Guidelines for the Rehabilitation of Concrete Pavements Using Asphalt Overlays

Final Report

FHWA TPF-5(149)

Design and Construction Guidelines for Thermally Insulated Concrete Pavements

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Chapter 1. Introduction

The Federal Highway Administration (FHWA) Pooled Fund TPF-5(149) research project was initially focused on thermally insulated concrete pavements (TICP), a subclass of composite pavements that involve the construction or rehabilitation of Portland cement concrete (PCC) pavements with asphalt concrete (AC) overlays. In addition to the project's primary focus on TICP, much of the project research included a significant literature review and investigation into general composite pavements – i.e. PCC concrete pavements rehabilitated with AC overlays (henceforth AC-over-PCC or simply AC-PCC), as in Figure 1.

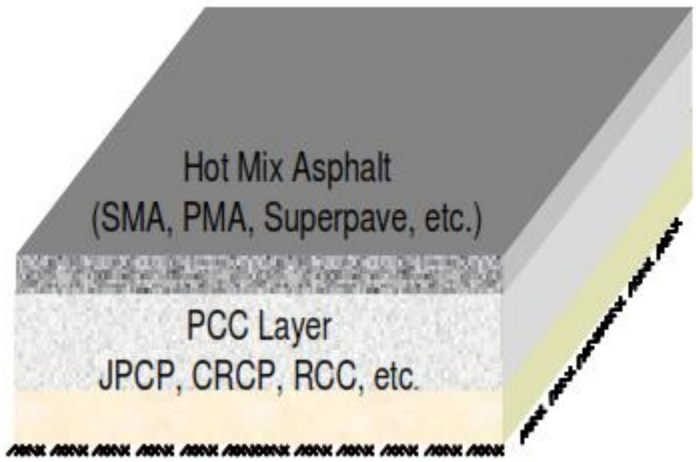

Figure 1. Typical cross section of an AC overlay over a PCC pavement (from Rao et al. 2011)

As this general knowledge would have been largely set aside in narrowing the project scope to solely consider TICP, the TPF-5(149) panel amended the project work to include a task to document the general AC-over-PCC knowledge developed during the course of TPF(5)-149. The result of that task is this synthesis.

1.1 Brief Overview of AC-over-PCC Implementation and Research

AC-over-PCC has been recognized as a viable means of rigid pavement rehabilitation for many years. As a result of the large number of existing AC-over-PCC pavements, pavement engineers are very familiar with the process of rehabilitating rigid pavements using AC overlays. The AC overlay practices of many state departments of transportation (DOT) in the United States can be found in Gopal (2010). This and other resources detail the breadth of the DOT experience designing and constructing AC overlays for existing rigid pavement.

In addition to familiarity through practice, some pavement research projects have included AC-over-PCC as a focus. One such research study is the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) program. The LTPP pavement test sections are divided into General Pavement Studies (GPS) and Specific Pavement Studies (SPS), and GPS and SPS sections that concern AC-over-PCC are:

GPS-7. Hot Mix Asphalt (HMA) Overlay of PCC Pavements
SPS-6. Rehabilitation Techniques Using HMA Overlays of PCC Surfaced Pavements

Each of these experiments includes one of four environments (1. Wet, Freeze; 2. Wet, No Freeze; 3. Dry, Freeze; and 4. Dry, No Freeze); traffic levels expressed in ESALs; layer structural thicknesses; and many other variables (Hall et al. 2005). Pavement researchers have used the LTPP GPS-7 and SPS-6 data to better determine the best design practices and performance of AC-over-PCC. Later sections will summarize a few of the relevant LTPP analyses that concern AC-over-PCC data

1.2 AC-over-PCC as Pavement Preservation, Preventative Maintenance, and Pavement Rehabilitation

Through observations gathered from implementation and research, pavement engineers have developed a number of definitions for pavement construction techniques and philosophies. The following definitions are relevant to the AC-over-PCC guidelines for TPF-5(149). Note that these definitions and discussion are adapted directly from the Federal Highway Administration (FHWA) 2005 memorandum that closely defines these terms for pavement engineers (FHWA 2005).

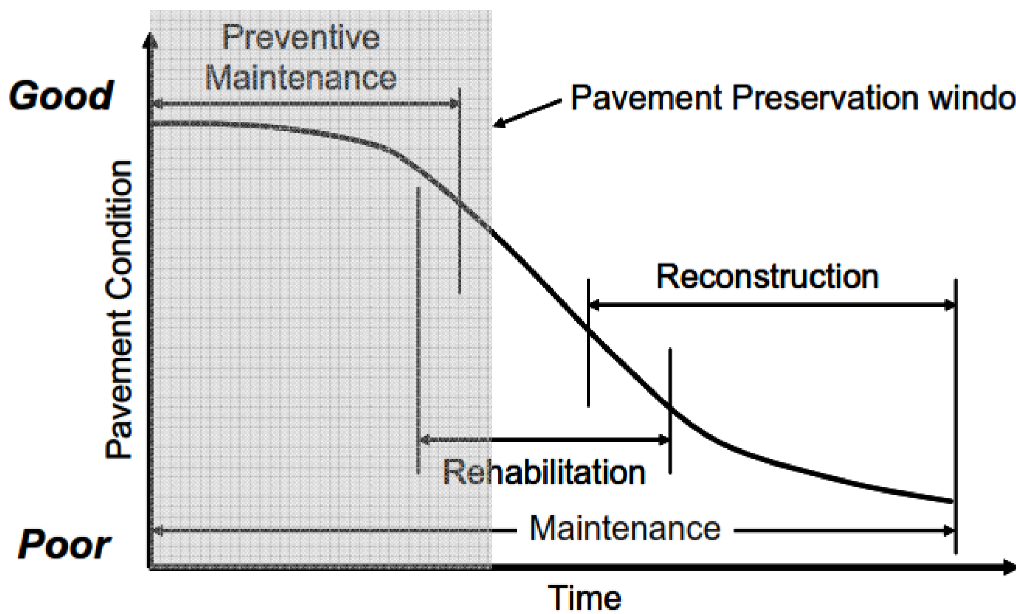

Figure 2. Terms for pavement improvement efforts relative to condition over time (from Smith et al. 2008)

Pavement preservation is a coordinated long-term effort to enhance pavement performance using practices that extend pavement life, improve safety, and provide a quality roadway for users. Pavement preservation involves the application of preventive maintenance, nonstructural rehabilitation, and other routine maintenance activities. While pavement preservation restores the function of the existing system and extends its service life, it does not significantly increase the system's load-carrying capacity or strength.

Preventative maintenance is a strategy of applying treatments to existing roadways that preserve the system, slow the rate of roadway deterioration, and maintain or improve the functional condition of the system without increasing the structural capacity of the pavements. Preventive maintenance is usually applied to pavements in good condition that still have service life remaining. As a major component of pavement preservation, preventive maintenance extends the service life by applying treatments to the surface or near-surface of structurally sound pavements. One obvious example of a preventive maintenance treatment can be thin or ultra-thin AC overlays. For more examples and discussion, consult the memorandum referenced above (FHWA 2005).

Pavement rehabilitation refers to "structural enhancements that extend the service life of an existing pavement and/or improve its load carrying capacity" (FHWA 2005). Relevant enhancements to the TPF-5(149) project are obviously structural AC overlays. These rehabilitations are designed to extend the life of the existing pavement by restoring structural capacity through the elimination of cracking or by increasing pavement thickness to accommodate existing or anticipated traffic loads.

Pavement rehabilitation strategies are divided into minor and major rehabilitation, the distinction being made by the change to the structural capacity as a result of the rehabilitation effort. *Minor rehabilitation* consists of non-structural enhancements made to the existing pavement sections to eliminate cracking. As minor rehabilitation techniques are non-structural, they are typically categorized as pavement preservation efforts. On the other hand, a *major rehabilitation* is a project that extends the service life of an existing pavement and/or improves its load-carrying capability through direct structural modification. For more information on this distinction, consult the FHWA memorandum referenced at the outset of this subsection (FHWA 2005).

Both preventative maintenance and pavement rehabilitation projects can be categorized as pavement preservation efforts. Furthermore, there can be overlap between maintenance and rehabilitation, and AC-over-PCC is a good example of the kind of project that can serve several functions depending on the particular needs of the existing pavement. Later sections will detail the differences in AC overlay structural and mix design for maintenance and rehabilitation efforts.

1.3 Use and Benefits of AC-over-PCC

Asphalt overlays of concrete pavements (AC-over-PCC) are not only used to improve the functionality of a distressed pavement. Overlays such as open graded asphalt friction courses or porous asphalt overlays can be used to reduce noise, improve skid resistance, and improve ride quality. Asphalt overlays, whether they are placed for structural reasons or noise/friction reasons, can be placed over pavements in various degrees of distress. No matter the condition, the overlay will perform better if the existing pavements are properly prepared, and an assessment should be made to determine that the overlay rehabilitation is more economical than rebuilding the pavement.

The benefits of using AC-over-PCC coincide with the benefits of any standard pavement preservation effort. For more information on these benefits, the reader may consult the FHWA

Pavement Preservation Compendium, whose collected articles detail the economic, infrastructure, and implementation advantages to using pavement preservation efforts (FHWA 2006). However, there are additional advantages provided by AC-over-PCC that have been observed as a result of the Long-Term Pavement Performance (LTPP) general pavement study of AC overlays of PCC pavements (GPS-7) and frequent application of AC-over-PCC by State DOTs.

The most notable benefit of AC overlays is that they provide a cost-effective method of rehabilitating an existing roadway. Another major benefit of AC overlays is the convenience and speed of overlay construction. Whether applied to flexible or rigid pavements, AC overlays can be constructed with little need for major traffic obstruction or redirection.

Furthermore, the use of the overlay itself, rather than more costly and involved alternatives (e.g. full-depth repair), is another benefit. Whereas more expensive options can provide the needed structural performance, often a major rehabilitation involving a simple AC overlay of the appropriate thickness can save an agency both time and money in the short term. As implied by the definitions above, another benefit of AC overlays is the versatility of the overlay, which can serve either as a maintenance technique to remediate unexpected environmental or traffic loading or as a rehabilitation technique to extend the service life of an existing pavement.

1.4 TPF-5(149) Project Summary

The FHWA Pooled Fund Project TPF-5(149) focused on the design, cost analysis, construction, and analysis of AC overlays of newly constructed PCC pavements. These pavements were termed “thermally insulated” in light of the benefits of the AC overlay relative to environmental/climatic effects on performance. Thermally insulated concrete pavements (TICPs) consist of a concrete pavement structure (jointed or continuously reinforced) covered by an asphalt layer during construction (before opening to traffic) or soon after construction to address ride quality or surface characteristic issues.

TICPs combine the structural longevity of PCC pavements with the serviceability of AC pavements. One of the perceived benefits of TICPs was the simplification of the PCC design and construction through a thinner PCC layer due to reduced stresses in the concrete from the insulating effects of the asphalt layer, simplified finishing and simplified joint formation techniques. The main objective of the TPF-5(149) research was to perform life cycle cost analysis comparisons and develop design and construction guidelines for TICPs. The study initially had the following secondary objectives:

1. Validation of the structural and climatic models of the Mechanistic-Empirical Pavement Design Guide (MEPDG) for asphalt overlays of concrete pavements.
2. Investigation of applicability of the MEPDG for design of TICPs.
3. Investigation of applicability of reflective cracking and asphalt rutting models developed in California.
4. Development of recommendations for feasibility analysis of newly constructed TICPs or thin overlays of the existing concrete pavements.

As discussed in the Introduction, an additional fifth objective, added as the TPF-5(149) project research was in process, is the development of an overview of the evaluation, design, and rehabilitation of concrete pavements using AC overlays. This synthesis is the fulfillment of that final objective.

1.4 Synthesis Outline

The issues relating to AC-over-PCC summarized in this synthesis are organized as follows.

- Chapter 2 identifies methods and tools used to evaluate an existing PCC pavement for rehabilitation.
- Chapter 3 details methods of repair and preparation of an existing PCC pavement ahead of AC overlay construction.
- Chapter 4 notes special considerations in the AC mix design of overlays to be used to rehabilitate PCC pavements.
- Chapter 5 summarizes the most popular mechanistic-empirical pavement design methods for AC-over-PCC.
- Chapter 6 describes the current practice of AC overlay construction.
- Chapter 7 summarizes performance evaluation studies conducted on AC-over-PCC, particularly those describing LTPP sections.
- Chapter 8 presents case studies in the construction and/or performance of AC-over-PCC as developed by state DOTs.

The synthesis report describes the evaluation, preparation, design, and construction of AC overlays of jointed plain concrete pavements (JPCP) in keeping with the focus of the TPF-5(149) project. However, many of the techniques summarized here are applicable to other kinds of concrete pavements (e.g. continuously reinforced concrete), and the resources pointed to in this summary can be consulted for more details on AC overlays of non-JPCP.

Chapter 2. Existing Concrete Pavement Evaluation

The purpose of the concrete pavement evaluation is to determine the condition of the existing pavement. This allows an engineer to decide if pavement preservation, maintenance, or rehabilitation is required. There are two major focuses of the evaluation process: the structural condition of the existing pavement and the functional condition. Based on the pavement's current condition, it can be determined which treatment would be most appropriate to prolong the pavement life and how that treatment must be implemented. It is important to properly determine the types of distresses in the current pavement and their causes because any repair or rehabilitation must address the causes of current distress to be successful. The initial evaluation should be conducted early enough in the pavement's life that it falls within the pavement preservation window, as shown in Figure 2. This ensures that there will be time to rehabilitate the pavement before it is so distressed that replacement is the only cost effective option

2.1 Evaluation Procedure

Both the structural and functional evaluations follow the same basic procedure. The evaluation begins in the office with an examination of historic records. From documents such as the original construction plans and field reports, as well as any previous damage surveys and pavement evaluations, an engineer can begin to form an overall vision of the project. Historical information can also be used to eliminate the need for certain types of tests. For example, if in the course of a previous evaluation, the thickness of the pavement has been determined at various locations, testing does not need to be conducted again to determine the pavement thickness unless the pavement is known to have experienced extensive repairs that warrant repeated measurement.

Similarly, if tests have already determined that certain issues exist, and these issues have not been addressed, then the engineer knows in advance that those problems will be present, though they may have increased in severity. In this case, special attention should be given to avoid neglecting previously known problems with the pavement. For example, if it was determined in prior evaluations that the pavement has poor drainage, and no remedial actions have been taken, special care should be taken to evaluate the severity of the lack of drainage, as it is unlikely to have improved on its own. In this way, careful consideration of historic documentation can help prevent unnecessary testing and alert engineers to potential problems already known to exist in the pavement (Miller and Bellinger 2003, NCHRP 2004a).

Once records of the pavement have been examined, an initial visit to the project site can be conducted. This visual evaluation can help to determine the extent to which testing will need to be conducted. As this is a very preliminary evaluation, it can be conducted by simply driving over the pavement and noting observations, including the quality of the ride and any very apparent distresses. A more thorough visual inspection can be performed as part of a damage survey (discussed in detail below) to identify the types of distresses present, the extent of the damage, and potential causes of the distress.

The initial site work may also include a review of the profile along the length of the roadway. By examining the profile of the entire roadway, areas with significant movement of the concrete panels can be detected for the benefit of close structural evaluation, as these movements can indicate unstable or non-uniform conditions in the layers below the concrete (Miller and Bellinger 2003).

The results of this initial visit, coupled with the examination of the historic documentation can be used to divide the pavement into discrete, similar sections based on their structure or distress type. Different sections may require different types of testing or test intervals based on their condition and the anticipated intensity of the testing regime required to collect a sufficient amount of data (Miller and Bellinger 2003, NCHRP 2004a).

Once the pavement has been divided into sections for testing, the structural and functional evaluations can begin. These evaluations both involve collecting data in the field through various forms of testing, which can include non-destructive testing (NDT) using a Falling Weight Deflectometer (FWD), skid testing for friction, etc. Figure 3 details various in-place evaluation methods used by 26 state DOTs surveyed to evaluate existing pavements, where abbreviations in the figure are coring and sampling (C&S); ground penetrating radar (GPR); dynamic cone penetrometer (DCP); visual distress surveys (VDS); traffic counts and vehicle classification (TRAF); and laboratory testing (LAB). The techniques in Figure 3 will be discussed in later subsections on evaluations for the rehabilitation of PCC using AC overlays. The data collected using these techniques is analyzed to determine the pavement condition

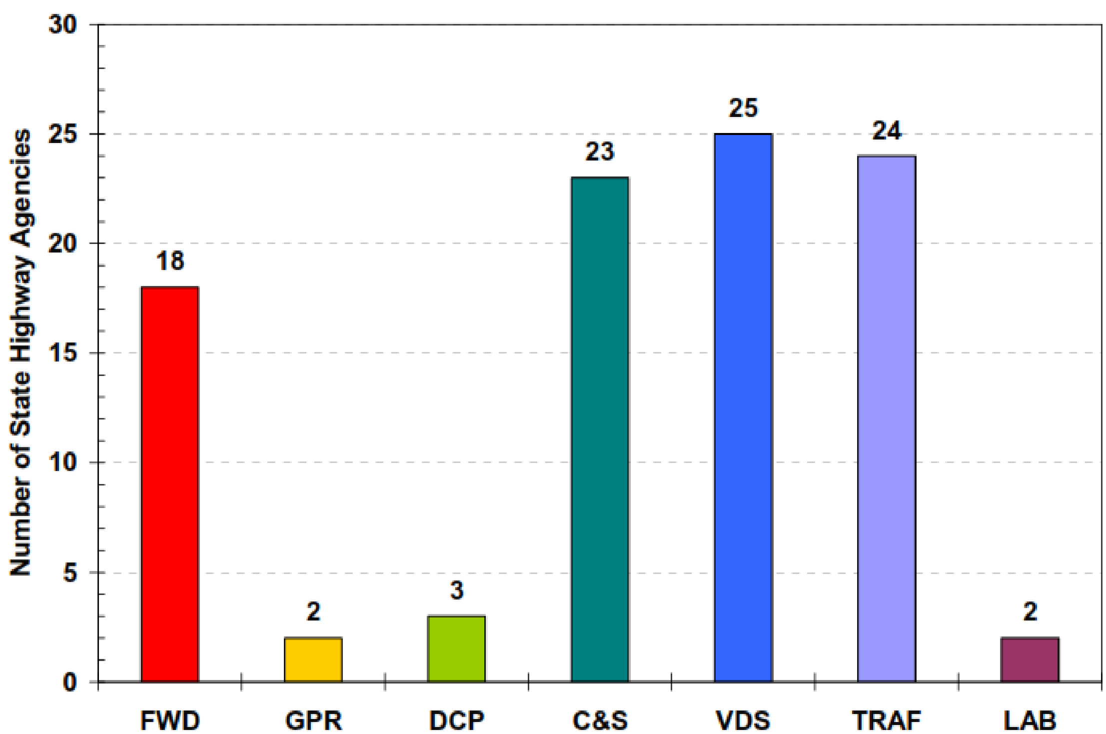

Figure 3. State agency NDT usage to determine the pavement condition as part of existing pavement evaluation (from Bennert 2009)

From the results of the structural and functional evaluations, decision metrics can be used to determine if rehabilitation is required. In some states, for the rehabilitation of JPCP, joint deflections in excess of a predetermined threshold indicate the need for dowel bar retrofits or undersealing, while a load transfer efficiency less than a certain amount indicates the need for joint rehabilitation. The results of the pavement evaluation can also be used in the design of the AC overlay. For example, 72% of states using FWD as part of their pavement evaluation used that information directly as a design input (Bennert 2009).

2.2 Structural Evaluation

The focus of the structural evaluation is to determine the structural condition of either the existing pavement, including the extent of damage, and a determination of the remaining life of the pavement (Hall et al. 2001). The two major categories of structural evaluation are surveys and testing. Generally, surveys are conducted to determine the extent of damage and the condition of drainage in the pavement based on visual surface assessment. Testing is conducted to determine the structural condition and extent of damage which cannot be established through a visual assessment of the top surface of and adjacent areas surrounding the pavement. Non-destructive testing can be used gather data without damaging the pavement. A structural evaluation can be conducted on both a concrete pavement and a pavement which has already been overlaid with asphalt.

2.2.1 Distress and Damage Survey

A distress survey on an existing concrete pavement notes any visible distress, along with its location, quantity, and severity along the length of the pavement. Numerous distress manuals exist to define the different distress types and how to rate their severity, such as the Long Term Pavement Performance (LTPP) (Shahin et al. 1976; Miller and Bellinger 2003) and Army Corps of Engineers (WSDOT 2004) distress manuals. Additionally, different states have their own distress rating standards. While the LTPP distress ratings are very detailed and intended for research purposes, the Army Corp pavement condition index (PCI) rating system is intended for pavement management. In this system, a perfect pavement is assigned a value of 100. The presence of distresses results in a lower PCI, depending on their type and severity.

To conduct a manual distress survey, an inspector must walk the entire length of the pavement, measuring and recording distresses along the way (Figure 4).

Figure 4. At left, inspector conducting a manual distress survey; at right, van outfitted to conducted a video distress survey (from Al-Qadi et al. 2009)

If the roadway carries a large amount of traffic, the section of pavement being evaluated may need to be closed (Hall et al. 2001). Distress surveys may also be conducted via video using a van which has been outfitted to record an image of the pavement as it is driven over the roadway, as shown in Figure 4 at right. An example of the visual documentation produced during a manual distress survey is given in Figure 5.

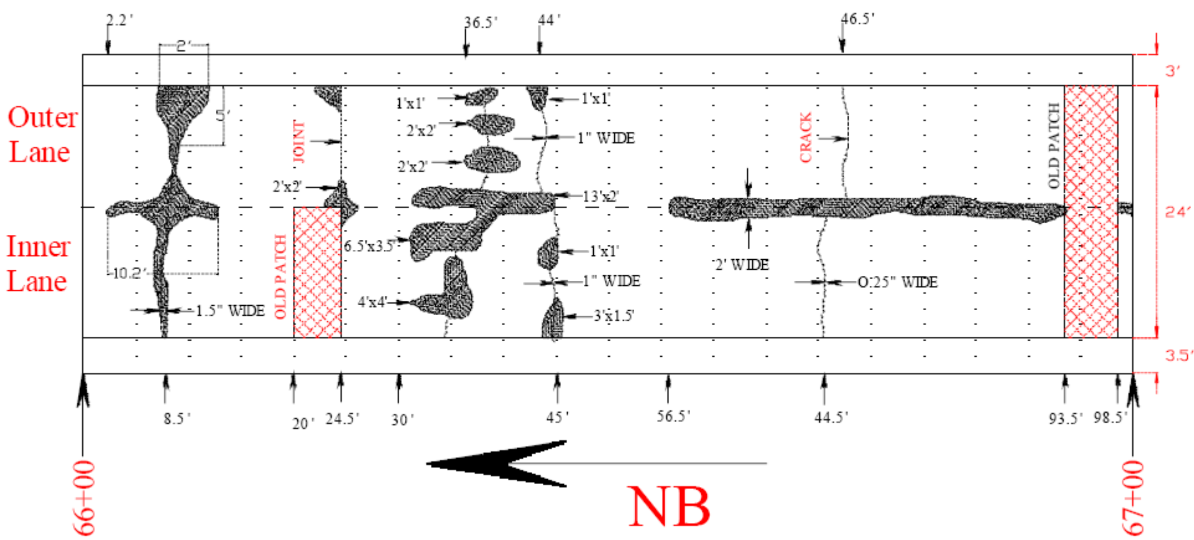

Figure 5. Example of a distress survey (from *Wen et al. 2005*)

A drainage survey is generally conducted in addition to a distress survey, as moisture is a frequent cause of damage in a pavement. It is important to understand the drainage system of the pavement, because, if the current distresses are caused by the presence of moisture in the pavement due to drainage problems, and these problems are not corrected when the pavement is repaired, they will reemerge.

The drainage survey involves walking along the length of the pavement while making visual assessments, so it can be conducted concurrent with the distress survey. A drainage survey should note the geometric properties of the pavement which induce or impede drainage, such as the topography, transverse slope, and adjacent ditches. Additionally the condition of ditches and any drainage inlets or outlets and edge drains should also be noted (Hoerner et al. 2001). In an advanced drainage survey, video equipment can be snaked down the drains to inspect their condition more thoroughly, as shown in Figure 6.

Figure 6. Video equipment for an advanced driainage survey (from Christopher and McGuffey 1997)

To present the data from both the drainage survey and the distress survey, results can be plotted on a diagram of the roadway. A diagram of the severity of damage along the length of the road way in addition to the local drainage condition and traffic levels can give insight as to the causes of different distresses, as well as indicate locations in need of more testing or maintenance. This information can also be used to refine the delineations of different pavement sections based on structural condition made in the preliminary investigation (Miller and Bellinger 2003).

In addition, plotting pavement distress, as in Figure 5, is useful to show specific distresses and their locations. However, to estimate the overall pavement condition for large-scale rehabilitation decisions, it is helpful to create strip charts. Strip charts can be used to display the potential relations between traffic, drainage, and pavement condition. A sample strip chart showing pavement cracking is shown in Figure 7.

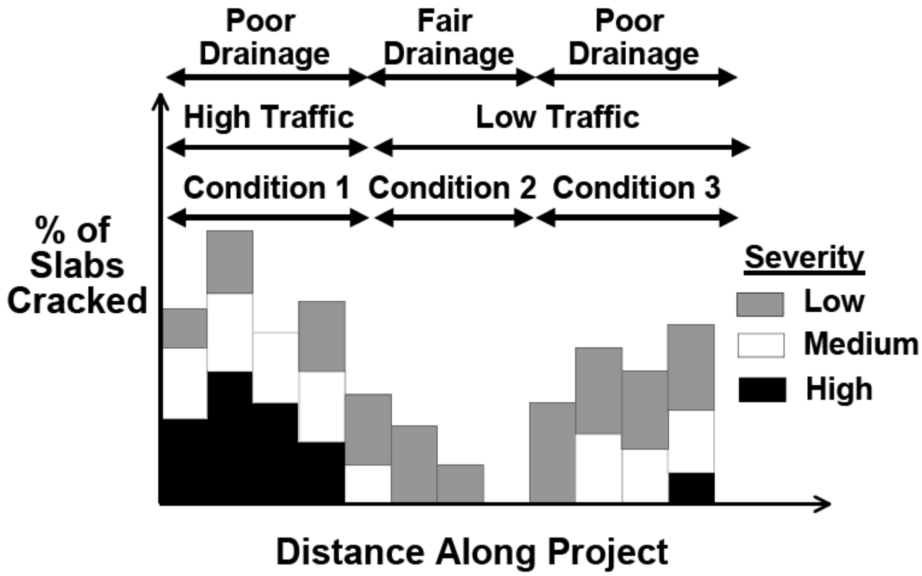

Figure 7. Example strip chart used to visual distress levels in different areas of a project (Smith et al. 2008)

Superimposed on Figure 7 are the delineations between traffic levels and drainage conditions. Similar plots can be made for additional distresses or measurements such as faulting, deflections, sunken slabs, etc. Plots can also be made showing functional characteristics such as roughness and friction as described in the following section. In reviewing these charts, it will be readily apparent how to delineate areas of varying pavement condition and to locate potential subsequent rehabilitation strategies on a project level.

