PERFORMANCE OF THE CRACK, SEAT, AND OVERLAY
REHABILITATION TECHNIQUE FOR CONCRETE
PAVEMENTS IN CALIFORNIA

A Thesis

presented to

the Faculty of California Polytechnic State University,
San Luis Obispo

In Partial Fulfillment

of the Requirements for the Degree

Master of Science in Civil and Environmental Engineering

by

Reed Calkins

June 2011

© 2008

Reed Calkins

ALL RIGHTS RESERVED

COMMITTEE MEMBERSHIP

TITLE: THE PERFORMANCE OF THE CRACK, SEAT,
AND OVERLAY REHABILITATION TECHNIQUE
FOR CONCRETE PAVEMENTS IN CALIFORNIA

AUTHOR: Reed Calkins

DATE SUBMITTED: June 2011

COMMITTEE CHAIR: Dr. Ashraf Rahim

COMMITTEE MEMBER: Dr. Gregg Fiegel

COMMITTEE MEMBER: Dr. Anurag Pande

ABSTRACT

THE PERFORMANCE OF THE CRACK, SEAT, AND OVERLAY REHABILITATION TECHNIQUE FOR CONCRETE PAVEMENTS IN CALIFORNIA

Reed Calkins

Research was performed to analyze the performance of the crack, seat, and overlay (CS&O) roadway rehabilitation technique in the Central Coast and Northern regions of California. This technique was evaluated through literature review to determine the state of practice and their conclusions. California highway sections rehabilitated using CS&O were selected for evaluation based on age and location. Pavement distresses and traffic data for these sections were collected and analyzed. Prior to beginning analysis this data was checked for errors, outliers, and omissions. The analysis consisted of checking the data for correlations among distresses and regions.

The focus of this research is to develop performance prediction models for pavement distresses in CS&O sections. Using data collected from Caltrans' Pavement Condition Reporting Software, performance models were developed based on dependent (distress) variables: alligator cracking, transverse cracking, longitudinal cracking, and International Roughness Index (IRI). And independent (explanatory) variables: age, traffic in the form of equivalent single axle load (ESAL), thickness of hot mix asphalt (HMA), thickness of Portland Cement Concrete (PCC), and cumulative traffic in the form of cumulative ESAL. Prediction models were then analyzed for preciseness and sensitivity to the variables included in each model.

Keywords: Alligator cracking, Transverse cracking, Longitudinal Cracking, International Roughness Index, IRI, Crack Seat and Overlay, Asphalt Overlay, Pavement Rehabilitation, Pavement Maintenance

Table of Contents

LIST OF TABLES	vii
LIST OF FIGURES	viii
Chapter 1 Introduction	1
1.1 Background.....	1
1.2 Techniques to Minimize Reflective Cracking.....	2
1.3 Purpose.....	4
1.4 Thesis Organization	5
Chapter 2 Literature Review	7
2.1 Introduction	7
2.2 Crack, Seat and Overlay (CS&O)	11
2.2.1 Equipment	12
2.2.2 Slab Size	21
2.2.3 Overlay Thickness.....	27
2.2.4 Pavement Performance.....	31
2.2.5 CS&O Performance Prediction Models	44
Chapter 3 Data Collection and Analysis	47
3.1 Introduction	47
3.2 Caltrans Data Collection and Analysis	48
3.2.1 Section Summary	48
3.2.2 Distress Data.....	49
3.2.3 Distress Identification and Measurement.....	51
3.2.4 Visual Survey	53
3.3 Data Cleaning	59
3.3.1 Data Cleaning Procedures	60
3.3.2 Central Coast Region Data.....	61
3.3.3 Northern California Region Data	67
3.4 Data Analysis	73
3.4.1 Central Coast Region	74
3.4.2 Northern California Region	78

3.4.3 Region Comparison.....	82
Chapter 4 Performance Models.....	91
4.1 Introduction	91
4.2 Regression Modeling	91
4.3 Data Models.....	94
4.4 Model Fit	99
4.4.1 Alligator Cracking	99
4.4.2 Transverse Cracking	102
4.4.3 Longitudinal Cracking.....	105
4.4.4 International Roughness Index (IRI).....	108
4.5 Sensitivity Analysis	111
4.5.1 Central Coast Region	112
4.5.2 Northern California Region	120
4.5.3 Summary of Sensitivity Analysis.....	130
4.6 Summary.....	131
Chapter 5 Conclusions and Recommendations.....	133
5.1 Overview	133
5.2 Conclusions	134
5.3 Recommendations	135
References	137
Appendix A	139

LIST OF TABLES

Table 2.1: Caltrans Pavement Condition States (Caltrans 2007)	9
Table 2.2: Change in PCC layer stiffness (Adapted from Al Hakim, 1999)	22
Table 2.3: Resilient modulus ranges for concrete slab size (Heckel, 2002)	23
Table 2.4: Minimum standard thicknesses for CS&O (Caltrans HDM, 2009)	30
Table 2.5: Medium/High severity reflective cracking prediction model (Carpenter and Darter, 1989)	44
Table 2.6: Low severity reflective cracking prediction model (Carpenter and Darter, 1989)	45
Table 3.1: Central Coast section locations and CS&O rehabilitation dates	49
Table 3.2: Northern California section locations and CS&O rehabilitation dates	49
Table 3.3: Summary of variables for Central Coast region	50
Table 3.4: Summary of variables for Northern California region	50
Table 3.5: Longitudinal cracking PCS & PCR equivalents	52
Table 3.6: Central coast region section IDs and visual survey dates	54
Table 3.7: Northern California region section IDs and visual survey dates	55
Table 3.8: CC Region layer thickness and type measured from extracted section cores	56
Table 3.9: NC region layer thickness and type measured from extracted section cores	57
Table 4.1: Alligator cracking performance models	95
Table 4.2: Transverse cracking performance models	96
Table 4.3: Longitudinal cracking performance models	97
Table 4.4: IRI performance models	98
Table 4.5: Central Coast region sensitivity analysis summary	130
Table 4.6: Northern California region sensitivity analysis summary	131

LIST OF FIGURES

Figure 2.1: Whip Hammer (http://www.keaslersjunk.com/Wolverine_Whip_Hammer.html)	13
Figure 2.2: Self Propelled Guillotine Drop-Hammer (http://www.antigoconstruction.com/specs-t8600.html)	14
Figure 2.3: Rubber Tired Proof Roller.....	15
Figure 2.4: Pneumatic Rubber Tire Roller (http://www.cat.com/cmms/13972553)	18
Figure 2.5: Whip-Hammer Striking Patterns (Osseiran, 1987)	24
Figure 2.6: Whip-Hammer Spider Web Cracking Pattern (Osseiran, 1987)	25
Figure 2.7: Striking Pattern Diagram for Iowa Test Sections (Marks, 1993)	26
Figure 2.8: Florida DOT CS&O Rehabilitation Cross-Section (Choubane and Nazef, 2005)	28
Figure 2.9: Typical California CS&O cross section	29
Figure 2.10: Amount & severity of reflective cracking for unreinforced CS&O projects (Carpenter , 1989).....	33
Figure 2.11: Crack Rating for Pavement Sections in Florida (Choubane and Nazef, 2005)	35
Figure 2.12: CS&O Transverse Cracking in Virginia (Freeman, 2002)	36
Figure 2.13: CS&O Transverse Cracking in Virginia (Freeman 2002)	36
Figure 2.14: Transverse Cracking for CS&O sections in Iowa (Harris 1993)	37
Figure 2.15: Longitudinal Cracking for CS&O sections in Iowa (Harris 1993)	38
Figure 2.16: Transverse cracking spacing versus time for Illinois test sections..	39
Figure 2.17: Rutting measurements for Florida CS&O sections (Choubane & Nazef 2005).....	41
Figure 2.18: Roughness data for Florida CS&O sections (Choubane & Nazef 2005).....	43
Figure 3.1: Austin Enterprises Core Extractor	58
Figure 3.2: Extracted cores from section SB_101N_15.24	59
Figure 3.3: Alligator cracking before cleaning for the Central Coast Region	62
Figure 3.4: Alligator cracking after cleaning for the Central Coast Region	62
Figure 3.5: Transverse cracking before cleaning for the Central Coast Region	63
Figure 3.6: Transverse cracking after cleaning for the Central Coast Region	64
Figure 3.7: Longitudinal cracking before cleaning for the Central Coast Region	64
Figure 3.8: Longitudinal cracking after cleaning for the Central Coast Region	65
Figure 3.9: IRI before cleaning for the Central Coast Region	66
Figure 3.10: IRI after cleaning for the Central Coast Region	67
Figure 3.11: Alligator cracking before cleaning for the Northern California Region	68
Figure 3.12: Alligator cracking after cleaning for the Northern California Region	68
Figure 3.13: Transverse cracking before cleaning for the Northern California Region	69

Figure 3.14: Transverse cracking after cleaning for the Northern California Region	70
Figure 3.15: Longitudinal cracking before cleaning for the Northern California Region	71
Figure 3.16: Longitudinal cracking after cleaning for the Northern California Region	71
Figure 3.17: IRI data before cleaning for the Northern California Region	72
Figure 3.18: IRI data after cleaning for the Northern California Region	73
Figure 3.19: Alligator Cracking for sections with different overlay thicknesses in the Central Coast region	75
Figure 3.20: Transverse cracks for sections with different overlay thicknesses in the Central Coast region	76
Figure 3.21: Longitudinal cracks for sections with different overlay thicknesses in the Central Coast region	77
Figure 3.22: IRI values for sections with different overlay thicknesses in the Central Coast region.....	78
Figure 3.23: Alligator Cracking for sections with different overlay thicknesses in the Northern California Region.....	79
Figure 3.24: Transverse cracks for sections with different overlay thicknesses in the Northern California region	80
Figure 3.25: Longitudinal cracks for sections with different overlay thicknesses in the Northern California region	81
Figure 3.26: IRI values for sections with different overlay thicknesses in the Northern California region.....	82
Figure 3.27: Alligator cracking for the Central Coast and Northern California regions.....	84
Figure 3.28: Transverse cracking for the Central Coast and Northern California regions.....	85
Figure 3.29: Transverse cracking for Central Coast and Northern California regions (vs. Age)	86
Figure 3.30: Longitudinal cracking for the Central Coast and Northern California regions.....	88
Figure 3.31 : Longitudinal cracking for the Central Coast and Northern California regions (vs. Age)	88
Figure 3.32: IRI for the Central Coast and Northern California regions	90
Figure 4.1: Actual versus predicted graph for alligator cracking in the Central Coast region.....	100
Figure 4.2: Actual versus predicted graph for alligator cracking in the Northern California region	100
Figure 4.3: Residual vs. Predicted plot for alligator cracking in the Central Coast region.....	101
Figure 4.4: Residual vs. Predicted plot for alligator cracking in the Northern California region	102
Figure 4.5: Actual versus predicted graph for transverse cracking in the Central Coast region.....	103

Figure 4.6: Actual versus predicted graph for transverse cracking in the Northern California region.....	103
Figure 4.7: Residual vs. Predicted plot for transverse cracking in the Central Coast region.....	104
Figure 4.8: Residual vs. Predicted plot for transverse cracking in the Northern California region	105
Figure 4.9: Actual versus predicted graph for longitudinal cracking in the Central Coast region.....	106
Figure 4.10: Actual versus predicted graph for longitudinal cracking in the Northern California region	106
Figure 4.11: Residual vs. Predicted plot for longitudinal cracking in the Central Coast region.....	107
Figure 4.12: Residual vs. Predicted plot for longitudinal cracking in the Northern California region	108
Figure 4.13: Actual versus predicted graph for IRI in the Central Coast region	109
Figure 4.14: Actual versus predicted graph for IRI in the Northern California region	109
Figure 4.15: Residual vs. Predicted plot for IRI in the Central Coast region	110
Figure 4.16: Residual vs. Predicted plot for IRI in the Northern California region	111
Figure 4.17: Age sensitivity analysis for alligator cracks in the Central Coast model	113
Figure 4.18: MESAL sensitivity analysis for alligator cracks in the Central Coast model.....	114
Figure 4.19: HMA/PCC thickness ratio sensitivity analysis for alligator cracks in the Central Coast model.....	114
Figure 4.20: MCESAL sensitivity analysis for transverse cracks in the Central Coast model.....	116
Figure 4.21: HMA/PCC ratio sensitivity analysis for transverse cracks in the Central Coast model.....	116
Figure 4.22: MCESAL sensitivity analysis for longitudinal cracks in the Central Coast model.....	117
Figure 4.23: HMA/PCC ratio sensitivity analysis for longitudinal cracks in the Central Coast model.....	118
Figure 4.24: Age sensitivity analysis for IRI in the Central Coast model.....	119
Figure 4.25: MESAL sensitivity analysis for IRI in the Central Coast model....	120
Figure 4.26: Age sensitivity analysis for alligator cracking in the Northern California model.....	121
Figure 4.27: MESAL sensitivity analysis for alligator cracking in the Northern California model.....	122
Figure 4.28: HMA/PCC ratio sensitivity analysis for alligator cracks in the Northern California model.....	122
Figure 4.29: Age sensitivity analysis for transverse cracking in the Northern California model.....	123

Figure 4.30: MESAL sensitivity analysis for transverse cracking in the Northern California model.....	124
Figure 4.31: HMA/PCC ratio sensitivity analysis for transverse cracks in the Northern California model.....	125
Figure 4.32: Age sensitivity analysis for longitudinal cracking in the Northern California model.....	126
Figure 4.33: MESAL sensitivity analysis for longitudinal cracking in the Northern California model.....	126
Figure 4.34: HMA/PCC ratio sensitivity analysis for longitudinal cracks in the Northern California model.....	127
Figure 4.35: Age sensitivity analysis for IRI in the Northern California model.....	128
Figure 4.36: MESAL sensitivity analysis for IRI in the Northern California model.....	129
Figure 4.37: HMA/PCC ratio sensitivity analysis for IRI in the Northern California model.....	129

CHAPTER 1

INTRODUCTION

1.1 Background

Roadway maintenance is the primary issue facing transportation agencies seeking to extend the life of existing roadways. Decreasing availability and increasing costs of construction materials is making new construction more expensive, which has forced roadway maintenance to be a major expenditure for transportation departments throughout the nation. Additionally, modern safety standards have dictated that roadway surfaces be maintained to a level that provides the safest travel possible for motorists. History has shown that highways constructed using Portland Cement Concrete (PCC) have provided long lasting durability with regular maintenance. Many PCC highways have outlasted their intended design lives, with some even doubling that figure. Despite the high durability of PCC pavements, they too eventually reach the end of their service lives. Once this point is reached, the maintenance and restoration costs become much higher. There are many common restoration treatments used by transportation agencies, and each have their own strategies and practices. Depending on the traffic volume, the most prevalent restoration treatment is the use of overlays. Although there are various types of overlays, hot-mix asphalt (HMA) is the most common.

The first use of overlays as a viable option for restoring PCC pavements can be traced to the 1950's. This is when engineers first realized that an overlay

thinner than conventional AC pavement could be used when rehabilitating an existing PCC slab supported by a high strength base. However, soon after overlaying these deteriorated slabs, it was discovered that cracks and joints present in the PCC were reflecting through the AC overlay. This type of cracking is known as reflective cracking. The cracks are “reflective” because they typically mirror the existing cracks and joints within the underlain PCC slabs. This cracking is considered to be a premature failure mechanism, and it severely impacts the performance and usability of the pavement. Reflective cracking was primarily due to differential horizontal and vertical movements along existing joints and cracks in the underlain PCC pavement. The movement of the concrete can be attributed to expansion and contraction due temperature changes, vertical movement due to a weak base materials, frost heave, and heavy vehicle traffic. Reflective cracking is most often caused by horizontal movement of the pavement along cracks or joints. The propagation of theses cracks is accelerated by vertical movement of the pavement due to vehicle loading. Unless additional rehabilitation methods are employed, experience has shown that reflective cracking will appear within one to two years after placing an HMA overlay over a PCC pavement base.

1.2 Techniques to Minimize Reflective Cracking

There are many different techniques used to minimize reflective cracking in PCC pavements rehabilitated using HMA overlays. Different methods are selected based on the condition of the PCC pavement, the type of PCC, the condition of the sub-grade, budget restraints, and designer experience and/or

preference. One of the most common techniques employed to minimize the potential for reflective cracking is Crack, Seat, and Overlay (CS&O). For CS&O to work properly it must be used for jointed plain concrete pavement (JPCP). This type of PCC pavement contains no steel reinforcement, except for small bars located at transverse and longitudinal joints. These are known as dowel bars and tie bars, respectively. The main goal of the CS&O technique is to crack the existing slab into smaller pieces while maintaining aggregate interlock between the sections. This is most commonly performed using a gravity or pneumatic type breaker. The purpose of cracking is to reduce the concrete slabs into sections small enough to reduce horizontal movement, but large enough to maintain structural integrity. After cracking, the new smaller sections must be seated into the existing base layer to restore contact and limit vertical movement. As mentioned before, care must be taken to insure that the structural integrity of the interlocking slab sections remains high to provide the best foundation for the HMA overlay. If too much force is used during the cracking process, aggregate interlock will be damaged and the effectiveness of the technique will diminish.

An alternative to the CS&O is to entirely destroy the existing slab by breaking it into loose aggregate pieces usually less than 9 inches in any dimension. This technique is often referred to as rubbilization. It is usually only used on slabs that have very little remaining structural strength.

In addition to mechanical measures taken to minimize reflective cracking, many transportation agencies employ the use of stress reducing membranes at the PCC/HMA overlay interface. PRF or pavement reinforcing fabric is commonly

used in this application. PRF's are generally 100 percent polypropylene staple fiber fabric, which help absorb and distribute stresses that cause reflective cracking.

Mechanically destructive techniques such as CS&O and rubbilization are a common alternative to full depth slab replacement. Because of the costs associated with the construction of a full depth slab and the issues it creates regarding lane closure. Closure time is an important consideration for rehabilitation projects as unsafe conditions for motorists and reduced productivity can generate unforeseen costs. In comparison to other rehabilitation methods, CS&O requires a lesser amount of time devoted to demolition, waste removal, pavement preparation, and construction time. It is for these reasons that the California Department of Transportation (Caltrans) prefers to use CS&O rehabilitation over other methods for PCC sections with the appropriate qualifying criteria.

1.3 Purpose

The purpose of this research is to evaluate the performance of the crack, seat, and overlay (CS&O) rehabilitation technique in the Central Coast and Northern regions of California. The performance evaluation assesses the impact of several pavement characteristics: age, PCC thickness, HMA overlay thickness, subbase type, vehicle traffic (equivalent single axel load). Performance models were developed correlating pavement responses to the aforementioned characteristics. The pavement responses considered within the

scope of this study are: transverse cracking, longitudinal cracking, alligator cracking, and the International Roughness Index (IRI). Data for this study were collected from Caltrans as-built drawings, field observations, and Caltrans' Pavement Condition Reporting software (PCR).

1.4 Thesis Organization

Chapter two of this thesis contains a review of previous research related to the use of the CS&O rehabilitation technique within California and the U.S. The review focuses on the factors that might affect the performance of these pavements with regard to minimizing reflective cracking.

Chapter three provides a description of the data collection and research methods used for this project. Data were collected using the Caltrans Pavement Condition Reporting (PCR) software, as-built drawings, and field observations. Prior to analyses, these were reviewed for erroneous or unreported points and outliers. Traffic data were then interpolated, and pavement thicknesses were verified. Cracking and IRI data were plotted as a function of the pavement characteristics mentioned previously (i.e. the independent variables for this study). These plots were evaluated for trends. Positive, negative, and extraneous trends were assessed to help produce and evaluate the mathematical relationships expressed in the performance model equations.

Chapter four explains the technique and methodology used to develop the regression empirical equations. Statistical analyses were conducted, including analysis of predicted values and their residual differences from actual observed

distresses. These methods were used to study the robustness and predictability of the empirical models developed.

Chapter five presents conclusions based on the analysis results developed and suggests recommendations for further research.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

According to the Federal Highway Administration's 2007 survey, there are 171 thousand miles of pavement in California (Public Road Length 2007). These pavement sections can be classified into three major categories: flexible (asphaltic), rigid (concrete), and composite. Flexible pavements consist of one or more layers of Hot Mix Asphalt (HMA) concrete laid over a base, subbase, and subgrade. HMA is made up of aggregate materials held together by bitumin. The aggregate provides the strength and rigidity of the pavement while the flexible bitumin bonds the aggregate together. Flexible pavements typically need stronger base layers to protect the weaker subgrade. In California, a common base used is called cement treated base (CTB), which consists of mixed aggregate held together with cement.

Rigid pavements are typically constructed with a layer of Portland Cement Concrete (PCC) over a subgrade. A base layer may be sandwiched in between. PCC is a much stiffer material than HMA, distributing traffic load over a larger area and making pavement deflection more uniform. Rigid PCC pavements may or may not contain structural steel reinforcing. California does not include structural steel reinforcing within rigid pavements, except at joints between slabs. Here transverse and longitudinal dowels are used to structurally tie PCC slabs together.

Composite pavements combine rigid and flexible pavements into one structural pavement section. Typically, an HMA layer is used as an overlay atop a PCC pavement layer. Composite pavements are rarely chosen for new construction. If they are, it is usually because of a poor subgrade. More commonly, composite pavements are used in a rehabilitation situation where the PCC layer is no longer suitable to support vehicle traffic. Overlaying this PCC layer with a smoother HMA pavement surface restores functionality while retaining the PCC pavement's ability to distribute stresses over the subgrade. These pavements are susceptible to reflective cracking, where cracks and joints within the PCC are reflected through the HMA overlay.

According to the 2007 Caltrans Pavement Condition Survey, there are 16,350 lane miles of rigid pavement within California, which account for 32 percent of all the lane miles. Many of these rigid pavements were constructed during the 1950's and 1960's after the passage of the Federal Highway Act of 1956. Some of these highways are still in service today, well outlasting their projected design life. In 2007 alone, 667 million dollars were spent rehabilitating and maintaining existing highways within California (State of the Pavement Report 2007).

As rigid pavements deteriorate they produce a lower level of service for traffic. A common measure for determining pavement quality is the International Roughness Index (IRI). This standard measure is calculated by determining a vehicle's up and down movement, in inches, over one mile of pavement. For a new, smooth road, a typical IRI would be less than 75. A road in which the IRI is

above 170 is consider a "rough ride" and is considered ready for repair or rehabilitation. In 2007, approximately 18 percent of all roads in California were found to be in the "rough ride" category (State of Pavement Report 2007).

Caltrans further categorizes pavements into five different condition states. These states are based on observed pavement distresses, as shown in Table 2.1. Since Pavement Condition States 4 and 5 require rehabilitation or reconstruction, only pavements in these categories are included in this study.

Table 2.1: Caltrans Pavement Condition States (Caltrans 2007)

	Description	Treatment Classification
Stage 1	Excellent condition with no, few potholes or cracks	Future Preventative Maintenance project
Stage 2	Good condition with minor potholes or cracks	Preventative or Base Maintenance project
Stage 3	Fair condition with moderate potholes and cracks	CAPM ¹ project
Stage 4	Poor condition with significant cracks	CAPM project or Rehabilitation candidate
Stage 5	Poor condition with extensive cracks	Long Life or Rehabilitation/Reconstruction candidate

¹ Capital Preventative Maintenance

For rigid pavements, there are only a few possible methods for rehabilitation. These include full-depth replacement, unbonded rigid overlay with HMA interlayer, or crack and seat slabs with a HMA overlay (Highway Design Manual 2009). The full-depth replacement option is often not considered due to the costs and time required for this type of rehabilitation. Additionally, unbonded

rigid overlays have been found to be less cost effective than cracking and seating slabs with a HMA overlay (Saraf 1991).

Once a rigid pavement has reached Pavement Condition State 4 or 5, Caltrans guidelines require that it either be overlaid or replaced. The replacement option is only chosen during special circumstances, due to the time and costs associated with reconstruction. Most often, rigid pavements are overlaid with an additional pavement layer to save costs and restore ride quality. Most transportation agencies choose to overlay their rigid pavements with Hot Mix Asphalt Concrete (HMA) surfaces. This is due to the desirable properties HMA provides, such as noise reduction, smoothness, and ease of construction. When a rigid Portland Cement Concrete (PCC) pavement is overlaid with a flexible HMA pavement, the resulting combined pavement is considered a composite pavement.

For a composite pavement to work effectively, the PCC provides a strong base while the HMA provides a smooth, non-reflective surface (Huang 2004). This alludes to the main problem and serviceability issue with composite pavements: reflective cracking. Reflective cracking is mainly caused by the movement of the PCC slab beneath the HMA surface. This is commonly caused by thermal or moisture initiated expansion and contraction (Huang 2004). When there is no overlay present, the expansion and contraction of the PCC pavement occurs at the pavement joints and existing cracks. A rigid pavement section in need of an overlay likely has numerous transverse, longitudinal, and even alligator cracks (fatigue cracking). Therefore, if the potential for expansion and

contraction of the PCC at these crack and joint interfaces is not reduced, reflective cracking is likely to occur.

There are several different techniques used to reduce the likelihood of reflective cracking. In this study, the Crack, Seat, and Overlay (CS&O) technique is investigated. The CS&O technique is used for PCC pavement sections that are considered jointed plain concrete pavements (JPCP), which represents the most common type of rigid pavement in California. The CS&O technique was introduced in the 1960's, and has gained popularity as an economical and environmentally friendly way to rehabilitate PCC pavements while reducing reflective cracking. The California Department of Transportation (Caltrans) currently provides CS&O design thicknesses for a design life of 20 years, after which, further maintenance is carried out in concordance with Caltrans composite pavement guidelines. As mentioned before, the CS&O technique is only to be considered for pavements that have reached Pavement Condition State 4 or 5, as described in Caltrans' State of the Pavement Report 2007, and in general, where more conventional pavement repair is economically prohibitive (Wixson, 1986).

2.2 Crack, Seat and Overlay (CS&O)

The focus of this study is the rehabilitation of Portland Cement Concrete (PCC) pavements by cracking, seating, and overlaying the PCC pavement with a layer of HMA. The CS&O technique is specifically used on PCC pavements that are not reinforced throughout, or Jointed Plain Concrete Pavement (JPCP). The

pavement may have dowels and tie-bars between slabs to transfer traffic loading. Ideal pavements that are candidates for CS&O are not worn or damaged to the point where they may lose their aggregate interlock during the cracking process. For the cracking portion of CS&O to work correctly, the slab must be entirely cracked from top of the slab to the top of the base, without losing its aggregate interlock. This ensures that the cracked slab will retain its structural integrity and load transferring capabilities when seated. The CS&O method is used throughout the United States on varying base materials, subbase materials, and pavement thicknesses, and many states have conducted relevant research regarding CS&O.

California PCC pavements are typically 8- to 12- inches thick and constructed over a 4- to 6-inch cement treated base (CTB), with a sand or earthen subgrade (Caltrans HDM, 2009). Survey cores of pavement rehabilitated using the CS&O technique have shown that these standards are not always followed (Rahim and Fiegel 2008). Some pavement sections do not contain a CTB, but rather a thicker, less dense aggregate base. Compared to an aggregate base, a CTB provides a much stronger and reliable structural support for the overlying pavement. However, a CTB may lose significant strength if it is accidentally cracked during the CS&O process.

2.2.1 Equipment

State transportation departments use various techniques for cracking and seating their PCC pavement, as many different types of mechanical breakers

have been used for cracking the PCC. Research conducted by Felter (1989) for the Michigan Department of Transportation found several types of cracking equipment, each of them producing adequate results. This equipment includes a modified diesel pile driver, wrecking ball, whiphammer, and a guillotine-type drop hammer. The most successful and widely used methods were found to be the whiphammer (Figure 2.1) and the guillotine drop hammer (Figure 2.2).

Figure 2.1: Whiphammer
(http://www.keaslersjunk.com/Wolverine_Whip_Hammer.html)

However, Sharpe (1988) found that the most common cracking equipment used is the modified diesel pile-driving hammer. The hammer is typically mounted to a rolling trailer and is towed by a tractor. The impact energy may be changed by adjusting the amount of fuel to the hammer. This equipment can produce 18- to 24-inch concrete fragments with 3 to 4 passes per lane by using a rectangular shaped impact head at the tip of the hammer (Sharpe, 1988).

Figure 2.2: Self propelled guillotine drop hammer
(<http://www.antigoconstruction.com/specs-t8600.html>)

In a study prepared for the Arizona Department of Transportation, a whiphammer was used to crack the existing PCC pavement (Osseiran, 1987). The impact footprint was approximately a 4.5-inch x 7.0-inch rectangle. The force developed by this type of whiphammer is a function of the pressure in the hydraulic system and the resiliency/number of leaf springs supporting the hammer head (Sharpe, 1988). The cracking pattern produced by the

whiphammer was not visually detectable on dry pavement, so water had to be used in conjunction with the whiphammer to observe the cracking pattern.

Osseiran (1987) found that proper seating is a crucial process for providing a stable supporting layer for the HMA overlay. Osseiran also found that seating will fill any possible voids in the subgrade, resulting in reduction of differential settlement at these points. For this case history, a "wagon-like" rubber tire roller filled with sand ballast (Figure 2.3) was used to seat the cracked PCC slabs. Similarly, the Michigan DOT also found that a 50-ton rubber-tired roller (Figure 2.3) is the most successful and widely used device for seating the cracked sections.

Figure 2.3: Rubber tired proof roller

(<http://www.antigoconstruction.com/images/p-cs-s-4.JPG>)

In Illinois, Schutzbach (1989) observed through coring pavement test sections that a hydraulic powered spring hammer or whiphammer did not properly crack the PCC sections. The study concluded that sections exhibiting

early reflective cracking were all cracked insufficiently by a whiphammer breaker. Test cores also showed that sections cracked using a guillotine style breaker were able to provide the desired full-depth cracking. It was also noted that pavement sections with a weaker subgrade should be seated using a 35-ton roller vs. a 50-ton to prevent possible damage to the subgrade that may aid in the formation of reflective cracking.

A study of CS&O sections in California by Wells et al. (1991) showed that whiphammers produce extensive shattering beneath the impact point, regardless of the head type. Additionally, impacts within a foot of any existing joint or crack cause severe spalling, especially at the crack intersections. Therefore, it was found that a guillotine drop-hammer best met the requirements set forth by Caltrans. The most common type of guillotine drop-hammer used is a 12,000 lb. self propelled model as shown in Figure 2.2. Wells et al. (1991) also found that California's experience with cracking and seating pavements goes as far back as the 1960's.

In the late 1960's, a project near Tracy followed procedures previously used by Minnesota, using a heavy roller to seat the pavement. Caltrans attempted to use a 50-ton pneumatic roller (Figure 2.4) to seat the PCC, before the HMA overlay was applied. After completion of the project, inspections found that the cracking process was not completed successfully. It was found that no cracking appeared in the PPC pavement surface, even after several passes of the roller occurred. The unsuccessfulness of this attempt at the CS&O technique was attributed to the fact that the project was conducted during the late summer

when the subgrade was dry and strong, providing abnormally good support. After studying CS&O projects in California and Indiana, Carpenter and Darter (1989) concluded that the use of excessively heavy rollers may diminish the beneficial characteristics of the seating process. The study also recommended measuring seated slab deflections in order to determine the best roller weight to use.

A study performed for the Florida Department of Transportation by Choubane and Nazef (2005) describes another piece of equipment known as a pneumatic breaker. A pneumatic breaker is similar to Michigan and Kentucky's pile driver; however, the equipment is more adjustable. A pneumatic hammer could be adjusted to increase or decrease striking force, depending on what is appropriate for a given project. The downside of pneumatic breakers is they cover a far lesser area than the guillotine style used in California.

The study by Choubane and Nazef (2005) focused on 7 CS&O sections within northern Florida. These sections were all jointed plain concrete pavements (JPCP), and were cracked using a 4,000 lb. gravity-type breaker. Their findings indicated that in order to obtain the optimal performance from CS&O sections, care must be taken not to damage underlying base layers. This damage may be caused by using too strong of a cracking force or too heavy of a seating device. The use of a light, 4,000 lb. breaker was successful in preventing damage to the 12-inch cement treated base (CTB) beneath the 9-inch PCC pavement surface. The seating process was performed using a pneumatic tire roller (Figure 2.4), and the weight was not specified.

Figure 2.4: Pneumatic rubber tire roller
(<http://www.cat.com/cmms/13972553>)

Freeman (2002) reported that cracking of the pavement is usually accomplished using modified pile driver, whiphammer, or guillotine. Freeman (2002) concluded that a Wirtgen AG guillotine drop hammer with a 6-ft. wide, 12,000 lb. free-falling blade consistently produced the desired cracking pattern. This equipment showed great versatility in producing satisfactory results in eight different study sites. Freeman (2002) attributes this to the device's ability to control the equipment speed and drop height. This study found that a drop height of 4-feet, with two passes of the 6-foot wide blade per each 12-foot wide lane produced the specified fracture pattern. For the seating portion of the rehabilitation, Freeman (2002) found that a 35- or 50-ton pneumatic tire roller (Figure 2.4) used by the state of Virginia seated the fractured PCC slabs into the base adequately, which also indicated that the purpose of seating was to ensure slab contact with the base layer and to locate damaged zones in the underlying base and subgrade. For this study, seating was of atypical importance, since

traffic was allowed to travel on the seated fractured PCC slabs before applying the HMA overlay.

Research conducted by Harris (1993) for a project in Fremont County, Iowa found that the Iowa Department of Transportation uses a guillotine style breaker very similar to the one shown in Figure 2.2. Before the project began, a test section was set up, and varying drop heights and cracking patterns were used to determine the best combinations. The study found that using a 3-foot spacing and 12-inch hammer drop produced a satisfactory cracking pattern. This combination proved to be successful for the remainder of the project. Once cracking was completed, the sections were seated using a 50-ton sand ballast roller (Figure 2.3) towed by a farm tractor.

In Shelby County, Iowa, Marks and Anderson (1993) had similar findings to those of Harris (1993), regarding the use of a 6-ton guillotine style breaker and a 2-inch wide blade on the striking edge of the hammer. For this study, a more extensive test section was set up where different striking patterns could be tested and observed. There were a total of five striking patterns tested on five separate 60-foot long test sections. It was found that a drop height of 16-inches, with two passes per lane at 4-foot intervals produced the best results for this project. Another test section using the same intervals found that excessive force was generated when using a 20-inch drop height, and caused unpredictable crack propagation in all directions. The other test sections were conducted at 12- and 16-inch drop heights with five even strike intervals for every 20-feet. These patterns were found to generate too much force as well, creating excessive

longitudinal cracking. Cores from the test section cracked using the chosen strike pattern of 16 inches at 4-foot intervals were extracted. These cores showed development of cracks through the full slab thickness, without loss of aggregate interlock. These observations then allowed for that technique to be used on the remainder of the project. Seating was accomplished using a 50-ton pneumatic roller similar to that in Figure 2.4. Marks and Anderson (1993) also mentioned that during the seating process it was hard to detect visible movement of the cracked slabs, but audible cracking noises could be heard.

Schutzbach (1989) studied six separate projects in Illinois that used the CS&O method. Five of these projects used a hydraulic whip-hammer (Figure 2.1), while the remaining projects used a 12,000 lb. guillotine breaker. Schutzbach (1989) found that cracking a full-lane width required numerous blows from the whiphammer, while the guillotine hammer was capable of cracking a full lane width in one blow. Hammer drop height and spacing had to be determined on a per project basis in order to obtain the desired cracking dimensions. In each project, cores were taken to ensure the cracks ran the full-depth of the slab and that aggregate interlock was not lost. After cracking, each project used a rubber tire roller (Figure 2.3) to seat the cracked slabs. Traffic was allowed to travel on the seated sections before the overlay was applied. If any soft spots or differential settlement was noticed during the seating process, these locations were immediately replaced to their full-depth with HMA.

2.2.2 Slab Size

Determining slab size is a crucial portion of the crack, seat, and overlay (CS&O) process. Larger-sized slabs are more susceptible to thermal expansion and contraction, thus increasing the risk of reflective cracking. The extent to which smaller slabs may reduce reflective cracking in the overlay has a limit. If slabs are too small, they will lose strength and may deflect under the weight of vehicles. Carpenter and Darter (1989) recommend that test sections be constructed prior to using the CS&O technique, allowing the contractor to investigate various equipment and striking patterns.

Despite past research on the CS&O process, slab size still remains a controversial subject. Huang and White (1995) found that North Dakota and Minnesota specify transverse strikes every 3-feet while California prefers strikes every 4- to 6-feet. Sharpe (1988) found that Kentucky prefers sizes to be nominally 24 inches. In a similar study, Wixson (1986) also found that California preferred cracked sections between 4- and 6-square feet

Al Hakim (1999) stated that reflective cracking is caused by horizontal and vertical movement of the seated PCC pavements. Horizontal movement caused by thermal strains can be reduced by decreasing the slab size. However, the slabs stiffness, which resists vertical movement, decreases as a function of the slab size. Al Hakim (1999) produced the data presented below in Table 2.2, documenting the change in the PCC stiffness before and after the CS&O process. A nearly 50% drop in stiffness after cracking and seating shows the

importance of choosing an appropriate slab size to avoid any excessive stiffness losses.

Table 2.2: Change in PCC layer stiffness (Adapted from Al Hakim, 1999)

Comparison between pavement layer stiffnesses before and after crack and seat operation					
Material	Direction	Statistical values	Before (MN/m ²)	After (MN/m ²)	Variation Ratios (%)
Concrete	N/B	Mean	36200	17400	-48%
		Standard Deviation	10300	6600	-
	S/B	Mean	31600	16300	-52%
		Standard Deviation	8400	7500	-

Felter (1989) also found that reflective cracking is caused by the horizontal or vertical movement of the underlying PCC pavement, attributing horizontal movement to thermal expansion and contraction, and in some cases, variations in moisture content. Vertical movement is generally due to traffic loading which can be amplified by a weak base, and frost heave or voids in the subgrade. Felter (1989) found little variation in reflective cracking for test sections with a crack spacing of 2, 3, and 4 feet.

In Illinois, Heckel (2002) was able to approximately determine the effect of the slab size on the resilient modulus (stiffness) of cracked and seated PCC pavements (See Table 2.3). This study was primarily conducted on rubbilized

PCC pavement which has around half the resilient modulus of cracked and seated pavement (See Table 2.3).

Table 2.3: Resilient modulus ranges for concrete slab size (Heckel, 2002)

Fractured PCC Layer Type	Typical Modulus Ranges, PSI		
Crack and Seat or Break and Seat	12 in. crack spacing	24 in. crack spacing	36 in. crack spacing
	200,000	250,000	300,000
Rubblized	50,000 to 150,000		

Osseiran (1987) conducted studies using a whiphammer as the device to crack the existing PCC slabs. This study experimented with cracking patterns of 3-feet x 3-feet, 4-feet x 6-feet, and 2-feet x 2-feet. Osseiran (1987) did not indicate what pattern was preferable, but he did show in Figure 2.5 what the expected cracking patterns would look like.

Figure 2.5: Whiphammer striking patterns (Osseirran, 1987)

Osseirran (1987) found that the whiphammer did not produce the expected cracking pattern, but more of a spider web crack pattern, as shown in Figure 2.6, with cracked pieces that were not square, but triangular or diamond shaped. Osseirran (1987) also found that the cracks were not visible on dry pavement; water had to be applied to observe the cracking patterns.

Figure 2.6: Whiphammer spider web cracking pattern (Osseiran, 1987)

In Iowa, Harris (1993) found that any slab size less than 3-feet longitudinally made it difficult to control spalling. Marks and Anderson (1993) conducted a separate study in Iowa where a large test section of pavement was established to observe five different cracking patterns. Each of these test sections were surveyed and cored to determine which pattern had the best crack penetration, with minimal slab destruction, and no loss of aggregate interlock. Each test section was 60-feet long and one lane width (12-feet) wide. The five varied striking patterns on the slabs are shown in Figure 2.7. Marks and Anderson (1993) found striking pattern No. 5 was the best technique for this particular Iowa project.

Also, it was mentioned that the following parameters should dictate striking patterns for varying slabs: desired cracking, slab destruction, and

aggregate interlock. Therefore, Marks & Anderson (1993) recommended that pilot test sections be conducted for individual projects.

Figure 2.7: Striking pattern diagram for Iowa test sections (Marks, 1993)

California standard specifications call for a cracking pattern measuring no more than six feet transversely and three to five feet longitudinally. Vertical cracks may not vary from vertical by more than six inches in between the surface and bottom of the pavement. In addition, the cracks must be continuous without

extensive surface spalling and excessive shattering of the pavement surface. Test cores must be extracted to verify cracking through the slab per the specifications before cracking of the entire slab can begin.

2.2.3 Overlay Thickness

Many studies have found that the overlay thickness plays a more significant role in preventing reflective cracking than the cracking and seating process. Felter (1989) found that varying the slab size had very little effect on reflective cracking. However, changing the overlay thickness from 4-inches to 6-inches significantly reduced reflective cracking. Felter (1989), mentions that further study is needed to determine if composite pavements with thicker HMA overlays and no added treatment perform as well as CS&O pavements. Some transportation departments require the use of a geo-synthetic or pavement reinforcing fabric material placed between the lifts of asphalt concrete. The fabric helps to absorb and reduce tensile stresses within the HMA overlay layers. Choubane and Nazef (2005) reported that Florida DOT installed one of these layers that included asphalt rubber as a binder. These types of membranes are known as asphalt-rubber membrane interlayers and are typically thicker than membranes used in other states. A cross section of the Florida test section before and after construction is shown in Figure 2.8. California and Florida use similar construction techniques. A typical cross section for a California CS&O section before and after construction is presented in Figure 2.9.

Figure 2.8: Florida DOT CS&O rehabilitation cross-section (Choubane and Nazef, 2005)

Figure 2.9: Typical California CS&O cross section

The California Department of Transportation (Caltrans HDM, 2009) calls for a minimum thickness of 0.45-feet for HMA overlays. Consisting of a 0.35-foot HMA base layer, stress absorbing membrane, and a 0.10-foot HMA leveling coarse. Table 2.4 contains the minimum standard design thicknesses for CS&O projects (Caltrans HDM, 2009). However, other research within California has led to slight modifications to these standards. A cooperative study between Caltrans and the University of California found that the optimum location for the PRF is

0.1-feet above the PCC pavement surface, and additionally, found that Dense Graded Asphalt Concrete (DGAC) could be used as a 0.1-foot leveling course, and 0.15-foot surface course (Wells 1991). By investigating current practice, Wixson (1986) found that Caltrans prefers a total overlay of 4-inches (put down in three lifts) with a reinforcing fabric between the first and second lift.

Table 2.4: Minimum standard thicknesses for CS&O (Caltrans HDM, 2009)

TI < 12.0	0.35' HMA SAMI-F or SAMI-R 0.10' HMA (LC)	0.20' RHMA-G SAMI-R 0.10' HMA (LC)
TI ≥ 12.0	0.50' HMA SAMI-F or SAMI-R 0.10' HMA (LC)	0.20' RHMA-G 0.15' HMA SAMI-F or SAMI-R 0.10' HMA (LC)

Notes:

(1) If the existing rigid pavement is not cracked and seated, add minimum of 0.10 foot HMA above the SAMI layer.

Legend:

HMA = Hot Mix Asphalt

HMA (LC)= Hot Mix Asphalt Leveling Course

RHMA-G = Rubberized Hot Mix Asphalt (Gap Graded)

SAMI-F = Stress Absorbing Membrane Interlayer (Fabric)

SAMI-R = Stress Absorbing Membrane Interlayer (Rubberized)

Carpenter and Darter (1989) found that overlay thickness plays a critical role in the development of reflective cracking when the overlay is less than 6-inches. In the same reference, it was reported that thicker overlays retard the appearance of reflective cracking and should produce lower severity cracks. However, along with overlay thickness, joint spacing, reinforcing steel, and other characteristics may be significant factors as well. Huang and White (1995) came to similar conclusions, finding that within their test sections the 4-inch overlays had the most cracking, while their 8- to 10-inch overlays had the least cracking.

2.2.4 Pavement Performance

The performance of pavements rehabilitated using the crack, seat, and overlay (CS&O) method has shown promise over the years. In general, the performance of CS&O pavements is based on the deferred development of reflective cracking in the HMA overlay. Studies have shown that the emergence of reflective cracking has been delayed, sometimes significantly, when compared to untreated overlaid pavements of the same thickness. In Iowa, Harris (1993) studied sections that were overlaid with a 3-inch HMA overlay, typically in two 1.5-inch lifts. Mark and Anderson (1993) studied the performance of pavements that had a range of 3- to 6-inches for overlay thickness. These overlays consisted of a binder course ranging from 1.5- to 4.5-inches, placed directly on the seated PCC pavement, as well as a surface course of consistent thickness (1.5-inches) placed atop that. Freeman (2002) found that of the eight projects surveyed in Virginia, five of them had a total overlay thickness of 4.5-inches, and the

remaining three had a total thickness of 6.5-inches. All eight of the projects used a 1.5-inch wearing coarse for the top lift of the overlay.

The following sections discuss pavement performance with regard to reflective cracking of the aforementioned CS&O projects.

2.2.4.1 Reflective Cracking

The most common concern with composite pavements such as crack, seat, and overlaid PCC pavements is reflective cracking through the surface of the new HMA overlay. Delaying the emergence of these cracks extends the design life of the overlay, and reduces maintenance costs. Overlays without reflective cracking will remain smooth and safe along with many other desirable characteristics. Most studies regarding reflective cracking focus on transverse cracking over longitudinal cracking. Transverse cracking is more prevalent since movement due to thermal and vehicle loads tend to shift the underlain PCC in the longitudinal direction. Reflective transverse cracking is most commonly observed at preexisting joints or cracks within the PCC pavement. However, as PCC slabs are cracked, the movement of the smaller pieces can cause tensile forces in the longitudinal direction as well.

A nationwide survey of 60 CS&O projects including reinforced and unreinforced PCC pavements was completed by Carpenter and Darter (1989). It was found that overlay thickness plays a critical role in the development of reflective cracking, particularly when the overlay thickness is less than 6-inches thick. Thicker overlays retard the appearance of reflective cracking, and result in

a lower severity cracking. The average overlay thickness was 4.25-inches, and the extent of reflecting crack observed on these sections is shown in Figure 2.10. The figure shows lengths (ft./1,000 ft.) of low, medium, and high cracking severities. The figure shows that for unreinforced sections, 100 percent of the projects showed zero feet of high severity cracking, concluding that, in general, thicker overlays will perform better for a longer period than thinner overlays placed over the same crack and seat sections (Carpenter and Darter 1989).

Figure 2.10: Amount & severity of reflective cracking for unreinforced CS&O projects (Carpenter , 1989)

Schutzbach (1989) found that in Illinois, premature reflective cracking can be attributed to insufficient slab cracking and seating. Insufficient cracking will produce cracks that do not extend through the full-depth of the slab. The seating process was found to be equally as important. If the cracked slab pieces are not

firmly seated to the subbase/subgrade, they will rock and produce reflective cracking (Schutzbach 1989).

Choubane and Nazef (2005) found that when compared to control sections, the CS&O method successfully reduces reflective cracking for the first few years after construction. However, the observed reduction in reflective cracking only lasts for 4 to 5 years, after which the reflective cracking in the CS&O sections rapidly increases to levels similar to that of the control sections. Choubane and Nazef (2005) studied 14 sections throughout Florida over the course of ten years. Figure 2.11 shows the cracking of these sections over the ten-year study. A crack rating of 10 indicates a pavement free of cracks and patches. Lower numbers indicate increasing crack extent, severity, and location (in or outside of the wheelpath).

Figure 2.11: Crack rating for pavement sections in Florida (Choubane and Nazef, 2005)

In Virginia, Freeman (2002) found that the use of the CS&O technique is an effective means to retard the occurrence of reflective cracking when compared to control (uncracked) pavement sections. Cracking within the CS&O sections remained relatively low for the first 3 years post construction. While the control sections showed cracking after 1 year of use. However, the data collected was too variable to produce any accurate time intervals for the retardation of reflective cracking due to the CS&O technique. Freeman (2002) also observed that the CS&O and control sections eventually reached the same crack density. It was mentioned that the possible delay in reflective cracking propagation due to CS&O would offset any costs associated with the added process. Figure 2.12 and 2.13 show the reflective transverse cracking measured in the Virginia test

sections. Unfractured data points represent the control sections, and fractured data points represent sections that utilized the CS&O process.

Figure 2.12: CS&O Transverse Cracking in Virginia (Freeman, 2002)

Figure 2.13: CS&O Transverse Cracking in Virginia (Freeman 2002)

In Iowa, Harris (1993) concluded that reflective cracking in CS&O pavements was reduced when compared to non-cracked and seated overlaid

pavements. Harris (1993) estimated that over 6 years, the CS&O sections with a 3-inch HMA overlay exhibited an average of 10 percent reduction in reflective transverse cracking when compared to two control sections. In the same sections, reflective longitudinal cracking was reduced by an average of 14 percent over the same 6 years. Harris (1993) also found that for CS&O sections using a thicker overlay would further reduce reflective cracking when compared to a 3-inch to 4-inch overlay. These findings are shown graphically in Figure 2.14 and 2.15. These figures show the degree of cracking for Section 1, 2, and 4 (CS&O with 3-inch overlay), Section 3 (CS&O with a 4-inch overlay), and Sections 5 and 6 (control sections with 3-inch and 2-inch overlays, respectively).

Figure 2.14: Transverse Cracking for CS&O sections in Iowa (Harris 1993)

Figure 2.15: Longitudinal Cracking for CS&O sections in Iowa (Harris 1993)

In Illinois, Schutzbach (1989) found that CS&O sections with 3-inch overlays had roughly doubled the transverse crack spacing when compared to a control (not cracked and seated) section with a 3-inch overlay. This means that the cracked and seated section had approximately half the cracks of the control section at the end of the study. These findings are shown graphically in Figure 2.16, showing that cracking and seating delayed reflective cracking for approximately 3-years.

FIGURE 4 Transverse crack spacing versus time for Illinois Routes 97, 101, and Rockton Road.

Figure 2.16: Transverse cracking spacing versus time for Illinois test sections (Schutzbach 1989)

Moody (1994) found that when CS&O sections perform poorly, with regard to retarding reflective cracking, it is due to insufficient slab cracking, or weak fractured PCC slabs. Moody (1994) found that in Texas test sections, the existing PCC pavement was not cracked entirely through its depth, limiting the

effectiveness of the CS&O technique, additionally finding that in some sections, fractured PCC slabs were not providing adequate strength to support traffic loads. This eventually caused pumping within the subgrade and accelerated the propagation of reflective cracking.

In conjunction with Caltrans, Wells (1991) found that out of 36 California CS&O projects selected for the study, only 10 percent of them exhibited initial reflective cracking after 5 years. Compared sections that were not cracked and sealed being overlaid; 75 percent of these sections showed at least initial stages of reflective cracking. Rahim and Fiegel (2008) found that increasing overlay thickness from between 4 and 6-inches to 8 inches would reduce reflective fatigue cracking up to 10 percent. In the same reference, it is reported that increasing the thickness of an overlays leveling course helps retard reflective transverse cracking.

2.2.4.2 Rutting

Rutting only occurs in flexible pavements with HMA surfaces. Rutting is indicated by permanent deformation of the pavement surface along the wheel-path. Rutting is typically controlled by limiting vertical compressive strain on the top of the subgrade, or limiting rutting depth to a tolerable amount (Huang, 2002). However, as tire pressure and traffic load increase, most rutting occurs in the upper pavement layers rather than the subgrade (Huang, 2002). For CS&O pavements, rutting typically originates in the HMA layers due to the existence of the much stronger PCC base.

Moody found that within the first 24 months after construction of CS&O projects in Texas, rutting remained minimal, ranging from 0.035 to 0.15-inches. Choubane and Nazef (2005) found that for the CS&O sections studied in Florida over a 10-year span, rutting developed slowly, and with a maximum depth of 0.35-inches. Figure 2.17 shows the collected data and verifies that rutting developed slowly for all but two sections: Jefferson County and Leon County (Section 2). Later investigation revealed that these sections had poorly laid asphalt, so they may be ignored.

Figure 2.17: Rutting measurements for Florida CS&O sections (Choubane & Nazef 2005)

Harris (1993) recorded rutting measurement at the beginning and end of a CS&O study in Iowa, finding that less than 0.25-inch of rutting occurred in any of the sections. While all of the sections had 3-inch overlays, it was found that none of the rutting was caused by reduced stiffness of the PCC pavement layer. The

control sections (not cracked and sealed) in this study exhibited similar behavior to the CS&O sections (Harris 1993). In Iowa, Marks and Anderson (1993) found in their test sections that over a 3-year period post-construction rutting increased by a maximum of 0.075 feet, which is well within acceptable values.

For CS&O pavement sections observed in California, Rahim and Fiegel. (2008) concluded that rutting depth is related to cumulative traffic level. It was also found that overlay thicknesses had no significant effect on rutting depth for the same sections.

2.2.4.3 Roughness

Moody (1994) studied the roughness of CS&O sections in Texas, recording the International Roughness Index (IRI) of the pavement for 24 months after construction. It was reported that the IRI for the inside (non-truck) lane increased steadily from 60 to 90 over the 24-month span, while the outside (truck) lane increased drastically from 60 to 140. Choubane and Nazef (2005) found that out of the seven projects surveyed over a 10-year span, five maintained a high level of ride quality. Figure 2.18 shows these recorded observations, where a ride number greater than 4 is consider a high level of ride quality. The daily traffic on these sections varied between 2,000 to 5,000 vehicles.

Figure 2.18: Roughness data for Florida CS&O sections (Choubane & Nazef 2005)

Rahim and Fiegel (2008) conducted a nationwide study of 61 CS&O sections extracted from The Long Term Pavement Performance (LTPP) database, observing IRI for different climatic regions, and California. It was found that for sections with bound bases (i.e. Cement Treated Base), thicker overlays provide for a smoother surface (lower IRI) over the life of the pavement, while overlay thickness makes no difference on pavement smoothness for sections with unbound bases. Based on prediction models produced by Rahim and Fiegel. (2008), in California specifically, it can be approximated that after a 10-year design life a typical CS&O section will have an IRI of 2.05 m/km. This is below the acceptable threshold for Caltrans (2.68 m/km), showing that CS&O sections within California perform adequately with regard to roughness.

2.2.5 CS&O Performance Prediction Models

Models to predict significant distresses in CS&O sections throughout the country were developed by Carpenter and Darter (1989). The report discussed reflective cracking and the use of regression models with a sample size of 107. Presented below in Table 2.5 and Table 2.6 are the models developed for low and medium-to-high severity reflective cracking using these independent variables: age of the overlay, PCC slab thickness, freezing index, seating roller weight, annual rainfall, average annual/monthly temperature, area of cracked slab pattern, and the traffic (ESAL).

Table 2.5: Medium/High severity reflective cracking prediction model (Carpenter and Darter, 1989)

$$\begin{aligned} \text{RFLCMH} = & 14.0523 + 2.928(\text{AGE}) + 0.04158(\text{FI}) - 10.677(\text{TPCC}) - \\ & 0.5853(\text{SWR}) - 13.583(\text{WDT}) - 6.55(\text{LT}) + 3.236(\text{AREA}) + \\ & 2.1345(\text{ANNPREC}) - 0.003928[0.14263(\text{ANNAVGT}) \\ & - 0.12123(\text{ANNPREC}) + 0.1955(\text{ANNRNG}) - 5.9531](\text{ESAL}) \end{aligned}$$

R^2 , correlation coefficient = 0.61;

SEE , standard error of estimate = 32.7; and

N , number of sample units = 107

where,

$RFLCMH$ = high-severity reflection cracking (ft/1,000 ft);

AGE = age of overlay in years;

$TPCC$ = thickness of original slab (in.);

FI = Freezing Index;

SWR = seating weight of roller in tons;

WDT = width of crack pattern (ft.);

LT = length of crack pattern (ft.);

$AREA$ = area of the cracked slab pattern (sq ft.);

$ANNPREC$ = annual precipitation (in.);

$ANNAVGT$ = average annual temperature (°F);

$AVGRNG$ = average monthly temperature (°F);

$ESAL$ = total 18-kip equivalent single-axle load on overlay (in millions).

**Table 2.6: Low severity reflective cracking prediction model
(Carpenter and Darter, 1989)**

$$RFLCL = 87.36 - 1.7074(JTS) + 3.3215(SWR) + 33.596(LT) - 1.5298(AREA) \\ - 47.438(SOIL) - 4.6739(ANNPREC) + 2.5865(ESAL) \times \\ [0.14623(ANNAVG) - 0.12123(ANNPREC) + 0.1955(AVGGRNG) - \\ 5.9531]$$

$$R^2 = 0.41,$$

SEE, standard error of estimate = 111.2, and

$$N = 107$$

where variables are all as previously defined, except:

SOIL = subgrade soil type, 1-coarse, 0-fine grained
JTS = joint spacing (ft.)

Developing these models allowed Carpenter and Darter (1989) to predict where CS&O would or would not be an effective rehabilitation technique. Their analyses also suggested including these additional variables in future research for improved accuracy: overlay thickness, any previous repair techniques used, HMA overlay mixture properties, and mechanistic data of the cracked slab.

Yu (2007) conducted a study to examine different types of mathematical prediction models that could be used to predict future pavement conditions. Yu (2007) mainly focused on a measure of pavement condition called Pavement Condition Rating (PCR), which is similar to IRI. Three different model types were used: Proportional Augmentation Method, Curve Shifting Method, and a Linear Mixed Effects Model (LMEM). The study found that a LMEM was the most robust method used, since its prediction is dependent on the entire known condition

trend instead of single measurements. The three most distinctive and desirable features of the LMEM are the model's ability to take into account correlations between repeated measurements, families of data, and individual sample history. Moreover, the prediction accuracy of the model improves with an increase in sample size and decrease with a variation in sample data (Yu 2007).

Rahim and Fiegel (2008) employed the use of rational regression models to predict future IRI values for CS&O projects nationwide and California. The independent variables considered were: age of overlay, thickness of AC overlay and PCC slab, traffic level (ESAL), and type of base layer. The models were created by selecting a set of desirable criteria; (1) the standard error of the estimate should be minimized; (2) the model should be as mathematically simple as possible; (3) models with the highest coefficient of determination (R^2) were selected; (4) relationships between dependent and independent variables must be relevant; (5) these models must always produce plausible results. One of the most important concerns when developing these kinds of models is watching out for multicollinearity between independent variables. In the same reference it was found that the prediction models provided adequate predictive capabilities (Rahim and Fiegel 2008).

CHAPTER 3

DATA COLLECTION AND ANALYSIS

3.1 Introduction

Data from CS&O sections built in two different climatic regions in California were collected to evaluate pavement performance and develop performance prediction models. These data were obtained for eight CS&O sections on the Central Coast, and nine sections in Northern California. The Central Coast Region (CC) spans along Highway 101 from Paso Robles in the North to Santa Barbara in the South. The Northern California Region (NC) spans along Interstate 5 from Corning in the South to Hornbrook in the North. Maps showing the core locations for both regions are presented in Appendix A.

Distresses investigated in this study include transverse cracks, longitudinal cracking, alligator cracking, and the International Roughness Index (IRI). The effect of several independent (explanatory) variables on CS&O section performance was investigated. These variables included: age (years since CS&O), asphalt overlay thickness, concrete slab thickness, and traffic level. Age for each section was obtained from the Caltrans database and verified from section as-built plans. In order to verify layer thickness core samples were extracted from the study sections. Traffic level data was extracted from the Caltrans Traffic Data Branch website (Caltrans Traffic Data Branch 2011). It is widely accepted that truck traffic is the major cause of damage inflicted on highway pavements; therefore regular automotive traffic was not considered in

this study. Truck traffic was used to calculate the Equivalent Single Axle Load (ESAL) values for the surveyed sections. Performance of these CS&O sections was measured in terms of the aforementioned pavement distresses

3.2 Caltrans Data Collection and Analysis

3.2.1 Section Summary

A total of 17 pavement sections throughout Central Coast and Northern California were identified as CS&O projects and selected for this study. Eight sections represent the Central Coast (CC) Region of California, and nine represent the Northern California (NC) Region. These sections are summarized in Table 3.1 and 3.2, respectively. The post mile range column represents the work extent of the rehabilitation project along U.S. 101 or I-5. The CS&O date represents the year in which a CS&O section was constructed and opened to traffic. In addition to being cracked, sealed, and overlaid, these sections were chosen based on the following criteria: sections must be constructed at least 10-years prior to this study (or oldest possible), as-built construction documents must be available, and sections cannot have been overlaid since the CS&O rehabilitation.

Table 3.1: Central Coast section locations and CS&O rehabilitation dates

County	Route	Bound	PM ¹ Range	CS&O Date
SLO ²	U.S. 101	N ³	55.8-58.8	2000
SLO	U.S. 101	N	58.9-63.6	2002
SLO	U.S. 101	S ⁴	58.9-63.6	2002
SB ⁵ /SLO	U.S. 101	N	88.1-91.0/0.0-0.1	1995
SB	U.S. 101	N	78.7-84.3	1995
SB	U.S. 101	N	27.2-28.6	1999
SB	U.S. 101	N	21.0-24.5	1995
SB	U.S. 101	N	14.2-21.2	1995

¹Post Mile ²San Luis Obispo ³North Bound ⁴South Bound
⁵Santa Barbara

Table 3.2: Northern California section locations and CS&O rehabilitation dates

County	Route	Bound	PM ¹ Range	CS&O Date
TEH ²	I 5	N ³	0.0-8.8	1998
TEH	I 5	N	8.77-22.4	1998
TEH	I 5	N	27.1-28.6	1993
SHA ⁴	I 5	N	18.1-23.3	1998
SHA	I 5	N	36.8-40.2	1998
SHA	I 5	N	56.2-60.5	1993
SHA/SIS ⁵	I 5	N	60.5-67.3/0.0-2.6	1994
SIS	I 5	N	36.7-43.1	1993
SIS	I 5	N	58.1-69.3	2001

¹Post Mile ²Tehama ³North Bound ⁴Shasta ⁵Siskiyou

3.2.2 Distress Data

Distress data were collected using Caltrans' Pavement Condition Reporting (PCR) Software. This software uses Caltrans Pavement Condition Survey (PCS) data to generate spreadsheets that include construction dates, traffic data, and distress survey results for pavement sections throughout the state. Currently, this software only reports data from the years spanning 1998-2007. Distresses obtained from the PCR Software included percentage of alligator cracking,

number of transverse cracks, length (extent) of longitudinal cracks, and IRI. A summary of the distresses observed for the CC and NC Regions is presented in Tables 3.3 and 3.4, respectively. For this study, all distress observations were recorded from the outermost travelled lane, also known as the truck or slow lane.

Table 3.3: Summary of variables for Central Coast region

Variables		Description	Range	Average	Units
Dependent	TRANS	Number of transverse cracks	(0.0 - 5.0)	0.6	%
	LONG	Length of longitudinal cracking	(0.0 - 3.0)	0.6	%
	ALLIG	Percentage area effected by alligator cracking	(0.0 - 100.0)	17.5	%
	IRI	International Roughness Index		75.8	in/mile
Independent	Age	Difference between CS&O rehab, and survey date	(1.0 - 12.0)	4.7	Years
	HMA ¹	Thickness of all layers of asphalt overlay	(4.75 - 6.0)	5.2	In
	PCC ²	Depth of crack and seated PCC slab	(8.0 - 9.5)	8.2	In
	ESAL	Equivalent Single Axle Loads per year	(0.57 - 1.75)	0.89	Axle (10 ⁶)

¹Hot Mix Asphalt ²Portland Cement Concrete

Table 3.4: Summary of variables for Northern California region

Variables		Description	Range	Average	Units
Dependent	TRANS	Number of transverse cracks	(0.0- 5.0)	2.0	%
	LONG	Length of longitudinal cracking	(0.0 - 3.0)	0.6	%
	ALLIG	Percentage area effected by alligator cracking	(0.0 - 100.0)	22.0	%
	IRI	International Roughness Index		82.0	in/mile
Independent	Age	Difference between CS&O rehab, and survey date	(0.0 - 13.0)	6.1	Years
	HMA ¹	Depth of all layers of asphalt overlay	(3.5 - 7.0)	5.3	In
	PCC ²	Depth of crack and seated PCC slab	(8.0 - 12.0)	8.8	In
	ESAL	Equivalent Single Axle Loads per year	(0.94 - 2.03)	1.57	Axle (10 ⁶)

¹Hot Mix Asphalt ²Portland Cement Concrete

3.2.2 Pavement Condition Survey and Reporting

The Caltrans Pavement Condition Survey (PCS) was conducted continuously between 1998 and 2007. During this time, the collected survey data were reported electronically via the Pavement Condition Reporting (PCR) software. Caltrans adheres to a strict process by which this data are collected. Caltrans defines the PCS as observing the severity and extent of surface distresses combined with a ride quality measurement, or International Roughness Index (IRI) for each traffic lane. Additionally, the way in which these distresses are observed varies for asphalt and concrete pavements. In this study, all of the observed sections are asphalt overlay composite pavements, putting them in the asphalt pavement category. Asphalt pavement distresses are observed within 100-foot long sample sections. The observer inputs data into a computer, which then uses software to select a distance between 0.5 and 1.5 miles, and proceeds down the lane for the next survey (drive ahead distance). The more distresses observed at a particular location, the shorter the drive ahead distance. At any time the operator may override the computer if they notice a significant change in distress concentration or type.

3.2.3 Distress Identification and Measurement

The distresses of interest for this study include: alligator cracking, transverse cracking, longitudinal cracking, and IRI. This study utilizes what is known as total alligator cracking. The PCS divides alligator cracking in two different categories: Alligator A Cracking, Alligator B Cracking. The PCR

Software combines these measurements and produces the total alligator cracking value. This measurement is represented in terms of a percentage area of the 100-foot sample section. All of the alligator cracking values included within this report utilize the total alligator percentage value.

Longitudinal cracking is often included within alligator cracking; however, reflective longitudinal cracks are always recorded separately. The extent of longitudinal cracking includes: the total length of all measured longitudinal cracks within the 100-foot sample section and is categorized into four categories: 0', <100', 100'-200', and >200'. The PCR Software represents these four categories numerically as a 0, 1, 2, or 3, respectively. For the purposes of this study longitudinal cracking was calculated as a percentage of total length. Category 1 was assumed to be 75-feet or 6.25 percent of the total 100-foot sample section, category 2 to be 150-feet, or 12.5 percent, and category 3 to be 250 feet, or 20.83 percent. Table 3.5 provides a side by side comparison of these pavement distress categorical equivalents.

Table 3.5: Longitudinal cracking PCS & PCR equivalents

Total Longitudinal Cracking Extent (ft.)	PCR Software Equivalent	Assumed Length (ft.)	Longitudinal Cracking %
0 (No Cracking)	0	0	0
< 100	1	75	6.25
100-200	2	150	12.50
>200	3	250	20.83

Transverse cracking is a common distress found in asphalt pavements, and typically appears at right angles to the centerline. In order for a transverse crack to be recorded, it must extend over 50 percent of the lane width. The number of transverse cracks within the 10- foot sample section are counted, with a maximum number of 5 per surveyed section. A "0" value would indicate no appearance of transverse cracking within the sample section.

The International Roughness Index was developed in part by the United States Cooperative Highway Research Program (NCHRP) and the World Bank. It is the most common measurement of a pavements ride quality or roughness. For the PCS, Caltrans records IRI using an automated laser device mounted to the front of the survey vehicle. This device measures the longitudinal road profile within each wheel path of a given lane. IRI measures the vertical movement of pavements longitudinal profile over a prescribed distance. The units used by the PCS are inches per mile. For example, in California, a pavement with an IRI less than or equal to 95 in./mile is considered to have good ride quality.

3.2.4 Visual Survey

The CS&O sections included in this study had a wide variety of lengths, ranging from 1.4 to 11.2 miles. For performance analyses to be meaningful, pavement thickness and type must be uniform throughout the entire section. As-built documents were obtained for all of the sections to help define layer thicknesses and specific locations of cracked and sealed pavement. Many of these sections were not cracked, sealed, and overlaid throughout the entire

length of their rehabilitation. The locations at which the CS&O technique is employed within these sections is up to the discretion of the designer. Therefore, as-builts were crucial in determining what locations within the rehabilitated sections were to be studied. As part of this study, all survey sections were cored to verify layer uniformity and agreement with as-built documents. In addition to coring, all sections were visually surveyed so that observed surface distresses could be mapped and quantified. Tables 3.6 and 3.7 present specific information regarding the sections surveyed in each region, including: county, route, bound direction, post mile of the first core, and the date when the sections were cored and visually surveyed. Every section was assigned a section ID for reference, as shown in Tables 3.6 and 3.7.

Table 3.6: Central coast region section IDs and visual survey dates

County	Route/Bound	PM ¹	# of Cores	Sec_ID	Date Cored/Surveyed
SLO ²	101/N ⁴	56.06	2	SLO_101N_56.06	9/8/2009
SLO	101/N	60.98	2	SLO_101N_60.98	9/8/2009
SLO	101/S ⁵	61.00	2	SLO_101S_61.00	9/8/2009
SB ³	101/N	90.06	2	SB_101N_90.06	9/9/2009
SB	101/N	82.85	2	SB_101N_82.85	9/9/2009
SB	101/N	27.59	2	SB_101N_27.59	9/10/2009
SB	101/N	21.66	2	SB_101N_21.66	9/10/2009
SB	101/N	15.24	2	SB_101N_15.24	9/10/2009

¹Post Mile ²San Luis Obispo ³Santa Barbara

⁴North Bound ⁵South Bound

Table 3.7: Northern California region section IDs and visual survey dates

County	Route/ Bound	PM ¹	# of Cores	Sec_ID	Date Cored/Surveyed
TEH ²	I 5/N ⁵	1.87	2	TEH_I5_1.87	3/16/2010
TEH	I 5/N	11.88	2	TEH_I5_11.88	3/16/2010
TEH	I 5/N	27.53	2	TEH_I5_27.53	3/16/2010
SHA ³	I 5/N	19.44	2	SHA_I5_19.44	3/17/2010
SHA	I 5/N	39.50	2	SHA_I5_39.50	3/17/2010
SHA	I 5/N	60.04	2	SHA_I5_60.04	3/17/2010
SHA	I 5/N	60.57	2	SHA_I5_60.57	3/17/2010
SIS ⁴	I 5/N	42.60	2	SIS_I5_42.60	3/18/2010
SIS	I 5/N	60.29	2	SIS_I5_60.29	3/18/2010

¹Post Mile ²Tehama ³Shasta ⁴Siskiyou ⁵North Bound

Each visual survey consisted of a 1,000 foot section of pavement. Cores were spaced at approximately 200 feet from the start, and end of the section. Therefore, the approximate minimum spacing between cores was 600 feet, depending on geography and obstacles. Each core was drilled to the depth of the base layer. The post miles listed in the Section ID column of Tables 3.6 and 3.7 represent the location of the first core taken, usually at the 200 foot mark north of the surveyed sections southern limit. After extraction, each core was examined to determine layer type and thickness to the nearest 1/4 inch. The results for the CC Region and NC Region are shown in Tables 3.8 and 3.9, respectively. Appendix B includes photographs of all cores removed from the test sections during the study.

Table 3.8: CC Region layer thickness and type measured from extracted section cores

Section ID	Core #	HMA			Surface Course		PCC ³ , in	Base	
		# of layers	Thick, in	PRF	OGFC ¹ , in	RAC ² , in		Thick, in	Type
SLO_101N_56.06	1	3	5.25	Yes	0.75	0	8.00	4.0	CTB ⁴
	2	3	4.75	Yes	0.75	0	8.00	4.0	CTB
SLO_101N_60.98	1	3	5.00	Yes	0	0	7.75	NA ⁵	NA
	2	3	4.50	Yes	0	0	8.00	NA	NA
SLO_101S_61.00	1	3	4.75	Yes	0	0	7.75	4.0	CTB
	2	3	4.75	Yes	0	0	8.25	4.0	CTB
SB_101N_90.06	1	3	5.00	Yes	0	0	8.00	4.0	CTB
	2	3	4.875	Yes	0	0	8.00	4.0	CTB
SB_101N_82.85	1	3	5.00	Yes	0	0	8.00	5.0	CTB
	2	3	5.00	Yes	0	0	8.00	5.0	CTB
SB_101N_27.59	1	3	4.75	Yes	1.00	0	8.25	NA	NA
	2	3	4.75	Yes	1.00	0	8.25	NA	NA
SB_101N_21.66	1	3	1.25	Yes	0.75	3.00	8.75	4.5	CTB
	2	3	1.25	Yes	0.75	3.00	8.375	3.0	CTB
SB_101N_15.24	1	3	5.00	Yes	1.00	0	8.00	4.0	CTB
	2	3	5.00	Yes	1.00	0	9.50	3.3	CTB

¹Open Graded Friction Coarse

²Rubberized Asphalt Concrete

³Portland Cement Concrete

⁴Cement Treated Base

⁵Not Available (base core could not be extracted)

Table 3.9: NC region layer thickness and type measured from extracted section cores

Section ID	Core #	HMA			Surface Course		PCC ³ , in	Base	
		# of layers	Thick, in	PRF	OGFC ¹ , in	RAC ² , in		Thick, in	Type
TEH_5N_1.87	1	3	4.25	Yes	1.50	0	9.13	4.5	CTB ⁴
	2	3	4.50	Yes	1.50	0	8.50	3.0	CTB
TEH_5N_11.88	1	3	6.25	Yes	0	0	8.50	NA ⁵	NA
	2	3	6.25	Yes	0	0	8.50	NA	NA
TEH_5N_27.53	1	3	5.00	Yes	0	0	8.50	4.0	CTB
	2	3	5.00	Yes	0	0	8.50	4.0	CTB
SHA_5N_19.45	1	3	5.00	Yes	2.13	0	8.00	4.0	ATB ⁶
	2	3	5.00	Yes	2.00	0	9.00	4.0	CTB
SHA_5N_39.50	1	3	5.00	Yes	0	0	8.00	5.0	CTB
	2	3	5.00	Yes	0	0	8.00	5.0	CTB
SHA_5N_60.04	1	3	1.75	Yes	1.00	1.75	8.50	4.0	CTB
	2	3	1.75	Yes	1.00	1.75	8.50	4.0	CTB
SHA_5N_60.57	1	3	2.00	Yes	0	1.75	9.00	4.0	CTB
	2	3	2.50	Yes	0	1.75	9.00	4.0	CTB
SIS_5N_42.60	1	3	5.00	Yes	1.00	0	8.00	4.0	CTB
	2	3	5.00	Yes	1.00	0	9.50	3.3	CTB
SIS_5N_60.29	1	3	5.75	Yes	0	0	12.00	4.0	CTB
	2	3	10.50	Yes	0	0	0	NA	NA

¹Open Graded Friction Coarse

²Rubberized Asphalt Concrete

³Portland Cement Concrete

⁴Cement Treated Base

⁵Not Available (base core could not be extracted)

Even though the cores were only 600 feet apart, the results show variations in layer thickness and type. Because of this variation, only distress data within approximately one-half mile of the cores were considered for analysis. This decision was made to limit errors associated with non-uniform layer thickness and type. Some sections had an Open Graded Friction Course (OGFC) added atop the AC overlay after the original construction. These sections were still included in the study even though surface distresses could not be documented. They remained included since data collected from the PCR prior to adding the OGFC layer were still applicable for use in this study.

3.2.4.1 Coring Procedure

The coring process was performed by Austin Enterprises of Bakersfield, California. Cores were extracted using a 6-inch diameter drill bit mounted to a mechanical press on the back of a pickup truck. A picture of the device used to extract cores in the CC and NC region is shown in Figure 3.1. Once the cores were extracted, samples of the base layer or soil at the bottom of the hole were collected for further examination. The cores were then wrapped for protection and brought back to the lab for measurement and further observations. Figure 3.2 presents a picture of cores extracted from section SB_101N_15.24.

Figure 3.1: Austin Enterprises core extractor

Figure 3.2: Extracted cores from section SB_101N_15.24

3.3 Data Cleaning

Data extracted from the Caltrans PCR software is convoluted and sometimes inaccurate. This is due to the thousands of miles of pavement that are surveyed and the lack of survey uniformity from year to year. It is highly unlikely that the exact same segment of pavement will be surveyed when comparing different survey dates. Therefore, distress data does not often conform to simple and manageable trends. Similarly, some surveyed sections initially report data that is impossible. For example, a section cannot have a reduction in cracking unless there is some sort of repair or rehabilitation performed on the pavement. Unwarranted improvement of distress values was encountered numerous times in this study. Due to these problems, data

extracted from the Caltrans PCR program must be analyzed and cleaned of erroneous data before a meaningful performance analyses can begin.

3.3.1 Data Cleaning Procedures

Raw data collected from the Caltrans PCR program contains 128 different variables. Almost all of these values are unnecessary for the research conducted in this study. The first step in cleaning the data was to eliminate spreadsheet columns containing unnecessary data. Once this was performed, the remaining data for each section included: postmile, HMA type, traffic (ESAL), survey date, total alligator cracking, transverse cracking, longitudinal cracking, and IRI.

The next task was to remove data from surveyed sections who's beginning and ending post-mile were not within one-half mile (plus or minus) of the location of the extracted cores. Additionally, half the length of these sections must be within the one mile (plus or minus), of the post-mile where the cores were extracted. After these steps, the data was then ready to be further analyzed looking for outliers, incomplete entries, and impossible values.

As part of this analysis, the independent variables collected from as-built documents (layer thicknesses and age) were added to the data. At this point, sections could be analyzed one-by-one, looking for outlier data points to remove. Engineering judgment was used to identify data points that did not accurately represent the real-life performance of the surveyed sections. Graphing the data using distresses as the dependent variable and age as the independent variable helped recognize outlier data points. These graphs helped estimate the logical

age at which various distresses would likely appear. Data points from the surveyed sections that contained zero reported distress values after these estimated ages were considered invalid and were deleted from the data set. Furthermore, data points that exhibited uncharacteristically high distress values before this estimated age were removed as well.

3.3.2 Central Coast Region Data

3.3.2.1 Alligator Cracking

Shown in Figure 3.3 and Figure 3.4 are alligator cracking data for the Central Coast Region. Before cleaning, a trend was difficult to discern from the data. Based on available data, it was assumed that alligator cracking started to develop after the pavement reached 5 years old. This age was chosen based on distress values reported by other surveyed sections with the region. After this time, any data reporting zero values of alligator cracking were considered invalid. Data from sections younger than 5 years old with alligator cracking greater than 60 percent were also removed. Based on this criteria, 32 of the original 92 data points were eliminated during the cleaning process. After cleaning, the data shows that alligator cracking increases as the age for surveyed CS&O sections increases.

Figure 3.3: Alligator cracking before cleaning for the Central Coast region

Figure 3.4: Alligator cracking after cleaning for the Central Coast region

3.3.2.2 Transverse Cracking

Transverse cracking data for the Central Coast Region prior to cleaning is shown in Figure 3.5. Based on the data, it was assumed that after 5 years transverse cracking distresses begin to appear in the in these CS&O sections.

Any data reporting a zero value for transverse cracking after 5 years was considered invalid and deleted. Additionally, any sample sections exhibiting more than 2 transverse cracks within the first 5 years were considered unrealistic, and discarded. These errors can be attributed to inconsistencies in the surveying process, differences in recorded post miles, or poor construction. The cleaning process removed a total of 30 data points from the original 82. The majority of this removed data were zero values reported after the assigned 5-year distress appearance cutoff. After cleaning, Figure 3.6 shows an increasing trend in transverse cracking as the sample sections age.

Figure 3.5: Transverse cracking before cleaning for the Central Coast region

Figure 3.6: Transverse cracking after cleaning for the Central Coast region

3.3.2.3 Longitudinal Cracking

Longitudinal cracking data for the Central Coast Region prior to cleaning are shown in Figure 3.7.

Figure 3.7: Longitudinal cracking before cleaning for the Central Coast region

Available data suggests that longitudinal cracking distresses appear in all pavements approximately 7 years after construction. Sample sections having a longitudinal cracking equivalent greater than 1 within the first 7 years were considered to be inaccurate, and removed during the cleaning process. The cleaning process removed 5 of the original 91 data points. Many of the 91 data points presented in Figure 3.7 have identical values. Preventing all 91 points from being visible within the figure. The cleaned data presented in Figure 3.8 suggest an increasing trend in longitudinal cracking extent as the surveyed CS&O sections increase in age.

Figure 3.8: Longitudinal cracking after cleaning for the Central Coast region

3.3.2.4 International Roughness Index (IRI)

IRI data for the Central Coast Region before cleaning are presented in Figure 3.9. The PCR survey IRI data provided by Caltrans appeared to be

Figure 3.9: IRI before cleaning for the Central Coast Region

more consistent than the other recorded distresses. Therefore, little cleaning was performed. Based on the data and prior experience, sections 7 years and older with an IRI below 70, were considered unrealistic and removed. Also, any surveyed sections younger than 2 years old with an IRI above 80 were removed. Only 11 of the 95 total data points were removed during the cleaning process. The cleaned IRI data presented in Figure 3.10 show an increasing trend in IRI as the age of surveyed pavements increased.

Figure 3.10: IRI after cleaning for the Central Coast region

3.3.3 Northern California Region Data

3.3.3.1 Alligator Cracking

Alligator cracking for the Northern California Region before cleaning is presented in Figure 3.11. There is an increasing trend in cracking as the sample sections grow older. However, numerous zero values at older ages eliminate the possibility of verifying this possible trend in performance. Based on available data for the Northern California Region, it was determined that alligator cracking distresses begin to appear 6 years. Any section survey returning zero values after this time was not considered in this study. In addition, any sections exhibiting alligator cracking above 95 percent before 10 years of service were considered unrealistic and removed. Subsequent cleaning of this data eliminated 37 of the original 105 data points. Figure 3.12 shows the data set after cleaning.

Figure 3.11: Alligator cracking before cleaning for the Northern California region

Figure 3.12: Alligator cracking after cleaning for the Northern California Region

3.3.3.2 Transverse Cracking

Transverse cracking data before cleaning for the Northern California Region are presented in Figure 3.13. These sections did not exhibit early

emergence of transverse cracking. This helped reduce the number of data points deleted during data cleaning. The data suggests that after 6 years, the majority of the sections will exhibit some level of transverse cracking. Therefore, any sample sections reporting zero transverse cracking after this time were removed from the data. This adopted criterion resulted in the removal of 19 of the total 101 data points. The cleaned data presented in Figure 3.14 show that once transverse cracking appears in a sample section, it increases rather quickly to the maximum value of 5 per 100 foot test section. A sample section may only report a maximum of 5 transverse cracks.

Figure 3.13: Transverse cracking before cleaning for the Northern California region

Figure 3.14: Transverse cracking after cleaning for the Northern California region

3.3.3.3 Longitudinal Cracking

Northern California region longitudinal cracking data, before cleaning, is presented in Figure 3.15. Unlike the Central Coast region, there are not any instances of early emergence of longitudinal cracking. However, zero values for longitudinal cracking were reported for older sections when similarly aged sections contained some level of observed distress. It was determined that for these sample sections longitudinal cracking distresses should appear after 7 years of service. Any data reporting zero longitudinal cracking after 7 years of service were discarded. Resulting in the removal of 17 of the original 99 data points reported by the PCR software. The data set after cleaning is presented in Figure 3.16.

Figure 3.15: Longitudinal cracking before cleaning for the Northern California region

Figure 3.16: Longitudinal cracking after cleaning for the Northern California region

3.3.3.4 International Roughness Index (IRI)

IRI data before cleaning for the Northern California Region are presented in Figure 3.17. Similar to the Central Coast Region, IRI data obtained from the Caltrans PCR software for the Northern California Region did not contain many

invalid data points. Cleaning of this data set required the removal of 3 data points. With an abnormally high IRI value of 100 at 1 and 2 years of service, and 168 at 3 years of service. These points are easily recognizable in Figure 3.17. The cleaned data set is presented below in Figure 3.18.

Figure 3.17: IRI data before cleaning for the Northern California region

Figure 3.18: IRI data after cleaning for the Northern California region

The purpose of these previous sections was to summarize the procedure and assumptions that were used to collect and clean the data. The collected data will be analyzed in the following sections to identify possible trends and correlations.

3.4 Data Analysis

Performance of the crack, seat, and overlay (CS&O) rehabilitation technique in the Central Coast and Northern California regions is analyzed in this section. Data collected for these analyses were presented in Section 3.3 of this report. The distresses investigated included: percentage of alligator cracking, number of transverse cracks, percentage of longitudinal cracking, and the International Roughness Index (IRI). The explanatory variables include: age, HMA thickness, PCC thickness, and traffic. To evaluate possible trends and correlations in the data, independent variables were plotted against dependent

variables. Independent variables were selected based on their relationship to the distress variables and their usefulness in data correlation. In order to better develop conclusions regarding trends in the data, trend lines were applied to each data set after plotting. These trends/correlations are discussed in the following sections.

3.4.1 Central Coast Region

Cumulative Equivalent Single Axle Load (CESAL) was used as an independent variable to combine the effects of age and traffic level. CESAL is a measurement of the cumulative traffic that a pavement will endure over its years in service. It has been found to be a more robust predictor for evaluating trends and correlations in this type of data, when compared to annual ESAL and age as separate variables.

This variable CESAL was plotted against all the available distress variables. The data was broken into two categories of HMA thickness (4- to 5-inches and 5- to 6-inches) and plotted separately. PCC thickness was not considered due to the small range of available thicknesses. Because of this small range of PCC thicknesses, it was assumed that PCC thickness would show little to no effect on the development of distresses between surveyed sections.

3.4.1.1 Alligator Cracking

Figure 3.19 presents the relationship between alligator cracking and cumulative traffic levels (CESAL) for different HMA overlay thicknesses. Trend

lines were applied to the two ranges of HMA thickness, in order to better estimate trends in the data. From Figure 3.19, it is noticed that alligator cracking tends to decrease as the HMA overlay thickness increases. For the same cumulative ESAL, test sections with a thicker HMA overlay reported lower alligator cracking percentages than those with thinner overlays. The trends approximately show that for a total CESAL of 6 million, which equates to about 6.5 years of service life, a 1-inch thicker overlay would reduce alligator cracking by 20 percent.

Figure 3.19: Alligator Cracking for sections with different overlay thicknesses in the Central Coast region

3.4.1.2 Transverse Cracking

The relationship between transverse cracking and cumulative ESAL (CESAL) is shown in Figure 3.20, along with their respective trend lines. These

figures show that overlay thickness (for the range of data analyzed) apparently has little effect on the appearance of transverse cracking. The trend for overlay thicknesses ranging from 4 to 5 inches is nearly identical to those ranging from 5 to 6 inches. It is generally believed that HMA thickness plays a role in retarding and minimizing transverse cracking. However, this was not reflected in the data collected for the Central Coast sections, mainly due to the narrow range of documented HMA thicknesses (4 to 6 inches). More data is needed to draw a definitive conclusion.

Figure 3.20: Transverse cracks for sections with different overlay thicknesses in the Central Coast region

3.4.1.3 Longitudinal Cracking

The trend lines in Figure 3.21 show that longitudinal cracks begin to appear at approximately the same time for the two overlay thickness ranges.

Overlays with a thickness ranging from 5 to 6 inches consistently reduced the percentage of longitudinal cracks over those with overlays thickness ranging from 4 to 5 inches. After 9-years of service, sections with an overlay thickness between 4 and 5 inches exhibit 5 percent more longitudinal cracking than sections with an overlay thickness between 5 and 6 inches (See Figure 3.21).

Figure 3.21: Longitudinal cracks for sections with different overlay thicknesses in the Central Coast region

3.4.1.4 International Roughness Index (IRI)

The effect of overlay thickness on IRI values for CS&O sections in the Central Coast region of California is presented in Figure 3.22. The trend lines suggest that sections with an overlay thickness 5 to 6 inches exhibited slightly lower IRI values than sections with an overlay thickness of 4 to 5 inches. For CESAL values up to 6 million, the trend lines show very little difference in IRI

between the two ranges of overlay thickness. Suggesting that overlay thickness only begins to effect IRI values at higher CESAL levels. As CESAL values increase, the difference in IRI values reported by the two overlay thicknesses increases as well. Therefore, for older pavement sections, thicker overlays will help reduce IRI values.

Figure 3.22: IRI values for sections with different overlay thicknesses in the Central Coast region

3.4.2 Northern California Region

Data for the Northern California region was analyzed using the same combination of the traffic and age as explanatory variables in evaluating the performance of CS&O sections. Although there is a much wider range of PCC thicknesses for the Northern California region, PCC thickness was still not

considered during this analysis to maintain consistency with the analyses for the CC region.

3.4.2.1 Alligator Cracking

The relationship between alligator cracking, CESAL, and different overlay thicknesses is presented in Figure 3.23. The figure shows that overlays over 5 inches thick exhibit much lower alligator cracking than sections with HMA overlays in the range of 3 to 5 inches. Figure 3.23 shows that thicker HMA overlays help to minimize the amount of alligator cracking. A similar trend was also observed for the Central Coast sections.

Figure 3.23: Alligator Cracking for sections with different overlay thicknesses in the Northern California Region

3.4.2.2 Transverse Cracking

The relationship between transverse cracks and CESAL for different overlay thicknesses in the Northern California region is presented in Figure 3.24. The trend lines show transverse cracks decrease as the overlay thickness increases. Also, compared to sections with an overlay thickness between 3 and 5 inches, sections with an overlay thickness between 5 and 7 inches take longer to begin developing transverse cracks,

Figure 3.24: Transverse cracks for sections with different overlay thicknesses in the Northern California region

3.4.2.3 Longitudinal Cracking

The relationship between longitudinal cracking, CESAL, and overlay thicknesses for Northern California CS&O sections is presented in Figure 3.25. The trend lines reveal that longitudinal cracks develop sooner in overlays

between 3 and 5 inches thick compared to overlays between 5 and 7 inches thick. In addition, once cracking has appeared, the propagation of these cracks increases at a faster rate for thinner overlay thicknesses, suggesting that thicker overlays retard the appearance of longitudinal cracks.

Figure 3.25: Longitudinal cracks for sections with different overlay thicknesses in the Northern California region

3.4.2.4 International Roughness Index (IRI)

The effect of overlay thickness on IRI values for CS&O sections in the Northern California region is presented in Figure 3.26. The trend lines within Figure 3.26 suggest that surveyed sections with overlay thickness in the range of 5 to 7 inches exhibited lower initial IRI values than sections with overlay thickness between 3 and 5 inches. As cumulative traffic levels increase, the IRI values for the sections with overlays between 5 and 7 inches increase until they

are equal to, or greater than, sections with overlays between 3 and 5 inches. This could be attributed to the cumulative densification of the thicker HMA overlays as cumulative traffic loading increases.

Figure 3.26: IRI values for sections with different overlay thicknesses in the Northern California region

3.4.3 Region Comparison

In addition to analyzing trends and correlations for the individual regions, performance comparisons between the two regions were investigated. For this portion of the analysis, region data was plotted and compared. Possible factors that may account for differences in performance included: weather, temperature, traffic characteristics, material type, subgrade type, and others.

3.4.3.1 Alligator Cracking

The relationship between alligator cracking and CESAL for the Central Coast and Northern California regions is presented in Figure 3.27. The trends reveal that in general, for the same CESAL level, alligator cracks develop more quickly in the Central Coast region. It must be noted that overall CESAL levels are much lower in the Central Coast region; therefore, the earlier development of alligator cracking must be caused by another, unaccounted for, explanatory variable. Generally, alligator cracking is thought to occur when transverse and longitudinal cracks intersect and propagate. This process is often triggered and hastened by increased traffic loads. However, between the two regions, Northern California has much higher traffic levels. Therefore, increased alligator cracking levels in the Central Coast region must not be attributed to traffic loads, but to some other factor.

Possible causes of the increase in alligator cracking is differing construction practices between the two regions, or different materials and pavement thicknesses used in both regions. Based off of the data available it is hard to determine which one of these is a likely cause. On average the PCC, thicknesses are thinner in the Central Coast region than in the Northern California region. Although PCC thickness was not analyzed in the previous sections, thinner PCC thicknesses in the Central Coast region may trigger increased alligator cracking. In addition, the different types of subgrade in either region may be partially responsible for differing trends shown in Figure 3.27.

Figure 3.27: Alligator cracking for the Central Coast and Northern California regions

3.4.3.2 Transverse Cracking

The percentage of transverse cracks in the Central Coast and Northern California region are plotted against CESAL in Figure 3.28. The trends show very little difference in the emergence of transverse cracking between the two regions. Both regions contain many sections that report zero transverse cracks, even for a CESAL as high as 10 million. It is difficult to understand why some of these sections exhibit high amounts of transverse cracks at lower CESAL and some do not. Transverse cracks in CS&O sections are typically created by thermal-induced horizontal strains and vertical traffic-induced strains in the pavement. However, Figure 3.28 does not show a definitive difference in performance between the two regions.

Figure 3.28: Transverse cracking for the Central Coast and Northern California regions

Since the relationship between transverse cracking and CESAL did not provide any discernable relationships, the emergence of transverse cracking was investigated as a function of age. This relationship is presented in Figure 3.29, clearly showing a definitive relationship for both regions. The trends show that CS&O sections within the Northern California region develop transverse cracks at a faster rate than similar sections in the Central Coast region. As mentioned before, transverse cracking is typically caused by thermal and traffic induced strains. The Northern California region has significantly higher traffic levels and a more varied climate compared to the Central Coast. It is believed that these inherent characteristics of the Northern California region account for the differences in evolution of transverse between the two regions.

Figure 3.29: Transverse cracking for Central Coast and Northern California regions (vs. Age)

3.4.3.3 Longitudinal Cracking

The relationship between longitudinal cracking percentage and CESAL for Central Coast and Northern California regions is presented in Figure 3.30. The trends show that for any CESAL value the Central Coast region exhibits a higher longitudinal cracking percentage than the Northern California region. Longitudinal cracking appears to begin at close to the same CESAL for both regions; however, as the sections experience more wear and tear, the longitudinal distresses in the Central Coast sections propagate at a faster rate. These trends are not in agreement with those previously established for transverse cracking. It is believed that longitudinal and transverse cracking should behave similarly. It is possible that the cause of this discrepancy is something that is impossible to quantify or rectify, given the data that was collected.

The longitudinal cracking relationships were also investigated as a function of age, as shown in Figure 3.31. Unlike transverse cracking, the change in explanatory variable from CESAL to age does not produce a clearer relationship between the two regions. It also does not provide a relationship that is any more similar to transverse cracking.

Despite this discrepancy, it is still expected that trends within transverse and longitudinal data behave similarly. Both types of distresses are commonly found as reflective cracking induced by thermal and traffic loading distresses. The cracking patterned followed during the breaking of the underlain PCC layer will effect this type of cracking as well. If the breaking of the pavement is performed inconsistently, with more force being applied in some directions, non-uniform cracking will result. Additional observations are needed in order to better understand why the transverse and longitudinal trends do not agree.

Figure 3.30: Longitudinal cracking for the Central Coast and Northern California regions

Figure 3.31 : Longitudinal cracking for the Central Coast and Northern California regions (vs. Age)

3.4.3.4 International Roughness Index (IRI)

Pavement roughness (IRI) has several different possible causes. These include: traffic loading, geographical/environmental effects, differing construction practices, varying construction materials, and built-in construction irregularities (Rahim and Fiegel 2008). Even a new pavement may exhibit abnormally high initial IRI values due to built-in construction irregularities or flaws in the pavement surface. All of these factors were kept in mind when comparing IRI for sections in the Central Coast and Northern California regions. The trends in Figure 3.32 reveal that initially the IRI for both regions is similar. However, as the sections age and are exposed to more cumulative traffic loading and weathering, the two regions begin to behave differently. After a CESAL of approximately 5 million, IRI in the Central Coast region begins to increase more rapidly than the Northern California region. This increase is possibly due to thinner overlays and different construction practices. The roughness of a pavement may be effected by factors other than age and traffic loading. Due to the more extreme conditions such as snow and ice found in the Northern California regions, it is possible that construction practices call for a more resilient HMA mix design with better materials. In addition, due to the roadway hazards created during these harsh conditions, roadway maintenance crews most likely repair and maintain these highways more often than those in the Central Coast region.

Figure 3.32: IRI for the Central Coast and Northern California regions

CHAPTER 4

PERFORMANCE MODELS

4.1 Introduction

Pavement Performance and the development of distresses are dependent on many different variables. As a result, models that predict the growth of pavement distresses are almost always empirical or semi-empirical in nature. The most common procedure for modeling pavement performance is to use age or a combination of age and traffic coupled with pavement layer thickness. Regression performance models are common.

In this study, regression models were developed for four different dependent (response) variables: alligator cracking, transverse cracking, longitudinal cracking, and IRI. The data used for these models were presented in Chapter 3.

4.2 Regression Modeling

The distress prediction models were developed using regression techniques. Initially, the scatter plots presented in Chapter 3 were used to show the relationships between response (distress) variables and possible explanatory (independent) variables. These scatter plots were examined to reveal likely relationships between the response and explanatory variables. As mentioned in Chapter 3, these scatter plots were evaluated for data errors and erroneous points were removed. In preliminary trials, many different model forms were

used. In the end it was found that non-linear regression provided the most accurate and reasonable models.

Regression models were created using the statistical software package SPSS (IBM, 2011). The non-linear regression tool within the software was used to relate a single response (distress) variable to multiple explanatory variables (age, traffic, HMA, PCC) in a variety of combinations. Many models were developed; the parameters used to determine the best models were R^2 -Value and Root Mean Squared Error (RMSE). R^2 -Value, also known as the coefficient of determination measures the difference between actual and predicted data points. R^2 -values range from 0 to 1; where a 0 indicates there is no correlation between the actual and predicted data points, and a 1 indicates that all the predicted points match the data used to develop the model. The R^2 -Value was used to determine which models predicted distress values closest to the actual distress data presented in Chapter 3. RMSE quantifies the absolute value of the average difference between the actual and predicted distress values. A smaller the RMSE indicates a better prediction model.

The following constraints were used when developing the regression models:

- Alligator, transverse, and longitudinal cracks must begin with zero values for pavement sections with zero age. However, for IRI there will be a minimum constant for pavements with zero age.

- Prediction models must be flexible and able to accommodate the representation of small distress values for younger pavements (<5 years) and larger distress values for older pavements.
- Prediction models must not return implausible values or scenarios such as: negative distresses, distresses which decrease over time or distresses that contradict trends agreed upon within the engineering community.

Expected behavior was taken into consideration during the development of these models. Expected behavior can be defined as any reasonable physical reaction of the pavement to a certain explanatory variable. For example, pavement distresses should increase with age and traffic loading. Also, distress levels should decrease as overlay thickness increases, better construction materials are used, or the presence of more ideal weather conditions. However, models were not automatically rejected if they did not exhibit expected behavior. For these cases, the models were often reworked, and evaluated against each other. A common technique used was to isolate one variable during the analysis, to determine if there is a statistical anomaly causing the unexpected trend.

The general form used to develop the models was basic polynomial equation: "Distress = $a_0 + a_1x^{b_1}x^{b_2}....$ " where a_1 and b_1 are coefficients and a_0 is the initial value of distress before the pavement was put into service (i.e. $a_0=0$ for cracking distresses). The goal of this empirical modeling was to choose a model that produced the lowest variation (lowest RMSE) and highest accuracy (highest R^2).

4.3 Data Models

The empirical models developed for this study are presented in the following sections. These empirical models predict distress values for the Central Coast and Northern California regions. Models were developed for the following four distresses: alligator cracking, transverse cracking, longitudinal cracking, and IRI. Presented with each model are R^2 , RMSE, and the number of data points used to develop it.

The performance models for alligator cracking, transverse cracking, longitudinal cracking, and IRI in the Central Coast and Northern California regions are presented in Table 4.1, 4.2, 4.3, and 4.4, respectively. Variable definitions, applicable units, and data ranges are found in Table 3.3 and 3.4, respectively.

Table 4.1: Alligator cracking performance models

Region	Alligator Cracking Models					
Central Coast	$ALLG = A1(AGE \times MESAL)^{A2} \left(\frac{HMA}{PCC}\right)^{A3}$					
	A1 = 0.349		A2 = 1.308		A3 = -3.465	
	R ² = 0.68		RMSE = 13.45		N = 59	
	95% Confidence Interval					
	A1		A2		A3	
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound
	-0.47	1.169	0.75	1.866	-7.953	1.024
Northern California	$ALLG = A1(AGE \times MESAL)^{A2} \left(\frac{HMA}{PCC}\right)^{A3}$					
	A1 = 0.233		A2 = 1.77		A3 = -0.729	
	R ² = 0.78		RMSE = 15.83		N =71	
	95% Confidence Interval					
	A1		A2		A3	
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound
	-0.089	0.555	1.369	2.172	-1.413	-0.045

Notes:

MESAL = Annual Equivalent Single Axle Load x 10⁶

R² = Coefficient of determination

RMSE = Root Mean Squared Error

N = Number of data points used to develop the models

ALLG = Percentage of Alligator Cracking

AGE = Years since CS&O rehabilitation

HMA = Hot Mix Asphalt Thickness (in.)

PCC = Portland Cement Concrete Thickness (in.)

Table 4.2: Transverse cracking performance models

Region	Transverse Cracking Models							
Central Coast	$TRANS = A1(MCESAL)^{A2} \left(\frac{HMA}{PCC} \right)^{A3}$							
	A1 = 0.129		A2 = 1.376		A3 = -0.262			
	R ² = 0.53		RMSE = 0.86		N = 52			
	95% Confidence Interval							
	A1		A2		A3			
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound		
	-0.036	0.294	0.964	1.788	-2.586	2.062		
Northern California	$TRANS = A1(AGE)^{A2}(MESAL)^{A3} \left(\frac{HMA}{PCC} \right)^{A4}$							
	A1 = 0.03		A2 = 1.251		A3 = 1.799		A4 = -1.711	
	R ² = 0.78		RMSE = 1.12			N = 82		
	95% Confidence Interval							
	A1		A2		A3		A4	
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound
	-0.004	0.064	0.856	1.646	0.64	2.959	-2.573	-0.849

Notes:

MCESAL = Cumulative Annual Equivalent Single Axle Load x 10⁶

MESAL = Annual Equivalent Single Axle Load x 10⁶

R² = Coefficient of determination

RMSE = Root Mean Squared Error

N = Number of data points used to develop the models

TRANS = Percentage of Transverse Cracking

AGE = Years since CS&O rehabilitation

HMA = Hot Mix Asphalt Thickness (in.)

PCC = Portland Cement Concrete Thickness (in.)

Table 4.3: Longitudinal cracking performance models

Region	Longitudinal Cracking Models							
Central Coast	$LONG = A1(MCESAL)^{A2} \left(\frac{HMA}{PCC} \right)^{A3}$							
	A1 = 0.063		A2 = 1.705		A3 = -2.633			
	R ² = 0.74		RMSE = 2.73		N = 85			
	95% Confidence Interval							
	A1		A2		A3			
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound		
	-0.007	0.133	1.304	2.107	-3.845	-1.421		
Northern California	$LONG = A1(AGE)^{A2}(MESAL)^{A3} \left(\frac{HMA}{PCC} \right)^{A4}$							
	A1 = 0.034		A2 = 1.701		A3 = 2.733		A4 = -0.31	
	R ² = 0.68		RMSE = 2.81		N = 81			
	95% Confidence Interval							
	A1		A2		A3		A4	
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound
	-0.015	0.082	1.091	2.311	1.205	4.261	-1.369	0.749

Notes:

MCESAL = Cumulative Annual Equivalent Single Axle Load x 10⁶

MESAL = Annual Equivalent Single Axle Load x 10⁶

R² = Coefficient of determination

RMSE = Root Mean Squared Error

N = Number of data points used to develop the models

LONG = Percentage of Longitudinal Cracking

AGE = Years since CS&O rehabilitation

HMA = Hot Mix Asphalt Thickness (in.)

PCC = Portland Cement Concrete Thickness (in.)

Table 4.4: IRI performance models

Region	IRI Cracking Models							
Central Coast	$IRI = 59.378 + 0.023(AGE)^{3.431}(MESAL)^{0.068}$							
	A1 = 59.378		A2 = 0.023		A3 = 3.341		A4 = 0.068	
	R ² = 0.62		RMSE = 20.99			N = 83		
	95% Confidence Interval							
	A1		A2		A3		A4	
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound
	52.23	66.53	-0.046	0.93	2.186	4.677	-0.989	1.125
Northern California	$IRI = 54.544 + (AGE)^{0.868}(MESAL)^{1.448}\left(\frac{HMA}{PCC}\right)^{-1.61}$							
	A1 = 54.544		A2 = 0.868		A3 = 1.448		A4 = -1.61	
	R ² = 0.63		RMSE = 13.52			N = 103		
	95% Confidence Interval							
	A1		A2		A3		A4	
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound
	50.09	58.00	0.627	1.108	0.894	2.003	-2.131	-1.089

Notes:

MESAL = Annual Equivalent Single Axle Load x 10⁶

R² = Coefficient of determination

RMSE = Root Mean Squared Error

N = Number of data points used to develop the models

IRI = International Roughness Index (in./mile)

AGE = Years since CS&O rehabilitation

HMA = Hot Mix Asphalt Thickness (in.)

PCC = Portland Cement Concrete Thickness (in.)

4.4 Model Fit

The models were checked for fit by comparing actual and predicted distress data values. Actual and predicted values were compared by plotting them on the same graph, with the predicted values plotted as the independent variable and the actual values as the dependent variable. A perfect fit would be represented as a straight line of points along the line of equality. If there is variation within the model, points will be distributed above and below the line.

To check for multicollinearity, residuals were plotted versus predicted values. Multicollinearity exists when two or more variables within a multiple regression model are highly correlated. This may cause coefficient estimates such as R^2 and the RMSE to change unpredictably in response to small changes in the model or the data. Trends within these plots indicate the existence of multicollinearity relationships. Another problem occurs when residuals form a "funnel" shaped pattern, where residual values grow farther or closer to zero as the predicted values increase (Ott 2010).

4.4.1 Alligator Cracking

The predicted versus actual alligator cracking plots for the Central Coast and Northern California region models are presented in Figure 4.1 and 4.2, respectively. Also shown on these figures are lines of equality. The plots indicate that the models are able to predict alligator cracking.

Figure 4.1: Actual versus predicted graph for alligator cracking in the Central Coast region

Figure 4.2: Actual versus predicted graph for alligator cracking in the Northern California region

To check for multicollinearity within the alligator cracking models, the residuals for these models were plotted against their predicted values. These plots for the Central Coast and Northern California region are presented in Figure 4.3 and 4.4, respectively. Both of these figures show no specific trend, which indicates that of multicollinearity does not exists in these models.

Figure 4.3: Residual vs. Predicted plot for alligator cracking in the Central Coast region

Figure 4.4: Residual vs. Predicted plot for alligator cracking in the Northern California region

4.4.2 Transverse Cracking

The predicted versus actual transverse cracking plots for the Central Coast and Northern California region models are presented in Figures 4.5 and 4.6, respectively. Figure 4.6 shows the model's inability to predict a zero value for transverse cracking as age increases. The problems may be due the majority of the surveyed sections reporting 0 or 5 values for transverse cracking. If additional data were collected that reported transverse cracking values of either 1, 2, 3, or 4, the goodness of fit plot would be more robust.

Figure 4.5: Actual versus predicted graph for transverse cracking in the Central Coast region

Figure 4.6: Actual versus predicted graph for transverse cracking in the Northern California region

To check for multicollinearity amongst the variables within the transverse cracking models, residuals were plotted against the predicted values. These plots are presented in Figures 4.7 and 4.8. The transverse cracking model for the Central Coast region shows no signs of the existence of multicollinearity amongst the variables (see Figure 4.7). The model for the Northern California region shows some recognizable trends (see Figure 4.8). These trends are due to the ordinal nature of the data. Most of transverse cracking observations fall within a narrow range. This lack of dispersion in the data is responsible for these linear trends.

Figure 4.7: Residual vs. Predicted plot for transverse cracking in the Central Coast region

Figure 4.8: Residual vs. Predicted plot for transverse cracking in the Northern California region

4.4.3 Longitudinal Cracking

The predicted versus actual longitudinal cracking plots for the Central Coast and Northern California region models are presented in Figures 4.9 and 4.10, respectively. These plots indicate that the model predictability is consistent throughout all levels.

Figure 4.9: Actual versus predicted graph for longitudinal cracking in the Central Coast region

Figure 4.10: Actual versus predicted graph for longitudinal cracking in the Northern California region

To check for multicollinearity amongst the variables within the longitudinal cracking models, residuals were plotted against the predicted values. These plots

are presented in Figures 4.11 and 4.12. Both figures show trends, alluding to the presence of possible multicollinearity within the models for the Central Coast and Northern California regions. As mentioned previously, it is hard to conclude what variable correlations cause the trends present within Figures 4.11 and 4.12. Further analysis of the data and models must be done in order to remediate this problem. Given the nature of that data used in this study, these issues will not affect the outcome of final conclusions regarding the predicting of longitudinal cracking distresses within the Central Coast and Northern California regions.

Figure 4.11: Residual vs. Predicted plot for longitudinal cracking in the Central Coast region

Figure 4.12: Residual vs. Predicted plot for longitudinal cracking in the Northern California region

4.4.4 International Roughness Index (IRI)

The predicted versus actual IRI plots for the Central Coast and Northern California region models are presented in Figures 4.13 and 4.14, respectively. Due to the increased sample size and the larger inference space of IRI distress values, the data in these plots is dense and varied. Clustering of the data appears in both figures due to the variation IRI responses throughout all values (See Figures 4.13 and 4.14). Figure 4.14 indicates better prediction of IRI values within the Northern California region. This model accurately predicts low and high IRI values for the entire range of values. The Central Coast region model presented in Figure 4.13 shows that it predicts IRI accurately for low range values; however, as the values increase, it begins to lose its predictive capability.

Figure 4.13: Actual versus predicted graph for IRI in the Central Coast region

Figure 4.14: Actual versus predicted graph for IRI in the Northern California region

In order to check for multicollinearity amongst the variables within the IRI models, residuals were plotted against the predicted values. These plots are presented in Figures 4.15 and 4.16. These plots show no signs of multicollinearity within the IRI models for Central Coast and Northern California regions

Figure 4.15: Residual vs. Predicted plot for IRI in the Central Coast region

Figure 4.16: Residual vs. Predicted plot for IRI in the Northern California region

4.5 Sensitivity Analysis

Sensitivity analyses were completed for the prediction models to determine which variables have the greatest and least effect on the distresses in CS&O sections within the Central Coast and Northern California regions. The tests were performed by changing each of the explanatory variables individually, by plus or minus one standard deviation of the average value of that variable, keeping the other explanatory variables constant. Analyzing the differences between these predicted distress values reveals how sensitive each model is to a particular explanatory variable.

The bar charts presented in this section are a visual representation of each explanatory variables sensitivity analysis. The vertical axis quantifies the distress being predicted by particular model. The left most column represents the predicted distress value when the explanatory variable of interest is decreased

by one standard deviation from its average. The middle column represents the predicted distress value when the average value for the explanatory variable is used in the model. The right most column represents the predicted distress value when the explanatory variable is increased by one standard deviation from its average.

4.5.1 Central Coast Region

4.5.1.1 Alligator Cracking

Sensitivity analyses were conducted for the following explanatory variables included in the Central Coast alligator cracking prediction model: age, MESAL, and the HMA/PCC ratio.

As shown in Figure 4.17, when age was changed by one standard deviation above and below the average value, it increased and decreased the percentage of alligator cracking by almost two- and three-fold, respectively. Therefore, age is considered a important predictor for the appearance of alligator cracking in the Central Coast region.

Figure 4.17: Age sensitivity analysis for alligator cracks in the Central Coast model

MESAL is another important predictor for the Central Coast alligator cracking model. The variation in alligator cracks caused by changing the average MESAL value by one standard deviation is approximately 40 percent (See Figure 4.18).

Changing the HMA/PCC ratio by one standard deviation above and below the average decreases, or increases the appearance of alligator cracking by roughly 42 and 27 percent, respectively (See Figure 4.19). Increasing or decreasing this ratio has the opposite effect on the predicted amount of alligator cracking when compared to MESAL and age. Increasing HMA thickness for the same PCC thickness will increase the HMA/PCC ratio, which increases the resistance to fatigue-related cracking (known as alligator cracking).

Figure 4.18: MESAL sensitivity analysis for alligator cracks in the Central Coast model

Figure 4.19: HMA/PCC thickness ratio sensitivity analysis for alligator cracks in the Central Coast model

4.5.1.2 Transverse Cracking

Sensitivity analyses were conducted for the following explanatory variables included in the Central Coast transverse cracking prediction model: MCEAL, and the HMA/PCC ratio.

Figure 4.20 shows that varying MCEAL by one standard deviation above and below the average value increases, or decreases, the predicted transverse cracking percentage by 131 and 92 percent, respectively.

Figure 4.21 shows that the HMA/PCC ratio has a smaller effect on the predicted amount of transverse cracking in the Central Coast region as compared to MCEAL. Changing the average ratio by one standard deviation above and below decreases and increases the predicted transverse cracking by 2 to 3 percent, respectively. the HMA/PCC ratio does not have as much of an effect on transverse cracking as that for MCEAL. This could be attributed to the small range of HMA/PCC data used to develop the model.

Figure 4.20: MCEsAL sensitivity analysis for transverse cracks in the Central Coast model

Figure 4.21: HMA/PCC ratio sensitivity analysis for transverse cracks in the Central Coast model

4.5.1.3 Longitudinal Cracking

Sensitivity analyses were conducted for the following explanatory variables included in the Central Coast longitudinal cracking prediction model: MCESAL, and the HMA/PCC ratio.

Similar to transverse cracking in the Central Coast region, MCESAL proved to be the most significant variable for the longitudinal model as well (See Figure 4.22). A variation in the MCESAL by one standard deviation above and below the average, increases and decreases the predicted percentage of longitudinal cracking by 138 and 84 percent, respectively.

Figure 4.22: MCESAL sensitivity analysis for longitudinal cracks in the Central Coast model

Figure 4.23 shows that the ratio of HMA/PCC is a significant explanatory variable in the predictive longitudinal cracking model for the Central Coast Region. Changing this average ratio by plus or minus one standard deviation,

decreases and increases, longitudinal cracking by 21 and 28 percent, respectively. This is not as significant of an effect as MCEAL has on the predicted longitudinal cracking model in the Central Coast region. This trend is in agreement with that discussed for the transverse cracking model.

Figure 4.23: HMA/PCC ratio sensitivity analysis for longitudinal cracks in the Central Coast model

4.5.1.4 International Roughness Index (IRI)

Sensitivity analyses were conducted for the following explanatory variables included in the Central Coast transverse cracking prediction model: age and MESAL.

The analysis resulted in that age is the most significant variable for predicting the IRI of an HMA overlay in the Central Coast region. Predicted IRI increases by 37 percent when average age is increased by one standard

variation (See Figure 4.24). Which equates to an IRI increase of approximately 22 inches per mile.

MESAL has a small effect on IRI values for the Central Coast model (See Figure 4.25). Increasing or decreasing average MESAL by one standard deviation only effects predicted IRI by 0.15 and 0.20 percent, respectively. This might be due to MESAL values having a relatively low standard deviation compared to the age values included in the model. It is also worth noting that as traffic increases IRI will increase as well.

Figure 4.24: Age sensitivity analysis for IRI in the Central Coast model

Figure 4.25: MESAL sensitivity analysis for IRI in the Central Coast model

4.5.2 Northern California Region

4.5.2.1 Alligator Cracking

Sensitivity analyses were conducted for the following explanatory variables included in the Northern California alligator cracking prediction model: age, MESAL, and the HMA/PCC ratio.

Out of all the explanatory variables included in the alligator cracking prediction model for the Northern California, age has the greatest effect (See Figure 4.26). Changing a pavements age by one standard deviation above and below the average value increases or decreases predicted alligator cracking by 148 and 86 percent, respectively. This equates to a threefold increase in alligator cracking from an average pavement in the Northern California region.

Figure 4.26: Age sensitivity analysis for alligator cracking in the Northern California model

MESAL is also an important predictor of alligator cracking in the Northern California region. The variation in predicted alligator cracking due to a increase in one standard deviation of average MESAL is 36 percent (See Figure 4.27). A decrease in one standard deviation of average MESAL results in 31 percent change in the models predicted cracking values. Showing that MESAL has a smaller effect than age on the model results.

Adjusting the average HMA/PCC ratio by one standard deviation decreases and increases alligator cracking by 24 and 4 percent, respectively. This is a less of an effect than either age or MESAL has on the model. However, the effect that HMA/PCC ratio has on predicted alligator cracking compared to age and MESAL, matches that mentioned in the Central Coast region.

Figure 4.27: MESAL sensitivity analysis for alligator cracking in the Northern California model

Figure 4.28: HMA/PCC ratio sensitivity analysis for alligator cracks in the Northern California model

4.5.2.2 Transverse Cracking

Sensitivity analyses were conducted for the following explanatory variables included in the Northern California transverse cracking prediction model: age, MESAL, and the HMA/PCC ratio.

The analysis resulted in that age has the greatest effect of all the explanatory variables for predicting transverse cracking in the Northern California region (See Figure 4.29). Changing average age by one standard deviation increases or decreases the predicted transverse cracking by 80 and 68 percent, respectively. This is important considering that range accounts for 46 percent of all the possible predicted transverse cracking values.

Figure 4.29: Age sensitivity analysis for transverse cracking in the Northern California model

Figure 4.30 shows the predicted amount of transverse cracking is increased by 35 percent and decreased by 30 percent when MESAL is varied by

one standard deviation from the average value. That range accounts for 20 percent of all possible predicted transverse cracking, making it a significant amount.

Compared to MESAL, the HMA/PCC ratio has a larger effect on predicted transverse cracking for the Northern California region predictive transverse cracking model. Figure 4.31 shows that an increase in one standard deviation of the average HMA/PCC value will decrease predicted transverse cracking by 31 percent. A decrease in one standard deviation of the average HMA/PCC ratio will increase the predicted amount of transverse cracking by 61 percent. This range accounts for nearly 30 percent of all possible predicted transverse cracking values. This trend is in agreement with that discussed for transverse cracking models in both regions.

Figure 4.30: MESAL sensitivity analysis for transverse cracking in the Northern California model

Figure 4.31: HMA/PCC ratio sensitivity analysis for transverse cracks in the Northern California model

4.5.2.3 Longitudinal Cracking

Sensitivity analyses were conducted for the following explanatory variables included in the Northern California longitudinal cracking prediction model: age, MESAL, and the HMA/PCC ratio.

Of the explanatory variables included in the Northern California longitudinal prediction model age has the greatest effect (See Figure 4.32). Increasing a pavements average age by a single standard deviation increases, or decreases the appearance of longitudinal cracking by 120 and 78 percent, respectively.

For the predictive longitudinal cracking model in the Northern California region, varying the average MESAL value by a single standard deviation results

in a 41 percent decrease and 55 percent increase in predicted longitudinal cracking (See Figure 4.33).

Figure 4.32: Age sensitivity analysis for longitudinal cracking in the Northern California model

Figure 4.33: MESAL sensitivity analysis for longitudinal cracking in the Northern California model

The predictive capability of the HMA/PCC ratio is low for the Northern California longitudinal cracking model. Figure 4.34 shows that for a change of

one standard deviation in the average HMA/PCC ratio value, the predicted longitudinal cracking may vary up to 9 percent. Equating to 0.5 percent variation in actual predicted longitudinal cracking percentage (See Figure 4.34). The trends exhibited in the figure agree with those discussed for transverse and longitudinal cracking in both regions.

Figure 4.34: HMA/PCC ratio sensitivity analysis for longitudinal cracks in the Northern California model

4.5.2.4 International Roughness Index

Sensitivity analyses were conducted for the following explanatory variables included in the Northern California IRI prediction model: age, MESAL, and the HMA/PCC ratio.

See Figure 4.35 shows that increasing or decreasing the average age value by one standard deviation results in a 12 and 13 percent change in predicted IRI values, respectively. This equates to an increase or decrease of

roughly 10 inches per mile of IRI in the HMA surface. The analysis also indicates that IRI increases as pavements age.

Figure 4.35: Age sensitivity analysis for IRI in the Northern California model

Changing the average MESAL value of 1.57 by one standard deviation causes a 8 percent change in the predicted IRI values (See Figure 4.36). This is nearly a 6 to 7 inch per mile change in IRI.

The HMA/PCC ratio has a greater effect on predicted IRI than age and MESAL for the Northern California region prediction model. Changing the HMA/PCC ratio by one standard deviation results in a 17 percent increase or decrease in predicted IRI values (See Figure 4.37). This equates to a 12 inch per mile range of predicted IRI values. In addition, as the HMA/PCC ratio decreases, IRI will increase. This relationship is in agreement with the IRI prediction model for the Central Coast region.

Figure 4.36: MESAL sensitivity analysis for IRI in the Northern California model

Figure 4.37: HMA/PCC ratio sensitivity analysis for IRI in the Northern California model

4.5.3 Summary of Sensitivity Analysis

Table 4.5 and 4.6 present summaries of predicted distresses calculated during the sensitivity analysis of the Central Coast and Northern California regions, respectively.

Table 4.5: Central Coast region sensitivity analysis summary

Distress	Average Predicted Distress Value	Explanatory Variable Analyzed	+ STD	Change in Predicted Distress	- STD	Change in Predicted Distress
Alligator Cracking, %	11.04	Age	21.17	92%	2.86	-74%
	11.04	MESAL	15.82	43%	6.71	-39%
	11.01	HMA/PCC	8.02	-27%	15.6	42%
Transverse Cracking, %	0.061	MCESAL	0.141	131%	0.005	-92%
	0.061	HMA/PCC	0.06	-2%	0.063	3%
Longitudinal Cracking, %	3.07	MCESAL	7.31	138%	0.48	-84%
	3.07	HMA/PCC	2.44	-21%	3.94	28%
IRI, (in/mi)	64.89	Age	88.96	37%	59.56	-8%
	64.89	MESAL	64.99	0.15%	64.76	-0.20%

STD = One Standard Deviation

Table 4.6: Northern California region sensitivity analysis summary

Distress	Average Predicted Distress Value	Explanatory Variable Analyzed	+ STD	Change in Predicted Distress	- STD	Change in Predicted Distress
Alligator Cracking, %	13.77	Age	21.17	92%	2.86	-74%
	13.77	MESAL	15.82	43%	6.71	-39%
	13.6	HMA/PCC	8.02	-27%	15.6	42%
Transverse Cracking, %	1.55	Age	2.79	80%	0.49	-68%
	1.55	MESAL	2.09	35%	1.08	-30%
	1.51	HMA/PCC	1.04	-31%	2.43	61%
Longitudinal Cracking, %	2.68	Age	5.89	120%	0.59	-78%
	2.68	MESAL	4.16	55%	1.58	-41%
	2.68	HMA/PCC	2.48	-7%	2.91	9%
IRI, (in/mi)	78.42	Age	87.77	12%	68.47	-13%
	78.42	MESAL	84.83	8%	72.51	-8%
	78.06	HMA/PCC	70.83	-9%	92.42	18%

STD = One Standard Deviation

4.6 Summary

The purpose of this chapter was to develop and analyze regression models to predict pavement distresses in the Central Coast and Northern California regions. Findings indicate that all models for both regions are successfully able predict distresses. Issues that arose with model fit and residual

analyses are due to inherent characteristics of the data that cannot be quantified or accounted for given the collected data. In general, the models for the Northern California region predict higher levels of distresses than models for the Central Coast region. The sensitivity analysis suggests that age has the strongest effect on the models out of all the explanatory variables. Annual MESAL also has a noticeable effect on pavement distresses. When appropriate, the cumulative traffic load (CESAL) of a pavement is also an important predictor of pavement distress. The HMA/PCC ratio has the least effect of any of the explanatory variables included in the models. With exception of models used to predict IRI and transverse cracking values in the Northern California Region.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Overview

The purpose of this study was to evaluate the performance of the crack, seal, and overlay rehabilitation technique and its use in California. Data were collected using the California Pavement Condition Reporting (PCR) software for two geographical regions in California, namely the "Central Coast" and "Northern California". Eight sections were located in the Central Coast region of California, and nine were located in the Northern California region. After the data were collected, visual surveys and core extractions were completed. Data were then cleaned of outliers and erroneous points. Analyses were completed to develop performance models.

Performance models were differentiated based upon distress type and region. Prediction regression equations were developed for possible incorporation with the Pavement Management System (PMS). The developed prediction models were analyzed to determine how well predicted data fit existing data collected as part of the study. The sensitivity of these models to each explanatory variable was also investigated. Conclusions from the current study and recommendations for further research follow:

5.2 Conclusions

1. The preciseness of the distress prediction models included within this study allow for comparison between CS&O sections in the Central Coast and Northern California regions.
2. The following pavement distresses were analyzed:
 - a. Alligator cracking percentages are higher in the Central Coast region.

The higher levels of alligator cracking in the Central Coast region may be attributed to thinner PCC pavement thicknesses, different subgrade types, and differing construction practices between the two regions.
 - b. Transverse cracking numbers are higher in the Northern California region. Thermal and traffic induced strains within the pavement are higher than those found in the Central Coast region.
 - c. Longitudinal cracking percentages are higher in the Central Coast region. This finding is not intuitive, transverse and longitudinal cracking are types of reflective cracking that have been found to behave similarly. Further research is required in this area. Possible causes of increased longitudinal cracking could be different types of breaking patterns used during the CS&O process, construction materials, or variation in data collection.
 - d. IRI levels are higher in the Central Coast region. Possible causes include thinner HMA overlays and PCC pavements in the Central Coast region, different construction materials in the Northern California

region, and increased roadway maintenance in the Northern California region.

3. Alligator, transverse, and longitudinal cracking prediction models for CS&O sections in the Central Coast and Northern California regions are most sensitive to age and the cumulative traffic stresses endured over a pavements service life. IRI prediction models for the Central Coast region are also most sensitive to age. However, in the Northern California region, IRI prediction models are most sensitive to the HMA/PCC thickness ratio. These differences are caused by variations within the collected data.
4. On average, the pavement distress prediction models predict higher levels of alligator cracking, transverse cracking, and IRI for the Northern California region. The models predict higher levels of longitudinal cracking for the Central Coast region. These predicted distress levels are consistent with the data collected from both regions.

5.3 Recommendations

1. Predicted versus fitted graphs indicate that there are some correlations within the collected transverse and longitudinal cracking data. Most of this correlation is caused by the ordinal manner in which the data is recorded and quantified. Further investigation into the effects that pavement survey techniques and methods may have on predictive model analysis is needed.
2. Longitudinal cracking percentages indicate that thermal strains do not increase the appearance of longitudinal cracking. Further research is recommended to investigate the possible causes of longitudinal cracking.

3. Research on materials, construction methods, and HMA mix designs used in both regions should be investigated to determine their effect on the alligator cracking, transverse cracking, longitudinal cracking, and IRI within these regions.
4. It is recommended that further CS&O sections within each region be surveyed and cored to produce more accurate distress prediction models.
5. Thorough evaluation of PCC pavements prior to implementation of the CS&O rehabilitation technique is recommended. Stringent evaluation criteria must be adopted to insure that the CS&O technique is used only in optimal circumstances, preventing low pavement performance and unnecessary costs.

REFERENCES

1. California Department of Transportation. Highway Design Manual (HDM). Sacramento: Caltrans, 2009.
2. California Department of Transportation Division of Maintenance. State of the Pavement Report. Yearly Report. Sacramento: Caltrans, 2007.
3. California Department of Transportation. Jointed Plain Concrete Pavement (JPCP) Preservation and Rehabilitation . Design Guide. Sacramento: Caltrans, 2008.
4. —. Traffic Data Branch. 2011. 4 April 2011
<<http://www.dot.ca.gov/hq/traffops/saferesr/trafdata/index.htm>>.
5. Carpenter, Samuel H. and Michael I. Darter. "Field Performance of Crack and Seat Projects." Transportation Research Record 1215 (1989): 219-231.
6. Choubane, Bouzid and Abdenour Nazef. "Performance Evaluation of Crack-and-Seat Rehabilitation Strategy: A 10-Year Perspective." Research Report. 2005.
7. Felter, Bob. "Concrete Pavement Cracking and Seating." MATES (1989).
8. Freeman, Thomas E. Evaluation of Concrete Slab Fracturing Techniques in Mitigating Reflective Cracking Through Asphalt Overlays. Final Report. The University of Virginia. Charlottesville, Virginia: Virginia Transportation Research Council, 2002.
9. Hakim, Al. "Concrete pavement strengthening using crack and seat." H & T: The Journal of the Institution of Highways & Transportation & IHIE 46.10 (1999): 20-22.
10. Harris, Gary K. Cracking and Seating to Retard Reflective Cracking - Fremont County. Final Report HR-279. Iowa Department of Transportation. Ames, Iowa: Iowa Highway Research Board, 1993.
11. Heckel, Laura B. Rubblizing with Bitumous Concrete Overlay - 10 Years' Experience in Illinois. Physical Research Report No. 137. Springfield, Illinois: Illinois Department of Transportation, 2002.
12. Huang, Haiming and Thomas White. Concrete Pavement Crack and Seat Performance Overlay Design. Final Report. Purdue University. West Lafayette, Indiana: Purdue University, 1995.

13. Huang, Yang. H. Pavement Analysis and Design. 2nd. Upper Saddle River: Pearson Prentice Hall, 2004.
14. Marks, Vernon J. and Anderson Chris. Crack and Seat PCC Pavement prior to Resurfacing. Final Report HR-527. Ames, Iowa: Iowa Department of Transportation Highway Division, 1993.
15. Moody, Eric D. "Field Investigations of Selected Strategies To Reduce Reflective Cracking in Asphalt Concrete Overlays Constructed over Existing Jointed Concrete Pavements." Transportation Research Record 1449 (1994): 209-217.
16. Osseiran, Abdallah H. Crack and Seat Concrete Pavement. Construction Report. Pheonix, Arizona: State of Arizona, 1987.
17. Ott, R. Lyman and Michael Longnecker. An Introduction to Statistical Methods and Data Analysis. 6th Edition. Belmont: Brooks/Cole, 2010.
18. Rahim, Ashraf M. and Gregg L. Fiegel. An Evaluation of the Crack, Seat and Overlay Method in California. Caltrans Research Project. California Polytechnic University. San Luis Obispo, 2008.
19. Saraf, C L, et al. "Life-Cycle Cost Anlaysia of Ohio Pavement Rehabilitation Demonstration Projects." Transportation Research Record 1307 (1991): 234-242.
20. Schutzbach, Amy M. "Crack and Seat Method of Pavement Rehabilitation." Transportation Research Record 1215 (1989): 197-211.
21. Sharpe, Gary W., Mark Anderson and Robert C. Deen. "Breaking and Seating of Rigid Pavements." Transportation Research Record 1178 (1988): 23-30.
22. U.S. Department of Transportation Federal Highway Administration. Public Road Length - 2007. 14 April 2011
<<http://www.fhwa.dot.gov/policyinformation/statistics/2007/hm10.cfm>>.
23. Wells, Gordon K., Joseph B. Hannon and Nelson H. Predoehl. "California Experience with Crakcing and Seating of Concrete Pavements." Transportation Research Record 1307 (1991): 169-179.
24. Wixson, Gerald E. "Does PCC Cracking and Seating Work?" Better Roads 56.1 (1986): 24-26.
25. Yu, Jianxiong, Eddie Y. J. Chou and Zairen Luo. "Developement of Linear Mixed Effects Models for Predicting Individual Pavement Conditions." Journal of Transportation Engineering 133.6 (2007): 347-354.

APPENDIX A

Central Coast region core locations

Northern California region core locations