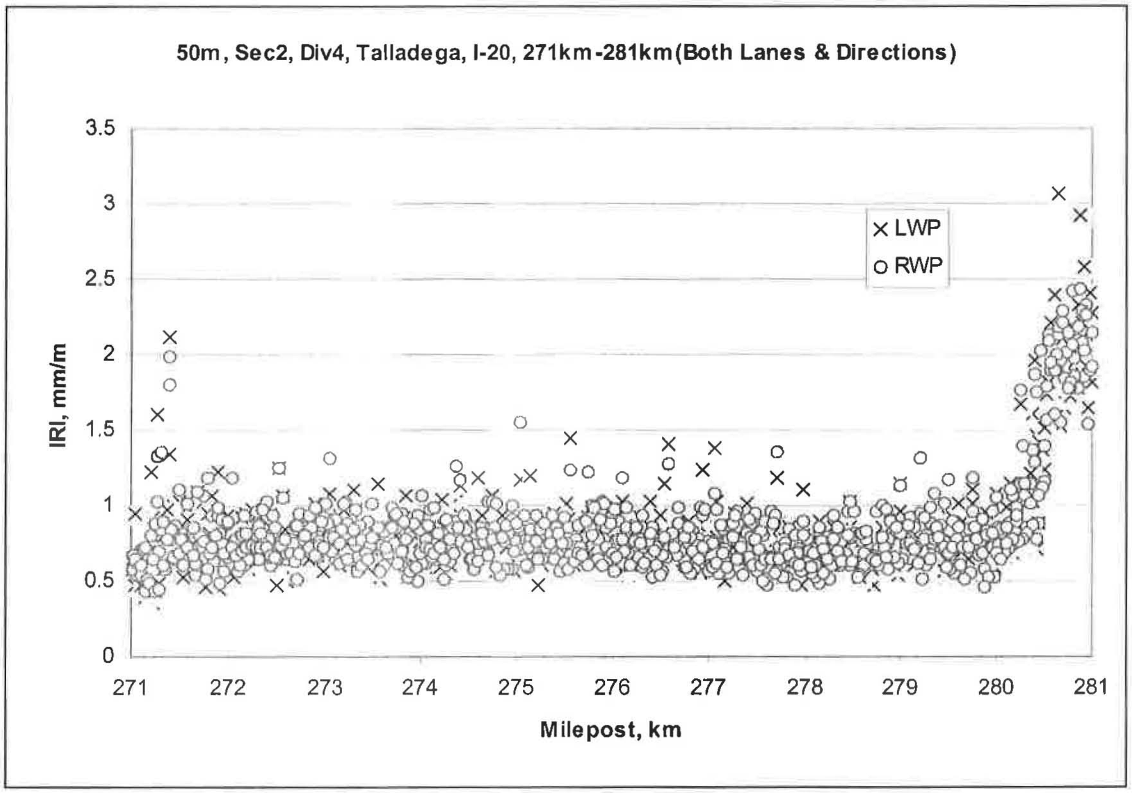

Figure 4.1 2001 IRI vs. Distance for Section 2.

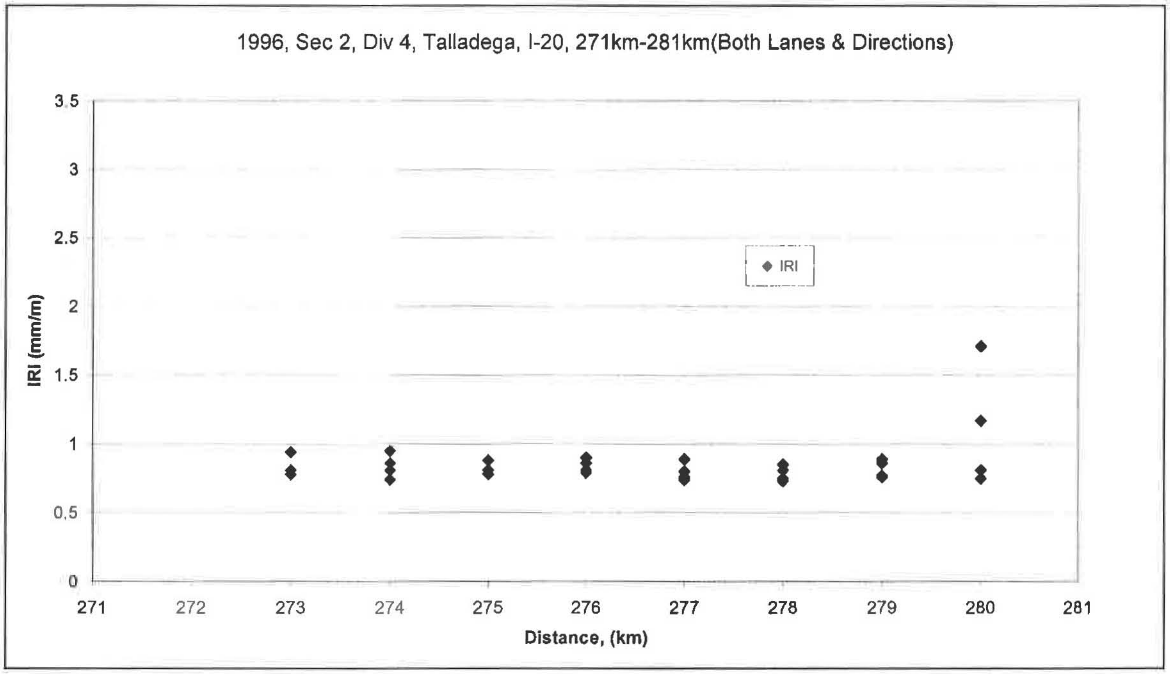

Figure 4.2 1996 IRI vs. Distance for Section 2.

Data Analysis and Presentation

Having discussed the amount of data and types of data available for analysis, the next step was to organize the information in a way to allow the comparison of each rubblized section. To begin the comparison of the rubblized pavement sections, the distresses obtained from ALDOT that were discussed earlier were plotted against the age of the section (Figure 4.3). The format used above proved to be effective in quickly determining the overall performance of the rubblized sections.

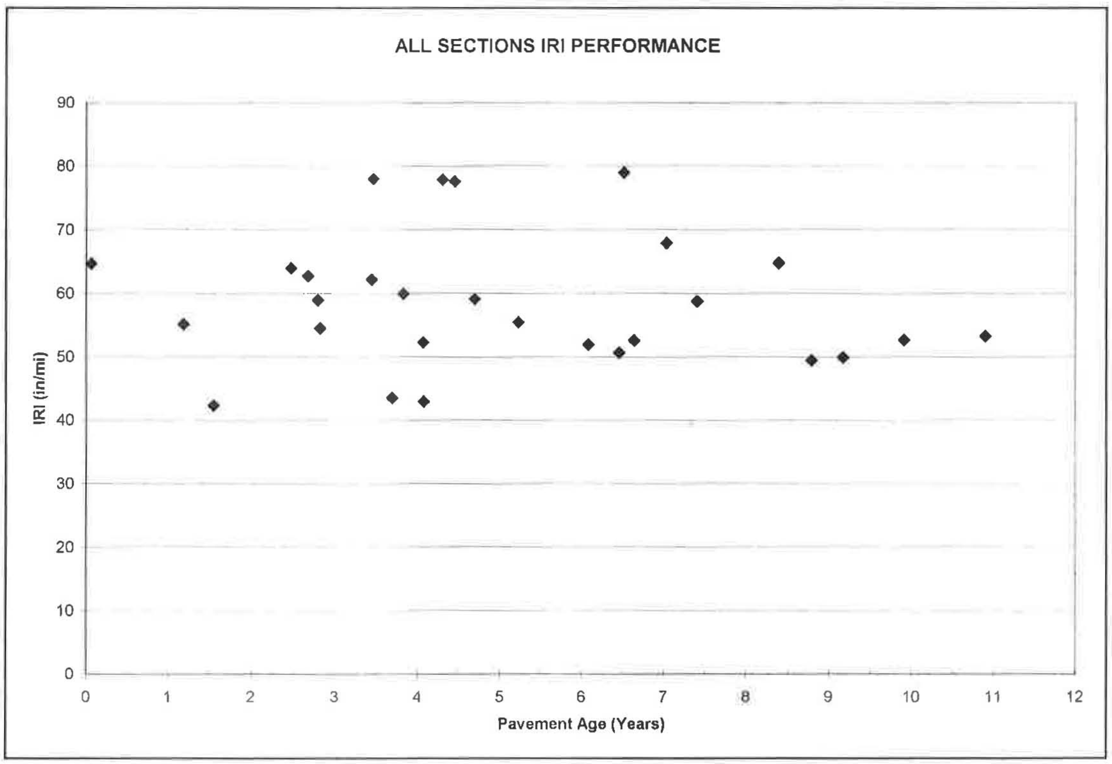

Figure 4.3 IRI vs. Pavement Age.

Upon further analysis, unique variables presented themselves for each section. This prohibited the comparison between each individual section. For example, the numbers of years the pavements were open to traffic and different construction thickness of each layer are just two of the factors that prohibited the direct comparison between sections. Also, because the rubblized sections were constructed at different times this meant that each section had different traffic volumes, thus providing another reason why it was not possible to directly compare the individual sections to one another. With the above-mentioned confounding variables, it was essential that a common element be identified to allow for the consistent and accurate comparison of the rubblized pavement performance. The next step was to calculate the Equivalent Single Axle Loads (ESALs) that served as a base line to compare the pavement sections.

Additional information, obtained from ALDOT's traffic database, was needed for the calculation of ESALs to compare the rubblized sections. The information consisted of the average annual daily traffic (AADT), the percent of heavy trucks and the truck factor was obtained from ALDOT. This information enabled the calculation of ESALs. The ESALs were found by converting mixed traffic information to a number of standard axles for design or in this case it establishes a baseline for comparison. ESALs are defined as the number of applications of a standard axle (general 80 kN single axle with dual tires inflated to 550 kPa) required to produce the same damage or loss of serviceability as a number of applications of one of more different axle loads and/or configurations (Yoder and Witczak, 1975). Equation 4.1, shows how ESALs were calculated for this project:

$$ESALs = AADT * \%Trucks * TF * 365 * LD * DD \qquad \text{(Equation 4.1)}$$

where:

ESAL = Equivalent single axle load

AADT = Average annual daily traffic

% Trucks = Percent vehicles that are heavy commercial

TF = Truck damage factor (ESALs/vehicle) (0.99 per ALDOT)

LD = Lane Distribution (typically 90%)

DD = Directional Distribution (typically 50%)

For the initial comparison, the calculated ESALs were compared to the IRI data from the PMS database. By comparing the ESALs to IRI, it was possible to investigate the effects of traffic on the rubblized pavement sections in terms of roughness (Figure 4.4).

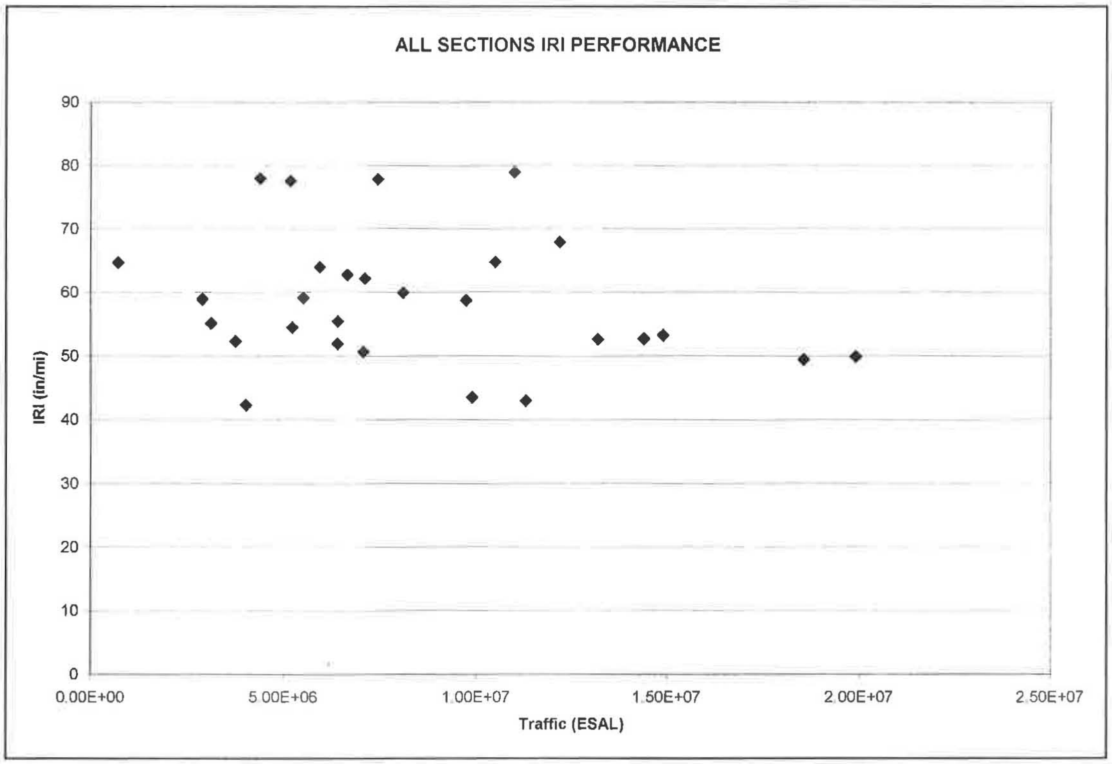

Figure 4.4 IRI vs. Traffic.

To gain a more accurate measure of the effect that traffic had on the rubblized sections, all of the distresses were plotted against ESALs. By graphing this relationship, the results provided a clear and concise method to observe the effects of traffic on the various distresses.

One distress that requires a special discussion is transverse cracking. In this project the transverse crack distress was reported as transverse crack density. Transverse crack density is a method of normalizing the transverse count. Dividing the number of transverse cracks by the total length of the pavement section yields cracks per mile. This number can then be used to readily compare each of the rubblized pavement sections. Freeman used this method of normalizing transverse crack density in his 2002 research of slab fracturing technology. One might be concerned with the fact that the severity of the distress is not considered in this approach. If the transverse crack is one-eighth or one-half of an inch, they are represented the same with the crack density approach. The important concept to remember is that with any slab fracture technique the purpose of this procedure is to eliminate the presence of reflective cracking (transverse cracking) in the pavement overlay. Therefore, the rate in which the transverse cracks occur, rather than the severity, is the main focus when evaluating rubblized pavement.

The above-mentioned methods of comparing the rubblized pavements were very helpful to understand the effects of traffic on rubblized pavements, with similar site conditions and construction specifications. However, comparing the distress directly to ESALs does not take in consideration the different thicknesses of the HMAC overlays. To account for this variation in overlay thickness the cumulative ESALs were divided by the pavement thickness to yield ESALs per inch (Figure 4.5).

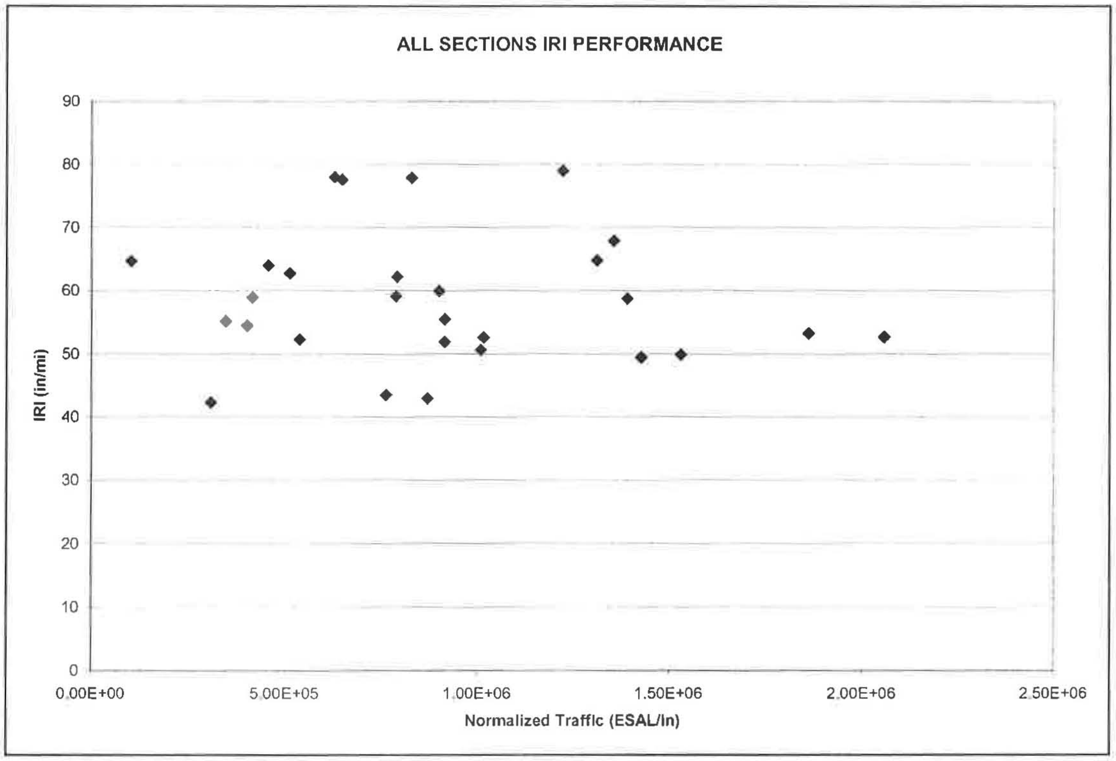

Figure 4.5 IRI vs. Normalized Traffic.

Now that the variation in the initial construction dates and overlay thickness had been accounted for with the IRI, transverse cracks, longitudinal cracks, alligator cracks and rutting, they were compared to the normalized traffic (ESALs per inch). This was the final stage of the comparison process; by using the ESALs per inch, it enabled the comparison of the performance of each pavement section to one another. As shown in Figure 4.2 through 4.5 with the IRI distress there was not a recognizable advantage to reporting all three methods (age, ESAL, ESAL/in) therefore only the graphs with ESAL/in are reported in Chapter 5.

After the pavement sections were compared to one another they were compared to established industry standards to gain a scale in which to rate the overall performance of the rubblized pavement sections. The nature of this project did not allow for the construction of a control section to be used as a direct comparison for evaluating the performance of the rubblized sections.

Initially, pavements that were constructed around the same time with similar conditions were sought out to act as control sections. These potential control sections would directly compare the performance of the rubblization process. In order to locate the possible control sections, the Federal Highway Administration Long-Term Pavement Performance (LTPP) database was explored. A LTPP pavement section was located on U.S. 280. It has been flexible pavement since the original date of construction. This LTPP test section was investigated and found to have different construction materials, varying construction procedures, years of traffic that would be difficult to accurately quantify, and finally the performance information had been reported over the years in a number of different formats. It is for these reasons that comparing the performance of the rubblized sections directly to the LTPP pavement was not an option, although the LTPP database did provide graphs of the distresses over time. These graphs served as a guideline to compare the rubblized performance and helped give a representation of performance of the pavement.

Having exhausted a number of different possibilities to accurately measure the rubblized pavement performance, it was necessary to explore other means of evaluating the performance of the rubblized sections. In the past, agencies responsible for the management of pavement systems have established thresholds based on knowledge of the materials used in the construction process and decades of experience studying past pavement performance. These thresholds were used to establish critical values for warranted pavements. For example, the Wisconsin Department of Transportation has established a five-year warranty specification that states that at the end of the five-year time frame, the total amount of alligator cracking will not exceed 10% of the total pavement area. In Table 4.2, there are a number of agencies that have established a various array of critical pavement thresholds. By comparing the performance of the rubblized pavement sections to these industry standards allowed for a direct assessment, which proved beneficial in the analysis of the performance. These thresholds served as a means of comparing the performance of the rubblized pavement sections.

Table 4.2 Pavement Rating Criteria.

<table>
<tr><th colspan="8">Pavement Performance Criteria</th></tr>
<tr><th></th><th>Source</th><th>Time</th><th>IRI (in/mi)</th><th>Transverse Cracking (cracks/mi)</th><th>Rutting (in)</th><th>Longitudinal Cracking (% of total length)</th><th>Alligator Cracking (% of total area)</th></tr>
<tr><td>1</td><td>Forthcoming AASHTO 2002 Design Guide</td><td>NA</td><td>252</td><td>88</td><td>.75</td><td>20</td><td>8</td></tr>
<tr><td rowspan="3">2</td><td rowspan="3">Wisconsin DOT Warranties</td><td rowspan="3">5 year</td><td rowspan="3">NA</td><td>251</td><td>.25 = mill & overlay with microtexture</td><td>190</td><td rowspan="3">10</td></tr>
<tr><td rowspan="2">251 cracks with 25% having band cracking or dislodgement</td><td rowspan="2">.52 = Remove & replace surface layer</td><td>95</td></tr>
<tr><td>If the above conditions having 25% band cracking or dislodgement</td></tr>
<tr><td>3</td><td>Oregon and Washington DOT's Warranties</td><td>3 year</td><td>133</td><td>Any Severity 2</td><td>.375</td><td>5.5</td><td>NA</td></tr>
</table>

Statistical Analysis

The rubblized pavement sections were surveyed to gather the distress information. This information was then compared to one another and to established industry standards to gain a better understanding of the performance of the rubblized sections. However, further investigation was needed to obtain a more accurate description of how each individual distress affected the deterioration of the rubblized pavement. To better understand the factors that contribute to the performance of the pavement, a statistical analysis was performed on the data using the MINITAB software package. A linear regression was calculated to determine the stochastic relationship between the pavement distress data.

The linear regression model assumes that the given data are linear and adjusts the values of the slope and intercept to find the line that best predicts Y from X. The model that the statistical package MINITAB follows is shown in Equation 4.2. The goal of the regression is to minimize the sum of the squares of the vertical distances of the data points from the line (curvefit.com, 1999). The process of linear regression is a simple mathematical relationship between two or more variables. It is used to obtain a quantitative measure of how extensively two or more variables are related. A variable to show the accuracy of this regression equation is R^2 (Equation 4.3). R^2 is referred to as the coefficient of determination and represents the fitting linear regression model. The regression process used the following form:

$$Y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \ldots + \beta_n x_n + \varepsilon \qquad \text{(Equation 4.2)}$$

Y is the response

x is the predictor

β_0 through β_n are the regression coefficients
ε is the error with a normal distribution with mean = 0 and standard deviation = σ

Note that:

R^2 = 1-(Sum of Square Error) / (Sum of Square Total) (Equation 4.3)

The first regression that was performed to predict IRI from ESALs, pavement age, HMA thickness, PCC thickness, type of PCC pavement (CRCP or JRCP), and process of rubblization (resonance or multi-head breaker). This combination resulted in coefficients that were too small because the magnitudes of the different parameters were not the same. With this information, a comparison of the same parameters was performed, but now changing the ESALs to million ESALs (MESALs). This reduced the magnitude of the inputs and adjustments and provided coefficients that were similar in magnitude and was used in the following regressions.

The remaining regressions examined the relationship between the distress average rutting, transverse crack density, percent longitudinal cracking, and average alligator cracking to ESALs, pavement age, HMA thickness, PCC thickness, type of PCC pavement (CRCP or JRCP), and process of rubblization (resonance or multi-head breaker). The above-mentioned regression analyses provide useful models that can directly support the findings in the literature. It is important to remember that the regression equations were based on a limited amount of data. The regression serves as a starting point for future studies.

All of the previous information concentrated on the procedure that was followed throughout this research. The procedure included how and what types of data available, how the data were processed, and the methods in which the rubblized pavements were evaluated. The specific results of the procedures will be discussed in greater detail in the following chapter, Results and Discussion.

CHAPTER 5 - RESULTS AND DISCUSSION

BACKGROUND

Having discussed the method for collecting and processing the information in the PMS database in Chapter 4, this chapter will concentrate on the comparison of the rubblized sections. The pavement distress data that were compiled from ALDOT's pavement management database allowed for the comparison of the rubblized sections and are presented in Table 5.1. This table will serve as a reference to aid in the comparison of the pavements' distresses in all of the Alabama rubblized pavement sections included in this study.

DISTRESSES BY SYSTEM

To begin the evaluation of the performance of the rubblized sections, recall from Chapter 4 the five distresses that were the primary focus. They were transverse crack density, average IRI, average rutting, percent longitudinal cracks per pavement section, and average alligator cracking; all of which were plotted against the three standards of measurement: age of the pavement, ESALs and ESALs per inch of overlay. However, since no additional information was obtained from examining distress as a function of these different parameters, only distress as a function of ESALs/inch are presented below.

Table 5.1 Performance Data.

Rubblization Performance Summary

Section	Year	Date Collected	Original Date Open to Traffic	Years Opened to Traffic	Length of Section (km)	Length of Section (mi)	ESAL's	Thickness (in)	Normalized Esals (Esal's/in)	Transverse Crack Density (Crack/mi)		Average IRI (in/mi)	Rutting (in)			Avg Longitudinal Cracking (ft)	% Longitudinal/ Length (ft/ft)	Avg Alligator Cracking (ft^2)
										Lane 1 (outside)	Lane 2 (inside)		RRUT	LRUT	Average			
1	1996																	
	2001	2/24/2000	8/1/1998	1.54	6.61	4.11	4.03E+06	13	3.10E+05	2.46	0.62	42.29	0.13	0.14	0.14	0.66	1	0
	2002 AL	4/29/2002	8/1/1998	3.69	6.61	4.11	9.89E+06	13	7.61E+05	0.12	5.85	43.51	0.17	0.15	0.16	0.50	1	0.3
	2002	9/18/2002	8/1/1998	4.07	6.61	4.11	1.13E+07	13	8.69E+05			42.97			0.14			
2	1996	4/10/1996	6/1/1993	2.82	8.66	5.38	5.23E+06	13	4.02E+05	0.69	0.23	54.50	0.28	0.19	0.24	6.56	12	2.91
	2001	2/24/2000	6/1/1993	6.64	8.66	5.38	1.32E+07	13	1.02E+06	18.42	2.23	52.58	0.20	0.22	0.21	1.31	2	1.24
	2002 AL	4/29/2002	6/1/1993	8.79	8.66	5.38	1.86E+07	13	1.43E+06	0.28	3.54	49.44	0.28	0.26	0.27	10.77	20	1.02
	2002	9/18/2002	6/1/1993	9.17	8.66	5.38	1.99E+07	13	1.53E+06			49.88			0.26			
3	1996	3/4/1996	9/1/1992	3.46	6.55	4.07	4.40E+06	7	6.29E+05	0.00	0.80	77.95	0.29	0.27	0.28	4.26	8	4.57
	2001	3/5/2000	9/1/1992	7.41	6.55	4.07	9.74E+06	7	1.39E+06	3.32	19.77	58.75	0.10	0.14	0.12	6.23	12	4.92
	2002	9/18/2002	9/1/1992	9.91	6.55	4.07	1.44E+07	7	2.06E+06			52.67			0.04			
	No ALDOT																	
4	1996	3/4/1996	9/1/1991	4.45	8.67	5.39	5.18E+06	8	6.48E+05	0.40	1.31	77.57	0.27	0.28	0.27	14.75	28	8.06
	2001	3/5/2000	9/1/1991	8.39	8.67	5.39	1.05E+07	8	1.31E+06	72.73	44.18	64.75	0.10	0.20	0.15	33.17	63	19.51
	2002	9/18/2002	9/1/1991	10.90	8.67	5.39	1.49E+07	8	1.86E+06			53.24			0.01			
	No ALDOT																	
6	1996																	
	2001	4/1/2000	6/1/1997	2.79	12.18	7.57	2.91E+06	7	4.16E+05	19.30	0.86	58.92	0.15	0.11	0.13	0.33	1	0.72
	2002 AL	3/4/2002	6/1/1997	4.69	12.18	7.57	5.51E+06	7	7.87E+05	0.59	0.00	59.13	0.20	0.18	0.19	0.00	0	0
	2002	9/18/2002	6/1/1997	5.22	12.18	7.57	6.40E+06	7	9.14E+05			55.45			0.05			
7	1996	3/23/1996	3/1/1996	0.06	11.56	7.18	7.22E+05	7	1.03E+05	0.51	0.15	64.64	0.25	0.24	0.24	0.98	2	0
	2001	4/16/2000	3/1/1996	4.07	11.56	7.18	3.76E+06	7	5.37E+05	0.84	0.00	52.29	0.21	0.12	0.17	0.00	0	0.11
	2002 AL	5/1/2002	3/1/1996	6.08	11.56	7.18	6.40E+06	7	9.14E+05	1.95	0.00	51.90	0.22	0.23	0.23	0.01	0	0.03
	2002	9/18/2002	3/1/1996	6.46	11.56	7.18	7.06E+06	7	1.01E+06			50.64			0.07			
9	1996																	
	2001																	
	2002 AL	7/3/2002	1/1/2000	2.47	8.05	5.00	5.94E+06	13	4.57E+05	0.00	0.00	63.99	0.22	0.18	0.20	0.00	0	0.02
	2002	9/18/2002	1/1/2000	2.68	8.05	5.00	6.65E+06	13	5.12E+05			62.74			0.08			
10	1996																	
	2001	1/13/2000	11/1/1998	1.18	9.33	5.80	3.13E+06	9	3.48E+05	24.84	19.56	55.14	0.10	0.07	0.09	0.00	0	0.2
	2002 AL	4/28/2002	11/1/1998	3.44	9.33	5.80	7.10E+06	9	7.89E+05	2.31	0.80	62.15	0.24	0.20	0.22	0.47	1	0.03
	2002	9/18/2002	11/1/1998	3.83	9.33	5.80	8.09E+06	9	8.99E+05			59.95			0.12			
11	1996																	
	2001	12/11/1999	8/1/1995	4.30	4.3	2.67	7.44E+06	9	8.27E+05	40.04	15.49	77.85	0.10	0.09	0.09	1.31	2	0.56
	2002 AL	3/8/2002	8/1/1995	6.51	4.3	2.67	1.10E+07	9	1.22E+06	2.44	2.26	78.94	0.16	0.19	0.18	0.10	0	0.1
	2002	9/18/2002	8/1/1995	7.03	4.3	2.67	1.22E+07	9	1.36E+06			67.88			0.15			
Average				5		5				8		58.63			0.16		8	2

The data from all of the sections were combined and analyzed with no distinction between the individual rubblized sections. The distresses were plotted for the combined data to better understand how the rubblized pavements were performing in general (See Figures 5.1 – 5.5). The first distress that was evaluated against the three standards is the transverse crack density presented in Figure 5.1.

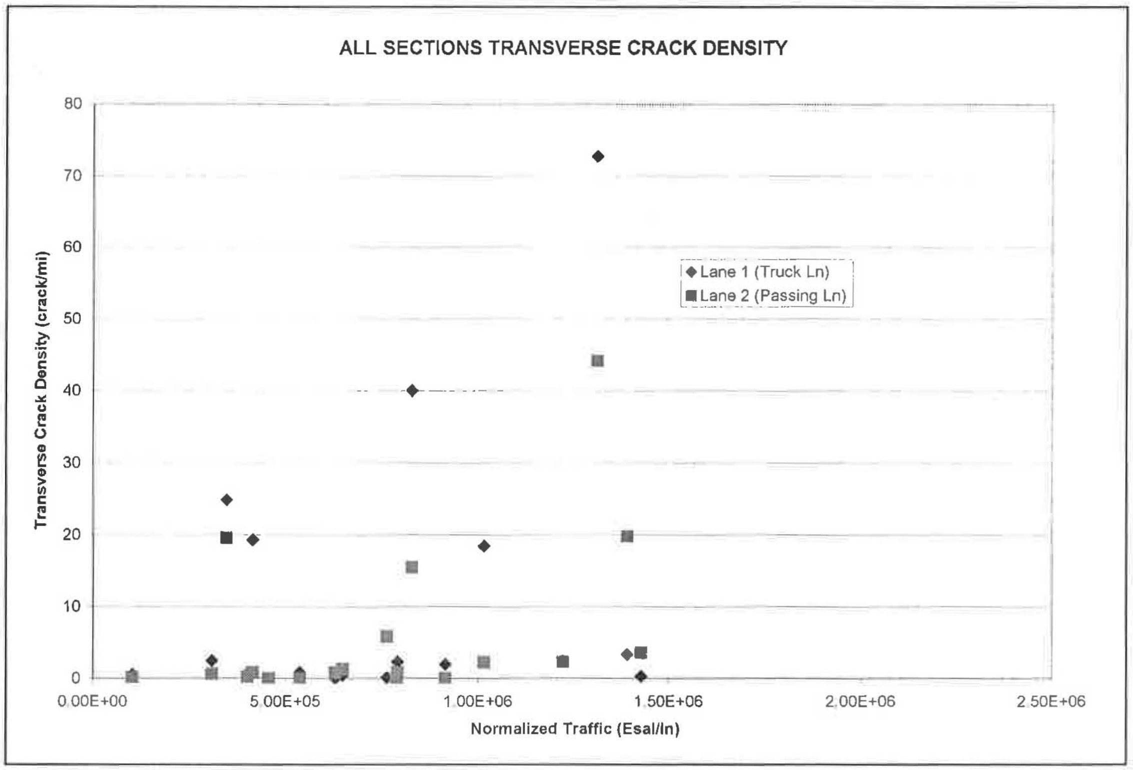

Figure 5.1 Transverse Crack Densities vs. Normalized Traffic.

When studying Figure 5.1, it is important to note that lane one has the highest levels of transverse crack density. This observation is logical because the heaviest vehicles tend to travel in lane one or what is commonly referred to as the truck lane. Another important observation is the magnitude of the data also seems to increase as the pavement age increases.

To aid in understanding of the y-axis in Figure 5.1, Table 5.2 was constructed to relate transverse crack density to motorist traveling down the highway. For example, there was an average of eight cracks per mile for the rubblized sections. This means that when traveling the posted speed limit of 70 miles per hour, the motorist would pass over a crack every 6.43 seconds. The average transverse crack density was compared to the original joint spacing. When compared to the original joint spacing of the PCC pavement, the rubblization sections are performing outstanding. This is discussed in the next paragraph.

Table 5.2 Converting Cracks per Mile to Seconds to Crack.

	Cracks/mi	cracks/second	seconds to next crack @ 70mph
Average	8	0.16	6.43
-	33	0.64	1.57
-	52	1.00	1.00
-	75	1.46	0.69
-	100	1.94	0.51
Highest	117	2.28	0.44
Original PCC Joint Spacing	264	5.13	0.19

After reviewing the transverse density graphs, one might argue that the rubblization process is a failure because as stated in Chapter 2 the goal of the process is to eliminate reflective cracking. It is important to note that the distress is measured as the number of transverse cracks and that just because transverse cracking is reported it does not necessarily mean reflective cracking. This similar situation was reported by Fitts (2001). Fitts said that although transverse cracking was reported, no reflective cracking was reported due to the fact that the transverse cracks did not correspond to the original PCC joint spacing. The transverse cracks were credited to have formed due to changes in environmental conditions (i.e., thermal cracking). Based on the finding of Fitts, and the comparison of the average crack density to the original joint spacing, Alabama's rubblized sections seem to have succeeded in the elimination of the reflective cracking to date (February, 2003).

The next distress that was plotted was IRI (Figure 5.2). As a whole, all of the sections appeared to have a distress level with a fair amount of scatter, neither increasing nor decreasing since the date of rubblization. The average IRI value for all the rubblized sections was 57 in/mile (0.9 m/km). To gain a better perspective of the IRI performance, the average IRI for the rubblized section was compared to the threshold values mentioned in Chapter 4. The AASHTO 2002 Guide establishes 252 in/mile (3.98 m/km) as a critical threshold and Oregon and Washington Departments of Transportation established a threshold of 133 in/mile (2.10 m/km) for a three-year-old pavement section. Comparing Alabama's rubblized sections with an average age of five years and an average IRI of 57 in/mile (0.9 m/km), it appears that the sections are performing well, surpassing industry standards.

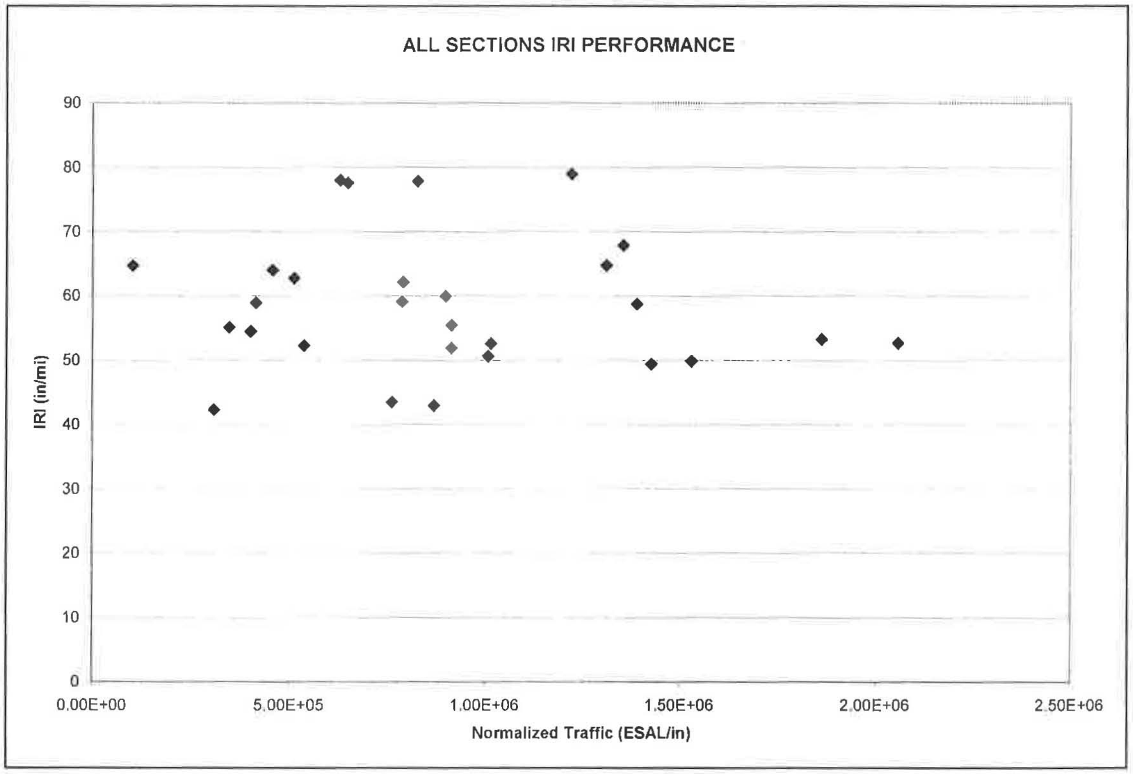

Figure 5.2 IRI vs. Normalized Traffic.

In Figure 5.3, the average rutting values were plotted against the normalized traffic of the pavement section. There appeared to be no recognizable pattern to the distribution of the rutting values. It is important to note that none of the rutting values exceed three-tenths of an inch, and all sections combined have an average rut value of 0.16 of an inch. Also noted is that the 0.16 of an inch, at an average pavement age of five years, is well below the 0.75 of an inch value established by AASHTO, 0.375 inch for a three year old pavement section in Oregon and Washington, and finally 0.25 of an inch in Wisconsin for a five year old pavement. This comparison suggests that the rubblized pavement sections are rutting at a minimum level. This minimum amount of rutting could possibly be contributed to the seating of the rubblized PCC pavement or densification of the mix under traffic. The fact that there is not an excessive amount of rutting present in any of the rubblized sections could be attributed to the significant structural cross sections. In other words, one would not expect to see deep structural rutting due to failure of the subgrade.

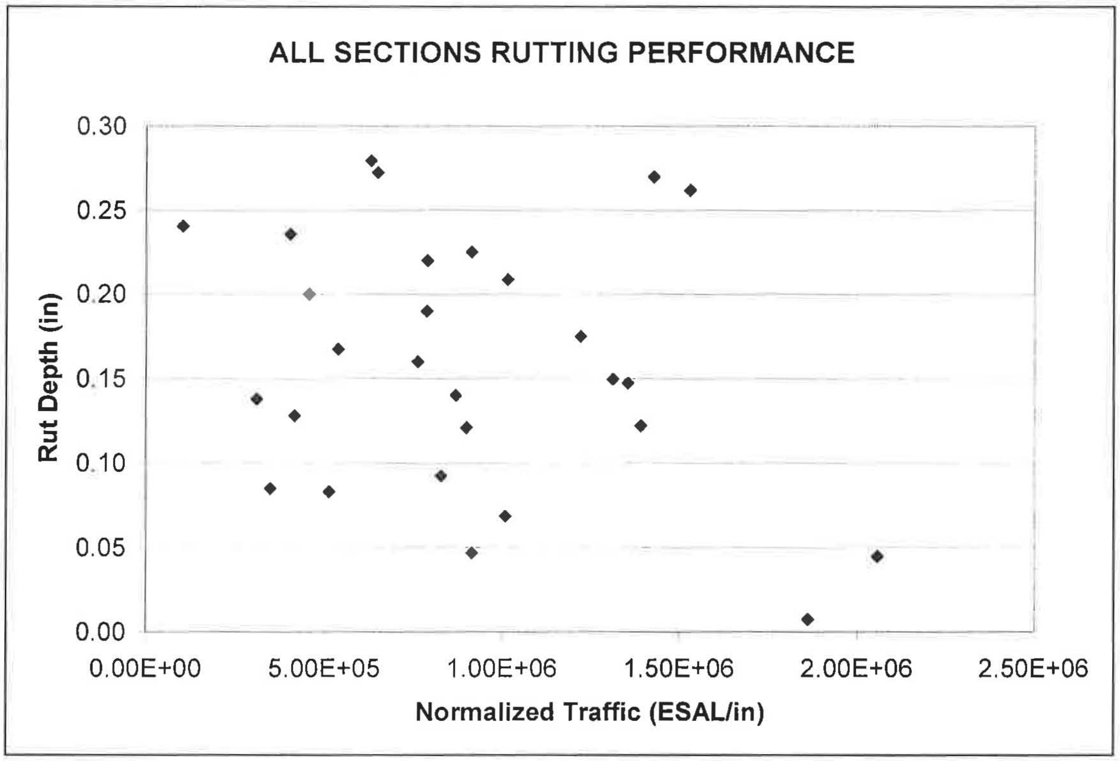

Figure 5.3 Rut Depths vs. Normalized Traffic.

The next performance graphs that were evaluated were the percent longitudinal cracking in the rubblized pavement sections. These data are provided in Figure 5.4. At first glance, the data present a trend that forms a logical pattern. This trend shows that as the pavement age increases, the magnitudes of the distress also increase.

When compared to the established standards in Chapter 4, the rubblized sections continue to surpass the set thresholds. The average percent longitudinal cracking for all of the rubblized section in Alabama was eight percent. This eight percent average is far below the 190 percent for a five-year-old pavement in Wisconsin and the 20 percent established by proposed AASHTO 2002 Design Guide. When compared to the thresholds established by Oregon and Washington, the eight percent average exceeds the 5.5 percent threshold. This comparison seems to suggest that the Alabama rubblized sections have surpassed the Oregon and Washington thresholds.

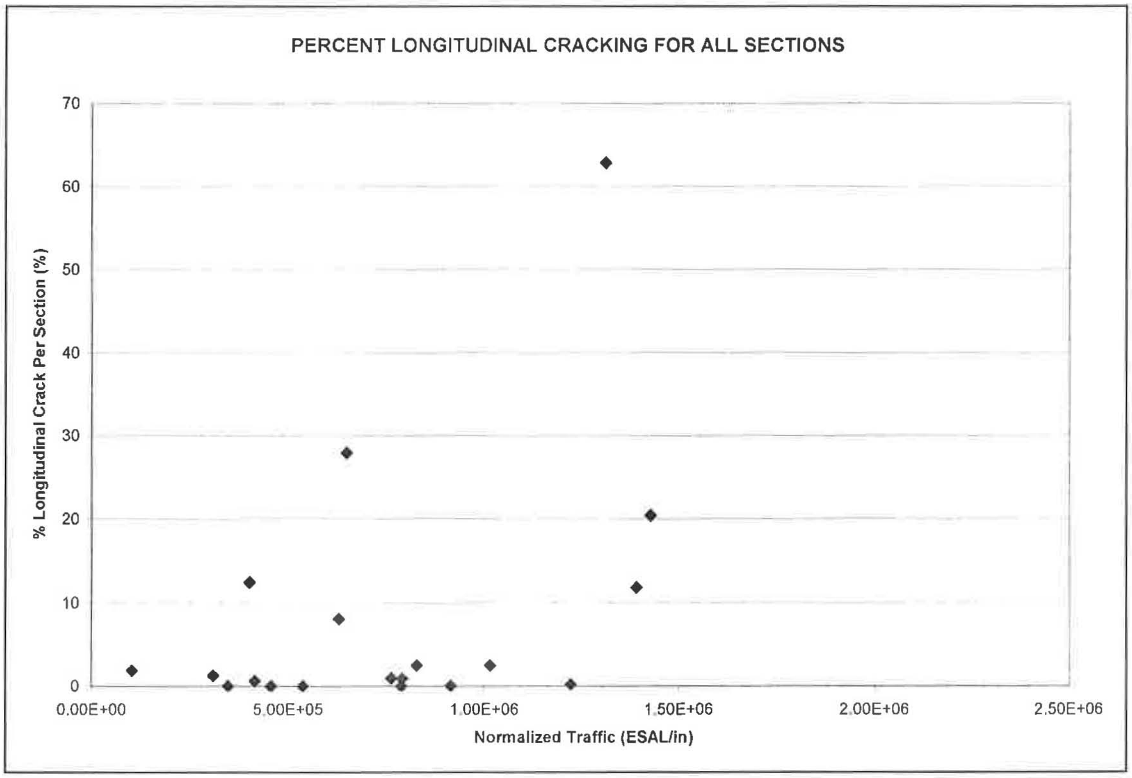

Figure 5.4 Percent Longitudinal Cracks vs. Normalized Traffic.

It is important to recall that the Oregon and Washington thresholds are established for a three year old pavement system and that the Alabama rubblized sections are an average of five years old. The Oregon and Washington DOTs, with the current specification, allow for 1.8 percent per year longitudinal cracking increase. If this same standard were increased to five years, to encompass the age of the Alabama rubblized pavements, the 1.8 percent currently allowed times the five-year average age would yield 9 percent. This is greater than the eight percent section average.

It is also important to recall from Chapter 2 that the rate in which the pavement condition deteriorates resembles a decreasing exponential function. With this increased rate of deterioration, Oregon and Washington would most likely, for a five-year specification, increase the allowed percent to reflect this trend. By considering the above-mentioned points, the conclusion can once again be reached that the rubblized pavement sections in Alabama are withstanding the deterioration modes that will result in longitudinal cracking distress.

The final pavement distress that was evaluated in this research was alligator cracking. As shown in Figure 5.5, the trend of deterioration seems to follow the pattern observed in Figure 5.4. As the pavement age increases, the magnitude of the distresses also increases. The average alligator cracking for the Alabama rubblized sections is only two square feet per rubblized section. If the two square feet average was compared to the thresholds of eight and ten percent, which are mentioned in Chapter 4, the two feet would be less. When the average two square feet is converted to a percent of total area, assuming 12-foot lanes, the value is less than 0.001 percent. This leaves a lot of room before any maintenance thresholds are met.

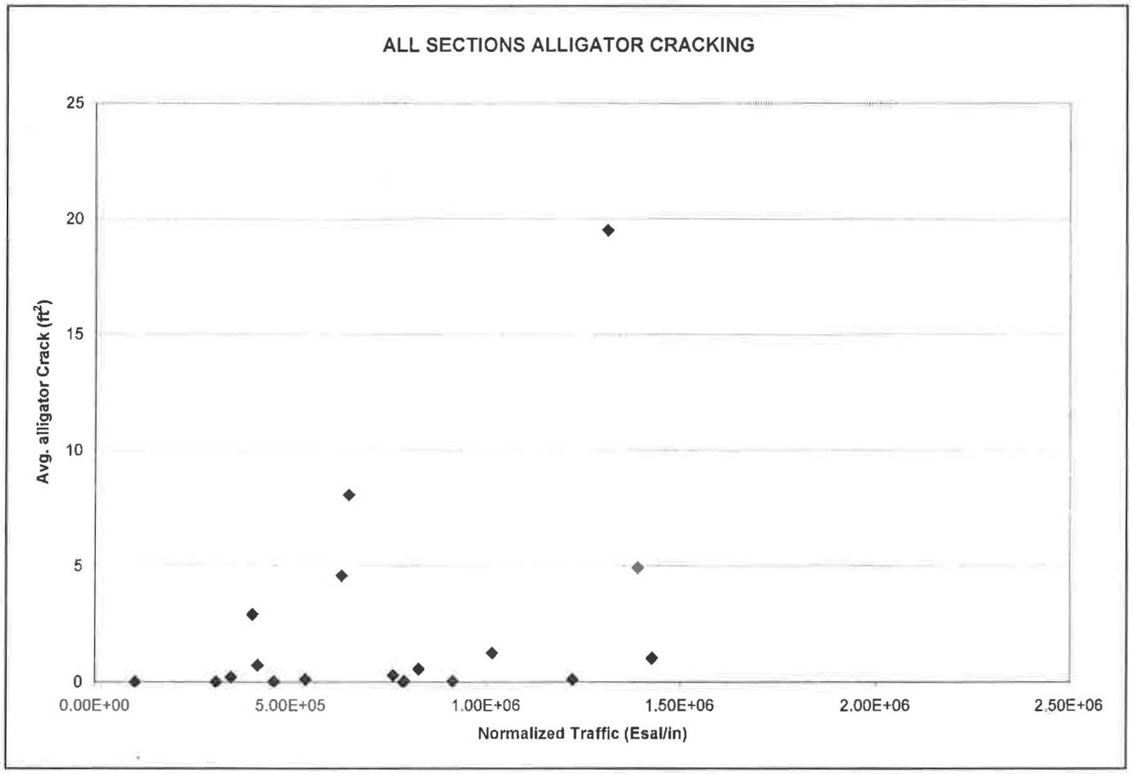

Figure 5.5 Average Alligator Cracks vs. Normalized Traffic.

DISTRESSES BY SECTION

To further investigate the performance of the rubblized pavement sections, the distresses were plotted in the same manner as above, except the distress data were divided into the individual rubblized pavement sections. This gave a clearer picture of how each individual section was performing. As mentioned in Chapter 4, the data were plotted in this manner to better understand the effects of traffic on the rubblized pavement and possibly identify causes of the deterioration. There was only distress that was plotted above in the system graphs that did not get reported in the next portion of the thesis, by section data. The rutting information was not reported because the data did not provide any recognizable trends. Figure 5.6 examines percent transverse crack density by section.

Upon analysis of the percent of transverse crack density graph, there were two main points to be noted. The first is that section four has the highest levels of distress. The possible reason for this high level will be stated later. The second point is that in most cases lane one exhibited the highest levels of distress. The fact that the higher levels of distresses are present in lane one seems logical because the higher volume of heavy vehicles travel in this lane.

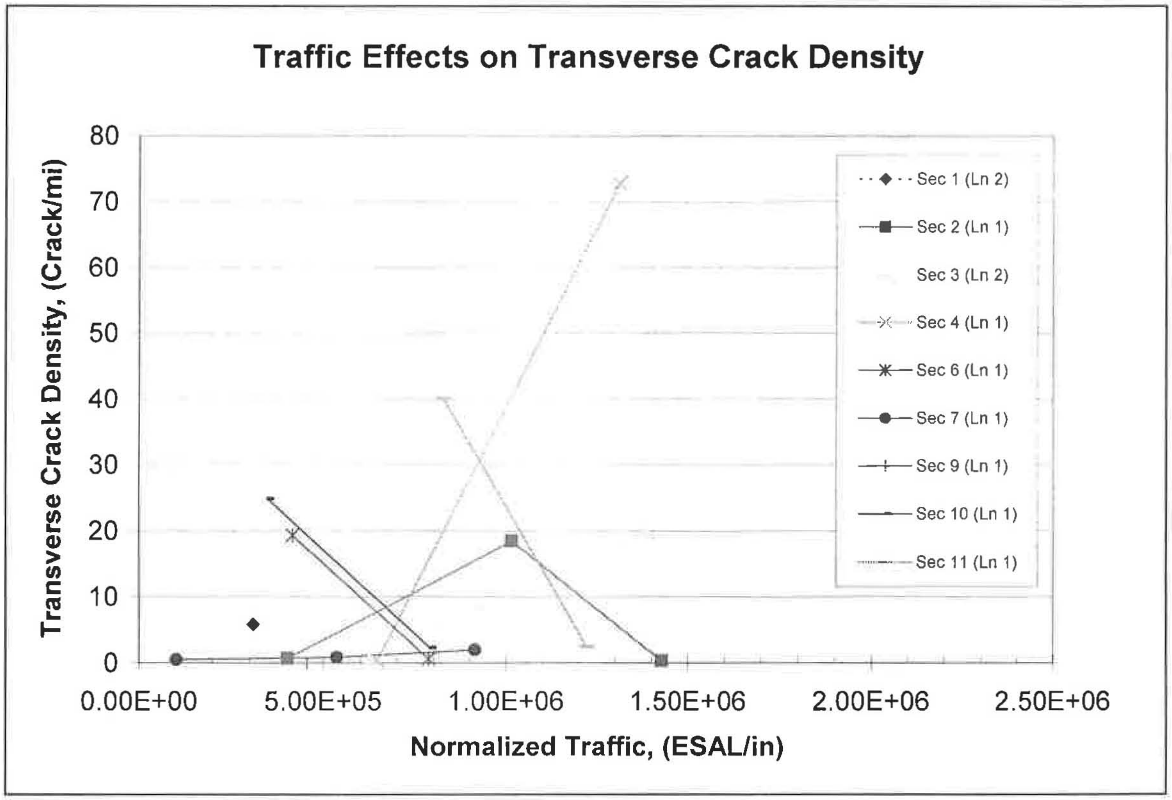

Figure 5.6 Transverse Crack Density vs. Normalized Traffic.

In Figure 5.7 the IRI distresses are evaluated for unique trends. There are two major points to recognize when analyzing the IRI graphs. The first is that in most of the sections the values of the given distress decreased initially. After the HMA overlay has been constructed, a certain degree of compaction under vehicular traffic evidently takes place. This reduction in IRI values could possibly be a direct result of the rubblized pavement seating from increased traffic loading. This trend is also shown in Figure 5.10 where the distress data from Highway 280 are presented. The second point to be made about Figure 5.7 is that sections three and four, the two CRCP, are among the sections exhibiting the highest levels IRI distress.

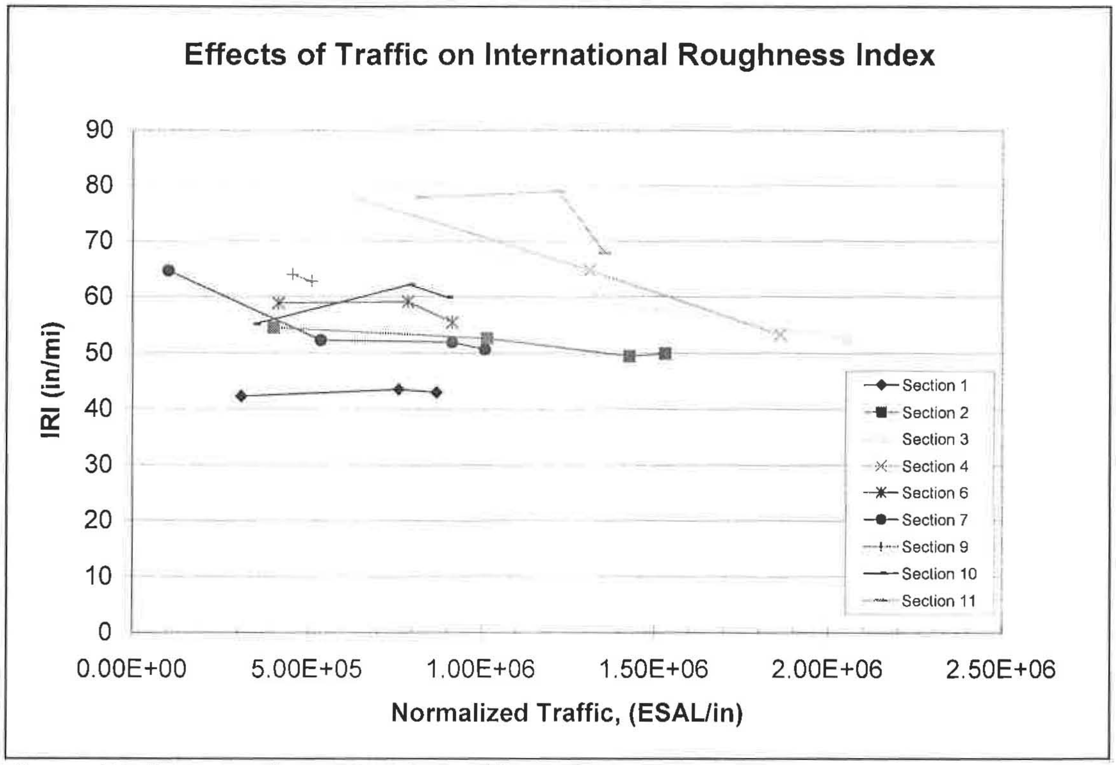

Figure 5.7 IRI vs. Normalized Traffic.

Next, percent longitudinal cracking and average alligator cracking were explored by pavement section in Figures 5.8 and 5.9. These two distresses will be discussed at the same time because they both have similar trends and are consistent throughout the data. Both distresses are present in low amounts throughout the Alabama rubblized pavements. As noted above, sections three and four are again among the sections that have the largest amounts of distresses present. Reviewing the distresses on section-by-section bases, provided an important perspective into the performance of the rubblized sections. Observing that sections three and four consistently had the highest levels of distress, raises questions about the possible causes for their elevated levels.

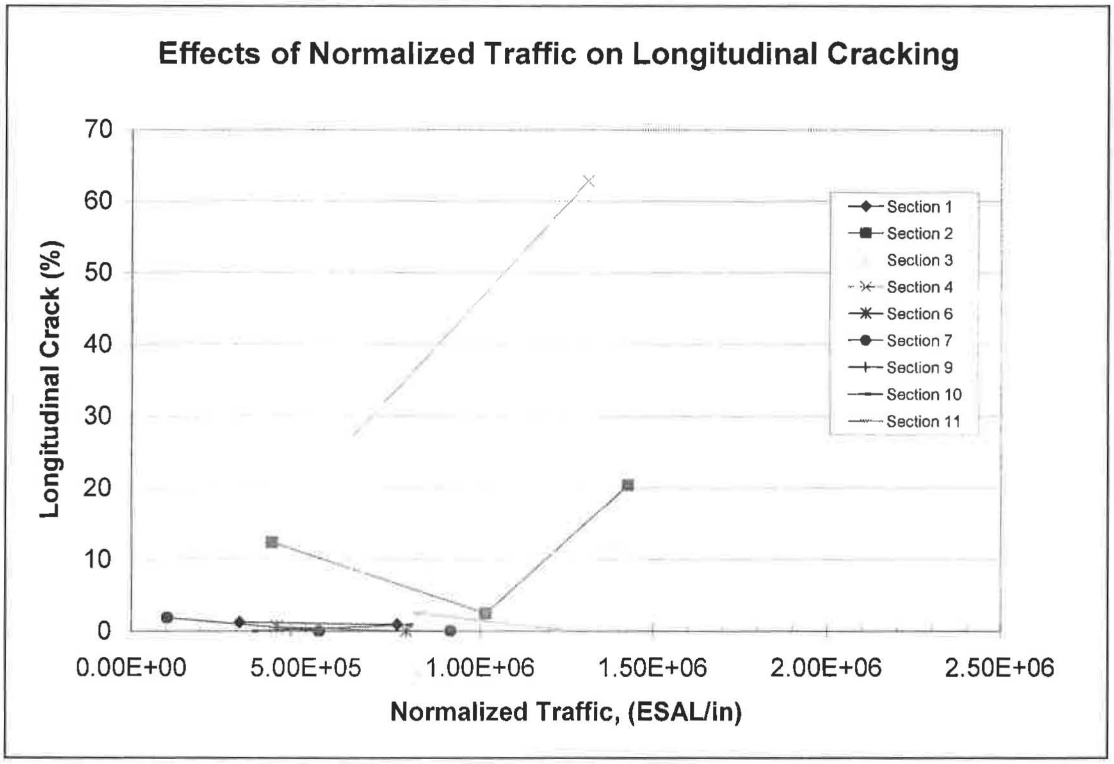

Figure 5.8 Percent Longitudinal Cracking vs. Normalized Traffic.

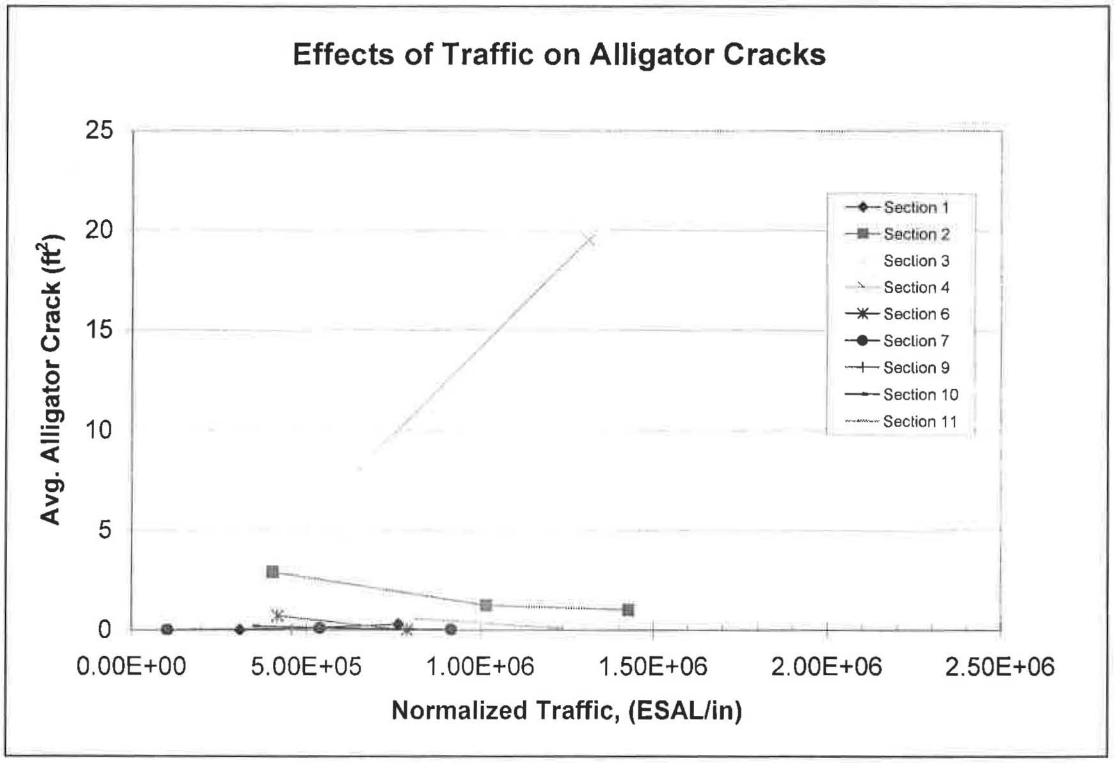

Figure 5.9 Average Alligator Cracking vs. Normalized Traffic.

When reviewing the construction information, sections three and four were the only sections that that were originally CRCP pavement. The fact that the reinforced pavements exhibited higher levels of distresses has been documented in the past by Witzack and Rada (1992). The elevated distress levels were credited to PCC pavement not fully debonding from the reinforcing steel in the pavement. Another unique element to sections three and four is that they both are constructed with a stabilized base. Though the issue of the PCC not debonding from the reinforcing steel and the fact that the sections have a stabilized base are not fully investigated in this research, it certainly could have been a contributing factor in the higher distress levels observed in these sections.

In Chapter 4, U.S. Highway 280 was mentioned as a tool that would serve as a comparison for the performance of the rubblized sections. Figure 5.10 is a graph from

LTPP database. This was included to illustrate the decrease in IRI levels. This decrease was also observed in the IRI levels in the above-mentioned rubblized pavements. Table 5.3 is shown below Figure 5.10 for a direct comparison of the two different measurement systems.

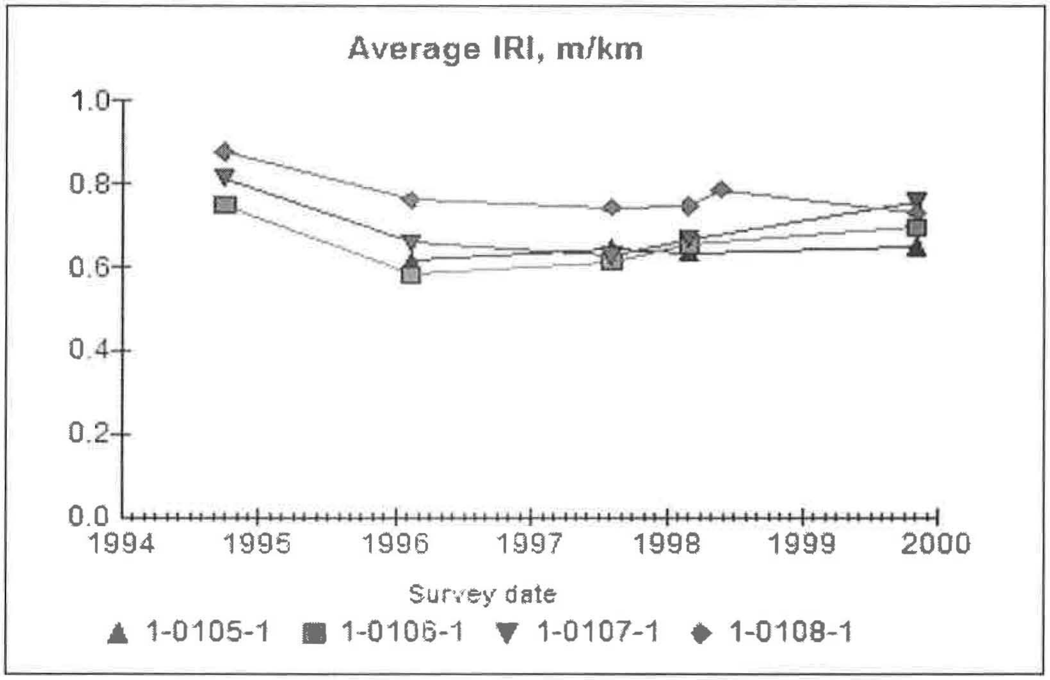

Figure 5.10 IRI vs. Survey Date for Highway 280.

Table 5.3 IRI Conversion.

Table 5.3 IRI Conversion.

IRI Conversion	
inches / mile	meters / kilometer
90	1.42
80	1.26
70	1.10
60	0.95
50	0.79
40	0.63
30	0.47
20	0.32
10	0.16
0	0.00

STATISTICAL ANALYSIS

Continuing the investigation into the performance of the rubblized sections, a regression was executed on the distress and construction information. This was done in hopes of possibly identifying elements of the process that might accelerate the deterioration process of the pavement.

The regressions were performed to predict the four distresses discussed previously. The independent variables included the age of the pavement, the type of existing PCC pavement (CRCP or JRCP), the type of rubblization process (resonance or Multi-head), the thickness of the HMAC overlay, the thickness of the existing PCC pavement and the amount of traffic in million ESALs. The results of the statistical regression yielded Equations 5.1 through 5.4.

These four regression equations yield some very interesting points. Before the results from the regression analysis were explored, the process in which the equations were examined had to be outlined. To begin the examination of the individual equations, the R^2 value, $F_{statistic}$, and the P_{value} were observed to determine the accuracy of the equation. R^2 values are reported as a fraction between 0.0 and 1.0. The closer the fraction is to one, the more accurate the equation. It also shows how well the predictor variable predicts the response variable. The $F_{statistic}$ is compared to $F_{critical}$ to determine the correlation of the equation. If the $F_{statistic}$ is greater than $F_{critical}$ then the data would have a strong correlation. This difference is represented by the P_{value}. When the P_{value} is less than 0.05, there is a strong correlation represented by the data.

With the above accuracy parameters for the entire equation, the coefficients and P_{values} were presented (Tables 5.4 through 5.7) and discussed for each of the predictors in the individual equations.

With IRI as the response, the regression equation is:

$$IRI = -60.9 - 1.45 * AGE + 26.9 * PCCType$$
$$-16.5 * RP - 2.26 * H + 15.5 * D - .06 * MESAL \qquad \text{(Equation 5.1)}$$

where:
IRI = average IRI, in/mile
AGE = number of years opened to traffic
PCC Type = type of existing PCC (0 for JRCP, 1 for CRCP)
RP = type of rubblization (0 for Resonant, 1 for Multi-head breaker)
H = thickness of HMA overlay, in.
D = thickness of existing PCC slab before rubblization, in.
MESAL = million ESALs since open to traffic
$R^2 = 0.604$
$F_{statistic} = 5.35$
$P_{value} = 0.002$

Table 5.4 IRI Statistical Information.

Predictor	Coefficient	P-Value
Constant	-60.87	0.298
AGE	-1.451	0.498
PCC Type	29.574	0.003
RP	-16.515	0.007
H	-2.258	0.068
D	15.463	0.023
MESAL	-0.055	0.965

Equation 5.1 was the first equation that was analyzed. The regression yielded an R^2 of 0.604, an $F_{statistic}$ of 5.35, and a P_{value} of 0.002. This indicated that the regression equation as a whole accurately predicts the IRI distress levels in the rubblized pavement system.

Upon a closer examination of Equation 5.1, the individual coefficients and P_{values} of each predictor were investigated (Table 5.4). AGE was found to have a negative coefficient with a P_{value} greater than 0.05, suggesting rather large amounts of variation. The negative coefficient proposes that as the pavement age increases, the IRI levels actually decrease. This negative coefficient could be attributed to the compacting of the HMAC and the rubblized particles. This compaction or seating of the pavement was also observed in U.S. 280 performance graphs.

The PCC Type coefficient was positive and the P_{value} was less than 0.05. This suggests that there was a minimal amount of variation. Recalling from Chapter 2, there were two types of pavements rubblized, including JRCP and CRCP. In the regression analysis, the JRCP was given a coefficient of 0 and the CRCP was given the coefficient of 1. With this in mind when reviewing the regression equations, the positive coefficient suggests that the higher levels of distresses were present on the CRCP in all of the equations. This may be a result of the concrete not completely debonding from the reinforcing steel. Whatever the cause, the data suggest that the rubblization of CRCP was not as effective as the rubblization of JRCP.

The RP coefficient found was negative, with a P_{value} less than 0.05. This implies an acceptable amount of variation. It is important to recall that Alabama rubblized pavements were constructed using two methods: the resonance pavement breaker and the multi-head pavement breaker. For the regression analysis, a zero was assigned to the pavements that were rubblized using the resonance breaker and a one was assigned to the pavement that utilized the multi-head breaker. The negative coefficient suggested that the lower levels of distresses were present on the pavement sections that employed the resonance pavement breaker. For rubblizing in Alabama, the regression suggested that the resonance pavement breaker produces a higher quality end product. Even though one machine is recommended over another, it is imperative to recall that there was only one rubblized pavement section in Alabama that employed the MHB. For this reason, it is suggested that further investigation be conducted to draw a more accurate conclusion.

The H coefficient was negative coefficient and had a P_{value} greater than 0.05, suggesting more variation than desired. This negative coefficient is not easily explained. It would seem logical that increased HMAC thickness would only extend the life of the pavement. One possible explanation for the negative coefficient is that the thicker HMAC pavements were designed for higher traffic volumes. These higher traffic volumes would also possibly increase the distress level. This needs to be further investigated to gain an accurate reason for its occurrence.

The D coefficient was positive with a P_{value} less than 0.05, implying acceptable amounts of variation. The positive coefficient implies that as the thickness of the PCC increases, the amounts of distress present also increase. This could possibly suggest that the PCC pavement was not being effectively rubblized throughout the entire depth.

The final coefficient and P_{value} for equation 5.1 was examined for "million of ESALs". The coefficient was negative with a P_{value} greater than 0.05, suggesting more variation than desired. The traffic levels were expressed as million of ESAL due to the original differences in magnitudes of the predictor data. The fact that the increased traffic levels seemed to reduce the IRI values was presented in the previous graphs (Figure 5-28). This information suggests that the seating process accomplish by the traffic also has an initial positive effect on the IRI distress levels.

The parameters for the transverse crack density regression equation (Equation 5.2) is presented and discussed below. When Equation 5.2 was analyzed, it was determined that the R^2 value was 0.276, the $F_{statistic}$ was 0.76, and the P_{value} was 0.612. These values do not meet the criteria mentioned above, which suggest that there is not a strong correlation between the distress data. This lack of correlation indicates that the transverse crack density is not accurately predicted with the regression equation. The equation is:

$$TCD = -169 + 5.40 * AGE + 31.3 * PCCType$$
$$+ 2.2 * RP - 1.46 * H + 19.1 * D - 2.48 * MESAL \qquad \text{(Equation 5.2)}$$

where:
TCD = average transverse crack density, cracks/mile
AGE = number of years opened to traffic
PCC Type = type of existing PCC (0 for JRCP, 1 for CRCP)
RP = type of rubblization (0 for Resonant, 1 for Multi-head breaker)
H = thickness of HMA overlay, in.
D = thickness of existing PCC slab before rubblization, in.
MESAL = million ESALs since open to traffic
$R^2 = 0.276$
$F_{statistic} = 0.76$
$P_{value} = 0.612$

Table 5.5 Transverse Crack Density Statistical Information.

Predictor	Coefficient	P-Value
Constant	-168.8	0.239
AGE	5.396	0.321
PCC Type	31.27	0.169
RP	2.2	0.876
H	-1.465	0.624
D	19.11	0.236
MESAL	-2.483	0.452

After reviewing the accuracy of the entire equation, the coefficients and P_{values} for each of the predictors of the transverse cracking density regression was reviewed. It was found that not one of the individual P_{values} was less than 0.05. This implies that there was an excessive amount of variation in all of the data.

The "number of years opened to traffic" had a positive coefficient. This positive coefficient seems to follow simple logic, by stating that as the pavement increases in age, the levels of distresses that pavement possesses will likewise increase.

The coefficient for the "type of existing PCCP" is positive. The positive coefficient suggests that the higher levels of distresses were present on the CRCP. This follows the trends shown in the previous Equation 5.1.

The coefficient for the "type of rubblization process" was examined and was found to have a positive value. This value suggests that the lower levels of distress were produced when the MHB was used in the rubblization process. This was the only case in which the coefficient for the rubblization was positive. It is important to recall that there was not a high level of correlation of the data in this equation.

The predictor "thickness of HMA overlay" was found to have a negative coefficient. The negative coefficient for the thickness of HMA overlay suggests that as the thickness of the HMA increased, the distress levels in the rubblized section decreased.

The next predictor "thickness of existing PCC" was evaluated and a positive coefficient was found. This positive coefficient implies that as the thickness of the PCCP increased, the amounts of distress present also increased.

The last coefficient that was examined for the transverse crack density regression equation was the coefficient for the "million of ESALs". This was found to be negative. The negative coefficient suggests that the increased traffic levels seemed to reduce the levels of transverse crack density in the rubblized sections.

In the third regression equation, the percent longitudinal cracking was the response variable (Equation 5.3), which yielded an R^2 of 0.599, an $F_{statistic}$ of 2.99, and a P_{value} of 0.050. These values imply that the regression equation accurately predicts the longitudinal cracking in Alabama's rubblized pavement sections.

$$LC = 16 + 3.35 * AGE + 16.9 * PCCType - 4.9 * RP + 2.74 * H - 4.7 * D - 1.08 * MESAL \qquad \text{(Equation 5.3)}$$

where:
LC = percent longitudinal cracking per section
AGE = number of years opened to traffic
PCC Type = type of existing PCC (0 for JRCP, 1 for CRCP)
RP = type of rubblization (0 for Resonant, 1 for Multi-head breaker)
H = thickness of HMA overlay, in.
D = thickness of existing PCC slab before rubblization, in.
MESAL = million ESALs since open to traffic
$R^2 = 0.599$
$F_{statistic} = 2.99$
$P_{value} = 0.050$

Table 5.6 Longitudinal Cracking Statistical Information.

Predictor	Coefficient	P-Value
Constant	16.2	0.883
AGE	3.354	0.432
PCC Type	16.93	0.336
RP	-4.86	0.664
H	2.738	0.258
D	-4.72	0.704
MESAL	-1.08	0.677

The P_{values} for predictors for Equation 5.3 all exceeded the 0.05 level, suggesting a greater amount of variance in the data than desired. This variance is important to keep in mind when analyzing the remaining data.

The first parameter was the "number of years of opened to traffic" (AGE). The coefficient of this predictor was positive. This positive coefficient for the age predictor seems to follow simple logic, by stating that as the pavement increased in age, the levels of longitudinal cracking that the pavement possesses will likewise increased.

The second parameter, "type of rubblized pavement", with a positive coefficient suggests that the higher levels of distresses were present on the CRCP. This value is consistent with the discussions of the previous equations.

The "type of rubblization process" with a negative coefficient suggested that the lower levels of longitudinal cracking were present on the pavement sections that employed the resonance pavement breaker, as previously stated.

The fourth parameter, "thickness of HMA overlay," was found to have a positive coefficient. This observation is not easily explained because it suggests that as the HMA overlay thickness is increased the amount of longitudinal cracking amounts also increase. The explanation for this trend, as previously stated, is that the thicker HMA pavements were designed for higher traffic volumes that could possibly increase the distress level.

The thickness of existing PCC for the longitudinal regression equation had a negative coefficient that suggests that as the thickness of the existing PCC pavement increased, the levels of longitudinal cracking decreased. This observation follows simple logic which shows that the larger amounts of existing PCC pavement present initially, the larger the granular base that is present to protect the subgrade after the rubblization process is complete.

The million ESALs since opened to traffic predictor was found to have a negative coefficient that indicated that as the amount of traffic increased, the amount longitudinal cracking decreased. This information suggests that the seating process accomplished by the traffic also has a positive effect on the amount of longitudinal cracks present.

The final equation predicted alligator cracking with an R^2 of 0.691, an $F_{statistic}$ of 4.48, and a P_{value} of 0.013. These parameters suggest that the regression equation is capable of accurately predicting the alligator cracking in the Alabama rubblized pavements.

$$AC = -9.1 + 1.5 * AGE + 8.12 * PCCType - 0.62 * RP + 0.678 * H + 0.24 * D - 0.752 * MESAL \quad \text{(Equation 5.4)}$$

where:
AC = average alligator cracking per section, ft^2
AGE = number of years opened to traffic
PCC Type = type of existing PCC (0 for JRCP, 1 for CRCP)
RP = type of rubblization (0 for Resonant, 1 for Multi-head breaker)
H = thickness of HMA overlay, in.
D = thickness of existing PCC slab before rubblization, in.
MESAL = million ESALs since open to traffic
$R^2 = 0.691$
$F_{statistic} = 4.48$
$P_{value} = 0.013$

Table 5.7 Alligator Cracking Statistical Information.

Predictor	Coefficient	P-Value
Constant	-9.11	0.758
AGE	1.496	0.201
PCC Type	8.122	0.098
RP	-0.615	0.837
H	0.6781	0.294
D	0.238	0.943
MESAL	-0.7524	0.289

By examining Table 5.7, it is clear that none of the coefficients are statistically significant in predicting the amount of alligator cracking. That having been said, increasing the age and rubblizing CRCP concrete seem to increase the amount of cracking. Also, increasing the thickness of the HMA and PCC, respectively, appears to increase the amount of cracking. This trend was observed with other forms of distress discussed previously.

To relate the predictors that were mentioned above to previous research, one must recall Witczak and Rada's research, discussed in Chapter 2. Their regression equations suggested that the following elements lead to the deterioration of the pavement system: increased age, increased air temperature, increased precipitation and decreased thickness of each section. It was found in this research that thickness had an opposite effect. However, this study was over a fairly limited number of sections with a narrow range of pavement thickness, so it is difficult to compare directly to the Witczak and Rada study.

SUMMARY

To conclude Chapter 5, the main observations from the system graphs, the graphs by rubblized section, and the regression analysis were reviewed. Beginning with the transverse crack density system graphs, there were two main observations made. The first observation was that lane one (truck lane) had the highest levels of distress. The second is the trend in which the data followed as time progressed. As the age of the rubblized sections increased, the amounts of transverse cracking also increased. The above-mentioned trend was also observed in the percent longitudinal cracking and the average alligator cracking. When dealing with the pattern that the IRI data presented, there was a fair amount of scatter present while the magnitudes of the distress maintained, neither increased nor decreased. Another important observation made with the IRI, rut, percent longitudinal cracking, and alligator cracking data is that each level of distresses met or surpassed established industry standards. The final observation that was made while dealing with the data on a system level was the lack of any recognizable trend made by the rutting data.

Next, the main observations when the data were evaluated on a system basis were presented. The first was that sections three and four consistently contained the highest levels of distresses. When examined closer, sections three and four were the only CRCP in the rubblization projects. Though not in the scope of this project, the fact that the CRCP were not fully debonding from the steel could contribute to the higher levels of distress and should be investigated further. Another important observation made, regarding the IRI data, was that after the initial survey the distress level recorded actually decreased. This was also found to be true on U.S. 280 and was attributed to the seating or compacting of the HMAC overlay and of the rubblized pavement.

The statistical analysis, discussed above, should be regarded with caution since it only represented a limited number of test sections. In the future, additional sections can be added to the data set to improve the scope of the investigation.

CHAPTER 6 - CONCLUSIONS & RECOMMENDATIONS

In conclusion, Alabama's rubblized pavement sections are performing very well. When the rubblization performance data were compared to the critical values established by AASHTO and other states' departments of transportation they met or exceeded all of the criteria. This investigation has concluded that the rubblization process is an effective and efficient rehabilitation technique for aged PCC pavements. For all intents and purposes, it can be concluded that there are certain types of pavements, equipment, and existing site conditions that will produce a higher quality product.

Further investigation into the performance of the individual rubblized sections suggested different catalysts for the pavement distresses. These catalysts were recognized based on the evaluation of the rubblized sections on both the analysis of an individual basis and statistical analyses. Examples of these elements are as follows:

- As age of pavement increases, distresses increases
- Higher levels of distress were present on rubblized CRCP. However, these also were the only two sections with stabilized base materials.
- Higher levels of distresses were present on thicker PCC, suggesting that close attention should be paid when rubblizing thicker pavements
- Initial observations show that traffic seemed to seat the pavement reducing the IRI levels.

Based on the observations of this study, when rubblizing CRCP, special attention should be given. As discussed in this report, the distress levels were higher on the CRCP pavement sections. Although the specific reasons the distress levels were higher were not examined in this report, the literature review suggested the following: It is possible that the reinforcing steel did not completely debonded from the PCC. Also, if thicker PCC slabs are to be rubblized it is important to ensure that the specified particle size is obtained.

The findings that are mentioned above are very interesting and informative. However, there is a rule of thumb that has been around for years that can be found in AI webpage that states, "Reflective cracking will propagate through HMAC overlay at a rate of one inch per year".

In the case of Alabama's rubblized pavements, the average thickness of the HMAC pavement overlays in each section is 10.56 inches, with an average age of five years. According to the above stated 'rule of thumb', even if the pavement sections had not been rubblized reflective cracking would not expect to be present with the existing site conditions for another five years.

With this information in mind, it is recommended that the progress of the rubblized pavements continue to be monitored, following the procedure established in this paper, to evaluate the future performance to see if they in fact surpass this projected life expectancy.

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