2.2.2 Non-destructive testing

After the drainage and distress surveys have been conducted, non-destructive testing (NDT) can be used to determine additional information about the pavement which cannot be found through visual means. This information includes the elastic modulus of the concrete, the modulus of subgrade reaction, the presence of voids under the slab, and the load transfer efficiency. Conducting these tests along the length of the roadway can show the variation of these parameters in different locations. This is important because it can influence the design of any repairs (Miller and Bellinger 2003).

2.2.2.1 Falling Weight Deflectometer

The main form of NDT used in structural evaluations of pavement is deflection testing, though other technologies can also be used. Deflection testing can be conducted using a variety of different devices, which differ in how they apply load to the pavement. The most common device is the falling weight deflectometer (FWD), illustrated in Figure 8 and Figure 9, which applies an impulse load to the pavement to simulate a wheel load, and then measures the induced deflections at different distances from the point of the applied load.

Figure 8. Falling weight deflectometer (FWD) machine (from Hall et al. 2001)

Figure 9. Close up of FWD load plate and deflection sensors (from Hall et al. 2001)

From these deflection measurements, a deflection basin is created, which can be used to calculate many different parameters (Miller and Bellinger 2003). A typical sensor arrangement is shown in Figure 10. Deflections are generally measured at intervals raging between 100 and 500 feet (30 to 150 meters), depending on the project. A pavement slab must be tested at a midslab location, at the middle of the longitudinal joint, and at a slab corner (the latter two in conjunction with the adjacent slab) to be able to fully characterize slab performance. Measurements at the center of the slab should be used to backcalculate the elastic moduli of the concrete and base layers and the coefficient of subgrade reaction. A detailed procedure for the backcalculation of rigid pavements is discussed in Hall et al. (1997) and Khazanovich et al. (2001). Test intervals and slab locations for FWD testing are illustrated in Figure 11.

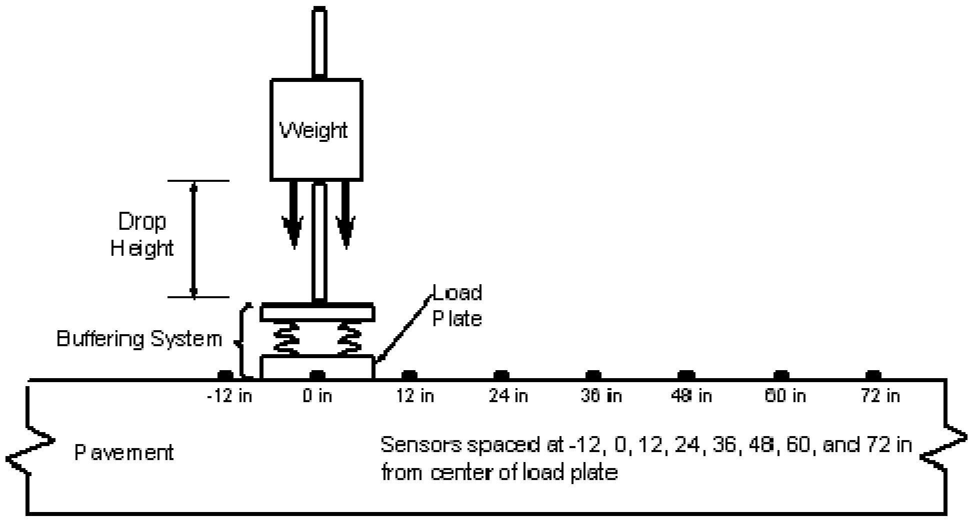

Figure 10. Diagram of typical FWD sensor and load locations (from Smith et al. 2008)

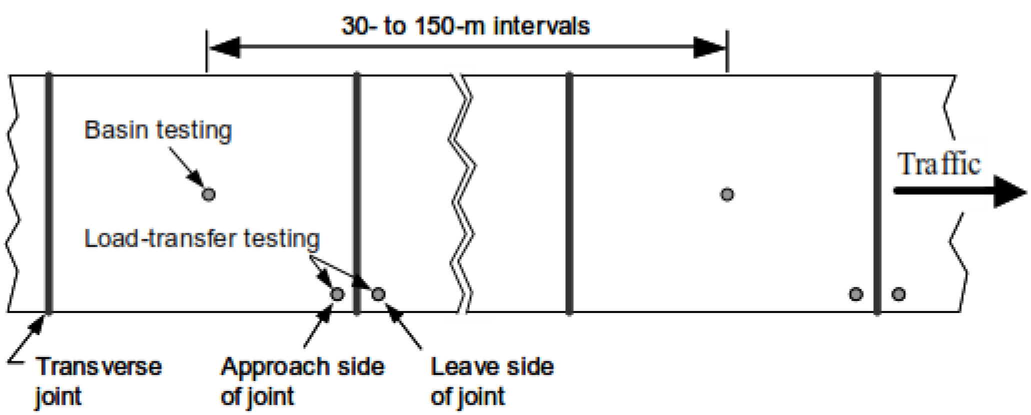

Figure 11. Recommended location for deflection basing and load transfer efficiency testing with an FWD (from Smith et al. 2008).

Backcalculation for AC overlays of PCC pavement is a more complex task. Higher compressibility of the AC layer, especially directly under the FWD load, makes the backcalculation procedures for developed for rigid pavements inaccurate. In some cases, backcalulation procedures developed for flexible pavements, such as WESDEF (Van Cauwelaert et al. 1989), MODULUS (Uzan et al. 1988), and MODCOMP (Irwin and Szebenyi 1991), but due to the presence of a stiff PCC layer they may necessarily lead to reliable results as well (Stubstad et al. 2006). In addition, the backcalculation procedures developed for flexible pavements cannot be used to determine the coefficient of subgrade reaction, which is often required for subsequent structural evaluation analyses and rehabilitation procedures.

A neural-network-based backcalculation procedure for composite (AC/PCC) pavement systems, DIPLOBACK, was developed by Khazanovich and Roesler (1997). The program is based on the forward analysis program, DIPLOBACK, which permits an analysis of the AC and PCC layers as compressible isotropic elastic layers and the subgrade using the Winkler foundation. The program is an attractive alternative to the traditional backcalculation tools, but it has not been widely used by practitioners.

Another approach for determination of the coefficient of sugrade reaction from the FWD deflections on composite pavements was proposed by Hall et al. (1997). It is based on the modification of the AREA method for rigid pavements (Ioanndides et al. 1985), but to eliminate the effect of compressibility of the AC layer it recommends ignoring deflections closer than 12 in to the center of the load.

If FWD testing is adopted for the evaluation of AC-PCC, precautions similar to those taken for rigid pavements should be followed. Testing should not be conducted in freezing conditions, or during the spring thaw, as the results will not be representative of year-round or more typical conditions (Miller and Bellinger 2003). Slab curling in the existing PCC layer, particularly downward curl, can affect deflection testing results and testing must be conducted at times and temperatures which would minimize these effects (Khazanovich et al. 2004).

For example, if the slab is curled in such a way that there is separation between the slab and the underlying layers, it is possible that that separation could be incorrectly detected as a void in the pavement. However, if the test were conducted when the slab was flat and not curled, the “void” would no longer be present. Determining the location of voids with FWD is difficult and depends on time of day and how tight the dowels are situated in the concrete. Upward curl is more prevalent at night, while downward curl is more common during the day. However, the effects of other differential volume change mechanisms, such as built-in curl and moisture warping, will affect when the slab experiences a flat slab condition.

Furthermore, testing at night will give good modulus of subgrade reaction numbers at mid-slab and good LTE results. In addition, LTE increases with increasing slab temperature, until there is aggregate interlock, at which point there is 100 percent LTE. If the same joints were tested just before the sun hits them in the morning, then the LTE might be extremely low (20 to 50 percent) as the slabs have contracted and the aggregate interlock is lost (Khazanovich and Gotlif 2003).

2.2.2.2 Rolling Dynamic Deflectometer

Another device used to measure deflections in the roadway is the rolling dynamic deflectometer (RDD), shown in Figure 12. Though its use is currently limited to a few states, RDD has been closely studied and proposed as an alternative to FWD to obtain deflection measurements for pavement evaluation (Scullion 2006), as it allows a continuous trace of deflections that can be gathered much more quickly than FWD measurements.

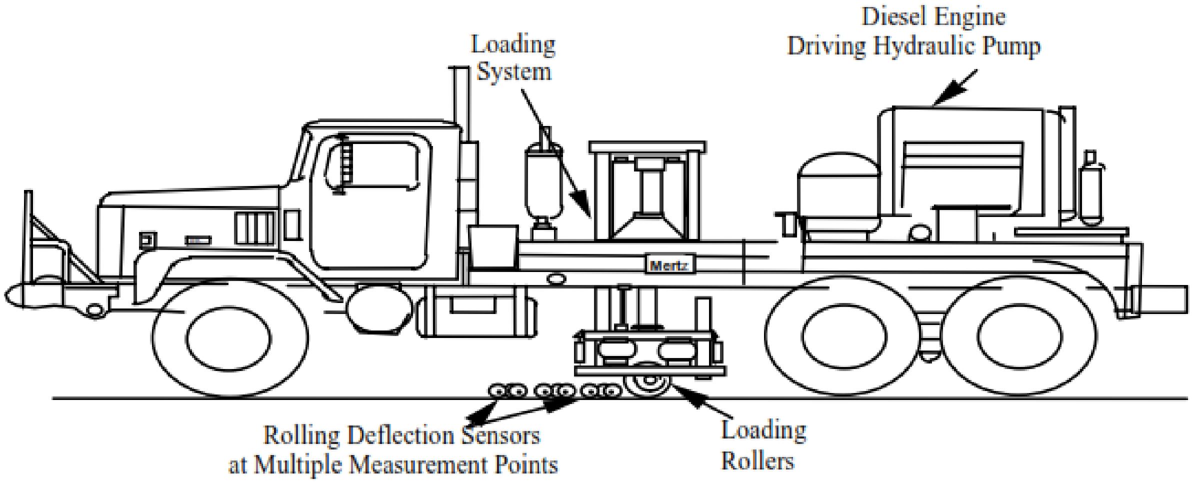

Figure 12. Rolling Dynamic Deflectometer (RDD) (from Lee et al. 2004)

The RDD is a specially outfitted truck which applies a cyclic load of fixed magnitude to the pavement as it travels along the road. Four rolling geophones attached in front of and adjacent to the load (see Figure 13) measure deflections induced by the load as the entire assembly is in motion. Typically the load has a magnitude of 10,000 lbs and is applied at a frequency of 30 Hz while the truck travels at 1.5 mph.

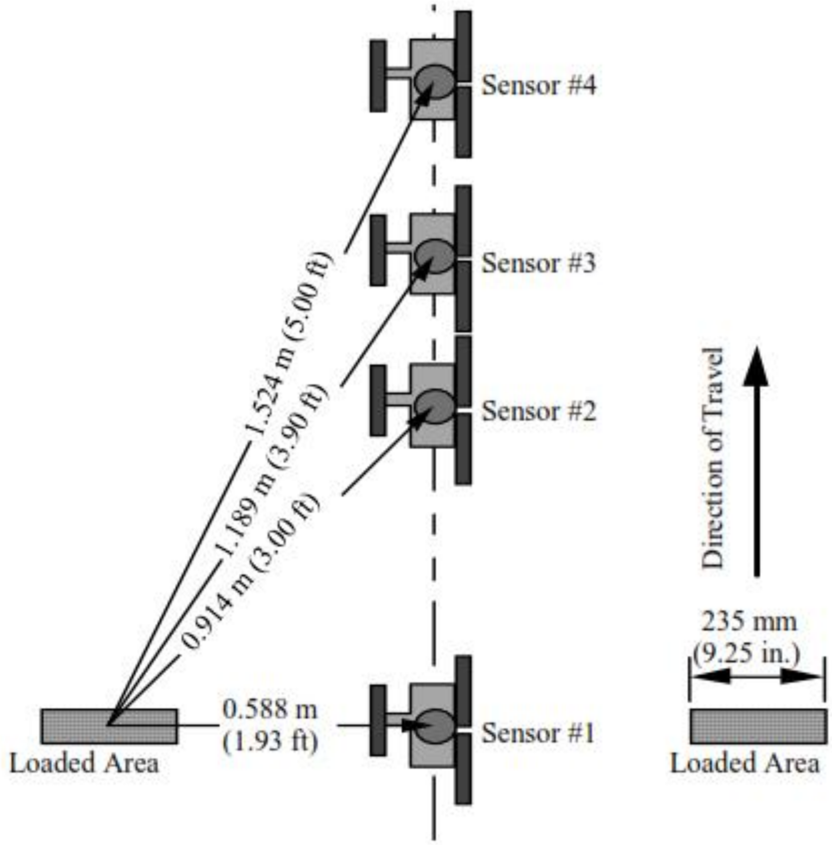

Figure 13. Locations of the load and sensors on the RDD (from Lee et al. 2004)

The RDD is used to measure the subsurface support of the pavement and the load transfer efficiency between slabs. The output of the RDD is average pavement deflections measured by the rolling geophones for each two-second time interval. The deflections from the sensor located between the loads (Sensor 1) are used to characterize the subgrade support while the difference in deflections from the sensor between the load (Sensor 1) and a sensor located away from the load (Sensor 3) can be used to calculate load transfer efficiency when the load is applied at a joint. A sample output of RDD data is shown in Figure 14, which illustrates the ease of locating "problem areas" using graphically presented data. With minimal training, it is possible to determine if the problem areas are indicative of low subgrade support, low load transfer efficiency, or both.

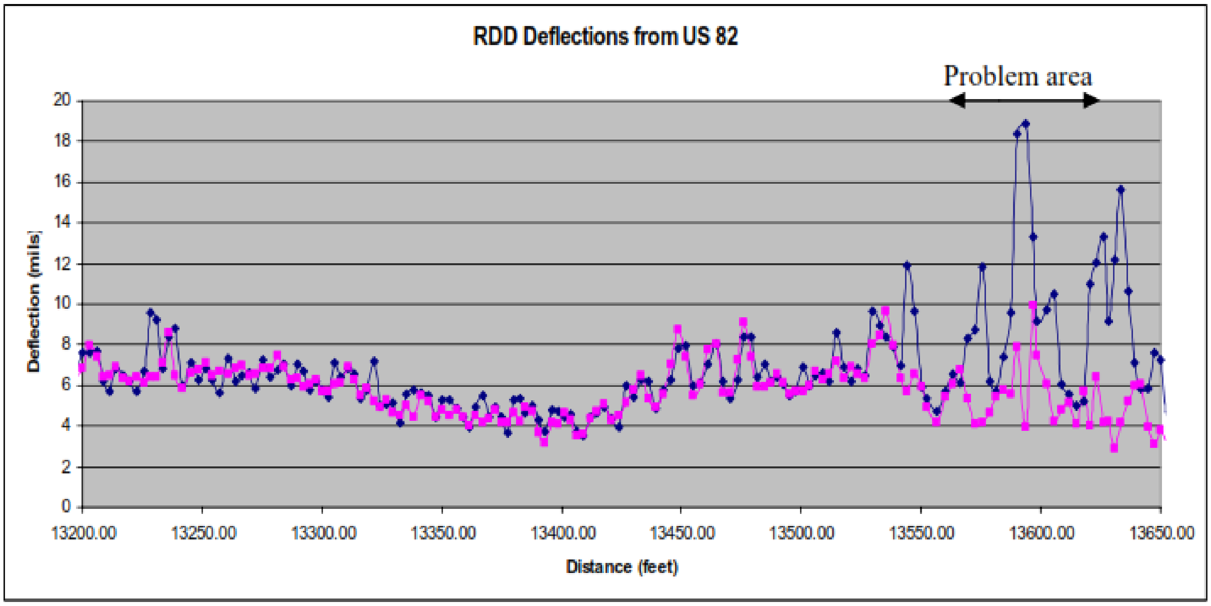

Figure 14. Sample RDD data identifying a "problem area" (from Scullion 2006)

While the RDD is a promising alternative to FWD in locating issues for a given on a roadway, it has a few limitations. Traffic must be diverted around the RDD to accommodate its slow speed. Also, the data collection system requires updating and increased functionality and accessibility for the user. Further study is required to determine how factors such as temperature will affect the deflections and the interpretation of results; however, RDD testing is another useful method for testing existing PCC prior to AC overlays.

2.2.2.3 Ground Penetrating Radar

In the last few decades, ground penetrating radar GPR has gone from being solely a QA/QC thickness verification device to being a widely used tool for forensic investigations and JPCP evaluations in addition to thickness detection As part of a structural evaluation of a jointed plain concrete pavement, GPR can be used to detect voids beneath the slab, particularly if they are filled with water (Scullion 2006). The GPR unit is shown in Figure 15. GPR can also be used as part of a network wide database for pavement management. In this case, info on pavement thickness and condition throughout a pavement network can be obtained using GPR and can be used to supplement as-built information (Harvey and Pyle 2009).

Figure 15. Vehicles equipped with ground penetrating radar (GPR) device (from Scullion 2006 (top) and Kohler et al. 2005 (bottom))

The van equipped with the GPR device (illustrated in both Figure 4 and Figure 15) travels along the roadway at highway speeds. From the GPR device mounted on the front of the van, an electromagnetic wave is emitted into the pavement, and the wave which is reflected back is received. These waves can penetrate up to 4.5 feet into the pavement structure, depending on the frequency of the signal. Higher frequency waves are used to obtain information near the surface of the pavement while lower frequency waves can penetrate deeper (Kohler et al. 2005; Maser et al. 2011). The waves are reflected differently at each interface between different materials in the structure (Zhou and Scullion 2007). Figure 16 briefly illustrates the signal interpretation underlying the assumptions made using GPR.

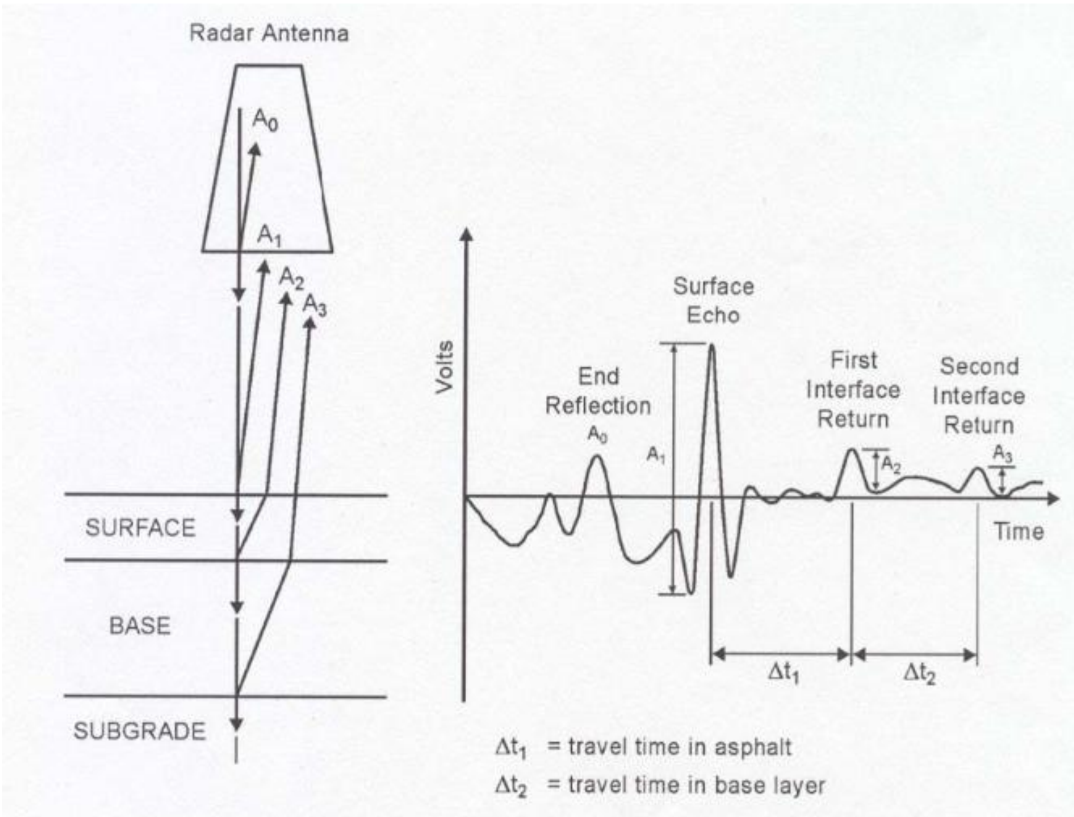

Figure 16. Reflected waves to GPR sensor and interpretation of signal (from Scullion and Saarenketo 2002)

The output of the GPR device must be processed in order to be easily interpreted. This can be accomplished through the use of commercially developed programs for GPR use. An example of this output is provided in Figure 17, which illustrates the interpretation of the data, as would be made by a trained operator, indicating areas beneath the slab which may be water filled voids.

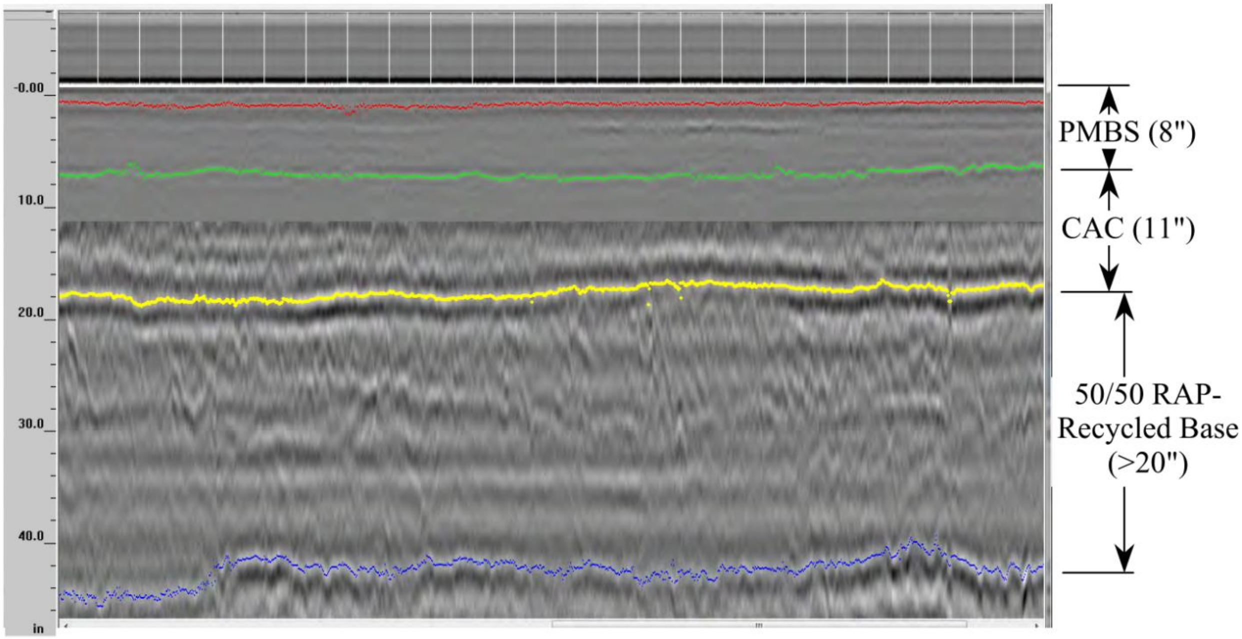

Figure 17. Example output of processed GPR data for pavement layer thickness analysis (from Maser et al. 2011)

Data interpretation similar to that of Figure 17 can be used in the rehabilitation selection procedure to determine which rehabilitation techniques are appropriate and what pre-rehabilitation repairs must be made. For example, if a concrete pavement has underlying water filled voids and it is overlain with an AC overlay, the moisture could become trapped. This moisture could then migrate up through cracks in the PCC and cause debonding between the PCC and AC, or lead to stripping of the AC (Zhou and Scullion, 2007).

Because the interpretation of GPR is dependent on the experience of the operator, it is very important to verify the condition of areas which are suspected to be damaged. Any areas of the pavement structure identified by the GPR as potentially containing water filled voids should be checked with pilot holes. In some instances, saturated clay can appear as a water-filled void under the pavement. One limitation of GPR is that it cannot detect small or thin voids (Zhou and Scullion 2007).

2.2.3 Special considerations for existing AC-PCC pavement

The structural evaluation of asphalt overlaid concrete pavements proceeds in much the same manner as the structural evaluation of regular concrete pavements. For the majority of AC-PCC, the pavement derives most of its strength from the PCC layer, however the asphalt overlay prevents a visual inspection of the structurally significant concrete layer. The condition of the asphalt can be indicative of the condition of the underlying concrete, as many distresses will propagate through (Miller and Bellinger 2003). In Figure 18, structural failure of the underlying concrete is suggested by observed cracking in the AC overlay.

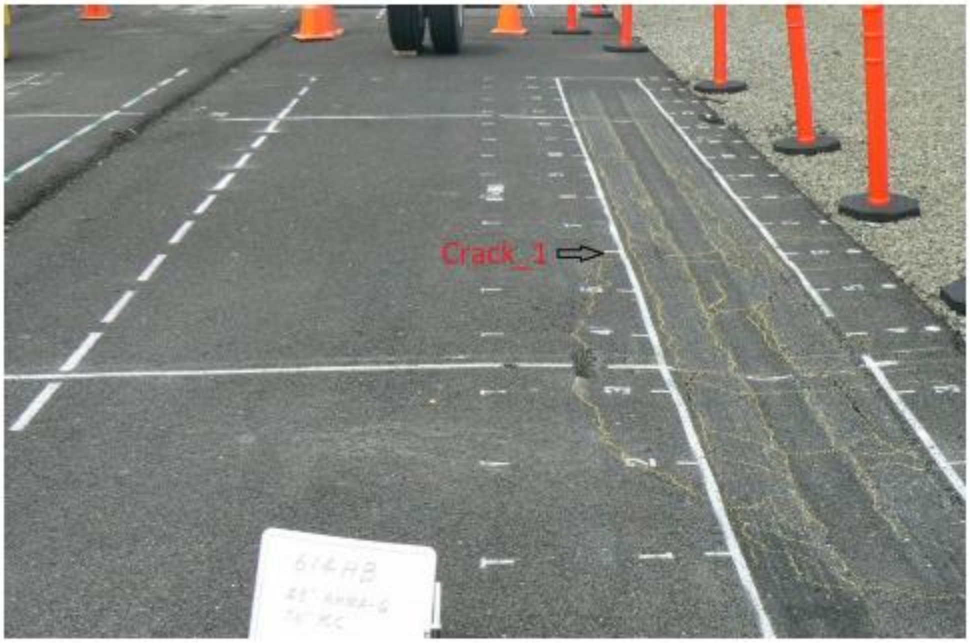

Figure 18. Structural failure of the underlying concrete layer appears as a crack in the AC overlay (from Rao et al. 2011)

Therefore, the visual distress survey is still of great importance for a concrete pavement with an asphalt overlay.

Drainage concerns remain unchanged from those of a regular concrete pavement, and the drainage survey must also be conducted. Certain types of testing, however, such as the back-calculation algorithms used in conjunction with deflection basin data obtained from FWD testing, are not compatible with pavements with asphalt overlays. As discussed earlier, the backcalculation procedure for AC-PCC is a more complex task due to the compressibility of the AC layer; for these specific cases, the reader is referred to the literature discussed in Section 2.2.2.3.

GPR can also be used in the structural evaluation of composite pavements to determine the thickness of the AC layer and to identify possible defects within the AC (Zhou and Scullion 2006). It can also be used to find water filled voids beneath the PCC, as was discussed above, and to differentiate between reflective cracks and transverse cracks (Al-Qadi et al. 2009).

2.3 Functional Evaluation

The functional evaluation serves to determine the functionality of the pavement rather than its structural integrity. A pavement that is structurally sound may not be functional if the rider experience is impaired by non-structural deficiencies of the pavement. The three major areas on which the functional evaluation of an existing pavement focuses are friction, roughness, and noise.

Friction on the surface of the pavement is important to ensure the safety of travelers. Without adequate friction for pavement/tire interaction, a vehicle may be unable to stop quickly enough when needed to prevent an accident, especially in wet conditions. The surface texture of the pavement determines the friction provided. There are four main types of surface texture: microtexture, macrotexture, megatexture, and roughness. As illustrated in Figure 19, these texture categories are differentiated based on the depth and frequency (wavelength) of the actual surface texture (Caltrans 2007).

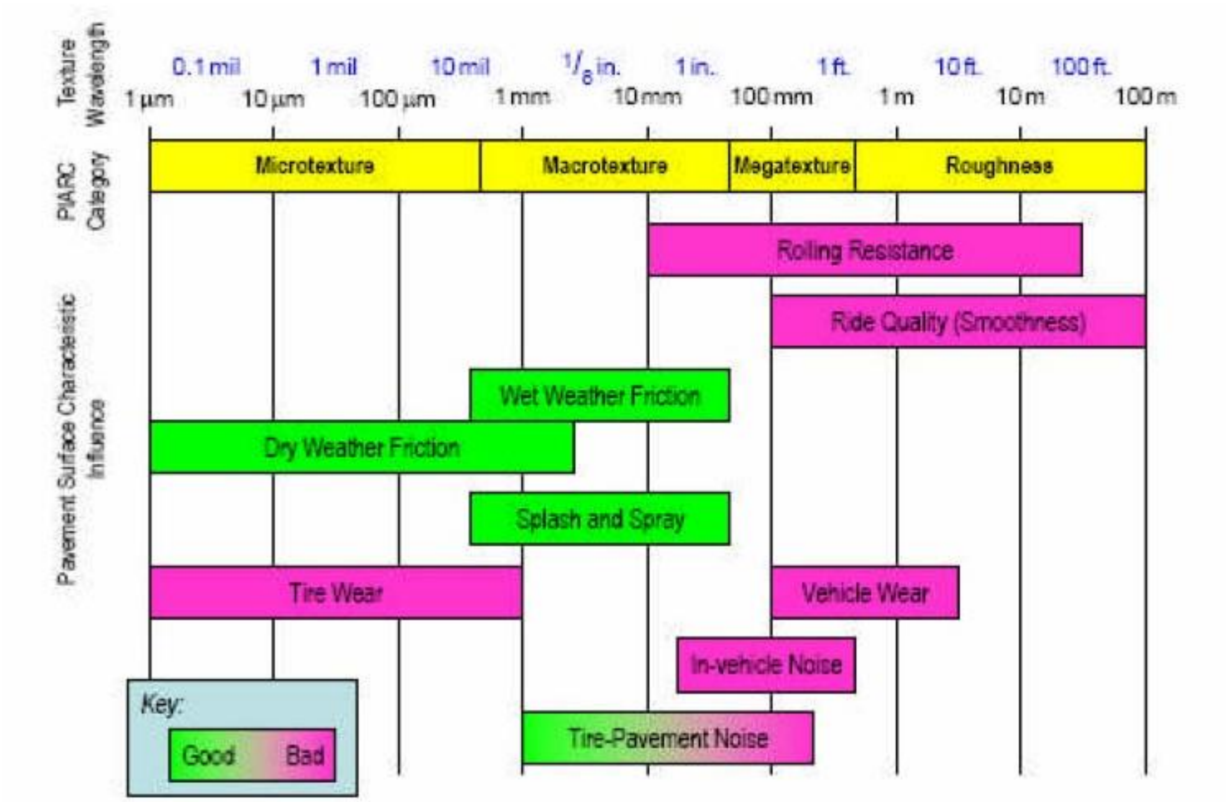

Figure 19. Surface texture categories (from ACPA 2006b).

The differences between microtexture and macrotexture are shown in Figure 20. Microtexture is inherently present in the concrete pavement due to the presence of fine aggregate, while macrotexture is the result of construction finishing techniques such as tining, dragging, grinding, grooving, or brushing. Both micro and macrotexture help to provide the surface friction needed for breaking. Macrotexture is also crucial to controlling splash and spray and preventing hydroplaning. Although megatexture and roughness are a result of surface defects and are not texture options considered in design, they contribute to the texture of the pavement and influence pavement-vehicle interaction (Caltrans 2007).

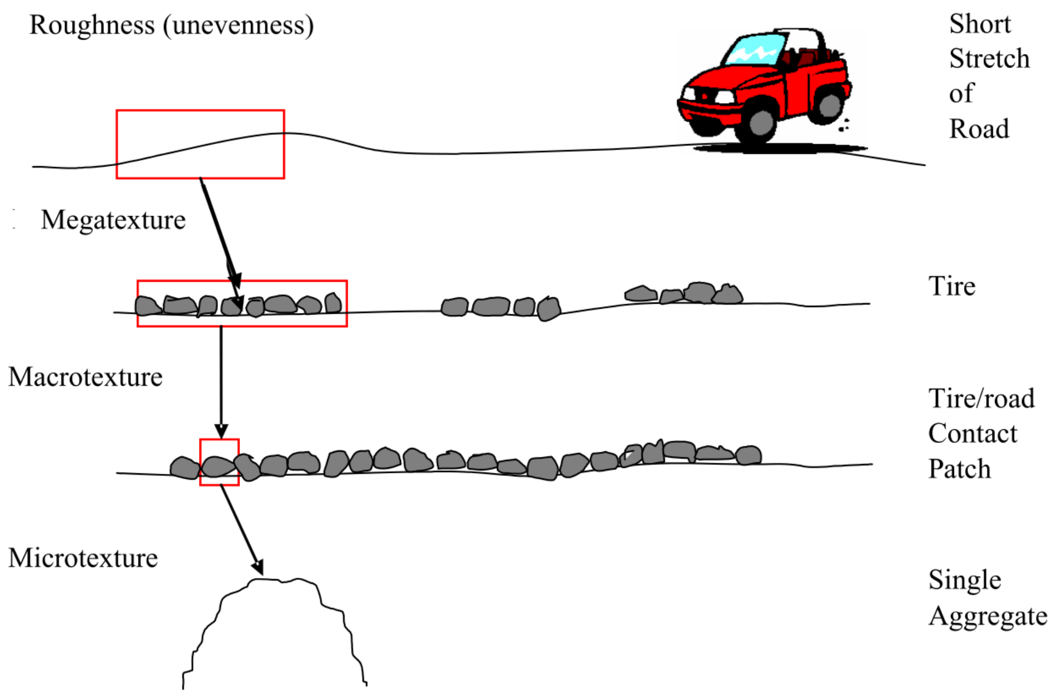

Figure 20. Differences between micro and macrotexture on concrete pavement (from Ongel et al. 2007)

During the functional evaluation of an existing pavement, the friction of the pavement is measured to ensure that there is adequate surface texture to provide sufficient friction for breaking and to minimize hydroplaning and splash and spray (Caltrans 2007). The friction of a pavement can be tested using a variety of devices, such as locked wheel, side force, fixed slip, and variable slip testers. Each of these devices test different aspects of the pavement friction, and simulate different vehicle actions. The friction of a pavement is measured at different locations along the project, generally in uniform intervals. While skid resistance is a general concern for the entire pavement, some tests should be conducted at any sharp turns on the roadway. The results of a friction test can indicate whether or not a surface needs to have more friction for safety purposes. While some state DOTs have programs with regular, network-level friction management, friction tests are generally not conducted unless it is suspected that there is a lack of friction on the roadway (Miller and Bellinger 2003).

If friction testing is deemed necessary, testing is conducted using full scale tires mounted on a trailer and locked in place, as shown in Figure 21.

Figure 21. Friction testing trailer (from Hall et al. 2001)

The trailer sprays water on the pavement in front of the test tires. The result of a friction test is the skid number, a commonly cited parameter for pavement friction that is equal to 100 times the measured coefficient of friction (Hall et al. 2001).

Roughness of a pavement is due to surface irregularities. These irregularities can either be built into the pavement during construction or can be due to different pavement distresses. In evaluation, the only irregularities of concern are those of sufficient magnitude as to affect ride quality or drainage. Roughness is generally measured in terms of the International Roughness Index (IRI), though other indices can also be used. The lower the value of IRI, the smoother the pavement is considered to be. The IRI of a pavement can help to determine if the pavement has a ride quality low enough to require repair. One important use of roughness measurements in overlay applications is to assess the effectiveness of the asphalt overlay. This can be accomplished by comparing the roughness before and after the overlay is installed (NCHRP 2004b).

A rudimentary roughness test can be conducted by simply driving over the road. From this, the relative roughness can be evaluated in terms of gross categories such as very rough, moderately rough, smooth, etc. Additionally, by observing if the roughness correlates to driving over a distress such as a transverse crack, the cause of the roughness can be surmised. Often, pavement management data collection systems include a camera mounted on the vehicle records a video of the road surface which can be used to more closely see distresses – the use of a camera for a video assessment was illustrated earlier in Figure 4.

A more detailed roughness test can be conducted using various vehicle mounted devices known as Inertial Reference (IR) profilers, shown in Figure 22. Data collected from IRI profilers can be analyzed to determine the IRI of the pavement. One popular software tool for this analysis is the FHWA's Pavement Profile Viewer and Analyzer (ProVAL). ProVAL allows users to view and profile a pavement given profile data; this analysis is not limited to IRI and includes ride indexes such as the Mean Roughness Index (MRI) and Ride Number (RN).

Figure 22. Examples of vehicles attached with profilometers for IRI assessments (from MnROAD 2008)

The noise level of a pavement can adversely affect those traveling in a vehicle, or people alongside a roadway, particularly residents of areas adjacent to roads with high traffic volumes and high speeds. Contact between vehicle tires and the pavement is a major source of noise emanating from a road. Factors affecting the loudness of a vehicle driving across a pavement include the tires themselves, and voids or joints in the pavement, and the surface texture of the pavement. Pavement noise can be measured as illustrated in Figure 23 by positioning a microphone near the roadway and measuring the decibel level due to the traffic, in this case using a trailer with microphones in it called the close proximity (CPX) method, which is widely used in Europe. Microphones can also be placed at the shoulder of the road and noise measured called a pass-by measurement. Pass-by measurements can be done for individual vehicles, or for a set of vehicles which is referred to as a Statistical Pass-By measurement (SPB) (Knuttgen 2008). Pavement noise may also be measured directly at the tire/pavement interface using the OBSI (On Board Sound Intensity) method (Donavan and Lodico 2009).

Figure 23. At left, trailer housing a microphone used to measure noise due to tire pavement interaction (from Hanson et al. 2004); at right, OBSI data collection using wheel-mounted probe (from Donavan and Lodico 2009)

The factors measured in the functional evaluation of the pavement do not indicate the structural capacity or condition of the pavement. However, they are important for safety and user comfort. Generally, these items can be used to determine if repairs or maintenance is needed, and by measuring levels of roughness, friction, and noise before and after the concrete is overlaid with asphalt, the effectiveness of the overlay can be demonstrated.

Chapter 3. Pre-Overlay Concrete Pavement Repair and Preparation

Prior to placing an AC overlay, it is important to repair certain types of distresses in the original concrete pavement or eliminate their causes. While the AC overlay itself can correct certain distresses in the pavement and restore ride quality, it cannot fix problems such as loss of support, poor drainage, full depth cracks, or low load transfer between slabs. Failure to address these issues will reduce the effectiveness of the overlay. Figure 24 shows the number of states which use common pre-overlay treatments prior to placing an AC overlay on a PCC pavement, based on a survey of 26 states.

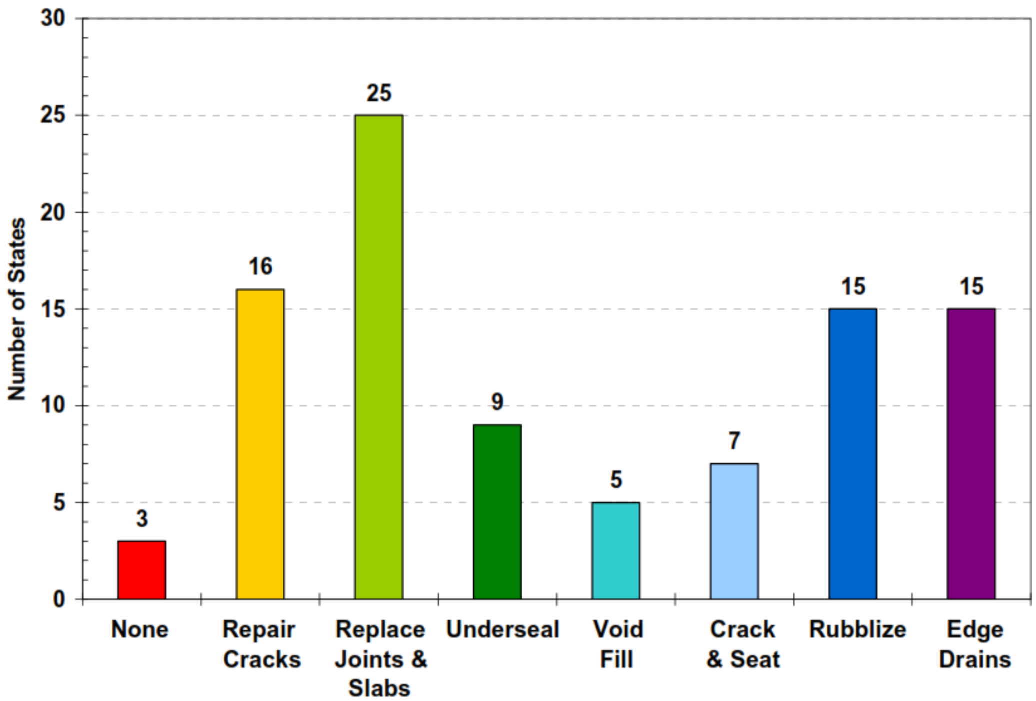

Figure 24. Pre-overlay treatments for PCC pavements used by state highway agencies, based on a survey of the practices of 26 states (from Bennert 2009)

The order in which these repairs are performed is important, so that the pavement can be prepared for the overlay in an efficient manner. Figure 25 describes a recommended order of repairs to maximize efficiency and minimize disruption to previous repairs by subsequent ones. While the later steps in Figure 25 do not apply to AC-PCC, and the steps taken individually are not mandatory given that pavement condition varies. However, the figure exemplifies the decision making required ahead of overlay construction, in which pavement preparation should be considered carefully and categorically.

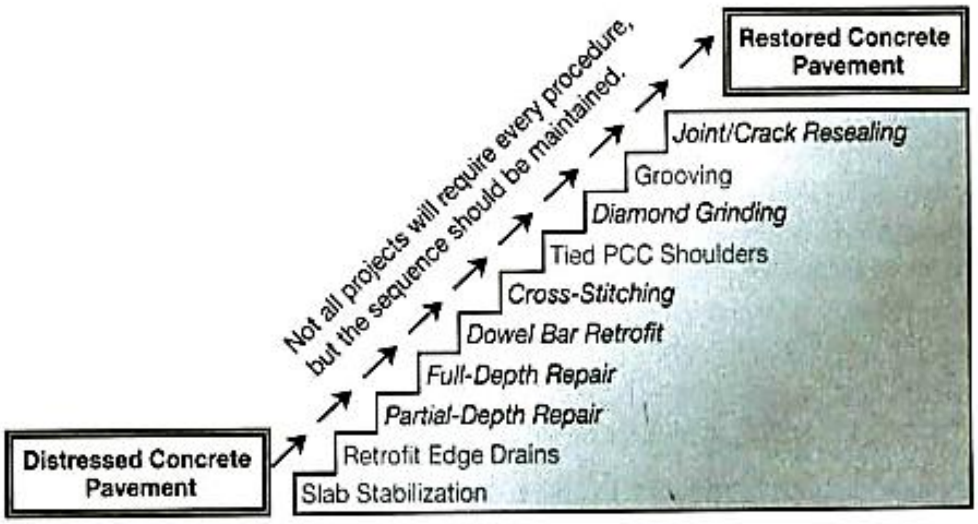

Figure 25. Recommended order for repairs of PCC pavement prior to overlay placement (from ACPA 2006a)

The following subsections describe pre-overlay methods to repair or otherwise prepare the existing rigid pavement for an AC overlay.

3.1 Restoring PCC Slab Support and Stability

Over time, slabs can lose support from underlying layers due to the creation of thin voids beneath the slabs. Several factors can cause loss of support; poor load transfer, pumping and erosion of the base, poor drainage, and localized settlement. One mechanism of loss of slab support and subsequent pavement damage is illustrated in Figure 26.

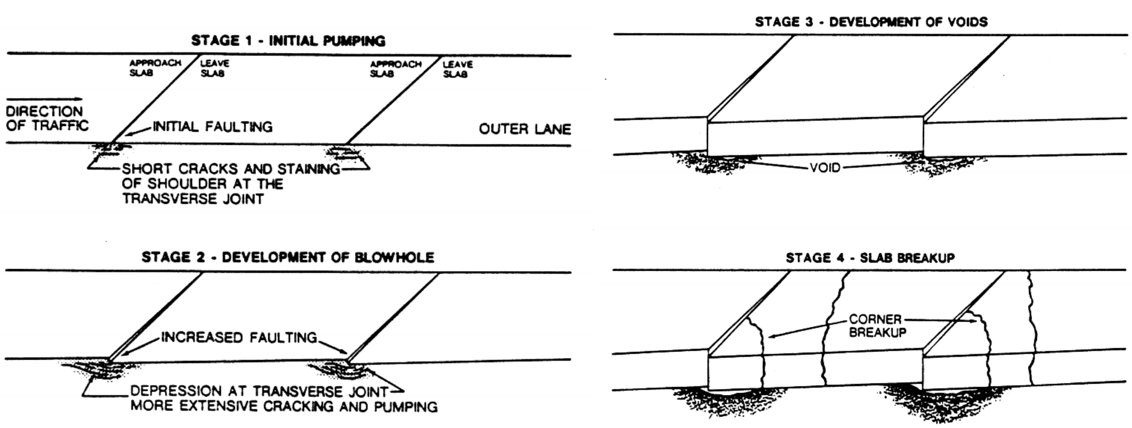

Figure 26. Loss of slab support due to poor drainage and associated cracking (from Darter et al. 1985)

Support can be restored to the slab through slab stabilization. However, it is important to note that slab stabilization will only restore support to the slab which is non-permanent; it will not prevent voids from developing in the future. Therefore, it is important that the cause of the loss of support be identified and addressed prior to placement of the overlay (NCHRP 2004a, AASHTO 2002), as slabs can also settle to varying degrees.

Slab jacking can be performed for slabs that show local settlement. Slab stabilization and slab jacking both involve the insertion of material under the slab; however, the techniques and materials differ, as do the applications. Some of these differences are evident in Figure 27.

Figure 27. At left, slab stabilization; at right, removal of material for slab jacking (from Smith 2009)

Slab stabilization is used to fill voids beneath the slab, while slab jacking is used to return a slab which has experienced local settlement to its original height. Slab jacking should only be used on slabs which have experienced local settlement, such as over soil which provides a poor foundation, or over a culvert. Though faulting can be considered as a minor type of local settlement, slab jacking or slab stabilization is not recommended to repair faults by lifting the leave slab. Diamond grinding is more effective to remove faulting. Slab stabilization does not lift the slab or return it to its original height (AASHTO 2002).

Locations in need of restoration of support should be determined as part of the structural evaluation (discussed earlier). Visual observation, deflection data, and non-destructive testing methods can all be used to detect the presence of a void or loss of support. Once the presence of a void is established, slab stabilization can be considered to fill that void. Areas of local settlement are also determined during the structural evaluation. It is important to ensure that non-uniform support or localized settlement and not a void, is the cause of distress. Only areas of local settlement should be considered for slab jacking (AASHTO 2002).

In both slab stabilization and slab jacking, the number and location of the holes for filler material is dependent on the distress being repaired, as that dictates where additional material is needed. Sample hole locations are shown for repairing voids via slab stabilization in Figure 28, while Figure 29 shows sample hole location for correcting slab settlement using slab jacking.

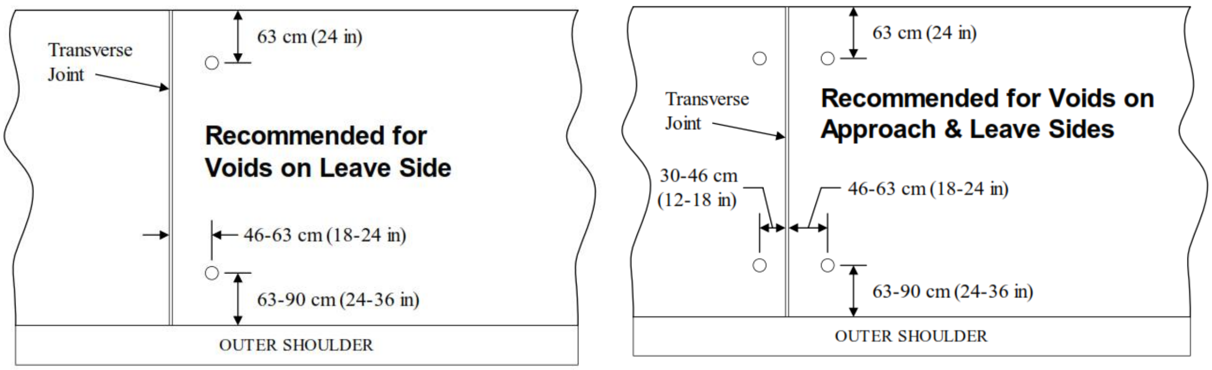

Figure 28. Sample pattern of grout holes to be used in slab stabilization for different distress types (from Darter et al. 1985)

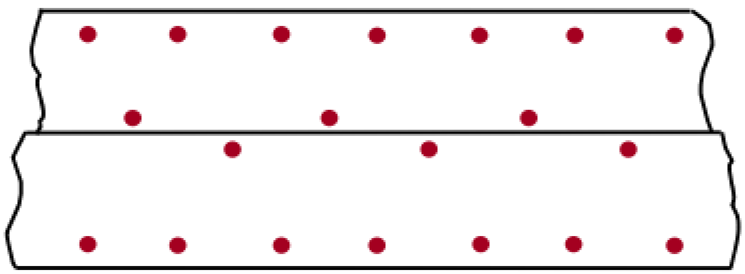

Figure 29. Location of grout holes used when correcting settlement with slab jacking (from Smith et al. 2008)

The type of filler material used is also dependent on the application. Cement based grouts are very common, and maintain constant volume during application, but urethanes are becoming increasingly popular however their potential for expansion beyond that expected can lead to slab cracking . Filler materials used for slab jacking are generally stiffer than those used for slab stabilization (ACPA 1994).

Overall, slab jacking and stabilization have been found to be effective in practice, however they can be highly contractor dependent. A before/after example of this effectiveness is illustrated in terms of FWD results in Figure 30, which suggests that slab stabilization efforts have restored the effective slab support.

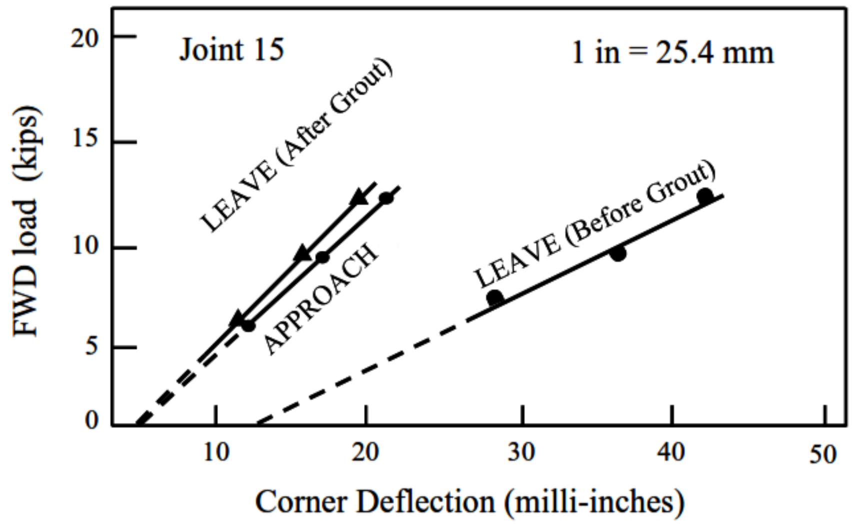

Figure 30. Effectiveness of slab stabilization to decrease corner deflections (from Darter et al. 1985)

Once slab stabilization and slab jacking have been completed, it is necessary to address the problems which originally caused the slab to lose support (AASHTO 2002).

3.2 Localized Slab Repair

Often, a concrete pavement to be overlaid has localized areas which exhibit higher levels of distress than the majority of the pavement. These areas must be repaired prior to placement of the overlay to prevent the distresses from propagating upwards into the overlay. Weak and deteriorated concrete on the top surface, which can cause spalls and scaling can be repaired with a partial-depth repair. Figure 31 shows spalls which were patched locally and have subsequently deteriorated further.

Figure 31. Spalled cracks which were previously patched; these areas will require either full- or partial-depth repair prior to placement of an AC overlay (Wen et al. 2005)

Distresses which extend into the thickness of the slab – that is, below dowels and tie bars – such as cracks and deteriorated joints, must be corrected with full-depth repairs. Both partial- and full-depth repairs involve removing a portion of the concrete slab and replacing it with new material (NCHRP 2004a, ACPA 2003, ACPA 2006a). Previously repaired distresses may still require full- or partial-depth repairs if the repair has deteriorated. These areas will require partial-depth repair before the AC overlay is placed. If the deterioration is found to extend beyond the upper third of the pavement, a full-depth repair will be necessary.

3.2.1 Partial-depth

Partial-depth repairs are those which do not extend more than the top third of the slab. Partial-depth repairs are generally used to correct spalls and surface scaling and deterioration. An example of a typical localized partial-depth repair of an existing PCC pavement is illustrated in Figure 32

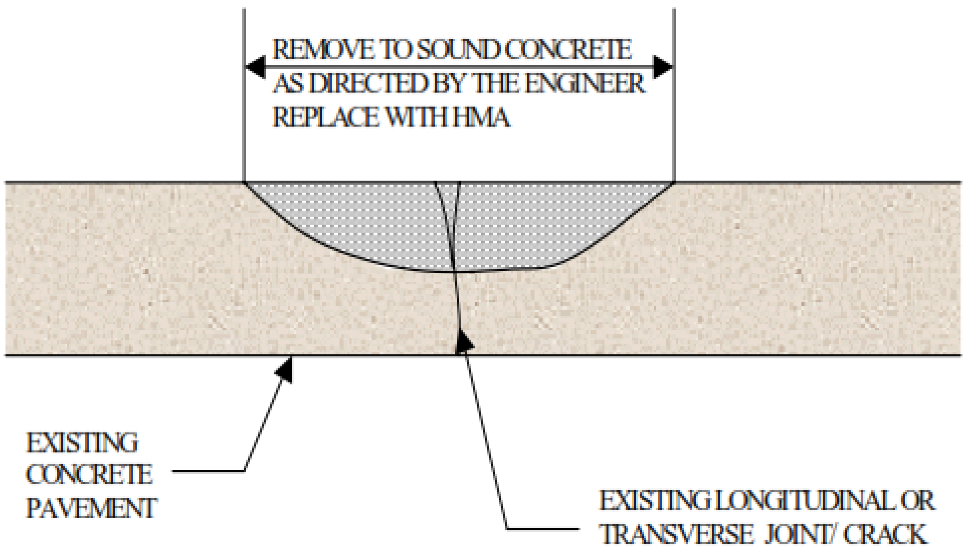

Figure 32. Partial depth repair of spalling around a joint or crack (from Wen et al. 2005)

It is important to note that partial-depth repairs cannot reliably transfer load, and care must be taken with partial-depth repairs adjacent to joints to ensure that the joint remain free of any backfill material, so as to ensure free joint movement and mitigate against future spalling. If these distresses are caused by factors which can be corrected by simply removing and replacing the damaged concrete, then partial-depth repairs can be used. Such situations generally arise when the top surface of the concrete has been weakened due to improper placement, inadequate air voids in the concrete, incompressible materials are lodged in the joints, or reinforcing steel is located too close to the slab surface. A partial-depth repair of a joint using a compressible insert is illustrated in Figure 33.

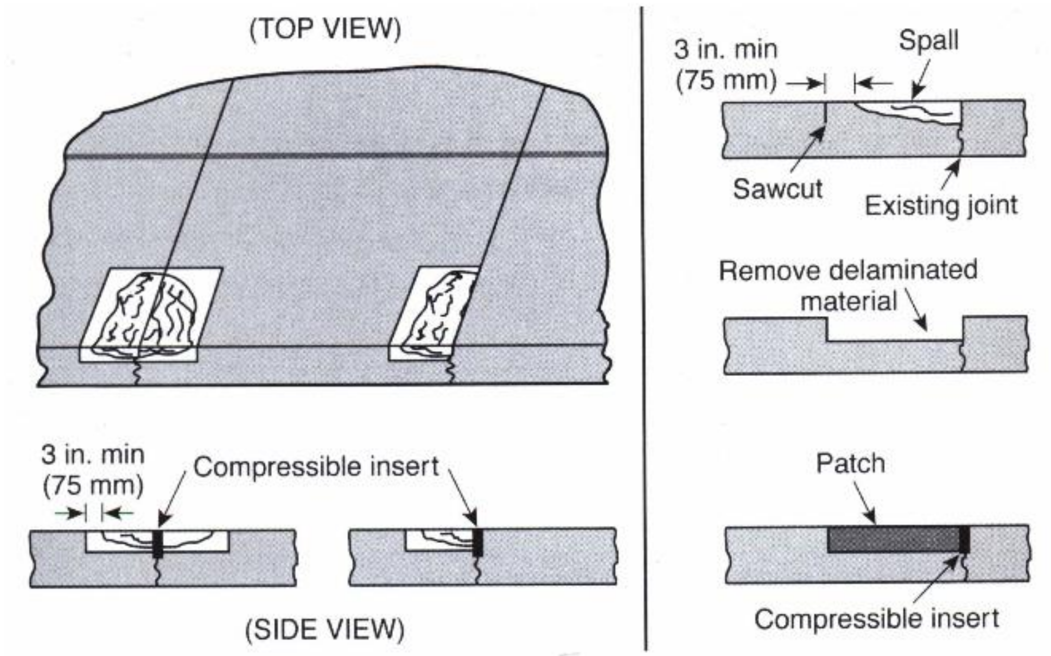

Figure 33. Partial depth joint repair (from ACPA 2006a)

Partial-depth repairs should not be used when spalling is caused by issues which cannot be resolved by replacing the damaged concrete, such as joint lockup, cracking or reactive aggregates. Spalls along a joint may be correctable with partial-depth repairs, but full-depth repairs are often more cost effective at joints (ACPA 2003).

Areas for which partial-depth repairs can be considered are identified during the structural evaluation using the methods described above. The area of the partial-depth repair should extend beyond the damaged area and only repairs of square or rectangular (but not with a high width to length ratio) geometry should be used. Often, it is more cost-effective to repair one large area than several small ones; therefore, adjacent partial-depth repairs should be combined if they are sufficiently close together (ACPA 2003). The damaged material is removed by sawing, chipping or milling, and the patch is applied after the area has been thoroughly cleaned to remove any loose debris.

The material used to replace the concrete which was removed must be carefully selected to ensure it will be compatible with the original concrete slab. Several different types of concrete can be used, such as cementitious and epoxy based concretes, depending on the application. It is important to select a repair material with a similar coefficient of thermal expansion compared to the original concrete to prevent cracking due to differential expansion and contraction. The shrinkage characteristics of the repair material are also important because cracking may result if the repair shrinks considerably during curing. Application of cement grout or epoxy prior to filler material is commonly used. Other factors which must be considered when selecting an appropriate repair material are the amount of time the roadway may be closed to traffic (which may eliminate materials which gain strength slowly), the required strength of the patch, and freeze-thaw durability, as dictated by climatic conditions (NCHRP 2004a, T ACPA 2003, ACPA 2006a).

3.2.2 Full-depth

Full-depth repairs involve removing the entire thickness of a portion of the slab and replacing it with new material. They are suitable for correcting many different types of deficiencies, such as cracks, corner breaks, blowups, and punchouts. Full-depth repairs can also be used to repair spalls for which partial-depth repairs were deemed unsuitable, and to repair areas which had previously been repaired and the patch deteriorated. While the applicability of partial depth repairs is quite limited, full-depth repairs are suitable for many distress types. The major factor in determining if full-depth repairs should be used in lieu of reconstruction is the extent of the damage.

If the entire length of the pavement is severely damaged, full depth repairs could be tantamount to reconstruction and with a shorter expected performance life. For most distresses in otherwise sound pavement, a full-depth repair is a viable relatively long lasting solution. The exception to this is material related distresses, such as D-cracking or spalling due to reactive aggregates, where the deterioration is likely to continue for any unrepaired portion of the pavement. Though full-depth repairs can often remedy most distresses, they are costly, and add joints to a pavement, which can increase roughness (NCHRP 2004a, ACPA 2003, ACPA 2006a).

Proper selection of the area to be replaced is critical to ensure the success of a full-depth repair. Often the damaged area extends beyond that which is visible at the surface; this is particularly true of distresses which are more prominent at the bottom of the slab, where very little damage may be seen on the surface, as seen in Figure 34.

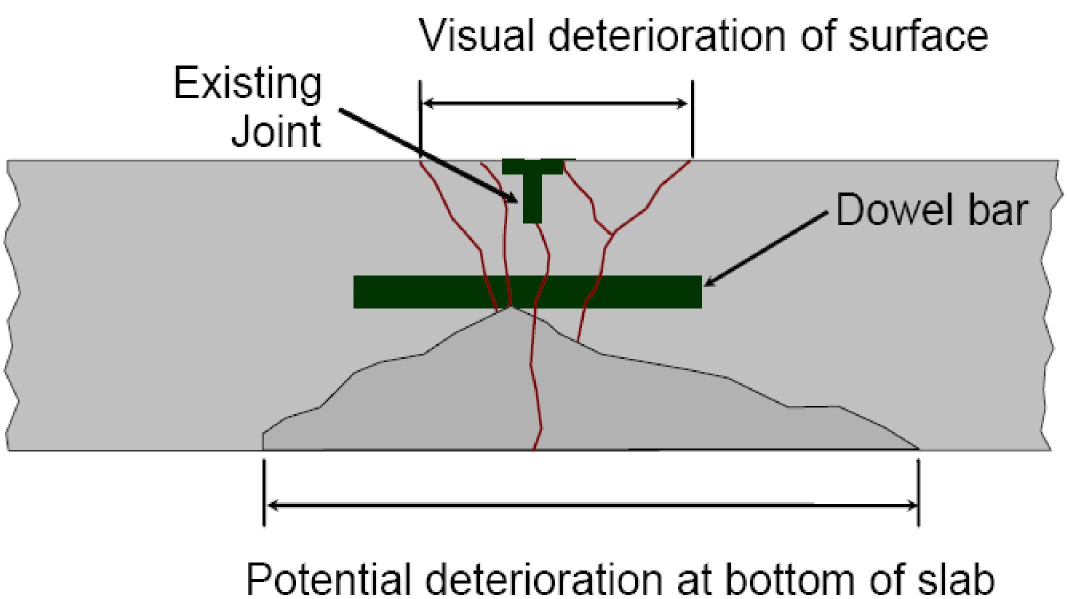

Figure 34. Deterioration at the bottom of the slab may be more extensive than is indicated by the distress at the top of the slab (Smith et al. 2008)

The boundary of the repair must encompass the entire damaged portion of the pavement, including distressed portions of the underlying layers, which may also require replacement. Special care must be taken to ensure the repair area is sufficiently large when it includes load transfer devices, such as dowel bars. Repair areas which are in close proximity to a joint should be extended all the way to the joint. As with partial-depth repairs, the repair area should be rectangular or square and should extend slightly beyond the damaged area. An example of boundary selection for full-depth repairs is illustrated in Figure 35. In assessing the boundaries of localized full-depth repairs, it is also important to recognize the number of repairs needed per slab; in cases where large portions of the pavement are damaged, replacement of the entire slab may be more cost effective than several large repairs (ACPA 2003).

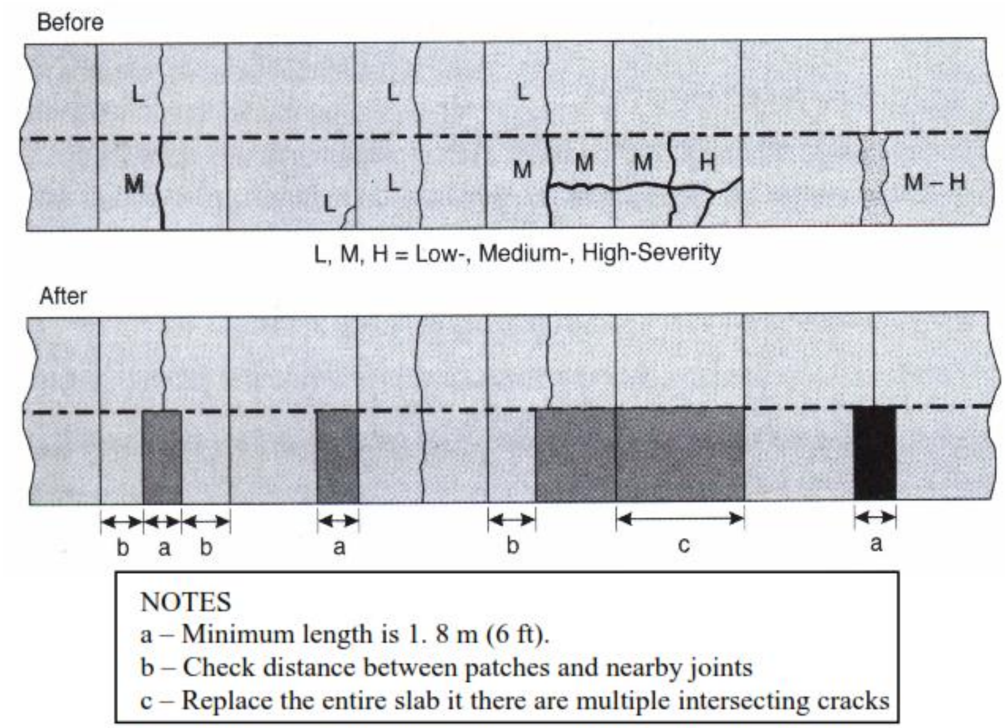

Figure 35. Proper selection of the repair boundary (from ACPA 2006a)

The boundary of the area to be removed is saw-cut, generally with a diamond blade, and the damaged material is removed. Ideally, the material should be removed as soon as possible, without allowing traffic on the saw-cut concrete, as this encourages pumping and erosion. The damaged concrete can either be broken into pieces and removed with a backhoe, or lifted out as a whole slab. It is easier to break the slab into pieces, but generally, the backhoe disturbs the underlying layers during the removal process and can spall slab edges. To prepare the area for replacement material, any disturbed base must be re-compacted, or removed, in which case it will be replaced with concrete during the placement of repair material. In areas where load transfer devices such as dowel bars were removed along with the damaged materials, these must be replaced prior to casting the repair concrete. Figure 36 shows an undoweled full-depth repair during the construction phase.

Figure 36. An undoweled full-depth repair in progress, the damaged concrete has been removed and the area has been cleaned. (Wen et al. 2005)

Once the patch has been cast, it must be allowed to cure. If the patch is at a different elevation than the surrounding original pavement, diamond grinding maybe used even out any elevation discrepancies. The new joints created by the repair should be appropriately sealed (NCHRP 2004a, ACPA 2003, ACPA 2006a). For higher volume roadways, traffic volumes demand that quicker methods are used. For instance, in California, concretes using Type III cements with admixtures to decrease the curing time are used so that the slabs can be opened to traffic within 4 hours of placement. These slabs must have flexural strength values of 400 psi at the time of opening.

The material used to replace the concrete which was removed must be carefully selected to ensure it will be compatible with the original concrete slab. Generally, PCC concrete is used as the repair material, but other, more expensive materials such as epoxies are also available if required.

One of the main selection criteria is the amount of time the roadway may be closed to traffic, which typically forces the use of high early strength cements. It is important to select a repair material with a similar coefficient of thermal expansion compared to the original concrete to prevent cracking due to differential expansion and contraction. The shrinkage characteristics of the repair material are also important because cracking may result if the repair shrinks considerably during curing. However, high early strength cements which must be used to reduce the amount of time the pavement is closed to traffic tend to result in greater shrinkage than ordinary cement. Other factors which must be considered when selecting an appropriate repair material are the required strength of the patch, and freeze-thaw durability, as dictated by climatic conditions (NCHRP 2004a, ACPA 2003, ACPA 2006a).

3.3 Drainage Repair

Edge drains can be used to improve drainage in pavements which were not originally fitted with drains, alleviating moisture damage. The effectiveness of edge drains, particularly those installed as a retrofit, is a matter of some contention. Studies have found that edge drains reduce pumping, faulting and joint deterioration (Yu et al. 1994), which contribute to extending the overall life of the pavement (Darter et al. 1985). However, other studies have found that retrofitted edge drains are often installed too late to prevent major damage, at which point their efficacy is low (Bradley et al. 1986), and that pavements retrofitted with edge drains do not perform better than those without edge drains (Baumgardner and Mathis 1989). Other studies have found that edge drains have been detrimental to pavement life and have exacerbated failure where surface cracking allows water to enter the pavement system (Harvey et al. 1999). Additional information on the selection and design of specific drain types is provided by NHI (1999), FHWA (1990, 1992), Baumgardner and Mathis (1989), and Christopher (2000)

Moisture related distresses observed during the structural evaluation can indicate poor drainage. However, poor drainage does not automatically indicate a need for edge drains, as edge drains are not appropriate for all pavements. Pavements should not be considered for edge drain retrofits if more than 10% of its surface is cracked or there is large number of other surface defects, if there is evidence of pumping, or the base contains more than 15% fines (Rutkowski et al. 1998). If it is not possible to retrofit the pavement with edge drains, the alternative is to wait until the pavement needs to be reconstructed; edge drains can be added when the pavement is reconstructed.

Edge drains work by shortening the drainage path of moisture through the base. Drains are installed longitudinally along the pavement at the edges. Water is collected from the base material at the edge of the pavement and an outlet is provided for the water to leave the pavement system. Thus the water does not need to travel through a clogged base to daylight. There are two main types of edge drains: pipes and prefabricated geocomposite edge drains (PGED). Aggregate trenches (also called French drains), can also be used, but are not recommended given their low hydraulic capacity and inability to be maintained (FWHA 1990).

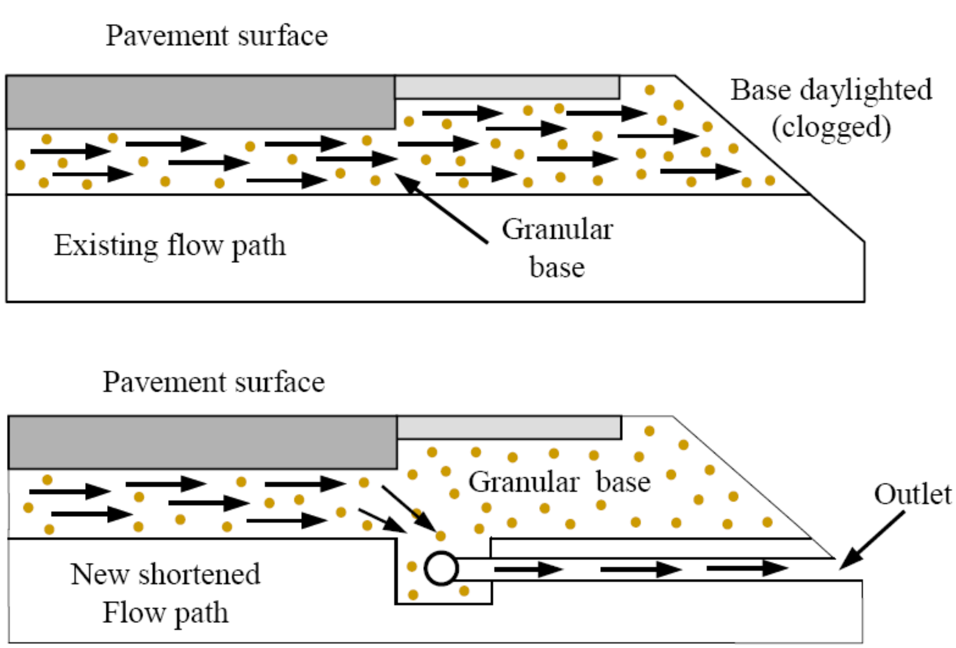

Figure 37. Drains work by shortening the drainage path and eliminating the need for water to flow through potentially clogged base material (Smith et al. 2008).

Pipe drains are simply perforated pipes placed in a trench lined with a geotextile to prevent fines from entering the drain. PGEDs consist of a prefabricated panel containing geotextiles wrapped around a drainage core (Figure 38). These panels are quite thin, allowing them to be installed in much narrower trenches than pipe drains. For this reason, PGEDs are often a much easier and less expensive option than conventional pipe drains. Care must be taken during the installation of PGEDs to avoid damaging them. Pipe drains are much easier to clean than PGEDs, which are nearly impossible to unclog (FHWA 1992).

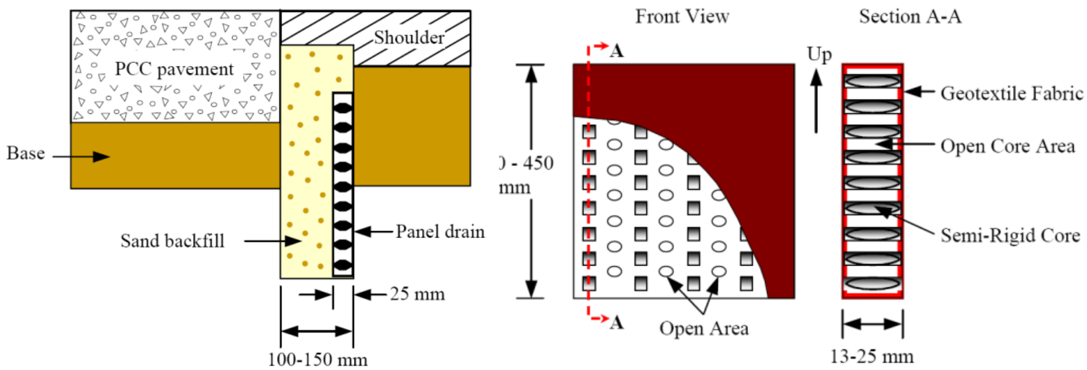

Figure 38. A panel drain installed (left) and isolated (Smith et al. 2008)

To determine which drain type is appropriate, the required flow rate to ensure that all infiltrating water can be discharged must be determined. The drainage capacity of PGEDs however, is lower than that of pipe drains, meaning that PGEDs are not suited for applications with high runoff. Once the collector type has been selected, it must be sized; often the ability to clean the drain necessitates a larger pipe than that required solely to handle the expected flow. The location of the collector and outlet pipes must be properly determined to ensure that water can exit the drain. A diagram of this determination is illustrated in Figure 39.

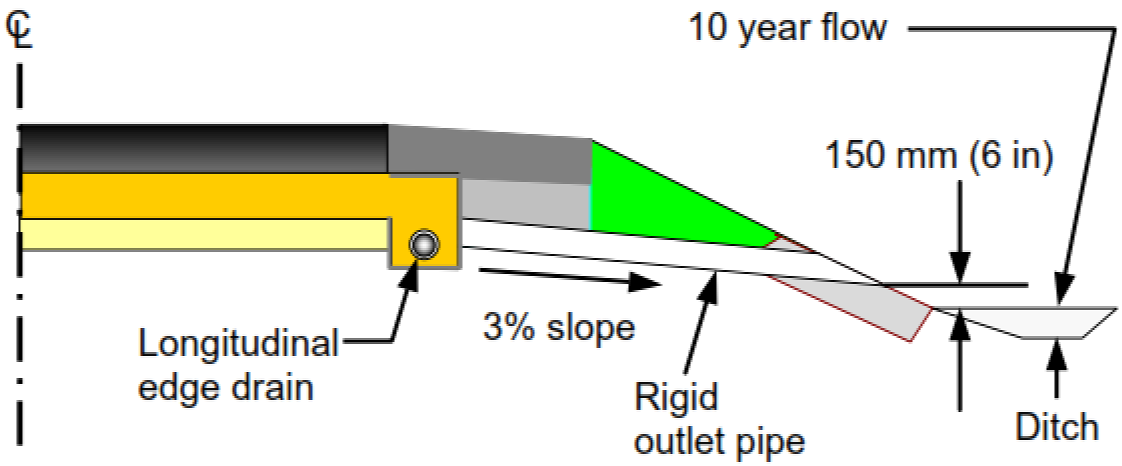

Figure 39. Diagram of drain configuration to ensure proper drainage (from FHWA 1992)

The collector pipe must be located below the frost line, while the outlet pipes must have an adequate slope to ensure that water can flow outwards. The daylight end of the outlet pipe must enter the ditch at an elevation above the waterline of the ditch (FHWA 1992).

For both types of drain, the trench surrounding the drain must be backfilled with a suitable material. The fill must be more permeable than the base to encourage water to flow into the drain, but must be stable enough to support the drain itself, as well as whatever is located above the drainage trench (often the shoulder). Another important role of the fill is to filter any fines out of the water before they enter and clog the drain. Location of the backfill in a retrofitted edge drain is illustrated in Figure 40.

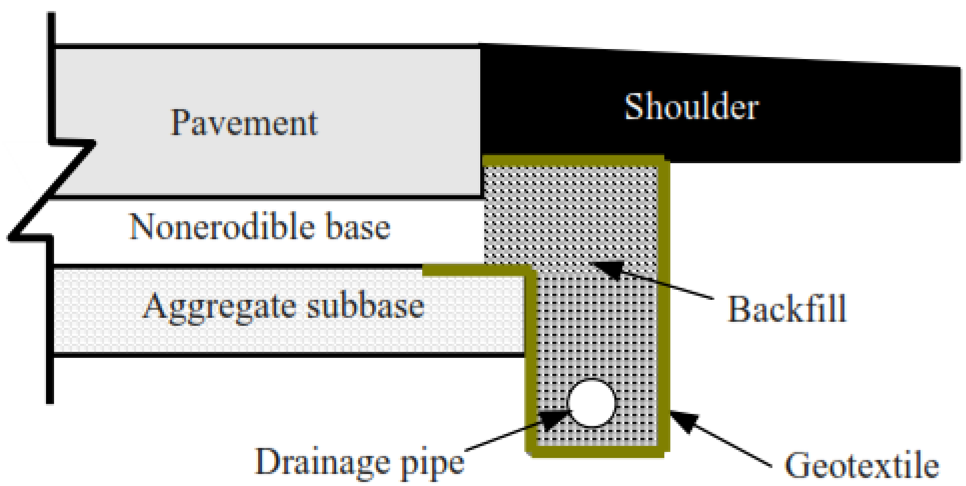

Figure 40. Schematic of retrofitted edge drain design (from NHI 1999)

Once the drains are installed, they must be properly maintained to ensure their functionality. Maintenance should be conducted at least twice a year, and includes removing any material such as vegetation or debris which may be clogging the drain. Additionally preventative measures should be taken, such as mowing the area around the drain, maintain adequate slopes in the ditches, and keeping drain outlets covered with screens.

3.4 Improving Load Transfer across Transverse, Longitudinal, and Shoulder Joints

Load transfer between adjacent slabs is important to prevent pumping and associated faulting and corner breaks. In doweled and tied pavements, load transfer is provided by the dowel and tie bars, which ensure that adjacent slabs deflect together (Figure 41). Dowel bars span transverse joints, while tie bars span longitudinal joints and the joint between the pavement and the shoulder.

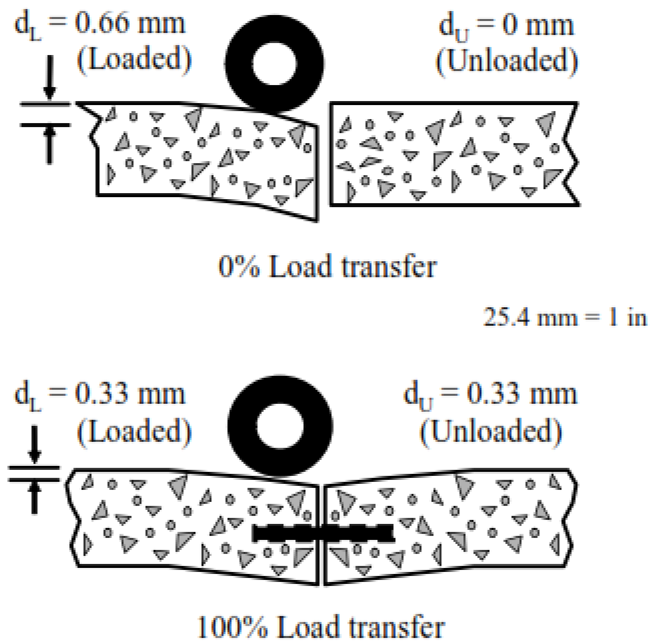

Figure 41. Mechanics of load transfer for adjacent slabs with no load transfer (top) and with complete load transfer (bottom) (from Smith et al. 2008)

Aggregate interlock is the main source of load transfer between adjacent slabs in undoweled pavements, and between faces of a crack. However, load transfer can only be provided by aggregate interlock if the faces of the joint (or crack) are in close contact. In cases where aggregate interlock no longer provides sufficient load transfer, a load transfer device, such as dowel bars, can be retrofitted across joints and cracks.

Ensuring that the original pavement has proper load transfer prior to overlay placement, can reduce reflective cracking in an overlay. Load transfer retrofits are an appropriate option for structurally sound pavements which do not yet exhibit many of the distresses associated with loss of load transfer, but are at risk of developing them. Several studies have found dowel bar retrofits to be effective at minimizing distresses due to load transfer (Christopher 2000; Bishoff and Teopel 2002; FHWA/ACPA 2003).

To repair transverse joints, the recommended load transfer device is a smooth, round dowel bar (Bishoff and Teopel 2002), which is placed in a slot cut perpendicular to the joint which is being retrofitted, except when retrofitting skewed joints, in which slots are cut parallel to the pavement edge stripe. The slot is cut larger than the dowel, and thoroughly cleaned to remove any loose material or debris. An illustration of the ACPA recommended procedure for retrofitting is shown in Figure 42.

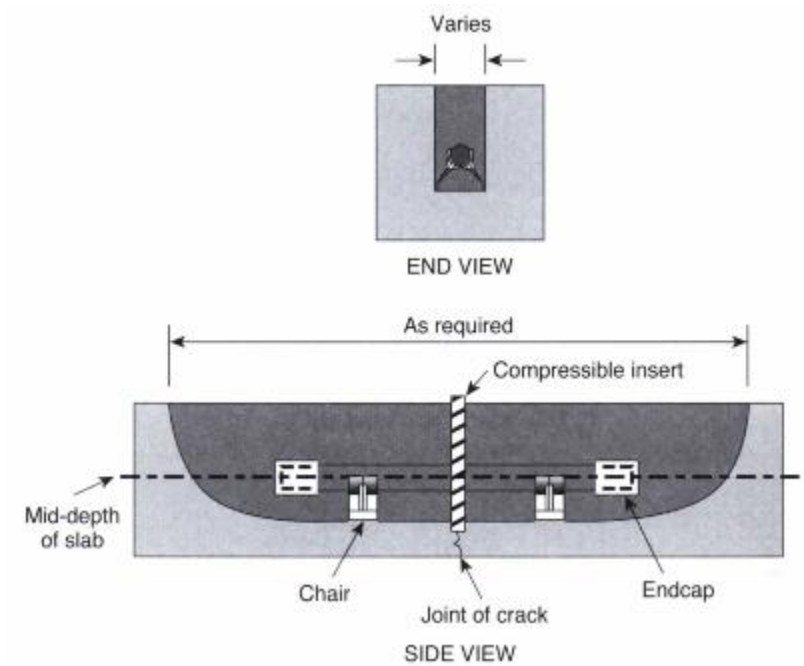

Figure 42. Detail for the installation of dowel bar retrofits (from ACPA 2006a)

Slab thickness dictates the required size of the dowel bar, and for recommendations the reader is referred to ACPA (2003). At least a six inch length of dowel bar should extend into each face of the joint, and dowel bars should be placed at every 12 inches along the length of the joint. For transverse joints, it is recommended to place three to four dowels in the wheel path, still at a 12 inch spacing (ACPA 2006a). A typical dowel bar retrofit pattern is illustrated in Figure 43.

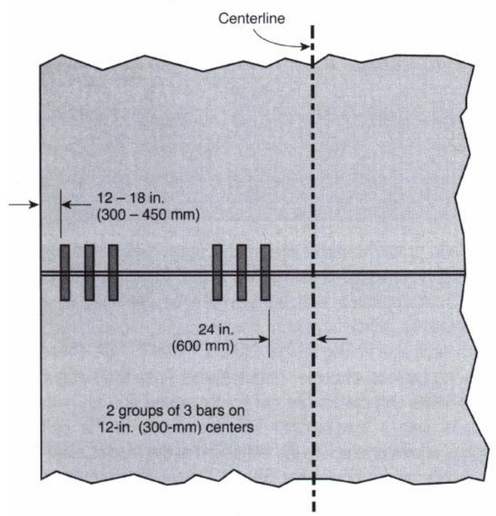

Figure 43. Recommended location of dowel bar retrofits (ACPA 2006a)

The entire dowel bar retrofitting procedure for transverse joints is summarized as:

1. Saw slot for dowel bar
2. Remove concrete to form kerf and rinse with water
3. Sandblast and vacuum clean slot
4. Seal or prime slot; seal cracks and joints
5. Place and align dowel bars and joint filler material
6. Place repair material (Larson et al. 1998).

For longitudinal joints, a technique called cross-stitching is used to hold the joint together and reduce the potential for faulting (Figure 44). In this method, deformed tie bars are used as the load transfer device instead of dowel bars.

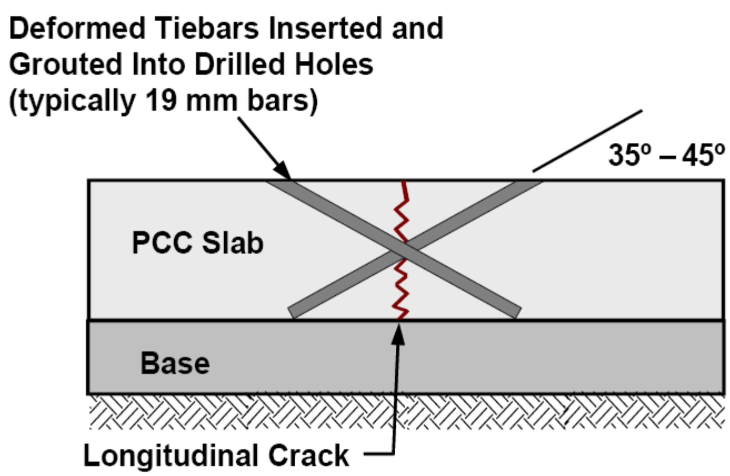

Figure 44 Configuration of tie bars to hold together a longitudinal joint, a technique known as cross stitching (ACPA 1995)

In cross-stiching a longitudinal joint, holes are drilled on either side of the joint to cross the joint at mid-depth at an angle of 35-45°, and tie bars are inserted and grouted in place. The size and spacing of the bars is dependent on the slab thickness, angle of the hole, and expected traffic loading (Figure 45). For further guidance on cross-stitching, the reader is referred to ACPA (1995).

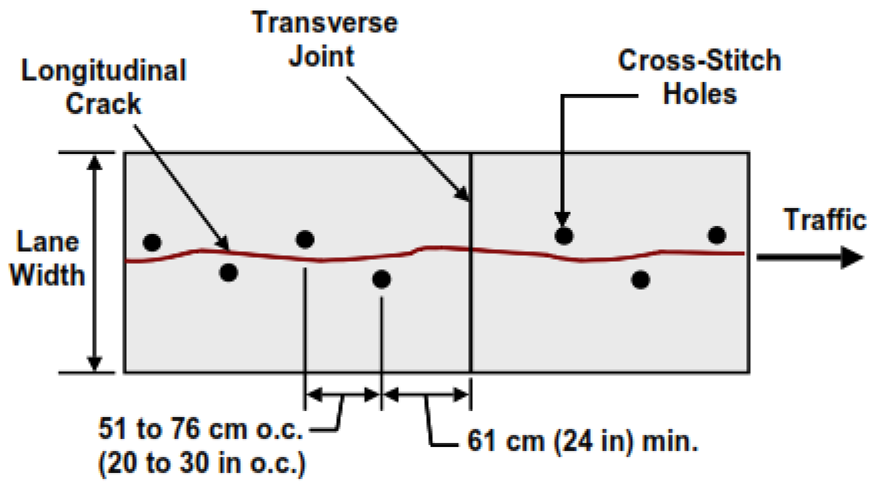

Figure 45. Location of tie bars for cross stitching (ACPA 2001)

The filler material used to patch the slot for the dowel bar or holes drilled for the tie bars is as important a selection as the load transfer device itself. The material used to replace the concrete which was removed must be carefully selected to ensure it will be compatible with the original concrete slab. For dowel bar retrofits, either a cement based concrete or epoxy filler can be used, while only epoxy is used in cross-stitching. It is important to select a repair material with a similar coefficient of thermal expansion compared to the original concrete to prevent cracking due to differential expansion and contraction. The shrinkage characteristics of the repair material are also important because cracking may result if the repair shrinks considerably during curing.

Other factors that must be considered when selecting an appropriate repair material are the amount of time the roadway may be closed to traffic (which may eliminate materials which gain strength slowly), the required strength of the patch, and freeze-thaw durability, as dictated by climatic conditions (NCHRP 2004a, ACPA 2003, ACPA 2006a, Bishoff and Teopel 2002). Following placement of the filler material, the joint should be resealed. Care should be taken to ensure that filler material does not enter the joint between slabs, otherwise slab spalling or cracking will likely occur. If there is a difference in elevation between the adjacent slabs, diamond grinding may be used to eliminate this difference and improve ride quality.

3.5 PCC Slab Preparation and Cleaning

Once all repairs discussed above have been made, the surface of the concrete slab needs to be cleaned and prepaired to receive the overlay. The surface can be cleaned by sweeping or compressed air to remove any debris (Wen et al. 2005). For areas with mud, or other difficult to remove substances, spraying with water may be effective for cleaning, but the area must be allowed to dry before the overlay is applied.

3.6 Fractured Slab Techniques for Existing PCC Pavement

As an alternative to traditional pre-overlay repairs, the existing concrete pavement can be fractured into many pieces to create a base layer on which to place the asphalt overlay. Crack-and-seating and rubblization are two types of fractured slab techniques that can be used prevent reflective cracking in asphalt overlays of concrete pavements (Figure 46). These techniques are particularly effective for deteriorated pavements (Thompson 1999).

In both techniques, the existing concrete slab is broken into small pieces before placement of the overlay. The difference between the two is the size of the resulting pieces of concrete. Crack-and-seating results in 12 to 24 inch "slablets" of concrete, while rubblization pulverizes the concrete until it resembles a granular material (Ceylan et al. 2005) . The rationale behind further breaking the concrete is that the probability of reflective cracking decreases as the crack spacing decreases (PCS 1991). This works because smaller crack spacing means that each individual piece of concrete is smaller and therefore deforms less in response to temperature changes, which produces lower critical strains in the asphalt overlay (Ceylan et al. 2005).

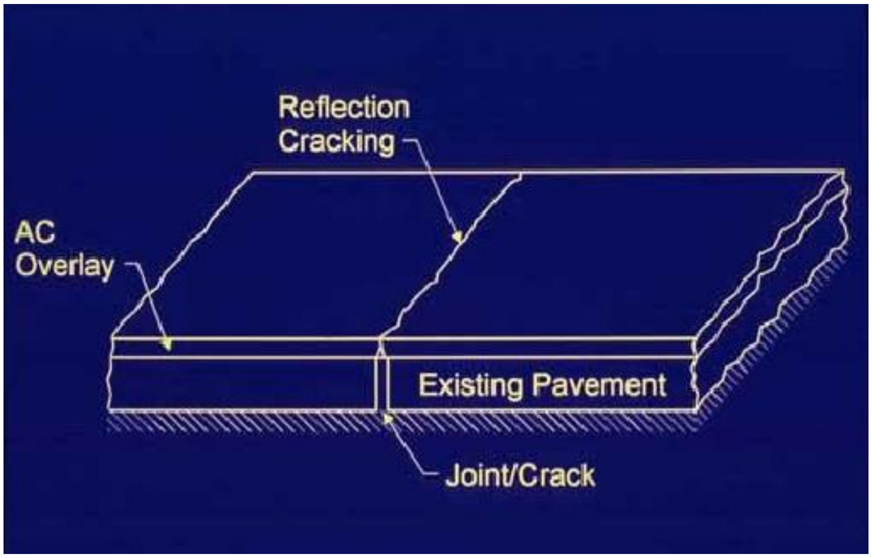

Figure 46. A reflective crack propagates from the existing concrete pavement into the new asphalt overlay (from Ceylan et al. 2005).

3.6.1 Crack-and-seat

Crack-and-seating is also called break-and-seating, though convention dictates that the term crack-and-seating is for jointed plain concrete pavements while break-and-seat is reserved for jointed reinforced concrete pavements. By cracking the slab into pieces roughly two feet in diameter, load transfer between the slablets is partially preserved, but the potential for reflective cracking is reduced. Cracking is accomplished via large construction equipment such as pile drivers or hammers, which raise heavy weights above the pavement surface and drop them to fracture the concrete slab (Ceylan et al. 2005). It is critical to performance to ensure that rubblization occurs throughout the entire slab thickness.

When determining the target size of the slablets, a balance must be struck between big enough (so that the slablets maintain interlock necessary to maintain a portion of the structural integrity) and small enough (so that the slablets mitigate reflective cracking due to reduced slab length) (PCS 1991). The cracking pattern achieved is dependent on the type of machine used to crack the concrete; Figure 47 shows a sample crack pattern produced by a whiphammer.

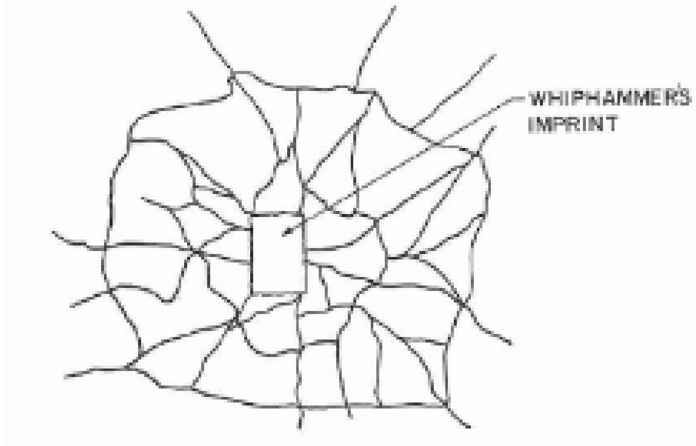

Figure 47. Cracking pattern produced by a whiphammer (from Osseiran 1987).

After the pavement has been cracked, a roller is used to seat the resulting pieces, which helps to create a firm and relatively level foundation on which to place the asphalt overlay. Seating reestablishes contact and support between the concrete slab and the underlying layers which may have been lost during cracking. During the rolling process, it is also possible to locate any soft areas in the base or subgrade which should be replaced before the overlay is placed (Freeman 2002).

3.6.2 Rubblization

Rubblization is similar to crack-and-seat, except that the resulting concrete pieces are much smaller, resembling granular material rather than slablets. Typically, the rubblized pieces are 2-3 inches at the surface of the PCC layer. Due to the rubblization process, pieces are larger on the bottom of the PCC layer, typically on the order of 9-12 inches (Thompson 1999). The smaller size of the rubblized pieces means that, in contrast to crack-and-seat pavements, rubblizing destroys the integrity of the slab, resulting in no load transfer between the pieces (Galal et al. 1999).

Rubblization is not a suitable pre-overlay treatment for pavements which have poor subgrade support (Ceylan et al. 2005). If rubblization is conducted when the underlying layers are saturated, the entire pavement system can be damaged. Therefore, if the drainage assessment of the pavement revels that edge drains are necessary, they should be installed prior to rubblization (Ceylan et al. 2005).

To rubblize a pavement, either a resonant pavement breaker (RPB) or a multi-headed breaker (MHB) is used to reduce the rigid pavement to a granular material. Figure 48 shows a rubblized concrete slab. While MHBs were primarily used in the past because they only required one pass to rubblize an entire lane and posed less risk of damaging the subgrade (Thompson et al. 1997), RPBs have been updated in recent years so that the methods both yield quality results (Ceylan et al. 2005). A test hole must be dug after rubblization to ensure that proper size pieces are being achieved with depth. More information on this can be found in the Wisconsin Department of Transportation Construction and Materials Manual (WisDOT 2004).

Figure 48. Existing concrete pavement after being rubblized using MHB method (from Ceylan et al. 2005)

After the pavement is rubblized, it must be rolled before the overlay can be placed. Rolling rubblized pavement is analogous to seating cracked pavement in the crack-and-seat procedure and is used to create a solid, stable base on which to place the overlay. Vibratory rollers, pneumatic (drum) rollers, or a combination of the two can be used to compact the rubblized pavement (Ceylan et al. 2005). When both types of rollers are used, the common practice is to make the first few passes with a vibratory roller equipped with a grid head (Figure 49) (WisDOT 2004), followed by one to two passes with a drum roller (Figure 50), and the final few passes with a vibratory roller with a smooth head (Ksaibati et al. 1999). If the stability of the rubblized pavement is in doubt after rolling is completed, the surface may be proof rolled to determine visually if proper compaction was achieved (WisDOT 2004).

Figure 49. Grid the rubblized pavement (left), and the surface created by grid rolling (from Ceylan et al. 2005)

Figure 50. Surface created by drum rolling (from Ceylan et al. 2005)

Chapter 4. Asphalt Concrete Overlay Mix Design

Rehabilitation of existing PCC pavements with AC overlays involves placing one or more layers of AC over the PCC. The AC may be placed directly on the existing PCC or placed over a broken or rubblized PCC layer (Figure 51).

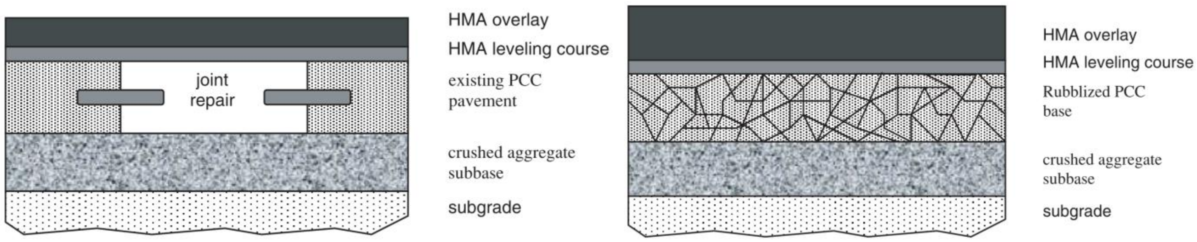

Figure 51. Examples of AC overlays to rehabilitate existing rigid pavements, here for a repaired PCC slab (at left) and rubblized PCC pavement (at right) (from Christensen and Bonaquist 2011)

The use of AC overlays on existing or rubblized PCC pavements involves placing an AC surface course, and possibly a thin leveling course of variable thickness to improve smoothness prior to placing the AC layers. The following subsections describe a few general details of asphalt concrete paving materials and details specifically describing materials/conditions of note for AC overlay surface or thin leveling layers. In general, much of the AC mix design selection process for AC overlays of PCC is identical to the process used for partial- or full-depth AC pavements. As a result, where appropriate, the guidelines will address special considerations for AC-PCC.

4.1 Important Performance Concerns for AC-over-PCC

Several important factors should be considered when selecting an AC mixture for the repair or rehabilitation of existing PCC pavements. These include:

- Rut resistance;
- Reflective cracking resistance;
- Raveling resistance;
- Noise suppression;
- Top-down cracking resistance to traffic loads and low temperatures; and
- Skid resistance.

Balancing these factors is necessary in AC mix design. Mix design properties for AC overlays will be considered in light of these important factors, though there may be overlap. For example, stiffer mixes have better rutting resistance but worse fatigue resistance for a given strain, while the reverse is true for softer mixes.

4.1.1 Rut Resistance

The required rut resistance of a mixture depends on the traffic level and the location of the mixture in the pavement structure. Pavements with higher traffic levels require greater rut resistance than pavements with low traffic volumes. Surface and intermediate layers require greater rut resistance than base layers. Extensive studies of the effect of asphalt overlay parameters, including properties of AC mix design, are detailed in Von Quintus et al. (2012).

4.1.2 Reflective Cracking Resistance

Reflective cracking is one of the primary forms of distress in AC overlays of PCC pavements. Reflected cracks degrade ride quality and introduce water and debris through these cracks into the pavement system, which exacerbates the deterioration of the overlay. The mitigation of reflective cracking is an important consideration in the mix design of AC to be used in composite pavements, and an increasing amount of research in this field is being developed to address this issue both in modeling and AC mix design (Lytton et al. 2010; Hu et al. 2010; Bennert 2009).

It should be noted that for thinner overlays, the need to mitigate reflective cracking, in a certain sense, competes with the need to mitigate rutting. Where a mix should be stiffer to resist rutting, it should also be more compliant and tougher to minimize cracks reflective through the overlay because for thin overlays, increases in the mix stiffness have little or no effect on the tensile and shear strains (Harvey et al. 2004). On the other hand, for thicker overlays, greater stiffness of the mix results in better rutting resistance and the greater stiffness combined with the thickness can reduce the tensile and shear strains. For thicker overalays, the effect of reducing the tensile strain has more effect on the fatigue life than does the lower fatigue for a given strain of a stiffer mix.

This concept is illustrated in Figure 52, which compares the logarithm of the tensile strain in a beam test with the number of load repetitions for permutations of beam specimens cut from pavements with thick and thin AC layers (*T* and *t* respectively) and mixes with stiff binders and soft binders (*S* and *s* respectively). It can be seen in the figure that for a given strain, the softer binder has a longer fatigue life, and that for the thin overlay increasing the stiffness of the binder does not change the tensile strain much, resulting in a lower fatigue life (y-axis) for the stiffer binder (NtS). However, it can be that for the thick overlay (T), the stiff binder has a larger effect on the tensile strain, and the reduced strain results in a greater fatigue life for the stiff binder (NTS). For many binders, one can substitute cold and hot temperatures for the same binder in place of stiff and soft binders, respectively, in the figure.

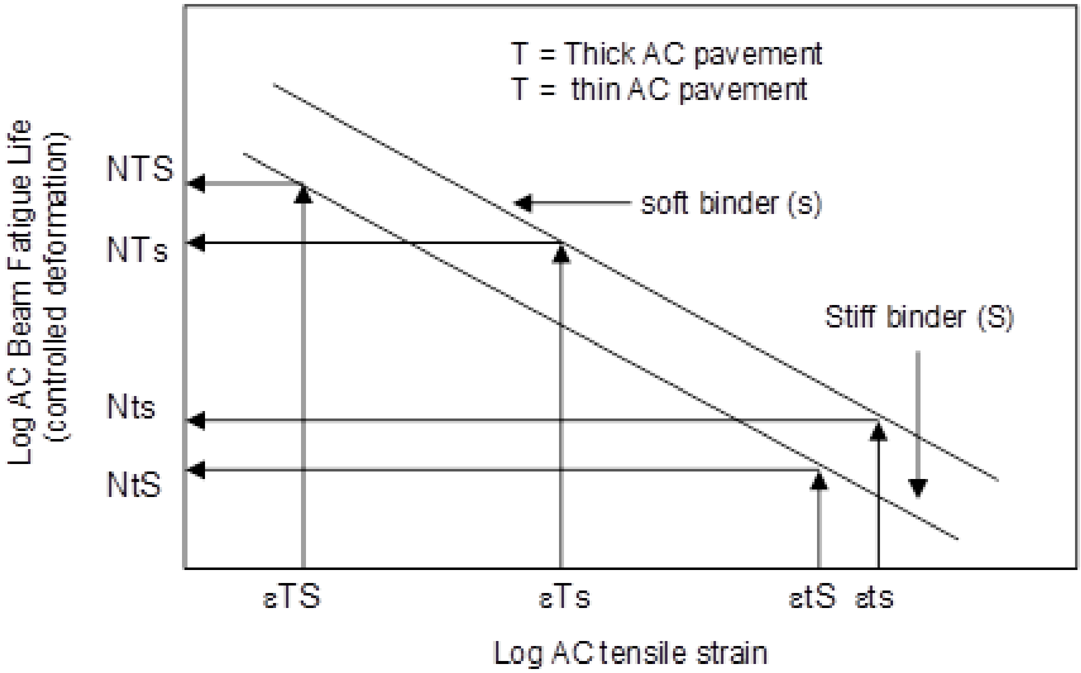

Figure 52. Illustration of net effects of binder stiffness and overlay thickness on fatigue life (from Harvey et al. 2004)

This consideration complicates the selection of the appropriate mix for an AC overlay, discussed later in Section 4.3.

4.1.3 Raveling resistance

Raveling resistance mitigates the disintegration of an AC mixture due to exposure to the combined effects of weathering and traffic. AC surface courses have the most severe exposure, because they are subjected directly to damage by both traffic loading and the environment. Mixtures subjected to more severe exposure conditions must have greater durability. One of the best ways to increase durability of dense-graded and SMA wearing courses is to get good compaction during construction, which limits the ability of water and air to enter into the mix.

4.1.4 Noise suppression

Noise reduction has been a major concern for pavement engineers in Europe for many decades, and in the past decade noise from roadway traffic has become an issue of growing concern in the United States. Roadway noise is generated by many sources, but the most predominant source (for highway speeds) is tire/pavement interaction (Bernhard and Wayson 2005). The design of AC mixes to mitigate noise considers factors such as permeability, macrotexture, roughness, and durability (Ongel et al. 2007).

4.1.5 Top-down cracking resistance

Another important consideration related to traffic loading is the resistance of the AC overlay to fatigue cracking at the surface, or top-down cracking. Pavements with higher traffic levels require overlays with greater resistance to fatigue cracking (Roque et al. 2010). In addition, for AC overlays in the northern United States and Canada, mix design must consider low-temperature cracking, which is the primary distress for AC pavements in these climates (Marasteanu et al. 2004). One of the most important mixture design factors affecting fatigue resistance in either regard is the effective binder content of the AC mixture.

4.1.6 Skid resistance

The skid resistance of a pavement is an important given the correlation between low pavement skid resistance and accident rates (Masad et al. 2007). Pavement skid resistance is typically measured in the field in terms of a friction or skid number, where higher values correspond to increased friction and reduced stopping distances. The mechanism of skid resistance is generally considered in terms of pavement microtexture and macrotexture (Ongel et al. 2007). Given that the skid resistance of a pavement can change over time, it is very important that the AC mix design carefully consider desired surface friction both at opening to traffic and throughout the service life of the pavement.

In this respect, aggregate gradation has been identified as the most important design parameter in providing adequate skid resistance at highway speeds (texture wavelengths between 0.5 and 50 mm), (Masad et al. 2007; Ongel et al. 2007), while microtexture is particularly controlled by microtexture (texture wavelengths less than 0.5 mm) which is largely controlled by aggregate source and crushing. Keeping binder contents at a level that results in no “bleeding” or expulsion of asphalt to the surface, is also very important for maintaining skid resistance, because thick films of asphalt at the surface in the wheelpaths become very slippery when they are cold and wet.

4.2 Asphalt Concrete Mixtures for AC-PCC

AC mixtures are typically classified by whether or not the mix must be heated prior to transport; the manner of placement; and the use of compaction. The most common type of AC is hot-mix asphalt (HMA), which must be thoroughly heated during mixing, transport, placement, and compaction. The asphalt binder used in HMA is quite stiff at room temperatures, so that once this type of AC cools it becomes stiff and strong enough to support heavy traffic. These guidelines summarize detailed design procedures for five types of HMA mixtures: dense-graded asphalt (DGHMA), polymer-modified DGHMA, stone matrix asphalt (SMA), open-graded friction course asphalt (OGFC), and rubberized gap-graded asphalt. Warm-mix asphalt (WMA) is also discussed, although warm mix is primarily a compaction aid and is expected in-service mix properties similar to dense-graded HMA.

4.2.1 Dense-Graded HMA

Dense-graded HMA mixtures are the most commonly used mixtures in the United States. They can be used in any layer of the pavement structure for any traffic level. Traffic level is a direct consideration in the design of dense-graded mixtures. Aggregate angularity, fine content, binder grade, compactive effort, and some volumetric properties vary with traffic level in the dense-graded mixture design procedure. Dense-graded mixtures also provide the mixture designer with the greatest flexibility to tailor the mixture for the specific application (Christensen and Bonaquist 2011).

For instance, dense-graded mixtures can also be designed as fine or coarse mixtures. Fine mixtures generally have a gradation that plots above the maximum density line (when the gradation is plotted with sieve sizes raised to the 0.45 power) while coarse mixtures plot below the maximum density line. The sieve sizes used in the definition of fine and coarse mixtures in AASHTO M 323 is summarized in Table 1.

Table 1. Sieve sizes used to define fine and coarse mixes in AASHTO M 323 (from Christensen and Bonaquist 2006)

Nominal Maximum Aggregate Size	Primary Control Sieve	Percent Passing
37.5 mm	9.5 mm	≥ 47
25.0 mm	4.75 mm	≥ 40
19.0 mm	4.75 mm	≥ 47
12.5 mm	2.36 mm	≥ 39
9.5 mm	2.36 mm	≥ 47

For each nominal maximum aggregate size, a primary control sieve has been identified. If the percent passing the primary control sieve is equal to or greater than the specified value in Table 1, the mixture classifies as a fine mixture; otherwise it classifies as a coarse mixture. Fine mixtures have smoother surface texture, lower permeability for the same in-place density, and can be placed in thinner lifts than coarse mixtures, a feature that might be advantageous for thin AC overlays of PCC. However, because of generally greater surface area, they often require more binder than coarse mixes, increasing the cost of the mix.

A survey of 26 state highway departments found that the majority of the states surveyed use either a 9.5mm Superpave Performance Grade (PG) mix over a 12.5mm Superpave mix or a 12.5mm Superpave mix over a 19mm Superpave mix for HMA overlays of existing concrete pavements. Generally, the LTPPBind recommendation for the PG grade of the binder was used by the states (Bennert 2009). The PG binder grade controls the high temperature properties related to rutting and the low temperatures properties related to top-down low temperature cracking.

4.2.2 Polymer modified dense-graded asphalt

Binder selection is a very important issue to improve the response of an AC mixture to climatic conditions, and in this regard conventional asphalts are not necessarily the most appropriate choice for surfaces exposed to extreme weather conditions. While climate determines the performance grade of binder that will be used for the mixture type, polymer modified asphalts can provide the additional flexibility needed to withstand thermal stresses in cold climates and still maintain adequate stiffness to help resist rutting (Terrel and Epps 1989; Shuler et al. 1987).

The use of polymer additives to modify the performance of an AC mixture is a common practice in paving, and these modified asphalts are as applicable to AC overlays of PCC as they are full-depth asphalt paving.

4.2.3 Open-Graded Friction Course (OGFC)

OGFC is an open-graded mixture with a high air void content. The high air void content and open structure of the mixture provides macrotexture and high permeability to drain water from the tire-pavement interface. This minimizes the potential for hydroplaning, improves wet weather skid resistance, and reduces splash and spray.

Other benefits of OGFC include reduced noise levels, improved wet weather visibility of pavement markings, and reduced glare. OGFCs are made with durable, polish-resistant aggregates and usually contain modified binders and fibers to increase the binder content and improve their durability (Christensen and Bonaquist 2011). OGFCs, though generally more expensive than dense-graded mixtures, are placed in a thin 30 to 60 mm thick lift over a dense-graded or gap-graded lift (Ongel et al. 2007).

4.2.4 Rubberized Gap-Graded Asphalt (GGHMA or ARFC)

Gap-gradations are primarily used for rubberized binders to provide empty space in the gradation to accommodate the rubber particles or heavily polymer modified binders. They are densely compacted to maximize rut resistance and durability. The principal design consideration in gap-graded AC mixtures is to maximize the contact between particles in the coarse aggregate fraction of the mixture. This fraction provides stability and shear strength to the mixture. The coarse aggregate fraction is then essentially glued together by a binder-rich mastic consisting of a properly selected asphalt binder, and mineral filler and/or fibers in SMA mixes. The fibers are included to minimize draindown of the binder from the mixture during transportation, handling and placement.

The advantages of rubberized gap-graded hot mix asphalt (GGHMA) mixtures over dense-graded mixtures include (1) increased resistance to rutting, cracking, and aging and (2) improved durability, wear resistance, low-temperature performance, and surface texture (Christensen and Bonaquist 2011). GGHMA mixtures generally cost more than dense-graded mixtures due to their higher binder content, high filler content, stringent aggregate requirements, and the use of polymer-modified binders and fibers.

The use of rubber reclaimed from waste tires is one means of both accommodating modifications to the binder and reducing the cost of the mix. These gap-graded rubberized hot mix asphalt (GGHMA) are often used in the top 60 mm (2.4 inches) layer of pavements in California (Coleri et al. 2012) and in thickness of 1.5 inches to 2.0 inches of composite pavements in Arizona, where they are referred to as an asphalt rubber friction course (ARFC) when used with a more open gap-graded gradation (Scofield and Donovan 2003; Kaloush et al. 2009).

4.2.5 Stone Matrix Asphalt (SMA)

During the past 20 years, stone-matrix asphalt (SMA) has become increasingly common in the United States and Europe. SMA is a special type of GGHMA designed specifically to hold up under very heavy traffic. SMA is composed of high-quality coarse aggregate, combined with a large amount of mastic composed of a high-performance asphalt binder, mineral filler, and a small amount of fibers. The aggregate used in SMA contains a large amount of coarse aggregate and a large amount of very fine material (called mineral filler), but not much sand-sized material. A well-developed coarse aggregate structure in combination with a relatively large volume of high performance binder helps ensure that a properly designed SMA mixture will exhibit excellent performance. SMA is usually only used on very heavily trafficked roadways, where its excellent performance makes it cost-effective despite the high initial investment required to construct SMA pavements (Rao et al. 2011; Christensen and Bonaquist 2011). They are typically placed as a highly durable surface course that is 60 mm or less in thickness because of their high cost.

4.2.6 Warm Mix Asphalts

Warm-mix asphalt (WMA)—has recently become increasingly popular. A hot mix becomes a warm mix when an additive is used during the mixing process that increases the workability of the mix at lower temperatures. Otherwise, there is little or no difference from hot mix. In this type of mixture, various different methods are used to significantly reduce mix production temperature by 30 to over 100°F. Warm mix additives are generally grouped into the following three categories:

1. using chemical additives to lower the high-temperature viscosity of the asphalt binder, referred to as chemical addtivies;
2. techniques involving the addition of water to the binder, causing it to foam, referred to as mechanical foaming when water is injected; and
3. using chemical additives that produce water in the binder causing it to foam, referred to as chemical foaming agents.

WMA has several benefits, including potentially lower costs (depending on fuel use and additive costs), lower emissions and improved environmental impact, and potentially improved performance because of decreased age hardening. There is some concern that WMA might in some cases be more susceptible to moisture damage, but this has yet to be clearly demonstrated. Furthermore, given its relatively recent adoption, the long-term performance of WMA is as yet uncertain and undocumented, but preliminary indications suggest there is much potential for the application of WMA in AC-PCC (Jones et al. 2009).

4.3 AC Overlay Mix Design Selection

The selection of an appropriate AC mixture for AC-PCC is a difficult decision that involves consideration of cost, traffic, climate, construction, and performance concerns. Although the types of mixtures to be used in a project are usually selected during the design phase, it is important that mixture designers understand the rationale behind the selection of mixtures for specific applications.

In general, the pavement designer should follow a process for the AC overlay mix design that is similar to the method described in NCHRP Report 673 for general asphalt design according to the Superpave method as follows:

1. Gather Information
2. Select Asphalt Binder
3. Determine Compaction Level
4. Select Nominal Maximum Aggregate Size
5. Determine Target VMA and Design Air Void Content
6. Calculate Target Binder Content
7. Calculate Aggregate Content
8. Proportion Aggregates for Trial Mixtures
9. Calculate Trial Mix Proportions by Weight and Check Dust/Binder Ratio
10. Evaluate and Refine Trial Mixtures
11. Compile Mix Design Report

Most of these steps are straightforward and easily accomplished. However, Steps 8 through 10 are more complicated and require some experience in order to perform them proficiently. The reader is referred to NCHRP Report 673 for more detail and references on this process, including a developed spreadsheet for AC mix design (Christensen and Bonaquist 2011).

Given that different applications require different mix designs, the pavement designer must consider desired performance (e.g. design life), climate, and traffic volumes together to best select an appropriate mix design for the AC overlay of a PCC pavement. The following subsections provide comments and recommendations on AC mixtures based on performance concerns for the constructed AC overlay, with comments addressing design life, traffic, and/or climate where appropriate.

4.3.1 Rutting

For dense-graded mixtures, aggregate angularity, binder grade, compactive effort, and some volumetric properties are selecting depending on traffic level and layer depth to provide adequate rut resistance. Crushed faces and rough surface texture are the most important properties controlling rutting. GGHMA and OGFC mixtures are designed to ensure stone-on-stone contact to minimize the potential for rutting. Binder grade for these mixtures is also selected considering environment and traffic level using the PG specification. Polymer modified binders will often improve rutting resistance. The PG binder specification is currently undergoing review to consider the greater elastic recovery under repeated loading of polymer modified binders compared to conventional binders. In general, thin overlays on concrete will generally not rut as much as thicker overlays on concrete or overlays placed on asphalt due to shear restraint from the underlying PCC slab (Coleri et al. 2012).

4.3.2 Reflective Cracking

Bennert (2009) found that the time for reflective cracking to appear is partially dependent on the low temperature PG grade of the binder used. Figure 53 illustrates the low temperature PG grade recommended by LTPPBind for locations in the United States.

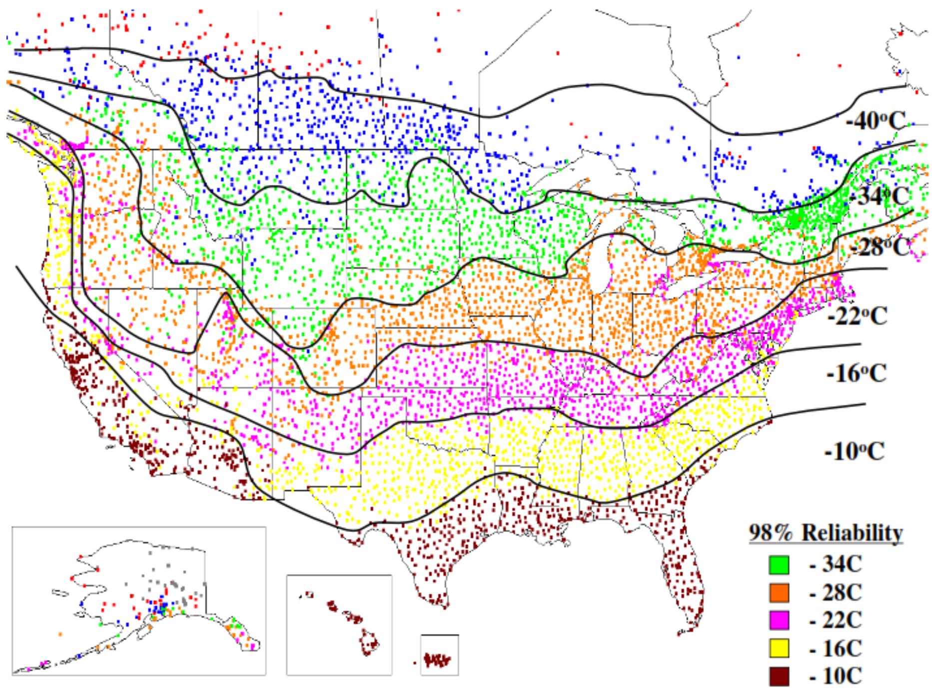

Figure 53. Recommended low temperature PG grade from LTPPBind (from Bennert 2009)

Likewise, Figure 54 describes the same recommendations as those of Figure 53, superimposed on a map illustrating the time until reflective cracking is generally observed in AC overlays of PCC pavements. From these figures, it can be seen that states using low temperature PG grades between -10°C and -16°C had the longest time before reflective cracking was observed, most likely because they had the warmest winters. Bennert (2009) observed that lower recommended low temperature PG grades for a given climate correspond to less time before the occurrence of reflective cracking.

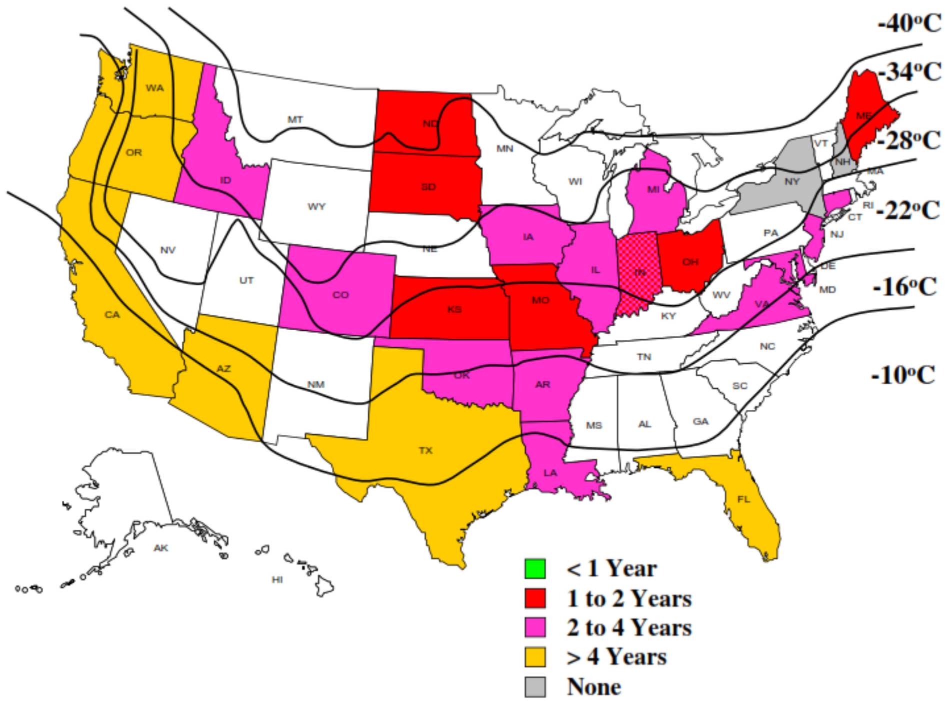

Figure 54. Time in years until reflective cracking occurs in AC overlays of PCC pavements and recommended low temperature PG grade from LTPPBind (from Bennert 2009)

Bennert (2009) also found that states using a binder with a PG low temperature grade lower than that recommended by LTPPBind by one or two grades had a longer pavement life before reflective cracking was observed. Conversely, composite pavements in states that used a binder with a PG low temperature grade higher than that recommended by LTPPBind by one or two grades had a shorter pavement life before reflective cracking was observed.

As noted in the overview of performance concerns for AC overlay mix design, rutting and reflective cracking are often considered alongside each other given their somewhat contradictory needs from a mix design point of view for thinner overlays. Recent work at the Texas Transportation Institute (TTI) involved the use of an AC overlay tester, which can be used to simulate horizontal opening and closing of joints or cracks in the pavement below the asphalt overlay (Figure 55). This allows the resistance of an asphalt mixture to reflective cracking to be measured using specimens that are easier to obtain than the currently used four point beam samples.

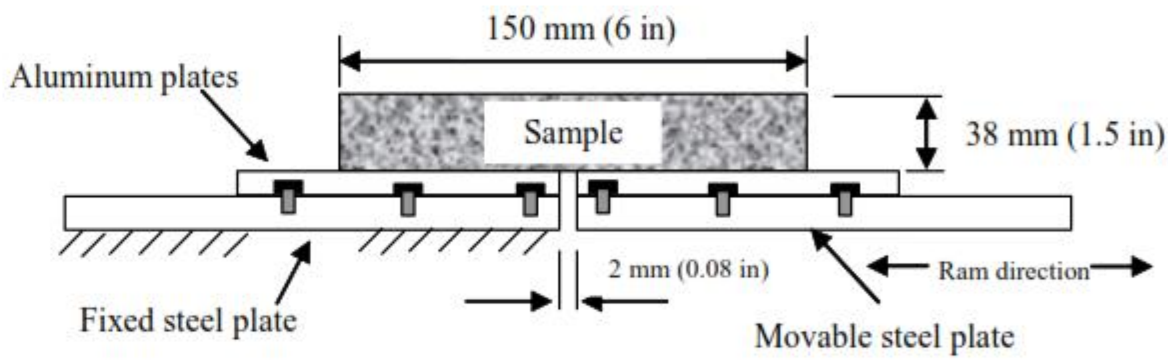

Figure 55. TTI overlay tester to investigate mix design influence on reflective cracking (from Zhou and Scullion 2003)

Using the overlay tester to examine in place overlays, Zhou and Scullion (2003) determined that overlays designed with stiffer binders had less resistance to reflective cracking than those using softer binders. Furthermore, Zhou et al. (2006) developed an HMA mix design selection procedure for overlays that accounted for both reflective cracking and rutting concerns. This procedure is outlined in Figure 56. Using this procedure, Zhou et al. developed recommendations for the binder content for HMA overlay mixes, illustrated in Figure 56.

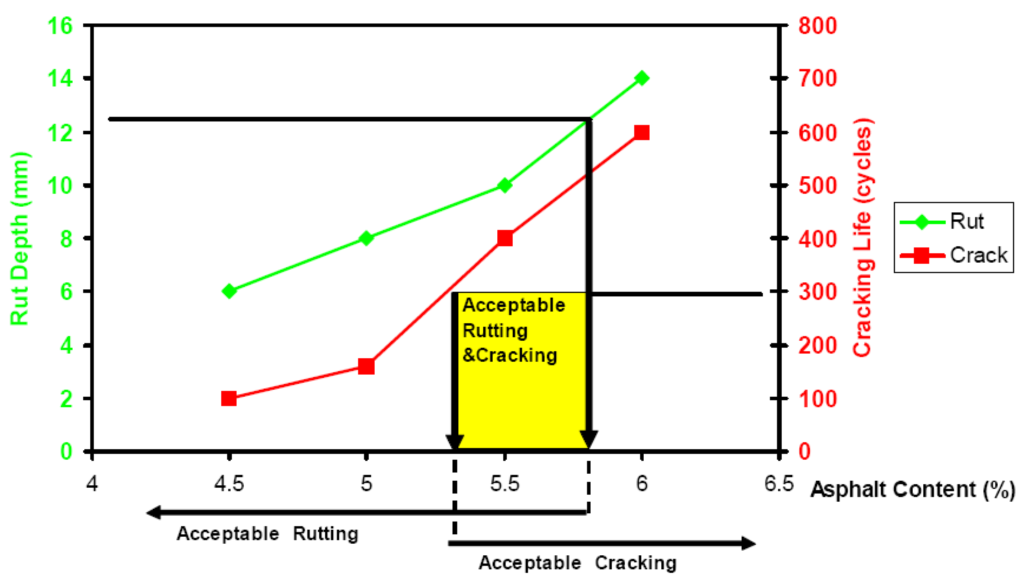

Figure 56. Overlay binder content to optimize resistence to rutting and fatigue cracking from Zhou et al. (2006)

4.3.3 Raveling

NCHRP Report 567 summarizes the relationships among AC composition and performance in raveling. For the most durable mixes—ones with good fatigue resistance and low permeability to air and water—high binder contents are needed, along with a reasonable amount of fine material in the aggregate. Perhaps most importantly, the mix should be well compacted during construction. In general, both the binder content and the amount of fines in the aggregate blend will increase with decreasing aggregate nominal maximum aggregate size (NMAS). This is one of the reasons that smaller NMAS mixtures are used in surface courses (Christensen and Bonaquist 2006). The effective binder content of GGHMA mixtures is very high due to the gap-graded structure of these mixtures.

OGFC mixtures typically incorporate modified binders and fibers to increase the binder content of these mixtures and improve their durability. If OGFC is selected, high-speed traffic is an important consideration because it helps keep the pores from clogging with debris, however in general raveling will become a problem for OGFC within about 8-12 years depending on traffic and winter maintenance (Ongel et al. 2007; Sandberg and Eismont 2002).

4.3.4 Noise and skid resistance

Both OGFC and rubberized gap-graded mixes perform equally well in reducing noise and improving skid resistance if their initial air-voids are similar, however, rubberized gap-graded mixes typically densify under traffic and lose some of the noise reducing properties (Ongel et al. 2007). Whereas OGFC can reduce hydroplaning and spray and splash and hence improve safety, its high permeability may lead to increased raveling relative to the performance of less permeable polymer-modified dense-graded and rubberized gap-graded mixes. OGFC mixes have been found to lose their noise-reducing advantage over a typical dense-graded mix within 7 years of placement due to clogging and distresses at the pavement surface (Ongel et al. 2007), which is improved if the open-graded mix has a rubberized asphalt binder (Lu et al. 2010).

Another important consideration for OGFC in mitigating noise and improving skid resistance is climate. The open structure of OGFCs causes these mixtures to freeze more quickly than dense-graded and GGHMA mixtures, resulting in the need for earlier and more frequent application of deicing chemicals. The SHRP2 R21 project discussed these difficulties in its tour of European composite pavements, where engineers from countries with freeze-thaw cycles discussed OGFC difficulties in AC-PCC applications (Rao et al. 2011). Additionally, sand should not be used with the deicing chemicals because the sand will plug the pores of the OGFC, decreasing their effectiveness in both noise and skid resistance. Finally, as SMA mixes are generally impermeable, they do not reduce the air pumping mechanism of tire/pavement noise. Their noise level primarily depends on their macro texture and the tire vibration mechanism.

4.3.5 Top-down cracking resistance

To resist load-related top-down cracking, Christensen and Bonaquist (2011) recommend dense-graded mixtures of smaller nominal maximum aggregate size and GGHMA mixtures should be considered for high traffic levels. The dense-graded mixture design procedure provides the flexibility to increase the design VMA requirements up to 1.0% to produce mixtures with improved fatigue resistance and durability. Increasing the VMA requirement increases the effective binder content of these mixtures over that for normal dense-graded mixtures. SMA mixes have also been shown to perform very well in terms of resistance to top-down cracking. For resistance to low-temperature cracking, established PG binder specifications can be followed to mitigate this distress for AC overlays in a manner consistent with that of full-depth AC pavements.

Chapter 5. Asphalt Concrete Overlay Structural Design

Chapters 1 through 4 describe how, for the most part, materials and construction practices for composite pavements utilize state-of-the-art techniques at all levels of application. In contrast are the structural design procedures currently utilized by state DOTs. A review of the AC overlay design procedures of 26 states for the overlay of PCC composite pavements found the following design practices (Bennert 2009):

- 77% of twenty-six state DOTs surveyed use the 1993 AASHTO design procedure (AASHTO93) to determine AC thickness. Of these states, most compare the thickness from this method to a minimum thickness as determined from their prior experience. One state compared the AASHTO93 results with the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) recommendations for thickness.
- 23% of state DOTs surveyed used a standard thickness for all AC overlays of PCC. This thickness was determined based on past experience, traffic, existing pavement condition, and cost.

In this respect, while contemporary mechanistic-empirical (M-E) design such as the MEPDG has been adopted widely for more conventional single-lift pavement design, these procedures have yet to be accepted for composite pavements. Some of this is to be expected, as the mechanistic-empirical models that form the basis of these design procedures have yet to be examined specifically for AC-PCC. (A first step in this direction is work described in the TPF-5(149) final report. While that work is for TICP projects, it can easily be generalized to AC-PCC.)

However, though structural models for composite pavements still require validation and verification, there is much in mechanistic-empirical design that can benefit the pavement engineer who wishes to design a composite pavement with optimal thickness and material use. The following subsections describe a brief introduction to available pavement design procedures for AC-PCC pavements, and in so doing briefly review the empirical design of AASHTO 1986 and 1993 and the M-E design of its successors. While there are many other design methods for AC overlays of existing PCC pavements, such as the Caltrans Overlay Design, US Army and Air Force Design, Illinois Department of Transportation Design, UK Pavement Design Guide, and the Danish Road Institute Design, these methods are not summarized here. The reader is referred to Gerado et al. (2008) for more detail on these design procedures.

For recent procedures such as MEPDG or SHRP2 R23, the procedures have been applied to examples to illustrate their use.

5.1 AASHTO 1986/1993 Design Guide

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, EPCC decreases; as a consequence, they recommended using a maximum value called EPCC 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).

5.1.1 AASHTO93 Procedural Overview for AC-PCC

The resulting outline to follow of 1993 AASHTO Design Guide method for rehabilitating a PCC pavement with an AC overlay is adapted from Gerado et al. (2008) and Fwa (2006). The procedural outline depends on whether fracturing of the existing PCC slab is required.

5.1.1.1 Fractured PCC slab

If the existing PCC pavement is fractured, the thickness of the AC overlay is determined by the Structural Number approach and is given as

$$D_1 = \frac{SN_{OL}}{a_1} \quad (5.1)$$

where D_1 is the required thickness of the AC overlay, SNOL 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

$$SN_{OL} = SN_T - SN_{eff} \quad (5.2)$$

where SNT is the structural number required if a new flexible pavement were to be constructed on the subgrade, and SNeff is the effective structural number of the existing pavement after fracturing. The structural number of the existing pavement after fracturing is defined as

$$SN_{eff} = a_2 D_2 m_2 + a_3 D_3 m_3 \quad (5.3)$$

where a_2 and a_3 are the layer coefficients of the fractured slab and the base layers, respectively; D_2 and D_3 are the layer thickness of the fractured slab and the base layers, respectively; and m_2 and m_3 are the drainage coefficients of the fractured slab and the base layers, respectively.

5.2.1.2 Intact PCC slab

If the existing PCC pavement is not fractured, 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, DOL, is calculated as

$$D_{OL} = A*(D_f - D_{eff}) \quad (5.4)$$

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. Furthermore, the conversion factor, *A* is defined as

$$A = 2.2233 + 0.0099*(D_f - D_{eff})^2 - 0.1534*(D_f - D_{eff}) \quad (5.5)$$

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

$$D_{eff} = CF * D \quad (5.6)$$

and *D* is the original slab thickness and conversion factor CF is

$$CF = F_{jc} * F_{dur} * F_{fat} \tag{5.7}$$

where F_{jc} is the joint and crack adjustment factor; F_{dur} is the durability adjustment factor; F_{fat} is the fatigue adjustment factor. Note that the *CF* is determined using either the remaining life of the pavement (estimated with the ratio of total ESALs for failure to the number which have already occurred) or Equation (5.7).

5.1.2 AASHTO93 example for AC overlay of existing PCC pavement

In the AASHTO-93 design method, the thickness of an asphalt overlay is selected based on the difference between the actual PCC pavement thickness and the computed PCC pavement thickness required to handle the future traffic loads. This value is then adjusted to account for the fact that the overlay will be asphalt instead of PCC.

For this example and other examples in this chapter, a 9-inch JPCP with minor distress cracking, 20% cracked slabs, and some faulting (0.15 inches) is assumed as the in-place PCC pavement to be overlaid. This pavement includes an 8-inch granular base and a 25-year design life.

To begin one determines D_F – through the standard AASHTO93 PCC design procedure – for the given conditions. The evaluation of D_F for this example requires the solution of

$$\log(W_{18}) = Z_R S_0 + 7.35\log(D_F+1) - 0.06 + \frac{\log\left[\frac{\Delta PSI}{3}\right]}{1+\frac{1.624*10^7}{(D+1)^{8.64}}} + (4.22 - 0.32P_t)\log\left[\frac{S_c' C_d\left(D_F{}^{0.75} - 1.132\right)}{215.63*G\left[D_F{}^{0.75} - \frac{18.42}{(E_C/k)^{0.25}}\right]}\right] \tag{5.8}$$

where W_{18} is the number of ESALs for the life of the overlay; Z_R is the standard normal deviate; S_0 is the standard error; ΔPSI is the change in pavement serviceability index; P_t is the terminal serviceability of the section; S_c' is the modulus of rupture of the concrete; C_d is the drainage coefficient; *J* is the load transfer coefficient; E_c is modulus of elasticity of the concrete; and *k* is the modulus of subgrade reaction. For the example of this chapter to be used for all design procedures, the values of Table 2 are assumed to specify the pavement system and conditions.

Table 2. Additional values for Chapter 5 AC overlay example

Parameter	Value
W_{18}	10,000,000 ESALs
Z_R	-1.645
S_0	0.5
ΔPSI	2.5
P_t	2.0
S_c'	690 psi
C_d	1
J	3.2
E_c	4,403,280 psi
k	165 psi/in

Applying these values to Equation (5.8) and solving for D_F yields a required PCC thickness of approximately 10.75 inches. This value for D_F is to be inserted into Equation (5.4).

The next step in the process is to determine the effective thickness of the existing pavement system, D_{EFF}, in Equation (5.4) using Equations (5.6) and (5.7) above. For this step, as detailed previously, the as-built thickness of the existing PCC, *D*, is 9 inches. For the adjustment factors to more closely describe the pavement condition, we select the values of Table 5.

Table 3. Adjustment factors to determine overall condition factor, *CF*, in AASHTO 93 overlay design

Condition factor	Value	Description
F_{jc}	0.85	Adjustment for joints and cracks
F_{dur}	0.96	Adjustment for durability
F_{fat}	0.90	Adjustment for of fatigue cracking

Using Equation (5.7), we calculate a conversion factor of 0.73. By inserting this value into Equation (5.6), our effective thickness for the existing pavement, D_{eff}, is 6.61 inches.

Having calculated both D_f and D_{eff}, we can now use Equations (5.4) and (5.5) to determine the required AC overlay thickness. Inserting these values into Equation (5.5) results in a value of 1.72 for the conversion factor, *A*. Using this value, along with D_f and D_{eff}, in Equation (5.4), we calculate a value of approximately 7.25 inches for D_{OL}, the AC overlay thickness.

5.2 MEPDG (DARWIN-ME)

The AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) allows the user to select and design AC overlays for existing PCC pavements. While this includes CRCP, only JPCP is assumed for this synthesis. The reader is referred to the MEPDG Manual of Practice for a discussion of AC-over-CRCP projects (AASHTO 2008). As with AASHTO93, the analysis first considers whether the existing PCC pavement is intact or has been fractured using crack-and-seat or rubblization techniques. The AC-PCC analysis for the MEPDG also considers damage in the PCC slab throughout the life of the AC overlay; this analysis uses the JPCP performance models.

Given the complexity of the MEPDG and the amount of data it accepts as input and produces as output, the MEPDG Manual of Practice (AASHTO 2008) should be consulted to describe the required information to build an AC-PCC project. The general procedure (detailed in the following subsection) for using the MEPDG to design an AC-PCC pavement is straightforward, this summary only emphasizes key concepts and is in no way comprehensive.

The AC overlay project for the MEPDG begins, as indicated in Chapter 2 above, with an evaluation of the existing PCC pavement. This includes its design features and thickness – from this information the MEPDG determines needed properties of the damaged modulus of the existing bound layers. All properties of the existing pavement layers and the overlay need represent anticipated conditions when the pavement is opened to traffic. The design procedure presented in the MEPDG considers distresses in both the overlay and existing PCC slab through the life of the pavement project.

For AC-PCC projects, in addition to general project inputs (reviewed in the general summary of the MEPDG procedure below), there are some important input parameters and issues to consider.

- The first is the number of AC layers in the overlay; according to the MEPDG Manual of Practice, the AC overlay itself is limited to three unique layers in the AC overlay, each requiring its own material inputs (AASHTO 2008).
- The user must also consider how the MEPDG treats reflection cracking. The empirical reflection cracking models included in the MEPDG have not been nationally calibrated and thus local calibration is even more important.
- It is also important to estimate the damage experienced by the PCC slab prior to traffic opening. This estimate influences performance in cracking for the rehabilitated pavement. More information on determining the input parameter for past damage can be found in the MEPDG Manual of Practice (AASHTO 2008).
- The dynamic modulus of subgrade reaction (or dynamic k-value) can be determined from resilient modulus inputs for the existing unbound sublayers including the subgrade soil or by backcalculating FWD measurements performed on the existing slab. More detail on converting the backcalculated elastic modulus of subgrade layer to a laboratory equivalent resilient can be found in Sadasivam and Mallela (2012), which also provides guidelines to adjust the resilient modulus according to as-constructed conditions.
- The value for the elastic modulus of the pre-existing JPCP is value for the slab just prior to rehabilitation. The input elastic modulus is that of the intact slab; it is not a reduced value due to observed cracking (as is done for unbonded PCC overlays). More considerations on this value can be found in the MEPDG Manual of Practice (AASHTO 2008).

Finally, while the MEPDG prompts the user to select a location for a given project, many users may choose to use locally available climate data rather than the MEPDG defaults. This decision requires the user to point the project file to a *.icm climate file corresponding to the desired location for the project. The selection of the climate file is an extremely important step, as seemingly insignificant errata or incomplete data in the *.icm file can create unrealistic results for an MEPDG project (Johanneck and Khazanovich 2010).

The following subsection illustrates the general process a user will go through to create and run an MEPDG project file, including specifying the pavement layers and properties and selecting terminal performance criteria.

5.2.1 Summary of MEPDG Procedure for AC-PCC project

The following describes a general overview of the MEPDG procedure for an AC overlay of an existing PCC pavement. The first step is the basic creation of an MEPDG project.

1. Design Type: Select “Overlay.”

2. Pavement Type: Select "HMA/JPCP" for AC-PCC project.
3. Design Life: Select desired life of pavement.

Once a project file has been initiated, the user must select all design inputs for a trial design. The unique inputs for AC-PCC are as follows.

4. Design reliability and performance for composite pavements:
    a. Design reliability should be based on traffic level of the highway. Higher traffic levels warrant higher reliability levels (95% to 99%).
    b. Structural fatigue cracking should range between 5% and 15% JPCP transverse fatigue cracking
    c. Smoothness (Terminal IRI) should be based on traffic level of the highway. Higher traffic levels warrant lower terminal smoothness levels (~150 in/mile).
    d. Permanent deformation (rutting of AC only which is total also) should be ~0.50-in mean wheel path
    e. Joint faulting for "bare" JPCP comparisons: 0.15 to 0.20 in.
    f. Initial IRI: The initial IRI for AC-PCC composite pavements can be very low due to the multiple layering of the pavement. Initial IRI values as low as 35 in/mile have been achieved, with routine values from 40 to 50 in/mile.
    g. Type and thickness of AC surface layer. The type depends on the design objectives. If reducing noise levels to a minimum are required, then some type of porous asphalt surface can be used. Thickness should be the minimum possible to provide durability and surface characteristics desired for a given truck traffic and climate. In warmer weather locations, a thinner surfacing is feasible, such as 1 in, but for colder weather and heavier traffic, up to 3 in total may be required.
5. Type (JPCP) and thickness of the PCC layer. This is the load carrying capacity layer for the composite pavement. The trial design should start with a typical thickness used for bare pavement. Depending on the thickness of the AC surface, the slab thickness may be reduced by 1 to 3 inches of concrete.
6. Joint design. Joint design includes joint spacing and joint load transfer.
    a. Joint load transfer requirement is similar to bare JPCP design in that dowels of sufficient size are required to prevent erosion and faulting for any significant level of truck traffic. The greater the dowel diameter the higher the joint LTE and the more truck loadings the pavement can carry to the terminal level of faulting.
    b. Simplified dowel design: the dowel diameter should be at least 1/8 the slab thickness
    c. Low volume roadways where dowels would not normally be used for bare JPCP do not require dowels for composite pavement. This is true for residential or farm to market streets where JPC or RCC is used as the lower layer. When dowels are not used, it is highly recommended to reduce the joint spacing to 10 ft to reduce reflection cracking severity and increase joint LTE.
7. Concrete slab recommendations:
    a. Typical concrete used in bare JPCP can be used for AC-PCC with no changes. There are no special requirements different than that for bare pavement.

b. Lower cost concrete based on local aggregates or recycled concrete. The strength, modulus of elasticity, CTE, and drying shrinkage of the concrete can be varied as inputs.
c. The SHRP2 R21 MnROAD test sections showed that recycled concrete from a local roadway or local aggregates can be used for the lower layer (Rao et al. 2011). Both of these alternatives provided sustainability advantages and cost savings.
d. Base layer and other sublayers should be selected similar to bare JPCP or CRCP designs based on minimizing erosion, construction ease, and cost effectiveness. No attempts should be made to reduce the friction between the slab and the base because as friction helps control erosion and pumping and reduces stress in the slab.

At this point, the user should have a full project file created. At this point, the user runs the MEPDG program for the created AC-PCC project file (or more generally, an AC-over-JPCP project file). The MEPDG software performs traffic and EICM (climatic) analysis and creates intermediate project files, which later will be used in the TPF-5(149) analysis.

5.2.2 MEPDG Example

Given that the MEPDG predicts performance for a given project overlay thickness, twelve project files for varying AC overlay thicknesses and two mix designs were created. The project files were otherwise identical in describing the existing 9-inch JPCP. Default (Level 3) parameters were used for all projects unless otherwise specified in Table 4.

Table 4. MEPDG inputs for chapter AC-PCC example

Parameter	**Input Value**
Pavement type	AC over JPCP
Initial two-way AADTT	1000
Number of lanes in design direction	2
Traffic growth	2.4%
Climate	Champaign, IL
Joint spacing	15’
Dowels	1 in diameter, 12 in spacing
AC Overlay Mix Design	PG 58-28 or PG 64-28
PCC	9 in
Base	8 in granular, A-1-a
Subgrade	Semi-infinite, A-6
Percent cracked slabs in existing JPCP	20%

For the various thickness overlays examined, distresses predicted by the MEPDG at the end of the 25-year design life are provided in Table 5.

Table 5. Predicted performance for various AC overlay thicknesses using the MEPDG for chapter example

AC Overlay Thickness (in)	Alligator Cracking (%)	Total Rutting (in)	
		PG 58-28	PG 58-28
4	8%	0.323	0.274
5	8%	0.329	0.279
6	8%	0.316	0.268
7	8%	0.296	0.250
8	8%	0.272	0.230
9	8%	0.247	0.209

Note that the projects were created with the existing JPCP having 20% cracked slabs, in keeping with the other methods exemplified in this chapter. If these cracks had been repaired prior to construction of the overlay, the MEPDG predicted 6.7% alligator cracking instead of 8%, though the results for rutting of the overlay remained unchanged. In order to compare the MEPDG results to those obtained using the other design procedures exemplified in this chapter, it is necessary to first decide how much damage will be acceptable.

The results of this example show that, unlike AASHTO93, MEPDG allows the designer to consider factors other than thickness. For example, rutting can be reduced with thicker overlay or different mix design. Finally, the designer is cautioned against relying on the MEPDG in its current form. The existing reflective cracking model in the MEPDG is not adequate, and extensive work, including NCHRP Project 1-41 and the work of the FHWA TPF-5(149) TICP project, has been developed to overcome this limitation (Lytton et al 2010).

5.3 SHRP2 R23

The SHRP2 R23 Design Tool performs basic flexible and rigid overlay design to be placed over existing rigid or composite pavements in various conditions. The required inputs are fairly basic, including the existing pavement type, distress condition, and traffic levels. The climate to which the overlay will be subjected is not considered. The output of the program is an overlay layer thickness, though the performance of the overlay is not predicted. Overlays must have a design life between 25 and 50 years.

The required input information consists mainly of information which is already known or is easily obtained or estimated and would be expected for the design of an overlay. The design tool can be used to design overlays of flexible, jointed plain concrete, continuously reinforced concrete, jointed reinforced concrete, and composite pavements. After the pavement type is selected, the number of layers in the pavement structure and their thickness is specified. The type of material for each layer is selected from several options: AC, PCC, granular base, cement stabilized base, and stabilized subgrade. The program can accommodate up to four layers in the original pavement structure, not including the subgrade. The only required material property input is the resilient modulus of the subgrade, which may be selected as 5000, 10000 or 20000 psi. The traffic information required is the number of ESALs per year and the AADT; a growth factor can be specified, or left at the default of 2.4%.

The existing pavement conditions inputs required vary based on the pavement type. All inputs are values which would typically be measured during a condition assessment or distress survey. For existing jointed plain concrete and jointed reinforced concrete, the required inputs are: the percentage of cracked panels, the average depth and deflection of faulted joints, the presence of low and moderate to severe D-cracking, the presence of ASR and the presence of pumping. For continuously reinforced concrete, the required inputs are: the number of punch-outs per mile, the presence of low and moderate to severe D-cracking, the presence of ASR and the presence of pumping. For existing composite pavements, the only required input is a classification of the surface condition as good, poor, or very poor.

A type of overlay (either rigid or flexible) is selected, and the program recommends various restoration techniques, based on the type and distress level of the existing pavement. The desired technique is selected and the modulus of the existing pavement or base is provided. From this information, a layer thickness for the overlay is generated. A summary report of the program inputs and outputs is also created.

5.3.1 Example of AC overlay of PCC design using SHRP2 R23 procedure

The example used in Section 5.1.2 and 5.2.2 is investigated here with the SHRP2 R23 procedure. All values assumed for the project were selected in such a way as to closely approximate values assumed for the AASHTO93 and MEPDG examples above. These values are indicated in Table 6, Table 7, and Table 8 below.

Table 6. Specifying existing pavement for SHRP2 R23

Parameter	**Input value**
Pavement type	JPCP
Number of through lanes in one direction	2
Number of Layers	2
PCC layer	9-inch PCC
Base layer	8-inch granular

Table 7. Inputs describing existing pavement condition using SHRP2 R23

Parameter	**Input value**
Percent panels cracked	20%
Average fault depth	0.15 inches
Average joint deflection	0.10 inches
Presence and extent of D-cracking (light, medium, heavy)	Light
Presence of ASR	No
Presence of pumping	No

Table 8. Inputs describing proposed, overlaid pavement using SHRP2 R23

Parameter	**Input value**
Design life	25 years

Subgrade resilient modulus	10,000 psi
ESALs per year	420,000
Percent traffic growth rate	2.4%
Current ADT all lanes, one direction	10,000
Number of through lanes, one direction	2
Presence of height restrictions	No

The input screen for the SHRP2 R23 procedure is illustrated in Figure 57.

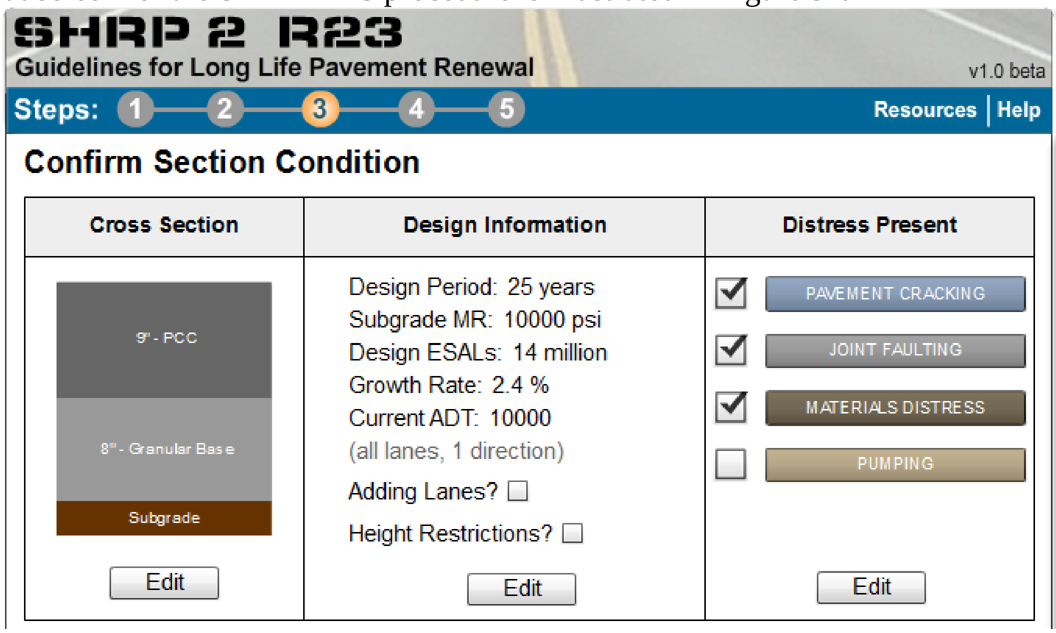

Figure 57. Following the five-step process to determine AC overlay thickness using the SHRP2 R23 procedure for the Chapter 5 example.

After the final step in the input process (Step 3), the SHRP2 R23 procedure then recommends options for the user to select in determining the extent of pre-overlay construction requires. This step is illustrated in Figure 58. The design tool allows the user to select either flexible or rigid renewal options, and for the flexible option described by this example, the R23 design tool recommended the use of crack-and-seat or rubblization with the AC overlay.

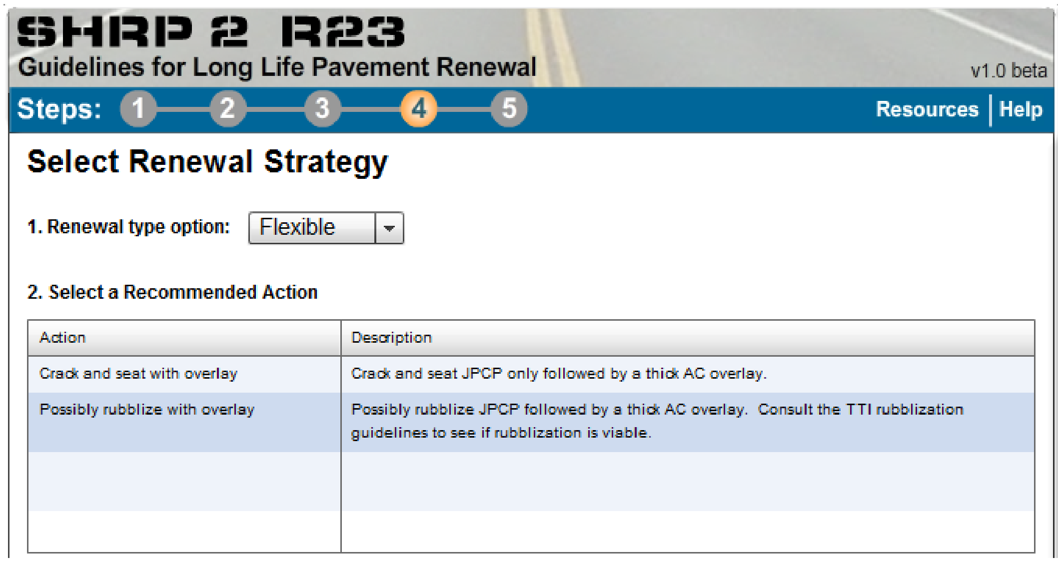

Figure 58. Selecting overlay type and pre-overlay repairs in the SHRP2 R23 design procedure

For either pre-overlay option, the R23 design resulted in an existing pavement or base modulus provided was 100,000 psi, and consequently a recommended AC overlay thickness of 6.5 inches. The project output screen is provided in Figure 59.

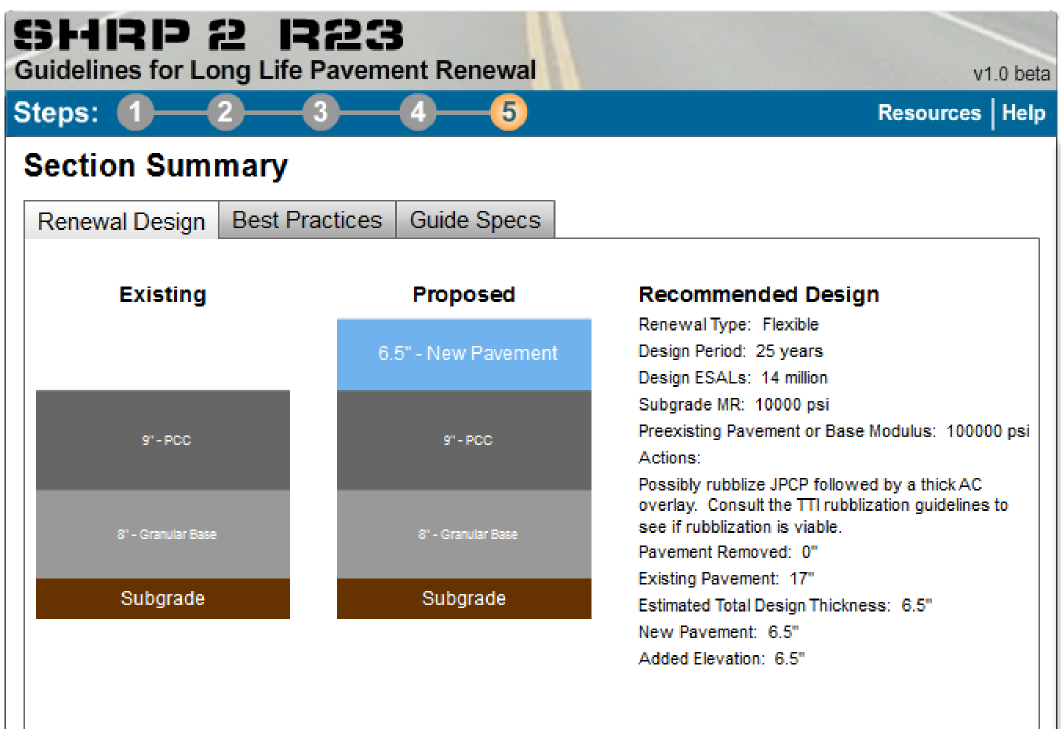

Figure 59. SHPR2 R23 example summary output

It is worth nothing that the SHRP2 R23 recommended thickness does not vary much from the design estimate produced using the AASHTO93 procedure. While SHRP2 R23 provides a quick an easy design estimate, it is recommended that the designer consider it an initial estimate given the limited inputs available to the R23 user. A final design should consider additional site conditions and utilize additional methods to ensure an appropriate structural design prior to construction.

5.4 Overall comparison and recommendations

At this stage, the designer is recommended to use MEPDG to predict rutting and develop a structural design to mitigate that distress. However, if reflective cracking is considered in the design, the MEPDG should not be used until its reflective cracking procedure is finalized, at which time MEPDG will be the tool of choice for AC-PCC design. Until then, it is recommended that the designer begin with an MEPDG design and use AASHTO93 and R23 to supplement the MEPDG design in terms of accounting for reflective cracking.

Chapter 6. Asphalt Concrete Overlay Construction

AC overlay construction on PCC requires careful attention to all phases of the process to achieve acceptable performance. Following proper AC mix selection, good construction practices such as proper mixing and placement temperatures, adequate compaction, and well defined quality control and quality assurance (QC/QA) procedures will help ensure the best possible overlay performance.

Due to regularly spaced PCC joints below and high stresses within the AC overlay, poor performance (reflective cracking and rutting) of the AC overlay is not uncommon. The performance of AC overlays on PCC is often a compromise between strategies utilized to minimize reflective cracking and AC rutting. These design strategies, which will affect construction, include selecting the mix to favor rutting or cracking performance, reducing the percentage of reflective cracks in the AC surface, retarding the rate at which the reflective cracks appear and propagate, and enhancing the appearance of the expected cracks at the surface. Agency policies such as utilizing thin AC surfacing on PCC pavements acknowledge the limited lifetime of the overlay and are made for non-structural purposes, such as noise or safety (Kaloush et al. 2009; Ongel et al. 2007).

The following elements of AC overlay construction on PCC pavements will be discussed: AC delivery and placement, mat compaction, joint compaction, and Quality Control and Quality Assurance (QC/QA). This section highlights pertinent factors to maximize AC performance when placed over PCC pavements. Many references are included for readers who seek additional information.

6.1 Construction Practices for AC-PCC Composite Pavements

This chapter discusses AC construction subsequent to pre-overlay repair. In-service PCC pavements will generally require pre-overlay treatment for best performance such as replacing and repairing slabs, stabilizing slabs, and preparing joints. AC overlays constructed on newly designed and built AC/PCC and in-service PCC pavements will generally follow comparable construction processes. However, depending on the design selection, the overlay may be conventional AC with the goal of maximum service life, or it may be constructed as a sacrificial layer for ride and noise purposes.

The AC overlays may be structural, consisting of dense graded or gap graded/stone matrix asphalt (SMA), or functional, consisting of open graded mixes for noise and/or splash-spray control, and may contain rubber or polymer modifiers. AC overlay performance can be enhanced through best practice measures throughout the construction process. These practices are aimed at maximizing cracking and rutting performance of the AC overlay, irrespective of any crack control measures, and the application of these best practices does not relinquish the contractor from using other best construction practices.

6.2 Pre-Overlay Treatments

Prior to placement of the AC overlay on existing PCC pavements, rehabilitation treatments should be performed to restore a smooth profile and minimize joint movement. Examples of these treatments include joint sealing, diamond grinding, dowel bar retrofitting, slab sub-sealing and jacking, full and partial depth slab repairing, crack-and-seating, and rubblizing, and were detailed in previous chapters.

6.3 Surface Preparation

The first step toward ensuring pavement performance is to provide the maximum bond between the PCC (new or existing) and the AC overlay. Poor initial bonding or the loss of bonding between the AC overlay and PCC pavement may result in several distresses, such as slippage cracking, as well as compaction difficulty (Leng 2009, West 2005). Strong interlayer bonding may be achieved through one or a combination of the following methods:

1. texturing the PCC surface to provide added bond potential;
2. cleaning the PCC surface; and
3. placing a tack coat with the proper type and quantity on the PCC surface (Leng 2009).

When combining these techniques, their interactions must be taken into account. For example, cleaning of the PCC surface is important prior to the placement of tack coat (Mohammad et al. 2012). A study by Tashman et al. (2006) found that applying tack coat to surfaces which had been milled did not increase bond strength significantly. However, tack coat was found to be very important for ensuring appropriate bond strength in sections which were not milled.

Surface texturing can be provided by transverse or longitudinal tining or diamond grinding. Tining does not allow for significant connection between PCC and AC and milling was found to provide the highest shear strength in laboratory tests (Leng 2009).

Subsequent to texturing and other pre-overlay repairs, clean the surface with mechanical brooms and air blowing and use water flushing where needed (LRRB 2012; USACE 2000). Other studies have shown that air blasting is preferred to mechanical brooms if only one method is chosen (Leng et al. 2009). A statistically significant difference has been found in the interface shear strength of overlays which were placed on properly versus improperly cleaned surfaces; therefore it is important to ensure that cleaning is adequate. It is also recommended that the surface be dry to avoid any negative effects of water on bonding. (Mohammad et al. 2012).

Tack coat contributes to adherence between the asphalt overlay and the concrete pavement so that they move as one unit. Using an excess of tack coat should be avoided as it will encourage interfacial shear slippage (Mohammad et al. 2012). However, it is important to use enough tack coat to ensure a good bond. The amount of tack coat required depends on the surface to which it is being applied. Open-textured surfaces, such as those which have been milled or experienced raveling, will require more take coat than closed-textured surfaces. The texture of the asphalt used in the overlay must also be considered. For example, Caltrans has observed that open-graded HMA requires more tack coat than dense or gap graded HMA (Caltrans 2003).

Various studies have been performed for tack coat optimization for overlays on PCC (Al Qadi et al. 2008) and AC (Mohammad et al. 2005), the latest study being the recently completed NCHRP 9-40 project (Mohammad et al. 2012). These studies found that SS-1h (or SS-1hP) spread at a rate of 0.2 gal/yd^2 (residual) provided the greatest shear strength between layers and best overall pavement performance. Uniform coverage of the tack coat without ribbons or “zebra stripes” is critical to achieving maximum bond strength (Figure 60). There is no consensus among DOTs on the optimal amount of cure time for tack coat, with some agencies finding cure time detrimental to bond and others stating that bond is increased by curing. However, a study by Tashman et al. (2006) found that cure time did not have a significant effect on the bond.

Figure 60. Improperly applied tack coat which leaves zebra stripes (from Mohammad et al. 2012).

Mohammad et al. (2012) developed two new tests for evaluating tack coat materials. The Louisiana Tack Coat Quality Tester (LTCQT) is used to measure bond strength in the field, while the Louisiana Interlayer Shear Strength Test (LISST) determines the interface shear strength of the tack coat in the lab (Figure 61).

Figure 61. The Louisiana Interlayer Shear Strength Test (LISST) is an attachment to a standard load frame used to test the interlayer shear strength (From Mohammad et al. 2012)

Using the LISST, different materials and application techniques can be tested in the lab prior to field placement in order to select the appropriate tack coat system for a project. Once the tack coat has been placed, the LTCQT can be used to ensure that sufficient bond strength was achieved in the field, see Figure 62. Field tests are particularly important because laboratory prepared specimens have been found to achieve higher interlayer shear strengths than field-obtained cores.

Figure 62. The Louisiana Tack Coat Quality Tester (LTCQT) is used to measure bond strength in the field (From Mohammad et al. 2012)

6.3.1 The use of geosythetic interlayers

Another construction technique with the intent of mitigating reflective cracking is the application of a geosynthetic fabric interlayer between the AC overlay and existing, prepared PCC. A fabric interlayer is used to "arrest" reflective cracking by dissipating the high stresses in the overlay that initiate reflective cracks in the overlay (Figure 63). The construction of composite pavement utilizing a fabric interlayer is illustrated in Figure 64.

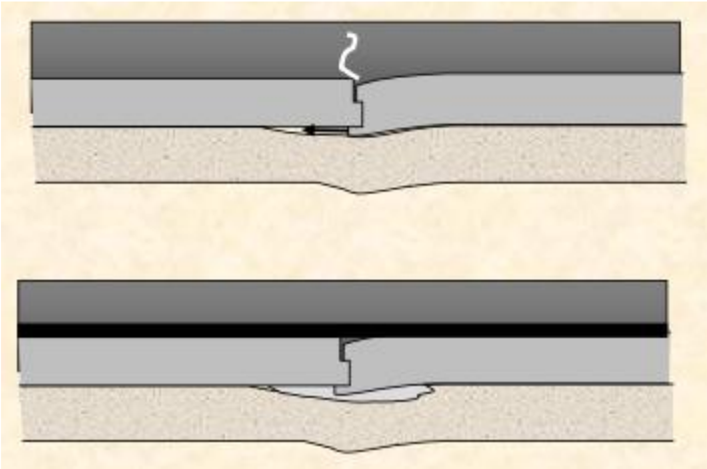

Figure 63. Composite pavement without (top) and with (bottom) fabric interlayer to arrest reflective cracking (From Al-Qadi et al. 2009)

Figure 64. Placement of a geosynthetic interlayer fabric (from Harrington et al. 2008)

While some practitioners recommend the use of interlayers, the success of these techniques is difficult to assess at this time given the limited application of interlayers and its relative cost. However, the available literature can guide those practitioners who wish to apply interlayers to the rehabilitation of existing rigid or composite pavements using AC overlays. A discussion on the effectiveness of interlayers is provided in Chapter 7.

Cleveland et al. (2002) review six different types of geosynthetic used to mitigate reflective cracking in AC overlays of both rigid and flexible pavements. The TTI overlay tester described in Zhou and Scullion (2003) was used to evaluate lab samples of overlaid pavements which used geosynthetics. This apparatus is illustrated in Figure 65.

Figure 65. Texas Transportation Institute overlay tester (from Cleveland et al. 2002)

Based on the results of these tests, fracture mechanics were used to develop a reinforcing factor to characterize geosynthetics. Field tests of overlays with interlayers were also constructed. A synthesis on the current state of knowledge surrounding geosynthetics (circa 2002) was also included in Cleveland et al.

Finally, Button and Lytton (2007) present a summary of guidelines for the use of geosynthetics to reduce reflective cracking in various types of asphalt overlay. In addition to definitions of the different types of available geosynthetics, the different uses, Button and Lytton provide guidance as to when and how a geosynthetic should be used.

Al-Qadi et al. (2009) developed a model to predict the how different interlayer systems perform for given environmental loads and traffic conditions. This model uses the performance benefit ratio (PBR) to compare the predicted performance of the interlayer to the predicted performance of a control section without an interlayer. Based on the PBR, practitioners can decide if the increase in performance justifies the increase in cost of a project. The output can also be used to compare the potential effectiveness of different interlayer systems in order to choose the system which is most appropriate for a particular project.

6.3.2 The use of stress arresting interlayers

Some agencies utilize a stress arresting material interlayer (SAMI) composed of a chip-seal alone or in combination with a leveling course for use on the PCC surface to act as a reflective crack control strategy prior to the AC overlay. Use of a reflective crack relief interlayer mix RCRI, consisting of one inch of fine aggregate bound with a highly elastic polymer modified binder was shown to be statistically the best performer in mitigating reflective cracking, in an extensive study of composite pavement performance in New Jersey (Bennert 2011).

6.4 AC Delivery and Placement

Guidelines for delivery, placement, and compaction of AC overlays on either existing or new PCC vary little from sound practice recommendations for paving AC on AC. The goal is providing a uniform spread quantity and quality of AC to maximize smoothness and minimize segregation during placement, prior to compaction.

Maintaining adequate mix workability is primary to proper placement and compaction, so the following factors should be considered when developing a delivery strategy: delivery haul length; truck type; mix type; ambient temperature; and wind. Lower than desired temperatures make placement more difficult, and can lead to mix segregation, which has been shown to affect mix fatigue performance adversely (Khedaywi 1996). This is very important over moving PCC joints, as reflective cracking models have been developed that are based on fatigue failure modes.

The following highlights several best construction practices for AC placement (LRRB 2012):

1. The truck bed should be raised before opening the tailgate if using end-dump trucks so the mix slides against the tailgate as shown in Figure 66. This minimizes segregation.

Figure 66. End dump of AC into paver hopper (Mahoney, Pavement Interactive.com)

2. Let the paver move forward toward the AC delivery truck to eliminate mat indentations. Avoid having the truck back up into the paver. The paver should stop during the exchange of material. Following emptying the AC, the truck should drive away smoothly without jerking the paver.

3. Bring the paver to paving speed as quickly as practicable to maintain constant head in front of the screed. Maintain sufficient mix in the hopper so that slat conveyors are never visible, but do not overflow the hopper. Only raise up paver wings when necessary to eliminate buildup of cold mix in hopper corners.

4. Maintain a constant head of material on the paver augers through constant speed and continuous operation. Material level should be at the same height or slightly above the auger shaft as shown in Figure 67.

Figure 67. AC head in auger (Mahoney, Pavement interactive.com)

5. A constant head at the augers results in minimal movement of the screed and helps maintain consistent mix height and mat smoothness. Variations in head affect screed movement and mat thickness as shown in Figure 68.

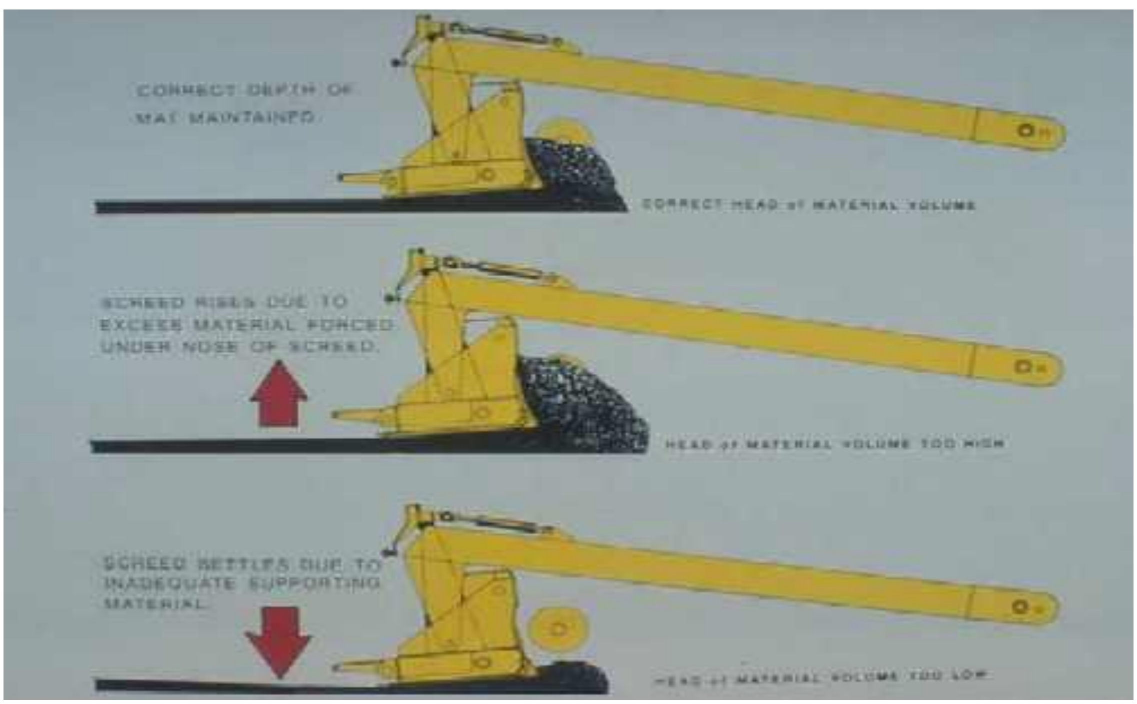

Figure 68. Screed movement with material head variation

6.5 Mat Compaction

Mat compaction (density) is considered the most important factor in determining AC performance. Compaction to six percent or less air voids increases fatigue life, reduces rutting, decreases oxidation and aging and moisture damage (Benson and Scherocman 2006). Lift thickness, mix properties, and environmental conditions are the key factors that affect the ability of the contractor to achieve density in the AC layer (Decker 2006).

With AC overlays on PCC, thin functional overlays (< 1.5 in.) may be placed as well as thicker structural overlays (> 2in.) which require greater attention during compaction, particularly in colder weather. Properties of the aggregate and asphalt binder (including use of modifiers) have an impact on the ability of the contractor to achieve density. It is well known that mixes made with coarse, angular aggregates may be more difficult to compact than mixes made with rounded materials. As a result, the coarser mixes may cool before density can be achieved (Decker 2006).

Affected by mixing temperature and delivery, the AC mat temperature must be sufficient to obtain the required AC density. Mat temperature is affected by numerous factors including haul time, mix type, ambient temperature, ground/PCC temperature, and wind.

When placed on PCC, AC overlays are subject to higher levels of compressive energy due from truck traffic. The highly rigid PCC (modulus greater than 4,000,000 lb/in2) leads to the AC layer absorbing more load energy due to higher confinement effects and lack of dissipation throughout the pavement system. With the increased stresses placed on AC in composite systems, makes meeting compaction specifications even more critical than in "conventional" AC over AC overlays or full depth paving.

6.6 Joint Compaction

Proper AC joint compaction is critical in composite pavements with AC surfacing due to the high degree of PCC movement beneath the AC. Poor compaction of longitudinal joints typically results in joints cracking, opening, and raveling of the adjacent material. Joint density is determined by three primary factors 1) density on the outside edge of the first paved lane (cold side – free edge) 2) degree of compaction of the joint which requires some overlap to ensure adequate material and 3) degree of compaction of the second paved lane (hot side) (Brown 2006). In composite pavements, even if saw and seal will be performed, having as high density as possible in the AC surrounding the sealed saw cut will facilitate longer life of the treatment. Figure 69 shows a typical poor performing longitudinal joint.

Figure 69 Raveled longitudinal joint (Brown, 2006)

Compaction of the free unsupported edge can be problematic. The type of roller used and its position in regard to the unsupported edge of the pavement significantly affects the amount of density that can be obtained. A pneumatic (rubber) tire roller normally cannot be used within about 150 mm (6 in.) of the unsupported edge of the lane without pushing the mix sideways due to the high pressure in the rubber tires (Benson and Scherocman 2006, Brown 2006). It is recommended that a steel wheel roller extend over the edge of the lane by approximately 6 in. as shown in Figure 70 (Benson and Scherocman 2006).

Figure 70 Proper rolling of free edge (Benson and Scherocman 2006)

The second factor for durable longitudinal joints relates to the amount of AC placed at the interface between the two adjacent mats. Two factors are involved in this – the height of the uncompacted hot mat and the amount of overlap onto the cold mat. The height of overlap onto the cold side depends upon the thickness of the uncompacted mat on the hot side. Given that mix compacts approximately 25 percent (1/4 in. per in. thickness) the overlap needs to be high by the amount of compaction expected to occur so adequate material is present (Benson and Scherocman 2006). The amount of overlap onto the cold side (transversely) is critical. There should be no gaps by the paver and even given variability of the paver, there should always be some overlap. Augers must adequately push AC against the free edge when placing the second lane. Any screed extensions must be assessed for sufficient material placement at the joint (Brown, 2006). The amount of transverse overlap needed is in the range of 25 mm (1 in.) to 40 mm (1½ in.) for proper longitudinal joint construction. If proper overlap is established, raking or luting the joint is not necessary (Benson and Scherocman 2006). Figure 71 shows proper overlap.

Figure 71 Proper overlap of hot mat onto cold mat (Benson and Scherocman 2006)

The final issues for constructing well performing longitudinal joints is placement of adequate material on the hot side and proper rolling of the joint. Sufficient material must be placed to allow for 20 to 25 percent compaction of the hot mat. Unfortunately, the tendency by contractors to place less material along the hot mat, to produce a smooth joint, results in too little material along the joint. Once the hot mat is compacted to the level of the cold mat, a steel wheel roller will bridge over this material and compaction will cease at the joint. A rubber tire roller can be used successfully to further densify the mat in low spots (Brown 2006).

It is recommended to place either a steel wheel roller or a pneumatic tire roller, a short distance (6 in.) over the top of the joint from the hot side of the joint. For a rubber tire roller, the center of the outside tire of the roller, at the end of the roller with an even number of tires, is placed directly over the top of the longitudinal joint. Placing the roller in this position permits proper compaction of the mix at the joint as well as compaction of the mix on the hot mat (Benson and Scherocman 2006). Figure 7 shows this roller placement.

Figure 72 Rolling of hot mat for joint compaction (Benson and Scherocman 2006)

6.7 Sawing and Sealing AC Overlays at PCC Joints

As part of the recent SHRP2 R21 project, R21 research engineers assisted the Illinois Tollway Association in developing a specification for the sawing and sealing of joints in the AC overlay of a newly constructed PCC pavement (Rao et al. 2011). This specification describes the saw cutting, cleaning, drying, and sealing of transverse joints in new AC overlay surfaces. More information on the saw and seal operations promoted by the SHRP2 R21 project can be found in Rao et al. (2011). A few illustrations of the saw-and-seal procedure are provided in Figure 73.

Figure 73. Sawing joints in the AC layer which correspond to the joints in the underlying PCC layer (left) and sealing the newly sawed joints (from Rao et al. 2011)

For the Illinois Tollway AC-PCC, the joints cut for the 3-inch AC overlay were to be ½ inch wide by ⅝ inch deep. Each sawed AC overlay joint was required to be within 0.5 inches of the transverse joints in the JPCP below – as illustrated in Figure 74 and later described below in discussing the effectiveness of sawing and sealing, sawed joint locations have been shown to be critical to well-formed joints in AC-PCC.

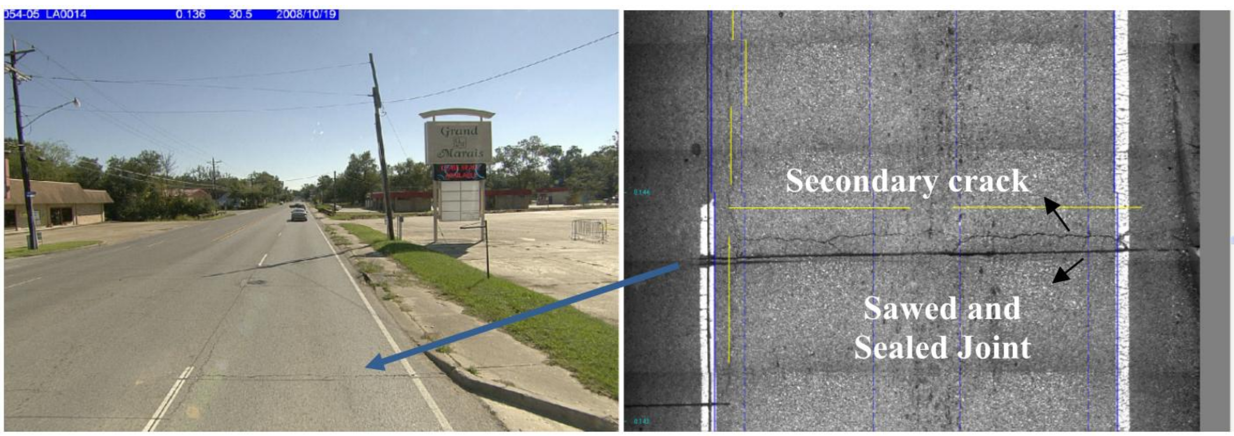

Figure 74. Illinois tollway saw-and-seal (Elsefi 2011)

The sealant used was required to meet ASTM D-3405 with the modifications that penetration at 77 deg F be 90-150 and that the sealant pass bond testing at -20 deg F. Furthermore, the specification required that the sealant density be between 9.0 and 9.35 lb/gal. The bond breaker tape used was required to be not more than 1/8 inch narrower than the joint saw cut. The SHRP2 R21 report can be consulted for specifications on the saw, compressor, and heat lance used (Rao et al. 2011).

The construction specification requires that saw joints were cut in the AC overlay no earlier than 48 hours after paving of the overlay. Saw cutting was performed as indicated above. Of particular note in the construction specifications was the requirement for the finished joint to be well-cleaned upon completion. This ensured an acceptable seal on the joint. Sealing was conducted during daylight hours and only in favorable weather. More information on the saw and seal operations promoted by the SHRP2 R21 project can be found in Rao et al. (2011).

6.8 Quality Control and Quality Assurance

Quality Control and Quality Assurance (QC/QA) for AC paving in composite pavement systems is comparable to that performed for conventional AC full depth and overlay paving. However, with the added environmental stresses and strains and the increased traffic energy imparted to the pavement by the truck traffic, sufficient mix density, minimal segregation, and adequate joint density become even more important.

Quality assurance specifications (also called QA/QC specifications or QC/QA specifications) are defined by the TRB glossary as a combination of end result specifications and materials and methods specifications (Burati 1995). The contractor is responsible for QC (process control), and the highway agency is responsible for acceptance of the product. [QA specifications typically are statistically based specifications that use methods such as random sampling and lot–by–lot testing, which let the contractor know if the operations are producing an acceptable product.] Detailed descriptions of the QC/QA process are found in (Hand and Epps 2006; Burati 1995).

Statistically based QC/QA is used by most states to evaluate mix quality during construction. Constructed characteristics such as smoothness, binder content, in-place density, and gradation are measured and the contractor is paid for the quality provided. To quantify the measurements of these mat and mix properties, the percent within specification limits (PWL) for each parameter is commonly used. The contractor is then paid based on performance through Pay Factors and incentives and disincentives which proportion the contracted pay depending on quality. To make the process more manageable, QC/QA specifications typically utilize a composite pay factor that includes many quality measurements, and often in-place density is given the most value (Hand and Epps 2006).

For composite pavements, joint saw and seal quality can be measured and rejected if quality does not meet specifications, such as joint not properly filled, sealant not bonding, and sealant contaminated (Rao et al. 2011). The main components of a QC/QA statistical system include

- acceptance sampling,
- comparison testing (f-testing (mean) and t-testing (variance)),
- quality-level analysis (PWL determination), and
- pay factor determination.

QC sampling and testing is normally the responsibility of the contractor (or the representative) and is performed randomly at a large number of intervals based on sublot size. Sublot size is determined from lot size (for example lot size is 5000 tons and sublot is 1000 tons). QA sampling and testing is performed by the agency, or the contractor samples and the agency (or representative) performs the assurance tests. A view of the process is shown in Figure 75. Variability is present in all stages of this process and adds risk to the agency and contractor. The goal is to minimize this variability and balance the risk to the contractor and agency.

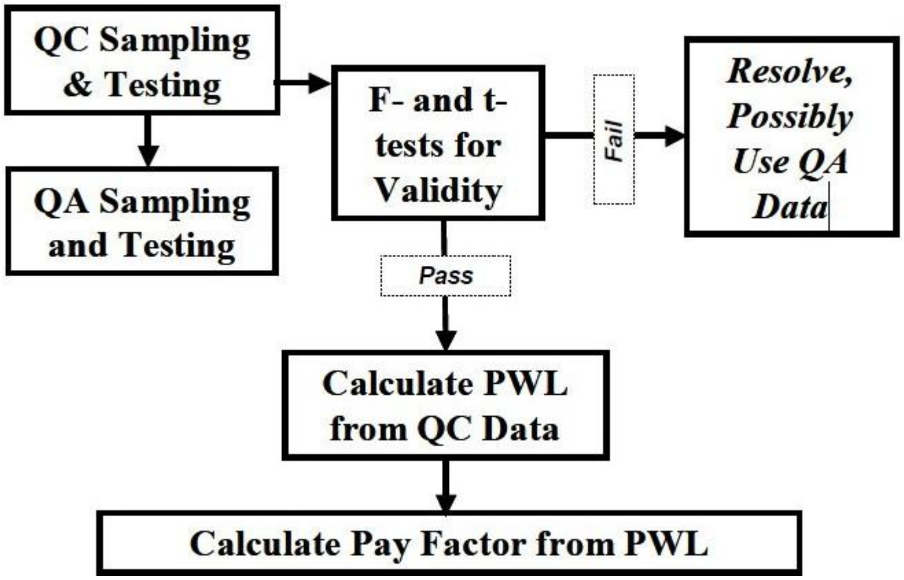

Figure 75 View of statistical QC/QA process (Hand and Epps 2006).

A key issue in QC and QA is the sampling process. This process has to be conducted properly so that the sample is representative of the placed mat. There are two phases of the sampling operation – selecting the sampling location and acquiring the material. FHWA supplement 23CFR637 recommends obtaining a sample as close to the work as possible. This means potentially sampling in the completed mat to ensure the mixture is most representative of the paved roadway. However, this disturbs the paved surface and contractors want to avoid potential penalties for smoothness (Elseifi 2009). For example, Michigan DOT has a procedure that presents little damage to the paved surface while obtaining representative samples. Plates with affixed wires are put randomly beneath the to-be-placed lift (Figure 76).

Figure 76 Plates randomly placed under AC mat

The material is shoveled out with a specially designed shovel from above the plates until sufficient quantities are obtained and back-fill AC is replaced (Figure 77). Overall, many states successfully sample from the back of the paver and this is recommended (Elseifi 2009).

Figure 77 Shoveling material from above plates

For PWL determination, some specifications use just QC data, some use QA only, and others use a pooled combination of QC and QA data. Using QC data alone is recommended. Specification limits are normally selected using engineering judgment and a current limitation is the tentative understanding of the relationship between measured quality of the pavement and long term performance of the pavement. This makes it impossible to rationally determine life cycle costs and equitably develop pay factors (Hand and Epps 2006).

Hand and Epps recommend the following to minimize variability throughout the sampling and testing process:

- Require all laboratories conducting QC and QA testing to be AASHTO accredited;
- Require all technicians working of QC/QA projects to be certified, preferably by a national (NICET), regional (WAQTC), or state agency;
- Select sampling locations and sampling/splitting methods that result in the lowest amount of variability;
- