DESIGN AND CONSTRUCTION GUIDELINES FOR THERMALLY INSULATED CONCRETE PAVEMENTS

TPF-5(149)

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Task 1

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Chapter 1. Introduction

1.1 Background

Currently there are several thousand miles of composite pavement consisting of Asphalt Concrete (AC) overlay of Portland Cement Concrete (PCC) on the interstate highway system with an AC layer usually between 1.5 and 6 in. thick (AASHTO 1993, Huang 1993). This document will review the current construction methods that are in practice to place an AC overlay on a PCC pavement both as new construction and as a means of rehabilitation. Many of these methods aim to control reflective cracking, which represents the main distress in AC overlays (Owusu-Antwi et al. 1998). Reflective cracking can be defined as cracks in the AC overlay that are directly above joints or above existing cracks in the underlying PCC pavement structure. This distress dramatically decreases the service life of the pavement through the infiltration of moisture and debris, thus increasing agency costs. The control of this distress is critical to the success of AC/PCC composite pavements.

In addition to a review of the different construction techniques, the design methods and analyses to obtain design thicknesses will also be considered with an emphasis on the AASHTO 1993 design guide and the 2002 Mechanistic Empirical Design Guide. Finally, recent efforts aimed at modeling reflection cracking will be presented.

1.2 Objectives

The objectives of this comprehensive literature review are to examine the past and current practices of placing an asphalt mixture overlay on a Portland cement concrete pavement, both as a means of rehabilitation, and as new construction. The evaluation of the pavement, as well as the various design and construction methods used to place the overlay will be considered with a special emphasis on the mitigation of the reflective cracking distress.

Chapter 2. History and Distresses of Composite Pavements

2.1 History

Composite pavements consisting of an AC wearing surface course placed over a PCC base (AC/PCC) or an alternative cemented layer (i.e. lean concrete base [LCB], cement-treated base [CTB], or roller compacted concrete [RCC]) have been used since the 1950s both in newly constructed pavements or as part of overlay rehabilitation procedures. Many thousands of miles of AC overlays have been placed on PCC pavements considered unserviceable. Numerous cities have regularly used AC/PCC composite pavements as their standard design. These pavements were not constructed with a strong concern for reflective cracking. In the subsections to follow, a detailed history of the AC overlays and a brief review of selected projects from the SHRP R21 draft interim report will support the case that AC/PCC composite pavements can be designed and constructed to satisfy the requirements of a long-lived, quality pavement.

2.2 Reflective Cracking

Reflective cracks are the most common distress associated with AC/PCC composite pavements, and often occur early in the lifespan of the pavement, thereby decreasing the service life of the overlay (AASHTO 1993). It is believed that the cracks initiate at the interface of the concrete and asphalt concrete layer and propagate upwards. The cracks are initiated by the movement of the joints and cracks of the underlying PCC layer due to differential temperatures and discontinuities in the pavement system. In addition traffic loading can induce differential vertical displacements (Roberts et al. 1996). These movements induce strains in the HMA layer and eventually lead to the formation of cracks (PCS 1991) (see Figure 2.1). The cracks are a concern because they provide a path for water infiltration, which leads to water related distresses that include pumping and stripping of the asphalt layer. Although these distresses could be mitigated by performing crack treatments (Yoder and Witczak 1975), a solution to the reflective cracking problem would be ideal.

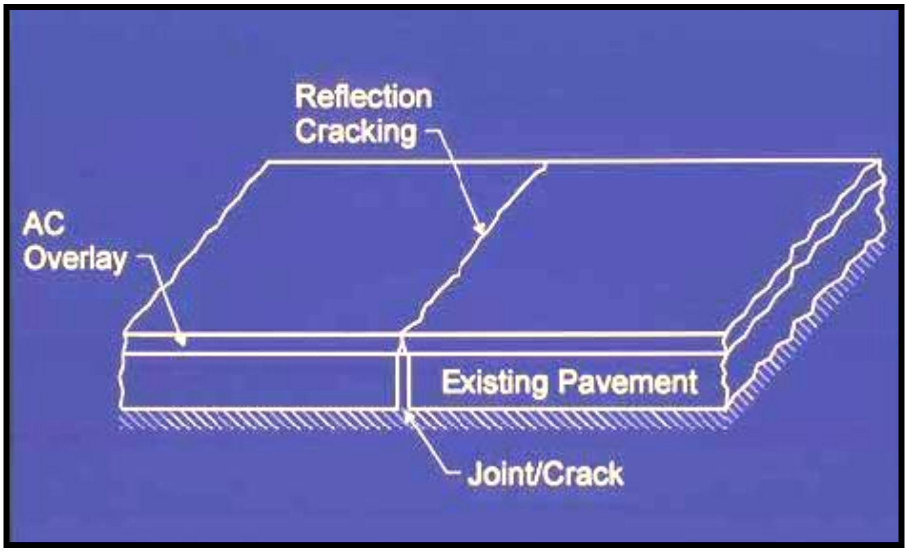

Figure 2.1. Reflective Cracking (Ceylan 2005).

2.3 PCC Distresses

The existing PCC layer is susceptible to various distresses namely, longitudinal cracking, transverse cracking, corner cracking, faulting, D-cracking etc caused by heavy traffic load along with curling and warping stresses. These distresses need to be corrected as they can cause excessive slab deflections, horizontal movements, as well as differential deflections of the PCC slab (Miller and Bellinger 2003). Therefore, the condition of the existing PCC pavement must be thoroughly analyzed and appropriate corrections should be adopted before placing the AC overlay. The following sections briefly describe modes of failure in PCC pavements that may influence the performance of AC-over-PCC pavements.

2.3.1 JPCP Transverse Cracking

When the truck axles are near the longitudinal edge of the slab, midway between the transverse joints, a critical tensile bending stress occurs at the bottom of the slab, as shown in Figure 2.2.

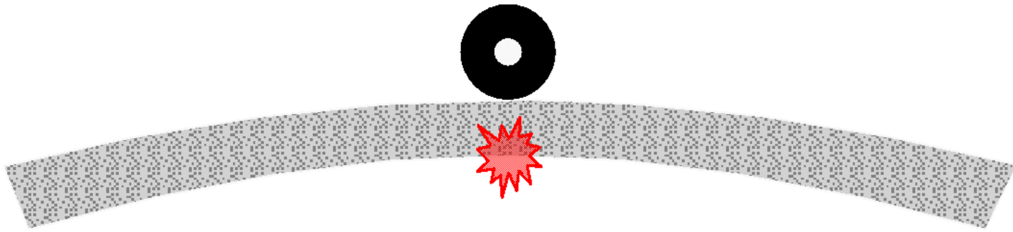

Figure 2.2. Curling of PCC slab due to daytime positive temperature difference plus critical traffic loading position resulting in high tensile stress at the bottom of the slab.

This stress increases greatly when there is a high positive temperature gradient through the slab (on a hot sunny day, the top of the slab is warmer than the bottom of the slab). Repeated loadings of heavy axles under those conditions result in fatigue damage along the bottom edge of the slab, which eventually result in a transverse crack that is visible on the surface of the pavement.

Repeated loading by heavy trucks when the pavement is exposed to high negative temperature gradients (during nighttime, the top of the slab cooler than the bottom of the slab) result in fatigue damage at the top of the slab, which eventually results in a transverse crack that is initiated on the surface of the pavement. The critical loading condition for top-down cracking involves a combination of axles that loads the opposite ends of a slab simultaneously. In the presence of a high negative temperature gradient, such load combinations cause a high tensile stress at the top of slab near the middle of the critical edge, as shown in Figure 2.3. This type of loading is most often produced by the combination of steering and drive axles of truck tractors and other vehicles. Multiple trailers with a relatively short trailer-to-trailer axle spacing are other common sources of critical loadings for top-down cracking. The top-down stress becomes critical when significant amount of permanent upward curl/warp is present.

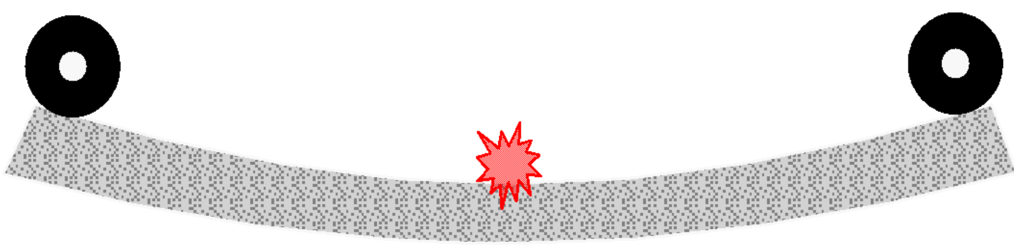

Figure 2.3. Curling of PCC slab due to nighttime negative temperature difference plus critical traffic loading position resulting in high tensile stress at the top of the slab.

2.3.2 *JPCP Faulting*

Repeated heavy axle loads crossing transverse joints creates the potential for joint faulting, as shown in Figure 2.4. Faulting can become severe and cause loss of ride quality and require premature rehabilitation if any of the following conditions occurs:

- Repeated heavy axle loads
- Poor joint load transfer efficiency (LTE)
- Presence of an erodible base, subbase, or subgrade beneath the joint
- Presence of free moisture under the joint.

Figure 2.4. Faulting in jointed PCC pavement (WSDOT Pavement Guide).

2.3.3 *JPCP Longitudinal Cracking*

Longitudinal cracking in JPCP pavements is a phenomenon which has received little attention in pavement engineering. Longitudinal cracking may precede development of transverse cracking if at least one of the two conditions are present: 1) a PCC pavement is very thin and undoweled and/or 2) a significant shrinkage and built-in curling occur. Since built-in curling highly affects transverse cracking as well, development of procedures for proper prediction and management of built-in curling during construction should be an important activity in this study.

2.3.4 CRCP Top-down Punchouts

One of the major distresses of continuously reinforced concrete pavements (CRCP) are cracks which propagate longitudinally across the slab from one transverse crack to another. Figure 2.5 illustrates CRCP punchout analysis for top-down initiated cracking.

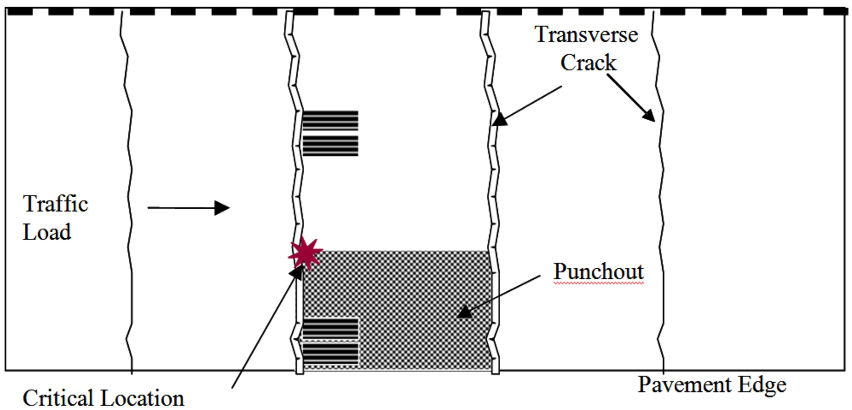

Figure 2.5. CRCP punchout analysis.

Conventional wisdom in pavement engineering often assumes that punchouts are uninitiated from the top of the bonded overlay. If the bottom layer of a composite CRCP is weaker than the top layer then the punchout may be initiated from the bottom surface of the lower layer. This mechanism of failure may be considered in the proposed study.

2.3.5 Built-in Curling/Warping

Although the fact of initial (“built-in”) curling was relatively well-known to researchers, only recently has this effect been acknowledged as a contributing factor to distresses and long-term pavement performance. Figure 2.6 illustrates the importance of considering built-in curl. Built-in curling is influenced by many factors, including material properties (PCC shrinkage, coefficient of thermal expansion, heat of hydration, etc.), placement conditions (ambient temperature, ambient relative humidity, wind, sunshine) and curing techniques. Excessive built-in curling can increase the initial roughness of concrete pavements and may increase top-down cracking and faulting in JPCP and punchouts in CRCP.

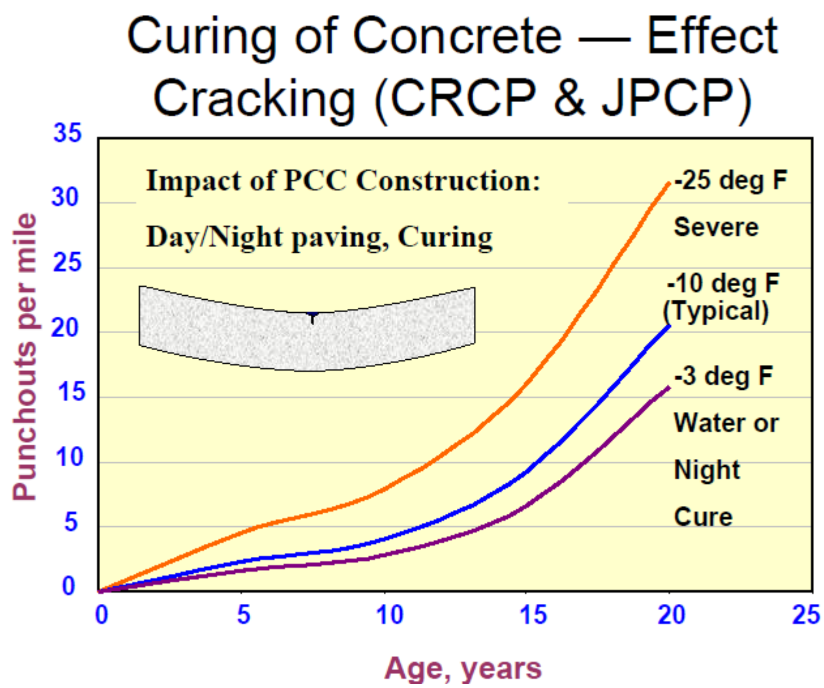

Figure 2.6. Built-in curl and pavement performance.

Chapter 3. Current Rehabilitation Construction Practices

This section discusses the construction practices employed by different state Department of Transportation (DOT) for rehabilitating existing PCC pavement with an AC overlay. Furthermore, different techniques employed for the purpose of rehabilitation are described in detail.

3.1 State Practices

Placing an asphalt concrete (AC) overlay on a distressed Portland cement concrete (PCC) pavement is a common technique to restore both ride and structural capacity. However this form of rehabilitation presents the significant problem of reflective cracking. The current practices for AC overlay of existing PCC pavements followed by various state DOTs such as Washington, Texas, Ohio, Virginia, and Michigan are outlined in this section. The following techniques describe widely adopted practices:

- Surface preparation – Before placing the AC overlay on the existing PCC pavement it is important to repair, clean, level, and coat it with a binding agent (Figures 3.1 and 3.2). These techniques help insure the bonding of the overlay to the existing surface and are critical for pavement performance.
    - Repairing a pavement is based on the type and severity of existing cracks; cracks that do not display structural failure should be cleaned out with pressurized air and filled with sealants, whereas for badly cracked pavement sections it is suggested to patch or replace them (WSDOT 2009). For a structurally acceptable PCC pavement with large amount of fine cracks it is suggested to use bituminous surface treatments or slurry seals.
    - Leveling is achieved though milling and/or diamond grinding the existing PCC surface to eliminated small surface distortions. Ohio DOT (2009) recommends lightly scarifying the PCC surface if the total overlay thickness is less than 5 in. (125 mm) such that the roughened surface can be used to provide a better interlock between the asphalt and the concrete, thereby reducing rutting and debonding.

    - Coating the prepared surface with a tack coat helps achieve a good bond between the asphalt and the concrete surfaces, ensuring long-term pavement performance (WSDOT 2009, TxDOT 2008).
- Pavement rehabilitation – Various techniques are employed for mitigating (prevent or delay) reflective cracking through the AC overlay. The techniques listed here are discussed in details in sections 3.2 to 3.4.
    - Crack/break and seat the underlying PCC pavement;
    - Rubblizing the badly distressed PCC pavement to serve as a base course;
    - Installing a crack-relief layer or laying a stress-absorbing interlayer;
    - Applying a fabric membrane interlayer; and
    - Providing a sufficiently thick layer of AC on the PCC pavement;
- Reflective cracking control – Along with proper surface preparation and rehabilitation techniques it is important to create joints in the AC overlay to match the transverse and/or longitudinal joints of the PCC pavement. These joints are then sealed so that water and foreign particles do not enter the layer. This process is termed "Saw and Seal" and is discussed further in section 3.5.

Figure 3.1. Milled road showing complete removal of the HMA overlay to expose the PCC slabs beneath (WSDOT).

Figure 3.2. Washing the Existing Surface Prior to Overlay (WSDOT).

3.2 Fractured Slab Approach

Thermal movements of the underlying concrete slabs at joints and working cracks induce excessive strains in the AC overlay to initiate reflective cracking. Three methods are commonly employed and will be examined in this section: 1. Crack and Seat, 2. Break and Seat, and 3. Rubblization. The intent of the Break and Seat and Crack and Seat methods (taken collectively as B/C&S) is to create PCC fragments that are small enough to reduce horizontal slab movements so that thermal stresses, which contribute to reflective cracking, are greatly reduced; to ensure that the fragments are large enough to maintain the original structural strength of the pavement (although many researchers do not consider cracked slabs as having the original structural capacity); and to provide aggregate interlock (Galal et al. 1999). Seating is completed after cracking with the goal of reestablishing support between the subbase and the slab (Galal et al. 1999). Crack and Seat is generally limited to unreinforced concrete pavements, while Break and Seat is usually applied to reinforced concrete pavements (Ceylan et al. 2005). Rubblization involves the conversion of the PCC pavement into a high quality aggregate base (generally pieces are less than 300 mm) thus eliminating joints and cracks that may reflect through the bituminous overlay (Galal et al. 1999, Wienrank and Lippert 2006).

3.2.1 Crack and Seat, Break and Seat

A performance survey conducted by the FHWA found that breaking/cracking and seating (B/C&S) as a rehabilitation alternative should be approached with caution, as a significant reduction in reflective cracking after 4 to 5 years due to B/C&S occurred on only 2 of 22 projects reviewed (Galal et al. 1999, Thompson 1989). The University of Illinois surveyed 70 projects in 12 states and found that B/C&S treatment reduced reflective cracking in the early years of the life of the overlay, however the study also concluded that the effectiveness of B/C&S diminished with age (Galal et al. 1999, PCS 1991).

Arudi et al. (1996) described a 1995 study in Ohio that involved rehabilitating eight miles of in-service composite pavements carrying heavy traffic by milling the original asphalt layer, breaking and seating the concrete slabs (on four of the eight miles), and constructing new asphalt overlays. After two and a half years, the study found that the type of breaking equipment used and the extent of breaking are the most significant factors affecting the performance of pavements.

Rajagopal et al. (2004) revisited the Ohio site and reported on the performance after 9 years of service. They found that breaking PCC slabs into 18-in. segments reduced flexural strength, increased surface deflections (50 – 100%), and decreased the area and spreadability (20 – 30%). Note that a lower spreadability, or more closely spaced cracks, of the B&S sections indicates a behavior more similar to flexible pavements. They observed that the spreadability of sections where a pile hammer was used were considerably lower than those sections broken with a guillotine hammer (Figure 3.3).

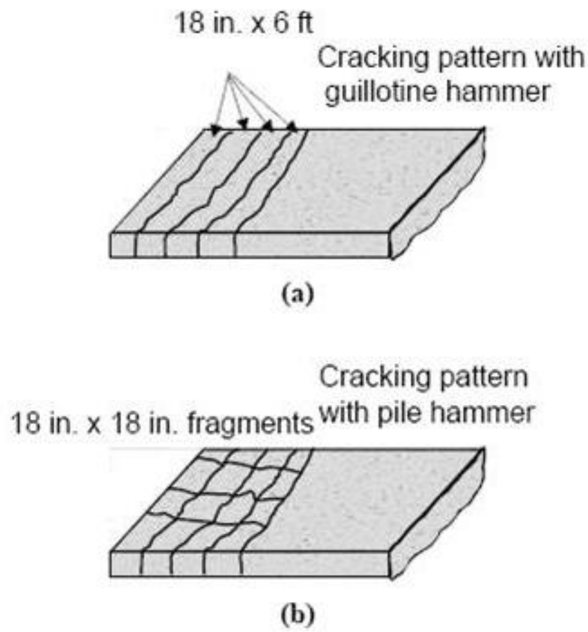

Figure 3.3. Cracking pattern with (a) guillotine hammer versus (b) pile hammer.

This reinforced the observation that the guillotine hammer is not an effective tool for breaking JRCP and helped them to come to the conclusion that the type of breaking equipment used (demonstrated in Figure 3.4) as well as the extent of breaking are significant factors in the performance of AC overlays. Finally, Rajagopal et al. (2004) observed that reflective cracks occurred in more than 80% of the joints in all of the control sections within 2 to 4 years, where the B/S sections were free of cracks at the joints after 9 years.

Figure 3.4. At left, guillotine hammer used for cracking and seating PCC; at right, PCC pavement after cracking and seating with guillotine hammer (WSDOT)

3.2.2 Rubblization

The rubblization process consists of converting a badly distressed PCC pavement into a high quality aggregate base. A National Asphalt Pavement Association (NAPA) study reported that the strength of the rubblized layer can be approximately 1.5 times greater than a conventional dense graded aggregate base (PCS 1991). This method of rehabilitation is usually considered cost effective when the PCC pavement has suffered structural distresses such as D-cracking, alkali-silica reaction, freeze-thaw damage, faulting, and patching of more than 10% of cracked slabs (Thompson 1999). Figure 3.5 presents a rubblized PCC pavement using resonant pavement breaker.

Figure 3.5. At left, resonant pavement breaker used to rubblize PCC pavement; at right, PCC pavement after rubblization with a resonant pavement breaker (WSDOT).

In 1989 a severely distressed PCC pavement, with reduced structural capacity, was overlaid at Mills County, Iowa. A survey conducted five years later revealed that no cracks were present. The report attributed the use of edge drains as a contributing factor to the success of the project (Ceylan et al. 2005). The Michigan Department of Transportation reported that AC overlays of rubblized PCC pavements designed for 20 years had an average service life of only 14 years. The poor performance of those sections was attributed to the equipment and methods used and/or poor subgrade or base support (Ceylan et al. 2005).

In 1992 Witczak and Rada (1992) located more than 500 fractured slab projects throughout the U.S. and selected 100 such sections to study in depth through visual distress surveys to assess pavement performance and non-destructive testing (NDT) to assess the in-situ characteristics of the pavement layers. They discovered that rubblization was the most effective fracture method to avoid reflective cracking—followed in effectiveness by crack and seat and break and seat. Using observed data, Witczak and Rada developed reasonable predictive models for E_{PCC} for both rubblized and crack and seat sections. The predictive models indicated that as the crack spacing and foundation support of existing PCC increases, so does E_{PCC} (Witczak and Rada 1992).

Galal et al. (1999) conducted a structural evaluation of a rubblized section of US 41 in Benton County, Indiana to estimate the AASHTO layer coefficient. They determined the coefficient to be (α_2 = 0.25), which was lower than the value obtained from a previous study conducted by PCS/LAW (PCS1991). As a result, Galal et al. (1999) recommended increasing the current Indiana Department of Transportation (INDOT) standard of (α_2 = 0.20) to (α_2 = 0.22), a conservative value set within two standard deviations of the mean. They concluded that the rubblization process provided a uniform, stable, high strength granular base and seemed to be an appropriate rehabilitation option for INDOT.

Ksaibati et al. (1999) published the results of a nationwide survey to determine agency rubblization practices and performance of rubblized sections across the country. Of the fifty surveys sent, 38 states responded, and of these 38, 21 had experience with rubblization (Table 3.1). Table 3.2 illustrates the predominant distresses associated with the rubblization method (note that most states reported no major distresses). The rutting failures can be attributed to poor subgrade support.

Table 3.1. Rubblization Survey Response (Ksaibati et al. 1999).

STATES WITH RUBBLIZING EXPERIENCE	STATES WITH NO RUBBLIZING EXPERIENCE	STATES WITH NO RESPONSE
AL, AR, IL, IN, KS, LA, MA, MD, MI, MN, MO, MS, NC, ND, NV, NY, OH, OK, PA, VA, WI	AK, AZ, CA, CT, FL, ID, GA, HI, ME, NE, NJ, NM, RI, SD, UT, VT, WA	TOTAL: 12
TOTAL: 21	TOTAL: 17	

Table 3.2. Predominant Distresses reported in Rubblized Concrete Sections (Ksaibati et al. 1999).

DISTRESSES IN RUBBLIZED PAVEMENTS	STATES	# OF STATES*
NONE	AR, IN, MA, MD, MI, MS, NV, NY, PA, VA, WI	11
CRACKING	AL, MI, MN, MO	4
RUTTING	KS, ND	2
FAILURE DUE TO SUBGRADE	OK, NC	2
OTHERS	IL, LA, OH, PA	4

* Pennsylvania had two answers.

Table 3.3 illustrates the overall performance of rubblization in subjective terminology. The vast majority reported excellent or good success with the technique, and no state reported poor performance (Ksaibati et al. 1999). The results of the survey also indicated that most states use the AASHTO 1993 design method and only three, including Minnesota, use the NAPA design procedure. The layer coefficients of the rubblized layer varied from 0.1 to 0.3, which in turn contributed to the large variability in asphalt thicknesses from 76.2 to 330 mm. Most states used a combination of steel vibratory and pneumatic tire rollers in compaction operations, and most states restricted the size of rubblized particles between 25.4 and 76.2 mm.

Table 3.3. Overall Performance of Rubblized Pavements.

PERFORMANCE OF SECTIONS	STATES	# OF STATES
EXCELLENT	IL, IN, MA, MD, MS, NC, NV, NY, PA	9
GOOD	AL, AR, KS, MO, VA, WI	6
POOR	None	0
MIXED SUCCESS	MI, MN, ND, OK	4
NO ANSWER	LA, OH	2

3.3 Interlayers, Grids, and Fabrics

An interlayer system can be applied between the underlying concrete pavement and the overlaid asphalt surface in order to mitigate the initiation and propagation of reflective cracking. The effectiveness of the interlayer to control reflective cracking is reduced considerably if the interlayer system is installed improperly and appropriate bonding between the interlayer system and surrounded layers is not achieved (Hakim 2002). Use of fabrics and interlayers has not generated the consistent performance associated with fractured slab techniques (Galal et al. 1999).

Blankenship et al. (2004) used a performance-related flexural fatigue test that addressed the tension, shear, and bending forces that result in reflective cracks. The researchers in this study designed the reflective crack relief system to consist of an impermeable, highly elastic interlayer that used fine graded aggregates and a quality HMA overlay specified by the flexural fatigue performance related test for crack resistance and Hveem stability performance related test for rutting resistance. Blankenship et al. (2004) analyzed preliminary cracking data from several projects that were specified with performance-based tests, and control sections showed that reflective cracking was delayed. Cores from the test sections revealed that when the overlay cracked, the interlayer seemed to remain intact and impermeable, further protecting the pavement structure from moisture intrusion. In addition the cores showed that, unlike those in the control sections, the reflective cracks were offset from the underlying joints which further disrupted the path for water and gave better rideability. Moisture intrusion is important to control because it can contribute to stripping of the AC layer (Kandhal et al. 1989).

Makowski et al. (20050) investigated the City of Milwaukee’s use of an asphalt rich, fine aggregate, polymer modified asphalt mix interlayer in composite pavements. The interlayer was placed on top of deteriorated or cracked jointed plain concrete pavements (JPCP), after which an HMA overlay was placed on top of the interlayer to complete the composite system. The purpose of the interlayer was to absorb joint movements, delay reflective cracking, and protect the existing underlying concrete pavement. The first project constructed in 1996 did not show any delay in reflective cracking in the first three years. However, projects that included performance-related designs—based on parameters including Hveem stability and flexural beam fatigue—that were overlaid with improved mixtures yielded more promising results. Those projects showed a 42% improvement in the time to appearance of surface cracks. The cores indicated that even though surface cracks were present, some of the interlayers were not cracked, in effect allowing the interlayer to provide further protection to the underlying pavement from infiltration. They also concluded that the type of pavement was a significant factor in the success of the projects (Makowski et al. 20050).

The Sand Anti-Fracture layer (SAF) was first developed in France and later imported to the U.S. by Koch Materials to be used in a project in Oklahoma in 1995 (Brewer 1997). SAF is a 1 in. thick fine graded asphalt mixture that utilizes a highly polymerized asphalt binder, a high VMA and asphalt content, and low air voids (Koch Materials Company 1997). The Oklahoma DOT placed a 1.5 in. wearing course over the 1 in. SAF layer. It is believed that the insufficient thickness of the wearing and the exposure of SAF to traffic for too long before the application of the wearing course contributed to the large amount of distresses observed (Brewer 1997).

In 2000 the Missouri Department of Transportation (MoDOT 2000a) published a report on the application of SAF on I-29 to retard the development of reflective cracking. The field test of the new technology involved eight test sections which had two degrees of pavement repair, two different overlay thicknesses, and two different grades of asphalt cement in addition to the sections which used the SAF layer. The initial review one and a half years after construction did not indicate a difference among the sections and consequently could not make a recommendation on either the use or disuse of the product (MoDOT 2000a). MoDOT (2000b) again applied the SAF on U.S. 36 and noted that the additional SAF cost of $34,000/lane mile was not offset by the reduction in pavement repair costs. The evaluation is currently ongoing, but the author was able to recommend the following based on preliminary construction performance:

- Do not use SAF on weak underlying layers
- SAF cannot replace full depth pavement repair
- Life-cycle cost comparisons should be completed to determine the economic feasibility of continued use of SAF

Recently Baek and Al-Qadi (2006) applied fracture mechanics in pavement analysis to investigate crack development. They used the finite element method with the cohesive zone model to simulate a single-edge notched beam test. They investigated the effects of steel reinforcement netting, interface, and hot-mix asphalt (HMA) properties on crack initiation time and crack propagation rate. Steel reinforcement netting is a relatively new interlayer product introduced by Al-Qadi and Elseifi (2004). It was first applied at the Virginia Smart Road in

1999 and has since been applied in numerous test sections throughout the country (Baek and Al-Qadi 2006). A damage value—an indication of the degradation of the initial stiffness of the material in cohesive elements—was used to define the degree of softening and to trace crack formation. On the basis of elastic analysis, the crack initiation time for the reinforced beam was five times greater than that of the unreinforced beam, and the crack propagation rate was reduced by 2.5 to 6 times at −10°C. Conversely, when viscoelastic analysis—a more realistic simulation of HMA behavior—was used the improvement in controlling reflective cracking was approximately one-half of the improvement for the same loading and temperature conditions. This improvement depends on shear stiffness at the interface, crack length, and HMA temperature. In general Al-Quadi and Elseifi (2004) found that steel reinforcement netting, when installed properly, enhanced the resistance of the overlay system to reflective cracking (Baek and Al-Qadi 2006).

South Dakota recently conducted an experiment on a 1.4 mile stretch of the southbound lanes of I-29 (Rumpca and Storsten 2000). The 1.4 mile stretch was divided into 12 test sections, each with 10 joints, and the sections were subjected to various treatments including the use of two different geogrid products, Strata Grid and Linq Tac, as described in Table 3.4 below.

Table 3.4. Order and Treatment of SD Test Sections (Rumpca and Storsten 2000).

Joints	Material	Rehabilitation	Asphalt Joint Treatment
615-624	Strata-Grid 200	Maximum	Sawed
625-634	Linq Tac - 711N	Maximum	Sawed
635-644	None	Maximum	Sawed
645-654	Strata-Grid 200	Maximum	Unsawed
655-664	Linq Tac - 711N	Maximum	Unsawed
665-674	None	Maximum	Unsawed
675-684	Strata-Grid 200	Minimum	Unsawed
685-694	Linq Tac - 711N	Minimum	Unsawed
695-704	None	Minimum	Unsawed
705-714	Strata-Grid 200	Minimum	Sawed
715-724	Linq Tac - 711N	Minimum	Sawed
725-734	None	Minimum	Sawed

The rehabilitation column in Table 3.4 refers to the amount of rehabilitation that the joints received prior to the application of the fabric and later the asphalt overlay. "Maximum" rehabilitation refers to joints that had four foot sections removed to the base, replaced with new material, and tied in to the existing pavement with steel reinforcement. “Minimal” rehabilitation refers to joints that were cleaned and had small holes filled prior to the placement of the geogrid and subsequent asphalt overlay. Figure 3.6 depicts the Linq-Tac and Strata Grid geogrid products after placement on the rehabilitated joint, and prior to the asphalt overlay.

Figure 3.6. Linq-Tac -711N (at left) and Strata Grid-200 geogrid products (Rumpca and Storsten 2000).

The results indicated that most of the joints reflected through to the asphalt overlay regardless of the use of a geogrid treatment. Table 3.5 illustrates the results of the study, obtained through the use of statistical software, with the mean amount of reflective cracks experienced by each treatment. The largest observed difference in performance was between sawed joints and unsawed joints, with 12% of the sawed joints experiencing reflective cracks and 25% of the unsawed joints experiencing reflected cracks. As the results indicate, the most economical and best performing option was the section that did not use a geosynthetic fabric, had minimal joint treatment, and had sawed joints (Rumpca and Storsten 2000). In support of this conclusion is the survey conducted by Bennert and Maher (2007), whose survey of DOTs on the use of interlayers concluded that the worst performance is given by paving fabrics and geotextiles.

Table 3.5. Mean amount of reflected cracks for different treatments (Rumpca and Storsten 2000).

Material	Variable	Mean amount of Cracks
Strata Grid-200	Unsawed	0.30
Linq Tac-711N	Unsawed	0.20
None	Unsawed	0.25
Strata Grid-200	Sawed	0.20
Linq Tac-711N	Sawed	0.10
None	Sawed	0.05
Strata Grid-200	Maximum	0.35
Linq Tac-711N	Maximum	0.10
None	Maximum	0.15
Strata Grid-200	Minimum	0.15
Linq Tac-711N	Minimum	0.20
None	Minimum	0.15

Buttlar et al. (2000) reported on the cost effectiveness of the Illinois Department of Transportation's (IDOT) practice of using paving fabrics as a method to control reflective cracking. IDOT uses a polypropylene paving fabric that is placed either longitudinally in strips over joints, or over the entire pavement as an area treatment. The effectiveness of the treatment was assessed by measuring the performance, in terms of crack mapping and serviceability data, of 52 PCC sections that were rehabilitated with a bituminous overlay. Serviceability data indicated that the longitudinal and area treatments improved the life spans of the composite pavement by 1.1 and 3.6 years, respectively. When placed in medium and large quantities, life-cycle cost savings were found to be 4.4 and 6.2 percent respectively, however a statistical analysis showed that these differences were insignificant. The researchers concluded that the treatments are marginally cost effective (Buttlar et al. 2000).

3.4 Interlayer Stress Absorbing Composite

Mukhtar and Dempsey (1996) carried out an in depth study of the causes and phenomenon of reflective cracking and the behavior and performance of overlays with interlayers, and they determined that neither a Stress Absorbing Membrane Interlayer (SAMI) nor a geotextile could completely stop the initiation of cracking. A stress relieving interlayer (rubber asphalt or thick geotextile with low stiffness) allowed for some deformation and reduced stress at the crack tip but often left undissipated stresses that would form cracks when the tensile strength of the asphalt concrete was exceeded. They found that a high stiffness interlayer provided reinforcement to the AC overlay and temporarily retarded the movement of the crack; but since it did not allow any relative movement between the overlay and the underlying pavement the upward moving crack changed direction and moved laterally along the interface between the reinforcement and the underlying material.

The researchers recommended the use of an interlayer stress absorbing composite (ISAC) system that consists of a low-stiffness geotextile as the bottom layer, a viscoelastic membrane layer as the core, and a very high stiffness geotextile for the upper layer. They proposed that the ISAC could be effectively designed to stop the upward propagation of a crack in the AC overlay and to also adequately reinforce the AC overlay (Mukhtar and Dempsey 1996). They published the results of both laboratory and field tests of an ISAC layer. First, they used various thermal/structural models and laboratory equipment to identify the properties of the materials intended to be used in their ISAC system. The researchers then selected a number of woven and nonwoven geotextiles and tested them for their engineering properties such as tensile strength, initial modulus, modulus at failure, and percent shrinkage. Several samples of rubber asphalt were then blended using different ratios of crumb rubber with various types and ratios of asphalt cements at 400°F, tested at different temperatures, and evaluated based on the rate of deformation at different temperatures. Finally, the researchers fabricated an ISAC layer in the laboratory using the appropriate materials.

Testing equipment was developed to evaluate the interfacial shear strength and laboratory testing was performed to determine the shear strength of the fabricated ISAC layer under an AC overlay. A laboratory pavement section with an AC overlay over a jointed PCC slab was constructed and placed in an environmental chamber. A mechanical device was used to simulate thermal strain in the slab and the joint was opened and closed at an extremely slow rate. The testing was conducted at 30°F and deterioration in the overlay was monitored using a sensitive LVDT device. The performance of the ISAC layer was evaluated by comparing the number of cycles to failure of an ISAC-treated overlay with the performance of a control section without an ISAC treated overlay.

The results from a laboratory evaluation testing program indicated that the ISAC layer was highly effective in preventing reflective cracking in a 2.5 in. AC overlay. In addition the researchers found that the ISAC behaved as a base isolation layer in a pavement system and vastly outperformed two commercial products now being used for reflective cracking control in AC overlays (Mukhtar and Dempsey 1996). In this experiment, the use of ISAC decreased the strain in the AC overlay significantly and dramatically increased the number of cycles to failure. The researchers found that the maximum strain in the overlay above the joint increased as the number of cycles increased, and the rate of increase of the maximum strain in the overlay above the joint was considerably higher for the control pavement section than for the section treated with an ISAC layer.

Dempsey later revisited the field test section constructed in 1994. The results of the performance of the ISAC layers compared with those of control sections are summarized in Table 3.6 below; note that the first measurement corresponds with the depth of the overlay (cm), and is followed by the number of reflective cracks (RC) for a particular section. Dempsey concludes that, based on several years of field performance, the ISAC system is highly effective for mitigating reflective cracking in AC overlays used on both airport and highway pavement systems (Dempsey 2002). The ISAC system has been employed on other highway and airport pavement projects.

Table 3.6. ISAC Field Performance.

Location	Control Section	Reflective Cracks (RC) ISAC Section	Average Joint and/or Crack Spacing Covered with ISAC
IL 38 Rochelle, IL Constructed Summer 1994 Last Inspected 8/14/2000	6.3 cm AC:20 RC 8.9 cm AC:18 RC	6.3 cm AC:5 RC	7.6 m transverse
US 67 Jacksonville, IL Constructed Summer 1998 Last Inspected 4/20/2001	5.7 cm AC:45-50 RC per kilometer	5.7 cm AC:3 RC in 1859 m	27.2 m transverse
Mattis Avenue, Champaign, IL Constructed August 2000 Last Inspected 7/28/2001	7.6 cm AC:21 RC in 1065 m	7.6 cm AC:2 RC in 1862 m	8.3 m transverse
DeKalb Airport, IL Constructed Summer 1998 Last Inspected 8/2000	No control	7.6 cm AC:0 RC	4.6 m longitudinal
Willard Airport, Champaign, IL Constructed July and August 1999 Last Inspected 7/25/2001	No control 5.1 cm to 15.2 cm wedge:4 RC	15 cm AC:0 RC 5.1 cm to 15.2 cm wedge:0 RC	7.6 m transverse
Rantoul Airport, Rantoul, IL Constructed September 1999 Last Inspected 7/25/2001	5.1 cm to 10.2 cm AC:0 RC in 27 m	5.1 cm to 10.2 cm AC:0 RC in 132 m	4.6 m transverse 3.8 m longitudinal

Note that the performance of stress absorbing interlayer composites can be improved by considering a rich bottom layer (asphalt mixtures in excess of asphalt to regular mixture design) and even the interposition of granular layers for reflective cracking control.

3.5 Bituminous Saw Cuts (Saw and Seal)

Bennert and Maher 0(2007) summarized the results of a survey conducted between January to May 2006 that outlined the current composite pavement practices and key hot-mix asphalt (HMA) overlay design features, designed mainly to mitigate reflective cracking in the HMA overlay, used by state highway agencies in the United States. Among the discoveries of this survey was the fact that that sawing and sealing the HMA overlay was only done by 31% of state agencies that responded. However, many of the states that reported using saw and seal described the method as being effective in minimizing the effects of reflective cracking. Only 12% of the states reported using the saw-and-seal method on second and third generation HMA overlays (Bennert and Maher 02007). The State of New York has received much attention due to its current practice of sawing and sealing transverse joints on many HMA overlays on Jointed

Concrete Pavements (JCP). As a result of the publicity, the procedure of sawing and sealing joints in HMA overlays of JCP is often called "New York Saw and Seal."

The Minnesota Department of Transportation (Mn/DOT) began experimenting with sawing and sealing bituminous pavements in 1969 and has constructed more than 20 separate projects, with more than 50 test sections, under various conditions including bituminous overlays of JCP and with several joint and sealant types. Janisch and Turgeon (1996) published a report that outlines the results of these tests and Minnesota's experiences with bituminous saw cuts. Overall, Mn/DOT found that the saw and seal method was an effective means of controlling both thermal cracking and reflective cracking for HMA overlays of JCP; however they noted that most of the sections that did not experience success had either one, or a combination of the following:

- a high frequency of midpanel cracks,
- badly deteriorated joints and/or lots of patching at or near the joints,
- badly deteriorated or stripped cracks that remain after milling.

In addition they recommend that saw cuts be made within 1 inch (longitudinally) of the underlying JCP joint. Furthermore they cited the need for additional research regarding the depth of the cut, but depths of the greater of 2.5 in, or one-third the depth of the bituminous layer, have been found in experiments to be successful (Janisch and Turgeon 1996). Minnesota's conclusions agreed with the Pavement Consultancy Services, Inc. (PCS) recommendation which stated that the success of saw and seal operations is dependent upon accurately locating all underlying joints, in addition PCS recommended sawing the new joints immediately after placing the overlay (PCS 1991).

Cumbaa and Paul (1988) compared three different techniques to mitigate reflective cracking on an urban LA expressway (PCC overlaid with AC). Those techniques were water proofing membranes (three different membranes total), a latex-modified asphalt binder, and sawing and sealing of transverse joints. They evaluated the pavement sections following construction and found that in most cases the cracks from the underlying pavement reflected during construction; however the use of the saw and seal technique was the most effective method of the three in controlling reflective cracking.

3.6 Modified Binders

As mentioned above Cumbaa and Paul (1988) published their research comparing the use of modified asphalt binders and joint treatments as reflective crack mitigation techniques. The modifier they chose was a styrene - butadiene (SBR) latex. It is purported that the SBR latex will increase mix stiffness at high temperatures, thereby increasing stability while reducing mix stiffness at low temperatures in order to reduce cracking and provide greater flexibility. They found that the SBR modified asphalt binder may indicate increased benefits over the conventional binder, but their results were only based on conditions following construction (Cumbaa and Paul 1988).

3.7 Ultra Thin Bonded Wearing Course

Ultra thin bonded wearing course (UTBWC) is a relatively new process that was developed in France in 1986 and has been in used in the U.S. since 1992. The primary difference between this treatment and those listed above is that, in the case of UTBWC, the asphalt surface does not provide any significant structural support; it can be applied to either a PCC or AC pavement. It consists of a layer of HMA laid over a heavy asphalt emulsion layer or membrane. The thickness of the UTBWC ranges from 9.5 to 19 mm (3⁄8 to 3⁄4 in.). The system is placed on a structurally sound rigid or flexible pavement that may exhibit minor surface distresses. Reporting on recently completed UTBWC projects in many locations indicate mixed performance of the UTBWC. A number of researchers claim that it provides a surface with excellent macro texture qualities, good aggregate retention, and excellent bonding of the very thin surfacing to the underlying pavement. However, one report notes that any cracks that exist in the underlying pavement reflected through the UTBWC (Hanson 2001).

3.8 Drainage Considerations

Kandhal et al. (1989) published a report that evaluated the phenomenon of stripping due to the subsurface drainage of the total highway pavement system. Stripping is a condition where the asphalt film is separated from the aggregate particles by the action of water. They analyzed three case histories of water damage to asphalt overlays over (PCC) pavements during the last ten years in Pennsylvania with detailed field observations. McKesson (1949) found that:

> "…[G]round water and water entering the roadbed from the shoulders, ditches and other surface sources, is carried upward by capillarity under a pavement. Above the capillary fringe water moves as a vapor and, if unimpeded at the surface, it passes to the atmosphere; this is known as drainage by evaporation. If the pavement or seal coat constitutes a vapor seal or a vapor barrier, the moisture during cool nights and in cool weather condenses beneath the surface. When the pavement absorbs solar heat, the water is again vaporized and, if not free to escape, substantial vapor pressure results because water as vapor has more than a thousand times the volume of water in liquid form. Vapor pressure forces the moisture up into the pavement and through the surface. Blistering in bituminous pavements is a well known example of the effect of entrapped moisture and moisture vapor]".

Kandhal et al. (1989) credits McKesson's findings as being the most likely explanation of the observed field behavior in the Pennsylvania case histories. The researchers obtained pavement layer samples using a jack hammer (instead of a core drill) to avoid the use of water, so that in-situ observations of water damage, actual moisture content determination in each layer, and study of subsurface water and/or water vapor migration in the pavement system could be accomplished. Cores from one project were also analyzed for tensile strength to assess the moisture induced damage (Kandhal et al. 1989). In many cases the researchers found that the stripping of asphalt pavements may not be a general phenomenon occurring on the entire project but rather a localized phenomenon in areas of the project which are oversaturated with water and/or water vapor due to inadequate subsurface drainage conditions. They recommend improving the existing subsurface drainage system of the PCC pavements prior to placing the asphalt overlays so that persistent problems of stripping and/or potholing do not occur. In addition their recommendations include the use of Asphalt Treated Permeable Material (ATPM), increased depth of longitudinal underdrains in cut sections, and lateral intercepting drains on grades (Kandhal et al. 1989).

Chapter 4. Composite Pavements as New Construction

More detailed information on the construction of new composite pavements is presented in Chapter 2 of R21 draft report. In the paragraphs below, the use of asphalt overlays over roller compacted concrete and jointed plain concrete is discussed.

4.1 Roller Compacted Concrete (RCC)

Roller Compacted Concrete (RCC) is a relatively new technology, although documents of its use in the reconstruction of estate roads in England go back to early 1940s. Over the past three decades RCC has been used in more than 100 projects (Delatte 2004). RCC is a durable, economical, and reliable technology in the construction of low speed heavy duty pavements (Delatte 2004), and has been employed successfully in the following purposes (Piggott 1999, Delatte et al. 2003)

- Storage areas, and roads,
- Highway weigh stations,
- Airport aprons,
- Docks and container ports,
- Multimodal facilities, and
- Heavy industrial facilities such as logging and automobile manufacturing

RCC has not been employed in high speed pavement construction as a surface layer because of the high surface roughness. It has been suggested that an asphalt overlay could considerably reduce the surface roughness and make RCC composite pavements a viable construction option.

Delatte (2004) noted that one of the obstacles to the use of RCC composite pavements is the lack of a proper design procedure, he presented a design procedure for that utilized simplified tables based on mechanistic analysis. The procedure considered only fatigue cracking, as RCC is very resistant to joint, or crack faulting. He included recommendations for selecting the proper HMA materials. Initial cost estimates indicate that, not considering the cost of the HMA overlay, RCC pavements could be 62% lower than the conventional cost of PCC paving (Delatte 2004).

4.2 Jointed Plain Concrete (JPC)

The following subsections provide a clearer picture of some design features and the performance of AC over PCC pavements. A number of the subsections are built upon work accomplished in the SHRP2 R21 Composite Pavements project, which will feature test sections at MnROAD similar to those of the TICP project. While this work is cited, the SHRP2 R21 project contributes a large amount of performance and design data that could not have otherwise been included, and the authors are indebted to SHRP2 for their support of the TICP project work.

4.2.1 Ontario Highway 401 (1959-1973)

Nine sections were built and instrumented with thermocouples, strain gauges, and devices to measure crack width. The materials (soils, base materials, concrete, AC, PCC) and construction are all typical of conventional practice in Ontario, although special precautions were taken to ensure that the subgrade and base were of consistent quality under each section. After 3 years under traffic, the pavement sections were assessed in a study by Smith (1963) by looking at the tightness of the crack distribution as expressed by percent of cracks greater than a given width. The best performing section was the 7-in CRC base reinforced with 0.44% welded wire and 1.5-in AC surface, closely followed by the an 8-in conventional CRC pavement with 0.38% reinforcement and no AC wearing course, considered for the purposes of that experiment a control. The worst performing section was an 8-in reinforced concrete base (0.18% steel) with 89-ft contraction joints and 3.25-in AC. From an economic viewpoint, in general the unreinforced bases were performing adequately and were considerably cheaper than the reinforced bases. Smith suggests that, in light of the performance of these sections, the performance of the unreinforced base pavements could be improved through sawing or joint forming contraction joints at very short intervals.

The pavements in the Ontario test site were evaluated again after 13 years by Ryell and Corkhill (1973). The authors found that the cracks were least frequent but widest in sections having unreinforced base or a reinforced base with formed joints. Cracks were most frequent and narrowest on sections with concrete base or concrete with the greatest percentage of reinforcing mesh. Increased thickness in PCC reduced cracking by a third in sections containing a heavier percentage of mesh. Increasing the percentage of mesh (from 108 to 140 lb per 100 sq. ft) for a given thickness of PCC resulted in an 80% increase in the number of cracks and a slightly lower average crack width. Although the thickness of the AC overlay varied from 1.5 in. to 4 in., it did not have any influence on the amount of cracking reflected through the base. Wherever a crack was observed in the AC, there was a crack directly below it in the base. While every section was affected by transverse cracking, little change occurred in either number or width of cracks after the first two winters. There was only one exception where the number of cracks had more than doubled after the first 2 years because of a breakdown of the macadam asphalt binder course.

Ryell and Corkhill (1973) considered performance, initial cost, maintenance requirements, and rideability as rating factors and concluded that the 3.25-in AC over 8-in plain (i.e. unreinforced) PCC with no joints was the most satisfactory of the experimental sections. Furthermore, the authors concluded that this section performs better than most other rigid or flexible pavements given its superior skid resistance and lower initial cost. As a result, the authors state that for heavily trafficked pavements, composite pavements such as TICP offer potential advantages over conventional reinforced concrete pavement.

4.2.2 New Jersey Composite Pavements

Another pioneering use of asphalt-over-concrete paving was took place in New Jersey on NJ-3 (Baker 1973). The pavement section consisted of the following design:

- 3.5-in AC (surface and binder)
- 5-in dry bound macadam (2.5-in stone choked with stone screenings)
- 3-in dense graded crushed stone
- 8-in plain undoweled concrete base (NJ Class D mix) with 15-ft joint spacing
- 6-in crushed stone base
- Subgrade was meadowlands, varied layers of silt, clay and sand.

The longitudinal joints between lanes did not include tie bars. The design included 8 in of granular material between the JPC slab and the AC surfacing to prevent reflective cracking. The design concept was to maintain the structural integrity of the pavement despite the unstable subgrade; to achieve a high load-carrying capacity; and to achieve continuity of surface and minimize reflective cracking, thus reducing maintenance costs. The section of highway had four lanes in the same direction with an AC shoulder adjacent to the outside lane. Two-way traffic existed on the four lanes for a short period of time. The AADT in 1970 was 88,780 in two directions at the Hackensack River Bridge. Trucks were 14.6% of all traffic. The pavement was monitored from 1963 until 1971 for smoothness, deflections, and distresses, summarized as follows.

Smoothness. The roughness index increased or decreased slightly on the various lanes over the 8-year period.

- Lane 1: 103 to 115 in/mile
- Lane 2: 108 to 91 in/mile
- Lane 3: 102 to 101 in/mile
- Lane 4: 110 to 128 in/mile

Rutting. Rut depth was measured at 50-ft intervals in both wheelpaths in lanes 1 and 4 using an 8-ft straightedge placed across one wheelpath to the bottom of the rut. Little increase in rutting occurred over the 8-year period.

Cracking. No reflective cracking was observed over the 15-ft spaced transverse joints or along the longitudinal joints. Little other random cracking was observed over the 8-year period.

Deflections. The deflections (measured with a Benkelman beam) were generally low and there was only slight increase in deflections over the 8-year period.

Baker (1973) concluded that the pavement functioned in accordance with design objectives and required no maintenance. A high load-carrying capability was achieved, as evidenced by low deflections. No localized differential settlement was observed, and no reflective cracking or other pavement cracking occurred. The ride quality of the pavement had not deteriorated to any significant degree. Baker notes that while the cost of this pavement was 15% greater than the typical New Jersey JRC pavement section, by the author's estimate the increased expenditure produces approximately 23% greater service life.

4.2.3 Zero-Maintenance Pavements Project

The Federal Highway Administration (FHWA) sponsored the Zero-Maintenance Pavements studies of the 1970s and 1980s, which included surveys of seven composite pavements, six of which were new construction AC-over-PCC. The NJ-3 pavement discussed above was included under the FHWA contract on Zero-Maintenance (Darter and Barenberg 1976) and was found to be providing excellent service at that time. Another section in New Jersey that included a 6-in. JPC base also performed well under lighter traffic. This section on NJ-20 did not require patching or other maintenance. Several composite pavement projects were built in the state of Washington from the 1930s through the 1990s. One such project was constructed in 1955 on I-5 north of Seattle. This 6-lane freeway project was initially evaluated for the Zero-Maintenance Study and had an AADT of 43,500. The design consisted of 4 in of AC placed on a new 6-in PCC slab. Over the years, the pavement exhibited good performance (three overlays over 50 years of service).

Table 4.1. Summary of information for six TICP projects included in the Zero Maintenance Study field survey (Darter and Barenberg 1976).

Route	City, State	Date Opened	1974 AADT	Number of Lanes	Pavement Section				
					Surface	Base	Subbase	Subbase	Subgrade
Hwy 401	Toronto ONT	1959	63,000	4	3.25 in AC	8 in PCC	6 in Gr. Base	6 in Gran. Base	Clay
Hwy 401	Toronto ONT	1959	63,000	4	3.25 in AC	8 in PCC 100-ft JS	6 in Gr. Base	6 in Gran. Base	Clay
QEW 10	ONT	1960	87,500	6	6 in AC	8.5 in PCC	6 in Gr. Base		
I-80	Ottawa IL	1962	18,900	4	3 in AC	8 in PCC	6 in Gr. Base		A-6
NJ-3	NJ	1963	96,500	8	3.5 in AC	5 in stone macadam	3 in Gr. Base	8 in PCC 15-ft JS	6 in Gr. Base, silt, clay, sand
I-5	Seattle WA	1966	43,500	6	4 in AC	6 in PCC	2 in Gr. Base	7 in Gr. Base	Glacial Till

Transverse cracking was the major distress type observed on these composite pavements (with the exception of NJ-3, detailed above, and its unconventional design. The only project without any transverse cracking after 11 years of service and heavy traffic loading was the NJ-3 project detailed above with the 8-in granular material between the AC surface and the PCC slab. Darter and Barenberg concluded that the occurrence of transverse reflective cracks in composite pavements is not critical if the cracks remain "tight." Three composite pavements (I-94, QEW 10, and I-5) exhibited cracking but did not receive any maintenance treatments because the cracks remained tightly closed. However, the two projects on Hwy 401 cracked and seriously spalled, resulting in maintenance. I-80 project near Ottawa, Illinois, had major spalling and was scheduled to be overlaid. The authors concluded from this distress that the AC surface thickness (for a corresponding PCC slab) is an important variable in predicting crack spalling.

Furthermore, the authors state that a relatively thin AC surface under heavy traffic is inadvisable. Rutting for all composite pavements surveyed was rated as only “minor” to “moderate” even under very heavy traffic—this may suggest that the existence of the PCC slab contributes to the minimization of AC surface rutting.

4.2.4 Premium Pavements Study

A follow-up study to Zero Maintenance titled “Premium Pavements” was completed by Von Quintus et al. (1980) that focused on flexible and composite pavement structures. In that study, the primary pavement structure recommended for use to satisfy the definition of “Premium Pavements” was a composite pavement consisting of an AC wearing surface and CRC base. That study reviewed composite pavements that had been built over 10 to 20 years prior to the study, but were still in service and providing good performance. Details from those projects were used in developing a preliminary design manual for both flexible and composite pavement types.

4.2.5 LTPP Information on TICP

The LTPP GPS-2 experiment includes several sections featuring AC surface layers over a lean concrete base (LCB) course or other low quality PCC mixtures. Details for this research are still being completed. However, some of the data and observations to be featured includes: in situ density and moisture content as measured from forensic trenches; transverse joint condition and joint width as observed by removing the AC surface layer; and temperature gradient measurements on the pavement as observed throughout the day (SHRP2 2009).

4.2.6 Reflective Cracking Survey Study

Bennert and Maher (2007) summarized the current flexible overlay design practices and design features for use on PCC or composite pavements across the United States. Of the 26 state highway agencies (SHAs) reporting that they overlay PCC pavements with AC, 22 (85%) reported that reflective cracking was observed within the first 4 years after the AC overlay was placed, while seven agencies (27%) reported observing reflective cracking within the first 2 years. Bennert and Maher conclude that, in general, the longest time before the onset of reflective cracking occurred in those states whose low-temperature performance grade is between -16°C and -10°C, based on LTTPBind software.

4.2.7 Globetrotter Engineering Corp. Study

A study was conducted by Globetrotter Engineering Corporation from 1983 to 1987 to review the performance of composite pavements. This study was a follow-on project to the FHWA-sponsored study on flexible and composite structures for premium pavements. One or more pavements from each of these agencies were included in the review for that project. The pavement condition at the time of the survey varied from poor to excellent. Only 1 of 11 projects was rated as poor. Most of the composite pavements were providing good to excellent performance. Longitudinal and transverse cracking were the primary types of distress observed on the projects visited. The following summarizes the reasons that five of those agencies decided to start constructing AC/PCC composite pavements.

- Florida. Florida started using composite pavements in the 1970s as a result on the oil crisis and built an experimental composite pavement project in 1975 as part of a widening project along US 41, just north of Fort Myers. This experimental project consisted of varying AC wearing surface types and thickness, econocrete base thickness, and different subgrade treatments. Composite pavements have been used in areas with heavy trucks and in areas with high water tables.
- Kentucky. Although the Kentucky Transportation Cabinet did not promote the use of composite pavements, the city of Louisville and other local areas have built composite pavement structures because they were perceived to be easier to maintain and are smoother. Two projects were surveyed in the premium composite pavement project; one along the Algonquin Parkway (built in 1964) and one defined as Breckinridge Lane (built in 1965). Both projects consist of 1.5-in AC surface over an 8-in JPC base followed by a relatively thin aggregate subbase.
- Ohio. Composite pavements have been used for all classes of streets and highways in the Columbus area, and two segments along I-670 were used in the premium composite pavement project. These original composite pavements consisted of a 3-in AC surface, a

9-in JPC base, and a 6-in aggregate subbase. One of these pavements was built in 1958 and the other one in 1961. Both segments performed without any rehabilitation or major maintenance for 24 and 27 years, respectively. The AC wearing surface of both projects was covered with Ralumac, a rubberized slurry. Both segments have also carried heavy truck traffic.

- Oregon. Similar to Louisville, state and local agencies in Oregon have used composite pavements because they were believed to be easy to maintain and cost effective. Two projects were used in the premium pavement project within the Salem area; one within the North Albany interchange section of I-5 and the other segment along Highway 30. Both projects were built in 1973 and consisted of a 4-in AC surface over a 6-in LCB. The only maintenance activity performed along both segments was to seal the cracks with rubberized asphalt in 1984.
- Texas. Texas started using new composite pavements in the late 1970s, because they were found to be easy to maintain. In addition, the AC surface was believed to be an effective seal against moisture intrusion that can cause the erosion of soil along joints and cracks and reduce curling stresses in the PCC layer.

4.2.7 State Transportation Agencies' Experience

Many other agencies have also constructed composite pavements consisting of AC wearing surfaces with various types of PCC bases, including Ohio, Kentucky, Florida, Oregon, Minnesota, New Mexico, and Texas. These agencies do so because of the perceived benefits regarding ease of maintenance from the AC wearing surface and better load carrying capacity of the PCC base.

New York. New York has been using composite pavements since the 1990s. New York has found that reflective cracking is the primary distress that limits the performance of this design strategy. In an effort to reduce maintenance costs and extend the life of this pavement type, the city sponsored and built an experimental project that included AC over JPC (new construction) with various treatments and techniques to retard and prevent the deterioration of reflective cracks in the AC wearing surface. This research was conducted by Applied Research Associates, Inc., and time history of reflective cracking is available along with fracture properties, volumetrics, and moduli of the asphalt material. The data are being utilized in NCHRP Project 1-44 for reflective cracking performance model development. In summary, the reflective cracking treatment that is most economical and has provided consistently good performance is the “saw and seal” method. Many other reflective cracking control treatments were used within the experimental project, but none of the test sections with specialized materials have performed consistently better than the “saw and seal” method.

In studying heat island effects of asphalt rubber (AR) overlays, Belshe et al. (2006) cited the importance of minimizing the urban heat island effect and the part that AR plays in it. In using the darker AR-OGFC there is an increased surface temperature during the daytime, but reduced surface temperature at nighttime (explained by the porosity and lower thermal mass). Traffic is felt to have an aeration effect, and the relative coolness of the pavement is due to the high air-void content and the lower thermal mass of the surface mix. Without the AR surfacing, the temperature of the PCC surface would be 3–8 ºF higher. The authors believe this increased temperature would result in increased truck and warping/curling damage due to greater slab stresses than would be experienced without the overlay. While the insulating effect of overlays on PCC is not a new concept (it is fully included in the MEPDG), the idea that the AR surface could provide such a benefit is crucial to the modeling of AC/PCC composite pavements. The increase in PCC life due to the reduced nighttime temperature gradients is unknown, and this is a complex issue that needs further research.

Arizona. The Arizona Department of Transportation has developed a Quiet Pavement Program (Zareh et al. 2006) where conventional AC and PCC surfaces are overlaid by a 1-in asphalt rubber friction course (ARFC). The ARFC was found to reduce noise by 3 to 5 dB compared to dense, fine-graded AC, and by 6 to 12 dB compared to harshly cross-tined PCC pavement. Based on public pressure, it was decided to overlay around 7,000 lane-miles of PCC pavement with 1 in of ARFC to reduce noise. The authors feel this program has been very successful, although some transverse reflective cracks have occurred and future maintenance may be a problem.

Montana. The Montana Department of Transportation made a decision in the mid-1990s to use the composite pavement strategy in upgrading many of their higher volume roadways. The construction process is to recycle the materials in place by using portland cement as the stabilizing material. The quality of the PCC base does vary and would be defined more as a semi-rigid pavement. However, the PCC base can be similar to a LCB or RCC layer on some of the projects. Most of these projects were used to calibrate the MEPDG for Montana's future use in designing and constructing this type of pavement (Von Quintus and Moulthrop 2007). The local calibration process was challenging because most of the pavements that were built in the 1980s have yet to exhibit any fatigue cracking and have low levels of rutting.

Washington. The Washington Department of Transportation (WDOT) has extensive experience in AC over PCC pavements going back to the 1930s. This experience includes:

- 4-in. AC over 6-in PCC sections along I-5 near Seattle, constructed in 1966;
- 3-in. AC over 7-in PCC sections along State Rte. 906, constructed in 1955;
- 2-in. AC over 9-in PCC sections along State Rte. 432, constructed in 1992;
- 6-in. AC over 6-in PCC sections along State Rte. 432, constructed in 1992;
- 2-in. AC over 9-in PCC sections along State Rte. 432, constructed in 1992;
- 6-in. AC over 6-in PCC sections along State Rte. 411, constructed in 1938;
- 6-in. AC over 6-in PCC sections along State Rte. 12, constructed in 1939.

Performance data for the I-5 North sections was provided to the SHRP2 R21 Composite Pavement projects (SHRP2 2009). This data for over 50 sections along I-5 North includes distress survey information conducted in 2005; reconditioning data for these sections; and traffic analysis for each section. While the individual results for each section cannot be reproduced here, it should be noted that all sections were typically subjected to a resurfacing after 5 and 17 years, and in the 2005 surveys the only distress of note was longitudinal cracking. However, the amount of longitudinal cracking was relatively low, suggesting that this design performed very well for WDOT along I-5.

4.2.8 International Experience in TICP

Several countries in Europe, including the Netherlands, Italy, France, United Kingdom, Austria, and Belgium have built a number of composite pavements. Many of these composites include relatively thin (2 to 3 in) porous AC over CRC. The thin porous AC layer is intended to reduce noise, splash and spray, provide good friction resistance and increased smoothness, and waterproof cracks in the CRC layer, which do not reflect through to the AC layer. Its limitations include a significant decline in the noise-reducing effect over time as the pores in the AC layer fill with materials, a relatively short life, and higher deicing salt requirements to prevent icing. Unsafe conditions can develop when icing cannot be prevented completely, as occurred in Austria.

France. France uses lower-quality local aggregates in concrete pavements for which an AC overlay is also to be placed during construction. This reduces overall pavement costs in areas where only poor aggregates exist. There is little freeze-thaw cycling in these areas. The French usually put SMA or a surface treatment on the surface to improve friction and smoothness, although the exposed aggregate surface is favored by many.

In recent years, France has made real progress with the AC/CRC composite pavements. The pavement structure consists of a very thin layer of AC surfacing (French designation BBTM) with maximum aggregate size of 1/4 in or 3/8 in (6 or 10 mm), a CRC in the pavement body, and a layer of Class 3 asphalt-treated base (French designation GB3) beneath the CRC (Christory et al. 2001). The Road Directorate of the French Public Works Ministry published a structural data sheet for an experimental pavement with this structure. Based on field and laboratory test results, the CRC and underlying bituminous materials are considered bonded for 15 years and unbonded until the end of the pavement's theoretical service life. Shotblasting or hydroregeneration (water blasting) is carried out on the asphalt-treated base and the CRC layer before placing the upper layer (SHRP2 2009).

The Netherlands. Holland has constructed about 10 projects with two-layer porous AC surfacing over CRC. Researchers from SHRP2 R21 Composite Pavement visited some of these projects:

- A12 west of Utrecht, 1998
- A76, Schopol Airport, 8 km, 1991
- A5, Am Sk, 5 km
- A50, Einhoven, 35 km, 2004/05
- A73, Province of Limburg, between Venlo and Echt-Susteren, 2007, 42 km

Specific details on the A12 sections are included in Table 4.2.

Table 4.2. Summary of A12 design and as-constructed pavement properties (SHRP2 2009).

Parameter	Design Objective	Actual Realized
Thickness sand sub-grade	250 mm	261 mm
E-modulus sand sub-grade	100 MPa	>200 MPa
k-value sub-grade	0.045 N/mm^3	~0.070 N/mm^3
Thickness Lean Concrete base	150 mm	not realized
E-modulus base	9.000 MPa	-
Thickness AGRAC base	250 mm	256 mm
E-modulus AGRAC base	not designed	4.000 MPa
Composed k-value	0.118 N/mm^3	~0.160 N/mm^3
Thickness asphalt inter-layer	60 mm	approx. 60 mm
E-modulus asphalt inter-layer	unknown	7,500 MPa
Thickness concrete	252 mm non-reinforced	250 mm min./261 mm av.
Strength class concrete	C45	C57 average

Transportation officials in the Netherlands believe that the porous AC reduces noise by about 7dB, and the performance observed by SHRP2 suggested as much. The project were noted to have performed well to date, though projects were noted to begin raveling after approximately 10 years. Hence, based on observations in the Netherlands, replacement of the porous surface layer would need to occur every 10 to 12 years.

Figure 4.1. A12 between Utrecht and Ede, Netherlands, May 2008 (SHRP2 2009).

Figure 4.2. Close-up of the porous asphalt surface of the A12 motorway. Note fines infiltrating the porous asphalt surface on the edge of the traffic lane (SHRP2 2009).

While older (10 years or more) designs feature surface courses that are typically refreshed after 10 years, transportation officials in the Netherlands currently design new porous AC over CRC pavements with the plan to renew the porous AC layer every 7 to 8 years. Overall, the SHRP2 survey found that the pavements performed very well both in terms of distresses and noise.

United Kingdom. As of 2005, over 40 percent of new roads in the UK utilized a thin AC surface layer over CRCP. These thin AC surface layers can be between 20 and 35 mm in thickness and are reported to have a service life of approximately 12 years. In addition to the thin AC overlay (which is similar to an SMA in design), the UK also utilizes porous AC over CRC in pavements along freeways. This porous AC surface layer is typically 50 mm in thickness. While no performance data is available on these pavements, given that the maintenance of UK (and EU pavements in general) is to renew surfaces regularly, these designs are held in high regard in UK for their performance in terms of distress incidence and noise (FHWA 2007).

Italy. Italy has constructed many miles of CRC with a porous AC surfacing. The projects on the ring road around Rome carry extremely heavy traffic. About every 7 years, the porous AC is scheduled to be milled off and replaced due to clogging from fines and other problems. The total structural design life is 40 years (SHRP2 2009).

Chapter 5. Design & Modeling of AC Overlays on PCC Pavements

5.1 AASHTO 1986 & 1993 Design Guides

Witczak and Rada (1992) conducted a nationwide evaluation study of the different rehabilitation types such as crack and seat, break and seat, and rubblization. They also conducted field evaluations of the performance and in-situ structural properties of more than 100 projects. Based on the results they developed design procedures for Asphalt Concrete overlays of fractured PCC pavements based upon the pavement performance methodology presented in the 1986 AASHTO Guide. They found that an AC overlay of PCC (even if sufficiently cracked) does not behave like a flexible pavement; in addition thin AC overlays were found to be more susceptible to reflective cracking. They recommended fracturing the pavement slab to eliminate or reduce reflective cracking. However, as fractured slab fragments become smaller, E_{PCC} decreases; as a consequence, they recommended using a maximum value called E_{PCC} critical = 1000 ksi, to ensure that reflective cracking does not occur (Witczak and Rada 1992).

Hall et al. (1992a) published a paper on the extensive revisions to the AASHTO Overlay Design procedures. Their goal was to make these procedures more adaptable to calibration by local agencies and more comprehensive. The revised overlay design procedures used the concepts of structural deficiency and required future structural capacity determined from the AASHTO flexible and rigid pavement design equations. Hall et al. (1992a) developed seven separate overlay design procedures encompassing all of the combinations of overlay and pavement types including AC overlays on fractured and unfractured PCC.

For AC overlays on unfractured PCC they recommended that discontinuities and distresses of the underlying PCC pavement—such as deteriorated joints, cracks, and punchouts—be corrected with full-depth repairs prior to overlay in order to control reflective cracking. Each of the design procedures follows a sequence of eight steps, by which the required future structural capacity for the design traffic, effective structural capacity of the existing pavement, and required overlay thickness are determined. In addition the procedures provide detailed guidelines on several important topics related to overlay design, including overlay feasibility, structural versus functional overlay needs, pre-overlay repair, reflective crack control, and overlay design reliability level. Also included in the recommendations of Hall et al. (1992a) are detailed guidelines for pavement evaluation for overlay design, including distress surveying, nondestructive testing, and destructive testing (coring and materials testing).

After the revised procedures were developed, Hall et al. (1992b) proceeded to conduct field tests of the new procedures. Based on this experience, Hall et al. concluded that the revised AASHTO overlay design procedures produced reasonable overlay design thicknesses that were comparable with industry and State agency recommendations. The field examples illustrated the importance of selecting appropriate inputs for overlay design, the use of NDT, and condition data. In addition the significance of design reliability level to overlay thickness was found to be an important factor. A 95% reliability level was found to produce reasonable results for AC overlay on JRCP & AC on JPCP. Designing AC overlay thicknesses by the "condition method" and "NDT method" produced similar design thicknesses, though the authors recommended the NDT method (Hall et al. 1992b).

A recent overview of composite pavements conducted at Virginia Tech Transportation Institute (Gerado et al. 2008) discusses various methods used to design composite pavements. The report outlines the 1993 AASHTO Design Guide method for rehabilitating a PCC (JPCP or CRCP) pavement with an AC overlay depending on whether fracturing of the existing PCC slab is required. The methods are outlined below (Gerado et al. 2008, Fwa 2006):

- For fractured PCC pavement – The thickness of the AC overlay is determined by the Structural Number approach and is given as:

$$D_1 = \frac{SN_{OL}}{a_1} \tag{1}$$

where: D_1 is the required thickness of the AC overlay,
SN_{OL} is the structural number of the overlay, and
a_1 is the layer coefficient of the overlay material.

The structural number of the AC overlay is given as:

$$SN_{OL} = SN_T - SN_{eff} \tag{2}$$

where: SN_T is the structural number required if a new flexible pavement were to be constructed on the subgrade, and

SN_{eff} is the effective structural number of the existing pavement after fracturing.

The structural number of the existing pavement after fracturing is defined as:

$$SN_{eff} = a_2 D_2 m_2 + a_3 D_3 m_3 \tag{3}$$

where: a_2 and a_3 are the layer coefficients of the fractured slab and the base layer, respectively

D_2 and D_3 are the layer thickness of the fractured slab and the base layer, respectively and

m_2 and m_3 are the drainage coefficients of the fractured slab and the base layer, respectively.

- For un-fractured PCC pavement – A conventional PCC pavement is designed for thickness based on the estimated future traffic demand. Once the PCC thickness is known, it is assumed that placing an AC overlay would allow for decrease in PCC thickness using a conversion factor. The thickness of the asphalt overlay, D_{OL} is calculated as:

$$D_{OL} = A * (D_f - D_{eff}) \tag{4}$$

where: A is the conversion factor,

D_f is the PCC slab thickness to carry future traffic, and

D_{eff} is the effective thickness of the existing slab.

The conversion factor, A is defined as:

$$A = 2.2233 + 0.0099 * (D_f - D_{eff})^2 - 0.1534 * (D_f - D_{eff}) \tag{5}$$

The effective thickness of the existing slab, D_{eff} is defined as:

$$D_{eff} = F_{jc} * F_{dur} * F_{fat} * D \tag{6}$$

where: F_{jc} is the joint and crack adjustment factor,

F_{dur} is the durability adjustment factor,

F_{fat} is the fatigue adjustment factor, and

D is the original slab thickness.

Other empirical design methods such as US Army and Air Force Design, Illinois Department of Transportation Design, UK Pavement Design Guide, and Danish Road Institute are also discussed in the Virginia Tech report (Gerado et al. 2008), which should be referred to for detailed information regarding the design methodology of these methods.

5.2 2002 Mechanistic-Empirical Pavement Design Guide

The design methodology of Mechanistic-Empirical Pavement Design Guide (MEPDG) for predicting reflective cracking in AC overlays of PCC pavement is currently based on an empirical approach defined as follows (NCHRP 1-40B):

$$RC = \frac{100}{1 + e^{a(c)+bt(d)}} \tag{7}$$

where: RC is the percent of cracks reflected,

t is the time in years,

a, b are regression fitting parameters defined through calibration process, and

c, d are user-defined cracking progression parameters, shown in Table 5.1.

The regression parameters, a and b, are defined as:

$$a = 3.5 + 0.75(H_{eff}) \tag{8}$$

$$b = -0.688684 - 3.37302(H_{eff})^{-0.915469} \tag{9}$$

where: H_{eff} is the effective AC overlay thickness.

Table 5.1. Reflection Cracking Model Regression Fitting Parameters (NCHRP 1-40B).

Pavement Type	**Fitting and User-Defined Parameters**			
	a and b	**c**	**d**	
	H_{eff}		Delay Cracking by 2 years	Accelerate Cracking by 2 years
Flexible	$H_{eff} = H_{HMA}$	---	---	---
Rigid-Good Load Transfer	$H_{eff} = H_{HMA} - 1$	---	---	---
Rigid-Poor Load Transfer	$H_{eff} = H_{HMA} - 3$	---	---	---

Effective Overlay Thickness, H_{eff}, inches	---	---	---	---
<4	---	1.0	0.6	3.0
4 to 6	---	1.0	0.7	1.7
>6	---	1.0	0.8	1.4

NOTES:
1. Minimum recommended H_{HMA} is 2 inches for existing flexible pavements, 3 inches for existing rigid pavements with good load transfer, and 4 inches for existing rigid pavements with poor load transfer.

The underlying layers in an AC over PCC pavement undergo fatigue damage due to repeated traffic loading. The fatigue damage accumulation is defined by a damage index (DI_m) for each critical location by summing the incremental damage indices over time, as shown below:

$$DI_m = \sum (\Delta DI)_{j,m,l,p,T} = \sum \left(\frac{n}{N_{f-HMA}} \right)_{j,m,l,p,T} \quad (10)$$

where: DI_m is the damage index for month m,

ΔDI is the increment of damage index,

N is the actual number of axle load applications within a specific time period,

$N_{f\text{-}HMA}$ is the allowable number of axle load applications,

j is the axle load interval,

p is the axle load type (single, tandem, tridem, quad, or special configuration),

l is the truck type using the truck classification groups included in the MEPDG,

m is the month, and

T is the median temperature for the five temperature intervals or quintiles used to subdivide each month in °F.

The allowable number of axle load applications, $N_{f\text{-}HMA}$ is defined as:

$$N_{f-HMA} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{HMA})^{k_{f3}\beta_{f3}} \quad (11)$$

where: ε_t is the tensile strain at critical locations, in/in,

E_{HMA} is the dynamic modulus of the HMA measured in compression, psi.

k_{f1}, k_{f2}, k_{f3} are the global field calibration parameters (k_{f1} = 0.007566,

k_{f2} = -3.9492, and k_{f3} = -1.281).

$\beta_{f1}, \beta_{f2}, \beta_{f3}$ are the local or mixture specific field calibration constants; for the global calibration effort, these constants were set to 1.0.

C_H is the thickness correction term, dependent on type of cracking.

$$C = 10^M \tag{12}$$

where: $$M = 4.84\left(\frac{V_{be}}{V_a + V_{be}} - 0.69\right) \tag{13}$$

V_{be} is the effective asphalt content by volume, percent.

V_a is the percent air voids in the HMA mixture.

The resulting area of fatigue damage in the underlying layer, CA_m at month m is given by:

$$CA_m = \frac{100}{1 + e^{6-(6DI_m)}} \tag{14}$$

With the increment of damage for each month, the cracking area will also increase incrementally. Therefore, the total reflected cracking area is defined as:

$$TRA_m = \sum_{i=1}^{m} RC_t\left(\Delta CA_i\right) \tag{15}$$

where: TRA_m is the total reflected cracking area for month m.

RC_t is the percent cracking reflected for age t (in years), and

ΔCA_i is the increment of fatigue cracking for month i.

Galal and Chehab (2005) examined the evaluation, calibration, and validation of the MEPDG by comparing the predicted performance using the Mechanistic-Empirical (M-E) procedure with the in-situ performance of a rubblized continuously reinforced concrete pavement overlaid with 13 in. of asphalt concrete. The 1993 design of the pavement section was compared with the MEPDG design, and performance was predicted with the same design inputs. In addition, design levels and inputs were varied to achieve the following: (*a*) assess the functionality of the MEPDG software and the feasibility of applying M-E design concepts for structural pavement design of Indiana roadways, (*b*) determine the sensitivity of the design parameters and the input levels most critical to the MEPDG predicted distresses and their impact on the implementation strategy that would be recommended to INDOT, and (*c*) evaluate the rubblization technique that was implemented on the I-65 pavement section.

Galal and Chehab (2005) found that distress model calibration coefficients must be verified or modified for AC overlays of PCC to reflect the local conditions. In addition they observed the largest difference between predicted and observed distresses was the longitudinal cracking (top down) and recommended further study to explain this discrepancy. Furthermore they found that selecting the appropriate level of input significantly affects the predicted distress, especially selecting the same binder grade with and without test data. In addition accurate characterization of the in situ conditions (existing structure and associated layer thicknesses, as well as material types and characteristics) significantly affects the predicted distresses of the rehabilitation strategy selected. The rubblization technique was evaluated based on comparing observed distresses with those predicted by MEPDG ten years after construction, and the analysis showed, there was agreement on rutting, and fatigue, but there was an 18% difference in the thermal cracking distress. The authors attribute this difference to the need for thermal calibration and the lack of a reflective cracking model. The authors concluded that the use of higher level inputs (from good test and material data) and calibration of the coefficients to local conditions is necessary for successful implementation of the MEPDG. These conclusions held especially true for INDOT because none of its test sections were used in the determination of nationally calibrated default coefficient values (Galal and Chehab 2005).

Rodezno et al. (2005) assessed the method in which distresses were predicted by the MEPDG by comparing the predicted distresses with those observed in the field for two pavement sections in Arizona. The two sections compared were a conventional HMA reconstruction and an asphalt rubber overlay on a PCC pavement. Of the three parameters evaluated—rutting, cracking, and IRI—only rutting was accurate. Fatigue cracking predictions did not match field observations, and consequently the IRI predictions were different as well (Rodezno et al. 2005).

5.3 Caltrans Overlay Design Governed by Reflective Cracking

The California Department of Transportation (Caltrans 2001) has developed multiple design procedures for determining the thickness of an AC overlay. These thickness determinations are governed by factor such as structural adequacy, reflective cracking, and ride quality. To achieve a 10 year design life, a minimum thickness of 4 in. (105 mm) of dense graded asphalt concrete (DGAC) is recommended. However for a longer design life, or for placement of DGAC on a PCC pavement which has not been cracked and seated, a minimum thickness of 5 in. (135 mm) may be required. To achieve a 5 or a 20 year design life the thickness should be approximately 75% or 125% of the 10 year design life, respectively. The manual (Caltrans 2001) stresses the importance of engineering experience, and notes that exceptions to the guidelines will occur. In addition the final design should incorporate the following factors:

- Existing surface condition including types, sizes, and amounts of surface cracks and the extent of localized failures
- The structural condition, and age of the existing material
- Historical data related to thickness and performance similar rehabilitation activities
- Anticipated future traffic loads (Traffic Index).

5.4 CalME Models

The California Mechanistic-Empirical Design procedure (CalME) was developed by the University of California Pavement Research Center (UCPRC). CalME includes models for fatigue damage and the resultant cracking for asphalt concrete layers; and rutting deformations for asphalt and unbound layers. Like the MEPDG, the CalME models account for seasonal variation in stiffness of the unbound layers and the effect of temperature on the stiffness of the asphalt layer. In addition to these factors, CalME accounts for the effect of fatigue damage on the asphalt layer stiffness and the effect of stiffness of the upper layers on the stiffness of the underlying base and subgrade. The following sections detail the reflective cracking and rutting models.

5.4.1 Reflective Cracking Model

The reflective cracking model was based on critical strains in asphalt overlays over joints and cracks in the existing concrete pavement. The critical strains are computed using regression equations developed based on several finite element simulations (Wu 2005). The model employs a recursive-incremental approach with a time increment of 30 days. The CalME reflective cracking mode is:

$$\text{Cr m/m}^2 = \frac{10}{1+\left(\frac{\omega}{\omega_0}\right)^{-6}} \tag{16}$$

where: $Cr\ m/m^2$ is the cracking in m/m^2

ω is the damage in the asphalt layer,

ω_0 is a constant.

Fatigue damage of asphalt is determined using the following equation:

$$\omega = \left(\frac{MN}{MNp}\right)^{\alpha 2} \tag{17}$$

where: MN is the number of load repetitions in millions

$\alpha 2$ is a constant

MN_p is the allowable number of load repetitions in millions for the given asphalt stiffness and strain level defined as follows:

$$MNp = A'*\left(\frac{\mu\varepsilon}{\mu\varepsilon_{ref}}\right)^{\beta 2} * \left(\frac{E}{E_{ref}}\right)^{\gamma 2} * \left(\frac{E_i}{E_{ref}}\right)^{\delta 2} \tag{18}$$

where: E is the modulus of damaged material,

E_i is the modulus of intact material,

E_{ref} is the reference asphalt modulus

$\mu\varepsilon_{ref}$ is the reference asphalt strain in microstrain

$\mu\varepsilon$ is the strain at the bottom of the asphalt layer computed on the top of the crack or joint, and

$\beta 2$, $\gamma 2$, and $\delta 2$ are constants.

The tensile strain at the bottom of the overlay is determined using the following regression equation:

$$\varepsilon = \alpha_1 * E_{an}^{\beta 1} * E_{bn}^{\beta 2} * (a1 + b1 * \ln(LS_n)) * \exp(b2 * H_{an}) * (1 + b3 * H_{un}) * (1 + b4 * E_{un}) * \sigma_n \tag{19}$$

$$E_{an} = E_a / E_s \qquad E_{bn} = E_b / E_s \qquad E_{un} = E_u / E_s \qquad \sigma_n = \sigma_0 / E_s$$

$$LS_n = LS / a \qquad H_{an} = H_a / H_a \qquad H_{un} = H_u / H_a$$

where: E_a , H_a are the modulus and thickness of the overlay, respectively,

E_u , H_u are the modulus and thickness of the underlying layer, respectively,

E_b is the modulus of the base/subbase,

E_s is the modulus of the subgrade,

LS is the crack spacing,

σ_0 is the tire pressure,

a is the radius of the loaded area for one wheel,

α_1 = 342650, $\beta 1$ = -0.73722, $\beta 2$ = -0.2645,

$a1$ = 0.88432, $b1$ = 0.15272, $b2$ = -0.21632, $b3$ = -0.061, and

$b4$ = 0.018752.

The modulus of the asphalt layer is updated at the beginning of each increment based on the asphalt temperature, age, and damage from the previous increments using the following equations:

$$\log(E) = \delta + \frac{\alpha(1-\omega)}{1+\exp(\beta+\gamma\log(t_r))} \tag{20}$$

where: E is the dynamic modulus,

t_r is the reduced time which depends on the duration of loading, pavement temperature, and asphalt weight,

ω is the damage in the asphalt layer, and

α, β, γ, and δ are constants.

Equation (20) is a generalization of the equations for the dynamic modulus developed for the MEPDG. Indeed, if ω is equal to 0, i.e. material does not experience damage, then equation (20) reduces to the equation for the dynamic modulus in the MEPDG.

The CalME reflective cracking model has been calibrated using accelerated loading test data from the Caltrans heavy vehicle simulator (Ullidtz et al. 2006a).

5.4.2 Rutting Model

The CalME permanent deformation model, or rutting model, for the asphalt layer is based on shear deformation approach developed by Deacon et al. (2002). It postulates that the rutting will occur at the top 100 mm of asphalt layers. If the top 100 mm consists of more than one material, then rutting is computed separately for each of the sublayers, and then summed up. The permanent deformation of each sublayer is determined using the following equation:

$$dp_i = K * h_i * \gamma^i \tag{21}$$

where: h_i is the thickness of layer i (above a depth of 100 mm), and

K is a calibration constant = 1.4

γ^i is the inelastic shear strain in the asphalt layer defined as follows:

$$\gamma^i = \exp\left(A_3 + \alpha 3 * \left[1 - \exp\left({}^{-\ln(N)}\!\big/_{\gamma 3} \right) * \left(1 + {}^{-\ln(N)}\!\big/_{\gamma 3} \right) \right] \right) * \exp\left({}^{\beta 3 * \tau}\!\big/_{\tau_{ref}} \right) * \gamma^e \tag{22}$$

where: τ is the shear stress calculated using the layered elastic analysis program at 50 mm below the tire edge,

N is the number of load repetitions,

τ_{ref} is a reference shear stress (0.1 MPa ≈ atmospheric pressure),

A_3, $\alpha 3$, $\beta 3$, and $\gamma 3$ are constants determined from the Repeated Simple Shear Tests at Constant Height (RSST-CH), and

γ^e is the elastic shear strain determined as follows:

$$\gamma^e = \frac{\tau}{E_i / (1 + \nu_i)} \tag{23}$$

where: E_i is the modulus of layer i, and

ν_i is Poisson's ratio for layer i.

The rutting model also employs the recursive incremental damage approach because the asphalt layer stiffness depends on the pavement temperature, age, and fatigue damage from the previous increments. The model was calibrated using performance data from WesTrack (Ullidtz et al. 2006b).

5.5 Modeling and Predicting Reflective Cracking

Reflective cracking in composite pavements is the result of horizontal and vertical movements of the joints and cracks in the underlying PCC pavement caused by temperature cycles and cyclic traffic loading (Owusu-Antwi 1998). Contraction of PCC due to falling temperatures opens the cracks and joints inducing horizontal stress in the AC overlay. Traffic loadings cause differential deflections that result in shearing and bending stresses. Repeated traffic and/ or environmental loadings allow for crack initiation at the bottom of the AC layer which leads to crack propagation and growth. Large cracks can allow infiltration which can reduce the load bearing capacity of the pavement (Owusu-Antwi 1998). Two modes of loading are critical in reflective crack propagation: mode I, and mode II. Mode I (i.e. loads applied normal to the crack plane) is horizontal movement of the slab induced by temperature gradients and Mode 2 (i.e. in-plane shear) occurs as the crack faces slide against one another (Witczak and Rada 1992).

Sousa et al. (2001, 2002) surveyed layered elastic properties and historical cracking data for numerous projects over a 6- to 15-year period and concluded that the flexural beam fatigue test (AASHTO T 321) is a good empirical tool for the prediction of reflective cracking. Texas Transportation Institute's overlay tester is also showing promise as an empirical tool for the analysis of horizontal-only joint movements (Roberts et al. 1996).

Mechanistic-empirical models take advantage of fracture mechanics principles and can be used to analyze the mechanisms of load-associated reflective cracking. Luther et al. recommend the general concept of the strain-energy-density factor as the method of analysis for mixed mode fracture, however they did not consider the influence of thermal stresses on reflective cracking, nor did they develop a practical model for use by practicing engineers (Luther et al. 1976). Jayawickrama and Lytton (1987) continued this work and used fracture mechanics principles and beam-on-elastic foundation theory to develop a theoretical mechanistic model for crack growth analysis. The mechanistic model of Jayawickrama and Lytton can be used to predict the stress intensity factors in an idealized pavement for the three mechanisms of crack propagation in an overlay: bending, shearing, and horizontal thermal stresses. They also developed mathematical expressions that relate pavement variables such as layer thicknesses, moduli, and subgrade support to the stress intensity factors. Using this information and a two dimensional (plane stress or plane strain) finite element model they were able to predict the bending stress intensity factor (kb), shearing stress intensity factor (ks), and the stress intensity factor due to thermal damage (kt) for actual AC overlaid pavements. Finally, using regression analysis they were able to predict the number of 80kN load applications until failure, and more importantly they developed expressions for predicting overlay life by relating the number of 80kN load applications to the observed field distress data.

Owusu-Antwi et al. (1998) developed a mechanistic based model for predicting reflective cracking in AC-overlaid PCC pavements. The procedure they used to develop the model involved the use of fracture mechanics principles for analysis of crack propagation in composite AC/PCC pavements. Using a three-dimensional finite element model, mathematical expressions were obtained that can be used to calculate the *J*-integral for temperature and traffic loadings. This allowed the number of applications of temperature and traffic loads to failure to be calculated using Paris's crack growth law. For 33 LTPP AC-overlaid PCC pavement sections, the number of load applications to failure and the total damage caused by traffic and temperature loading were determined. Using Miner's cumulative damage hypothesis, the accumulated damage corresponding to the observed distresses was estimated. Optimization techniques were then used to obtain a mechanistic-empirical model that can be used to predict the percentage of reflective cracks in an AC-overlaid PCC pavement. Sensitivity analyses of the model indicated that the approach can be used to obtain a model that predicts reflective cracking reasonably well. The results have applications in performance predictions, pavement management, and cost allocation [2]. Owusu-Antwi et al. (1998) also noted that some of the variables that have been used to predict the length of medium and high severity cracks include ESALs, thickness of the overlay, freezing index, PCC condition prior to overlay with AC layer.

Wagoner et al. (2005) published their work on the use of a Disk Shaped compact tension test and cohesive zone model approach to characterize the fracture resistance of asphalt concrete. They noted that historically the single edge notched beam SE(B) test had been used to characterize fracture resistance properties. The SE(B) is very effective at characterizing various fracture properties, especially mixed mode fractures, but the geometry of the test specimens is not well suited for in-situ investigation of pavements. The use of field specimens is necessary to calibrate crack propagation models to field performance. In their study, Wagoner et al. applied the test to investigate premature reflective cracking of an isolated pavement in Rochester, New

York. They found that the surface layer containing RAP had significantly lower fracture energy than the underlying base and leveling course. They recommended that the New York State Department of Transportation reevaluate its design procedures for mixtures containing RAP, specifically they recommended a softer binder grade to compensate for the stiffening and embrittlement caused by RAP; they also made a recommendation on the limitation of RAP in surface courses. One major benefit of the DC(T) test that was demonstrated during testing of field samples was “the ability to obtain mixture fracture properties as part of an efficient suite of tests performed on cylindrical specimens” (Wagoner et al. 2005).

Nesnas and Nunn (2004) recently reported that cracking in the asphalt layer of an as-built AC/PCC composite system generally initiates in the upper layer and travels downward, contrary to asphalt overlays of cracked pavements where reflective cracks initiate at the asphalt-concrete interface and propagate upwards. The researchers cited field evidence in the investigation of over 50 recent cracks in nine as-built composite pavements, such as Figure 5.1, to support their claim, and developed a response model which treats the concrete and asphalt layers as a composite system that acknowledged the following:

- The asphalt thermal expansion coefficient is much higher than concrete
- Larger temperatures occur in the asphalt surface
- The surface of the asphalt layer becomes brittle due to age hardening

Figure 5.1. Core sample from recently initiated top-down crack in as-built AC/PCC (Nesnas and Nunn 2004).

The 3-D finite element model represented multiple slab elements as individual slabs, with a gap to represent a crack, introduced friction between the slabs and the layer underneath, and varied stiffness to simulate aging and thermal gradients. The results of the analysis with the model provided evidence to field observations, and showed that reflective cracking in as built composite AC/PCC pavements can initiate due to temperature effects alone. The model also showed that the onset of reflective cracking was dependent upon the aging of the binder, suggesting that proper binder choices may reduce the onset of reflective cracking in as built composite pavements (Nesnas and Nunn 2004).

5.6 Utilizing the Finite Element Method

A rational design method for a composite pavement structure has not been universally established and adopted, due in part to the difficulty in modeling the complicated mechanical behavior of the system. In the past, elastic plate theory and the idea of an equivalent layer (with the same bending rigidity as a two-layer composite plate) have been used to model the mechanical responses of a composite pavement; however these methods do not apply to cases where the plane configurations are dissimilar in the upper and bottom layer, or locations where joints and cracks are mismatched between the layers. It is also impossible to handle the different curling deflections due to the separation at the interface, or different temperature gradients between the two layers (Khazanovich 1994).

Ioannides and Khazanovich (1994) suggested treating a composite pavement as a multi-layered system consisting of two plate elements connected in series with springs, resting on a Winkler foundation. This method can model the deformations of each layer independently in the unbonded condition, as the springs model the bond condition at the interface and compressive deformation through the thickness of the layers. However, this method cannot model the bonded condition, where two plates deform identically and only one neutral plane forms.

Nishizawa et al. (1999) continued the work done by Ioannides and Khazanovich (1994) and modeled a bonded interface by using the increased bending rigidity due to one neutral plane. They implemented the model using the finite element method and verified the results with a laboratory experiment involving strain measurements of a composite pavement in the field. The field results agreed with the model's predictions validating the model. The authors noted the main features of their model included the following:

1. Upper and lower layers were modeled with rectangular elements connected with link elements
2. account for structural differences between the two different layers
3. account for different deformations of each layer independently
4. account for the separation between the two layers (evaluation of bond strength)

They also noted that separation due to a negative temperature gradient was more severe than that from a positive temperature gradient and indicated a need to further study the phenomenon.

Chapter 6. Performance and Rehabilitation of Composite Pavements

6.1 Non-Destructive Testing (NDT)

Ground penetrating radar (GPR) was successfully used to evaluate the asphalt thickness and the condition of the underlying concrete of three composite pavements across the country (Maser 2001). The evaluations were conducted at highway speeds so as not to interfere with the normal operation of the corridor. The results were correlated with data from core samples (shown in figure 6.1 below) and used for planning and scoping of future rehabilitation activities (Maser 2001).

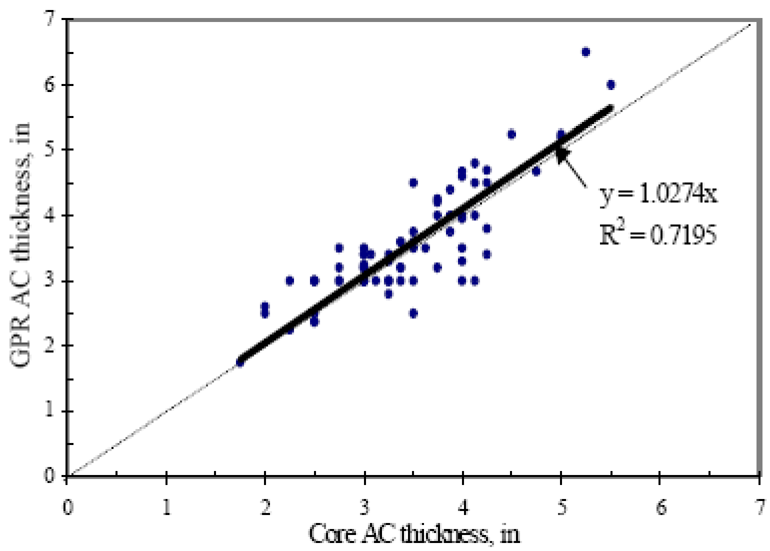

Figure 6.1. GPR vs. Core Thickness for Asphalt Overlay on I-90 (Maser 2001).

Williams et al. (2006) published Oklahoma's use of non-destructive test methods of the falling weight deflectometer (FWD) and ground penetrating radar (GPR) to analyze the structural capacity of the national highway system. The GPR and FWD data were used to determine homogenous sections enabling material to be extracted in representative locations. The NDT methods were also used to obtain thickness results which were compared with results from the materials sampling. When the AC layer was less than 1.5 in. thick, the predicted thickness of the AC layer (and sometimes of the PCC layer) differed from core sample thickness more often than when the AC layer was greater than 1.5 inches thick. This difference at times was more than one inch (Williams et al. 2006).

6.2 Performance & Rehabilitation

As with conventional pavements, composite pavements cannot be adequately rehabilitated unless rehabilitated unless the causes of the distress are identified and understood (Worting et al. 1998).

Table 6.1Table 6.1 describes the various distresses and causes, as identified by the Michigan Department of Transportation (MDOT), for composite pavement sections in various districts on Michigan state highways. Note that reflective cracking is listed a cause of six composite pavement distresses, this is the third most common cause of distresses preceded by temperature sensitive asphalt cement and low mixture tensile strength respectively (Worting et al. 1998). Tables 6.2 and 6.3 describe the various maintenance and rehabilitation alternatives available for MDOT engineers to use on composite pavements. Note that many of the preventive maintenance, rehabilitation, and reconstruction methods are similar to those used on conventional pavements, such as seal coats, patches, overlays, and crack sealing (Worting et al. 1998).

Table 6.1. Composite Pavement distresses and causes as defined by MDOT (Worting et al. 1998).

MDOT Composite Pavement Distress and Causes

Code	Distress	Causes of pavement distress																
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
0101	Transverse Tear									X						X		
0110	Transverse Crack (TC)									X					X	X		
0201	Longitudinal Crack - centerline									X					X	X		
0202	Longitudinal Crack - center of lane									X					X	X		
0203	Longitudinal Crack - edge of pavement									X					X	X		
0204	Longitudinal Crack - right wheel path									X					X	X		
0205	Longitudinal Crack - left wheel path									X					X	X		
0211	Zipper Crack	X								X								
0304	Repair Patch (white)												X					
0305	Repair Area												X					
0306	Shattered Area																	
0308	Longitudinal Repair												X					
0309	Overlapped Unsawn Patches												X					
0311	Miscellaneous Cracks		X				X					X						
0313	Repair Patch (black)												X					
0404	Surface Treatment Type																	
0405	Raveling								X									
0406	Flushing					X												
0601	Rutting (right)	X		X	X	X		X			X			X			X	X
0602	Rutting (left)	X		X	X	X		X			X			X			X	X

Causes
1. Base support insufficient
2. Delamination
3. Excessive amounts of filler material
4. Excessive amounts of non-angular aggregate
5. High asphalt content
6. High steel
7. Insufficient compaction
8. Low asphalt content
9. Low tensile strength mixture
10. Mix segregation
11. Poor freeze/thaw durablity
12. Poor materials
13. Reduction in air voids
14. Reflective Cracking
15. Temperature sensitive asphalt cement
16. Water sensitivity between aggregate and asphalt binder
17. Wrong grade of asphalt

Table 6.2. MDOT's various fix alternatives for Composite Pavements (Worting et al. 1998).

Fix type	Work category	Type of Work	Description	Fix Code
Composite to Flexible (CF)	Preventive Maintenance		Frost and Drainage Correction. Sub grade undercuts, replace underdrains and culverts, place aggregate base and bituminous surface.	1CF-01 {GA-KA}
	Rehabilitation	Overlay	Remove old bituminous, place OGDC interlayer on existing PCC slab, add underdrains, place bituminous concrete surface.	2CF-01 {JA-QB}
	Rehabilitation	Overlay	Remove old bituminous, rubblize the existing PCC slab, add underdrains, place bituminous concrete surface.	2CF-02 {KA-RC}
	Reconstruction		Remove existing asphalt, PCC slab, base, and subbase, place subbase, add drainable base and underdrains, place new asphalt courses.	3CF-01 {GA-SA}
	Reconstruction		Remove existing asphalt, PCC slab, base, and subbase, place subbase and filter cloth, add drainable base and underdrains, place new asphalt courses.	3CF-02 {LA-QA}
	Reconstruction		Reconstruct as a flexible pavement (grade lift). Place subbase, aggregate base, and new bituminous surface.	3CF-03 {GA-OA}
	Preventive Maintenance / Reconstruction		Frost and drainage corrections[1CF-01] and reconstruct as a flexible pavement [3CF-01].	6CF-01 {GA-SB}
	Preventive Maintenance / Reconstruction		Frost and drainage corrections [1CF-01] and reconstruct by grade lift [3CF-03].	6CF-02 {GA-OC}
Composite to Rigid	Rehabilitation	Overlay	Remove old bituminous surface, add underdrains, overlay with JRCP.	2CR-01 {KA-OB}
	Rehabilitation	Overlay	Remove old bituminous surface, sonic rubblize old PCC slab, add underdrains, overlay with JRCP (for transverse crack spacing >20 feet).	2CR-02 {OA-SC}
	Rehabilitation	Overlay	Remove old bituminous surface, add OGDC interlay and underdrains, overlay with JRCP.	2CR-03 {OA-SB}
	Reconstruction	Overlay	Remove old bituminous and old PCC slab, reconstruct as JRCP (place subbase, separator course, OGDC layer plus underdrains, and JRCP slabs).	3CR-01 {SA-UE}
	Reconstruction	Overlay	Remove old bituminous and old PCC slab, reconstruct as JRCP (place subbase, filter cloth, OGDC layer plus underdrains, and JRCP slabs).	3CR-02 {SA-WA}

Table 6.3. MDOT's various fix alternatives for Composite Pavements (cont. of Table 6.2)

Fix Type	Work Category	Type of Work	Description	Fix Code
Composite to Composite (CC)	Preventive Maintenance	Patching	Detail 6 Patches Detail 7 Patches Detail 8 Patches	1CC-01A {AA-AC} 1CC-01B {BA} 1CC-01C {CA}
		Type I	Concrete repair patches. Repair transverse cracks and joints in severe condition (distress > 4" wide and > 6' long). Repair severely distressed sawn patches. Repair overlapping unsawn patches.	1CC-02A {AA}
		Type II	Concrete repair patches. Repair transverse cracks and joints in severe moderate conditions(distress > 4" wide and >1" long). Repair severely and moderately distressed sawn patches. Repair overlapping unsawn patches.	1CC-02B {BA}
		Type III	Concrete repair patches. Repair transverse cracks and joints in severe, moderate, and low conditions (distress > 4" wide and > 1' long). Repair severely and moderately distressed sawn patches. Repair overlapping unsawn patches. All longitudinal cracks and joints distressed > 4" wide.	1CC-02C {CA}
			Prime and single seal existing pavement. Prime and double seal existing pavement. Seal coat. Microsurface (slurry seal). Thin overlay. Surface milling and thin overlay. Chip seal. Double chip seal. Crack seal. Crack fill.	1CC-04 {AA} 1CC-05 {AA} 1CC-06 {AA} 1CC-07 {AA-AD} 1CC-08 {AA-BC} 1CC-09 {AA-BL} 1CC-10 {AA} 1CC-11 {AA} 1CC-12 {AA} 1CC-13 {AA}
	Rehabilitation		Direct bituminous overlay. Remove old bituminous concrete, overlay with bituminous concrete.	2CC-01 {GA-LB} 2CC-02 {BA-LB}
	Preventive Maintenance / Preventive Maintenance		Detail 6 patches [1CC-01] and detail 7 patches [1CC-01BA]. Detail 6 patches [1CC-01] and detail 8 patches [1CC-01CA]. Detail 7 patches [1CC-01BA] and detail 8 patches [1CC-01CA]. Detail 6 patches [1CC-01], detail 7 patches [1CC-01BA], and detail 8 patches [1CC-01CA]. Crack seal [1CC-12] and crack fill [1CC-13].	4CC-01 {AA-AC} 4CC-02 {AA-AC} 4CC-03 {AA} 4CC-04 {AA-AC} 4CC-05 {AA}
	Preventive Maintenance / Rehabilitation		Detail 7 patches [1CC-01BA] and direct bituminous overlay [2CC-01]. Detail 7 patches [1CC-01BA] and remove old bituminous, overlay with bituminous [2CC-02]. Detail 8 patches [1CC-01CA] and direct bituminous overlay [2CC-01]. Detail 8 patches [1CC-01CA] and remove old bituminous, overlay with bituminous [2CC-02]. Detail 7and 8 patches [4CC-03BA] and direct bituminous overlay [2CC-01]. Detail 7 and 8 patches [4CC-03BA] and remove old bituminous, overlay with bituminous[2CC-02]. Type I repairs [1CC-02AA] and direct bituminous overlay [2CC-01] Type I repairs [1CC-02AA] and remove old bituminous, overlay with bituminous [2CC-02]. Type II repairs [1CC-02BA] and direct bituminous overlay [2CC-01]. Type II repairs [1CC-02BA] , remove old bituminous, and overlay with bituminous [2CC-02]. Type III repairs [1CC-02CA] and direct bituminous overlay [2CC-01]. Type III repairs [1CC-02CA], remove old bituminous, overlay with bituminous [2CC-02].	5CC-01 {GA-LB} 5CC-02 {BA-LB} 5CC-03 {GA-LB} 5CC-04 {BA-LB} 5CC-05 {GA-LB} 5CC-06 {BA-LB} 5CC-07 {GA-LB} 5CC-08 {BA-LB} 5CC-09 {GA-LB} 5CC-10 {BA-LB} 5CC-11 {GA-LB} 5CC-12 {BA-LB}

MDOT (Lee et al. 2001) also sought to establish a relationship between dynamic load-related distresses and surface roughness (IRI) in an attempt to establish roughness thresholds so that preventive maintenance treatments could be applied at a more optimal time. The study was able to establish a good linear fit for composite pavements with an R^2 value of 0.624 (Lee et al.

2001). However, it was shown that composite pavements would not benefit from this procedure as none of them were predicted to have a life extension greater than three years.

6.3 Survival of 1st, 2nd, 3rd AC/PCC Overlays

The Illinois Highway Research Study IHR 532 was a four year study conducted by the University of Illinois for the Illinois Department of Transportation (IDOT) which had the goal of developing project level guidelines for the evaluation, and the selection and design of rehabilitation strategies for existing AC/PCC composite pavements (Hall et al. 1995, Gharaibeh et al. 1997). The guidelines were based on the original pavement type (i.e., JRCP or CRCP) and the condition, including whether or not the pavement had D-cracking, and sought to find the most cost effective treatment options based on life cycle cost analysis. Figure 6.3 to Figure 6.5 depict the five suggested rehabilitation strategies. The cost analysis was conducted over a fifteen year analysis period, and did not include user costs associated with lane closures during construction, or those associated with increased pavement roughness.

Note that rehabilitation strategy four (Figure6.4) is an unbonded concrete overlay, and rehabilitation strategy three (Figure 6.3) is an asphalt overlay placed on a rubblized concrete pavement. These strategies are recommended only when the structural capacity of the underlying concrete pavement has diminished considerably, and were recommended over the second generation 3.25 in. AC overlay as the most cost effective option in the vast majority of cases. However, when the 3.25 in. AC overlay was compared with the 5 in. AC overlay, the additional structural capacity of the thicker overlay yielded a longer service life, which made it more cost effective in the long term (Gharaibeh et al. 1997).

The study also discovered that there was a strong relationship between the pavement type and the choice of the second generation rehabilitation strategy. The 5 in. AC overlay was chosen for the majority of JRCP and CRCP pavements; however for CRCP pavement that exhibited D-cracking, the 5 in. AC overlay was found to be cost competitive with the unbonded PCC overlay (Gharaibeh et al. 1997).

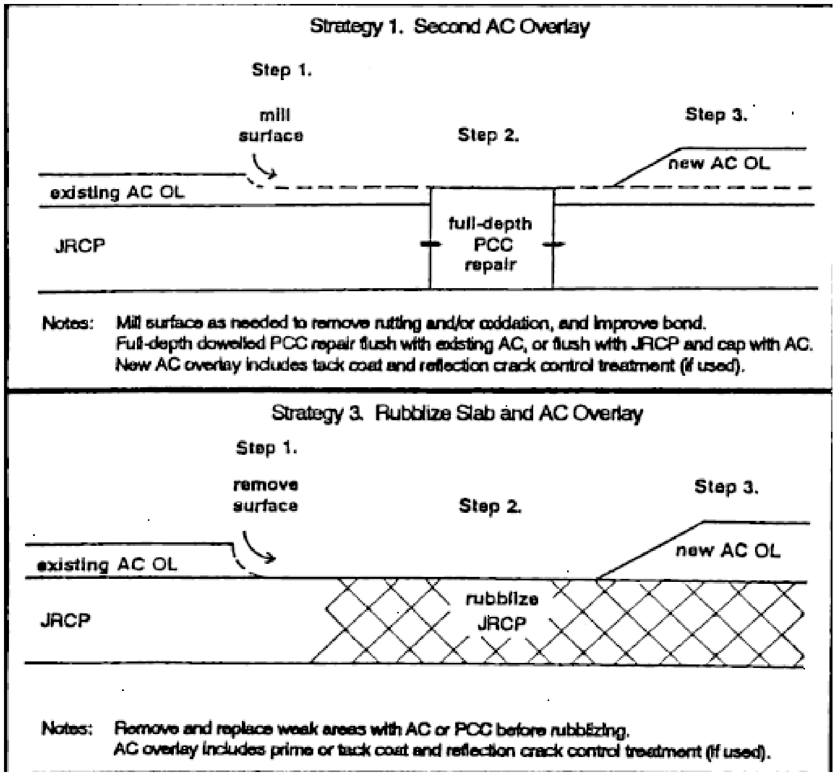

Figure 6.2. Composite Pavement Rehab Strategies 1 and 3 (Gharaibeh et al. 1997).

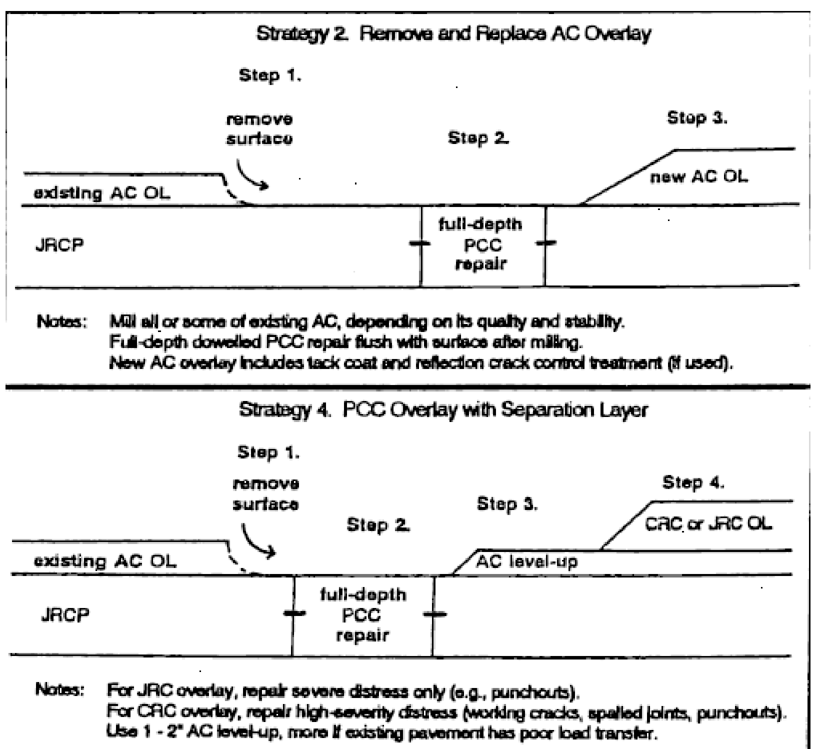

Figure 6.3. Composite Pavement Rehab Strategies 2 and 4 (Gharaibeh et al. 1997).

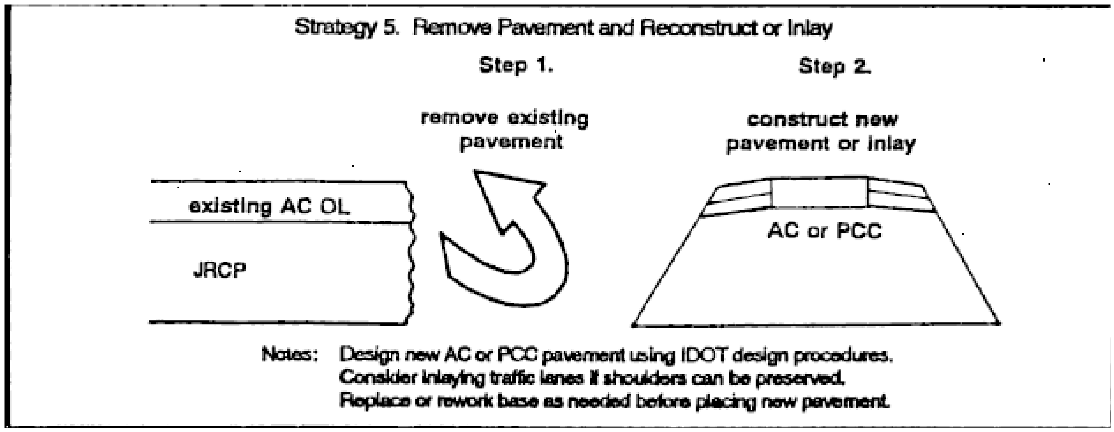

Figure 6.4. Composite Pavement Rehab Strategy 5 (Gharaibeh et al. 1997).

A report on life cycle cost analysis (LCCA) prepared for the Ontario Ministry of Transportation (MTO) (Smith et al. 1998) examined the most cost effective time to place an AC overlay over an existing JPCP and AC/PCC composite pavements. The report cited the use of a calibrated NAPCOM reflective cracking model to predict the time (in years) until a critical number of medium and high severity cracks would appear. The 1993 AASHTO design guide (AASHTO 1993) was then used to correlate the number of cracks to a serviceability loss, and using a critical serviceability level of 2.9 (45 medium and high severity cracks/km), an annual rigid ESAL level of 3 million the overlay was predicted to last 5.5 years without preventive maintenance, and 8.5 to 9 years with preventive maintenance. Conducting a failure analysis yielded a longer service life of 13.5 years, to offset the different results a mean value of 11 years with a standard deviation of 2.3 years was recommended for the LCCA analysis.

There was no historical data for AC overlays of AC/PCC composite pavements, however MTO typically mills off the 80 mm AC wearing course before placing the overlay, thus the first overlay of a composite pavement was predicted to last as long as the first overlay of a JPCP (11 years with a standard deviation of 2.3 years). The expected life of the second AC overlay, based on failure analysis, was found to be very close to that of the first AC overlay (13.5 years) even though the second overlay was expected to carry approximately 74% more ESALS a similar finding was also reported in Illinois (Hall and Schutzbach 1997). Consequently, the report recommended using 11 years with a standard deviation of 2.4 years as the expected service life of a second AC overlay placed over a JPCP. The report also noted the requirement of continual maintenance of the rehabilitation including patching and joint treatments to obtain the maximum benefit.

Chapter 7. Summary and Conclusions

In summary, while the construction and design methods, performance measures, and modeling tools presented in this literature review have informed the on-going work of the TICP research team, it is clear that this review is by no means comprehensive and final. As future task research is conducted and team members come into contact with other pavement engineers with AC overlay experience, the team members expect that this experience and the research that experience is built upon will prove valuable to the TICP project work. Hence, though the literature review officially concludes the "information gathering" stage of the project, the research team expects (and hopes) that the spirit of the literature review will continue through the duration of the project work.

The literature review work determined that there exists a lack of reliable models for composite pavements. In addition, the literature review points out that the mechanism of failure of as-built AC/PCC composite pavements is different than that of AC overlays of existing PCC pavement. It should also be noted that the construction and design challenges of AC-over-PCC pavements, whether rehabilitated or as-built, will also be addressed indirectly by the on-going work of the SHRP2 R21 Composite Pavement research project. The proximity of the SHRP2 R21 work and the TICP work both in subject matter and location (both feature test sections at MnROAD) make the projects natural partners. The research team anticipates that results from SHRP2 R21 will continue to be beneficial to the TICP work.

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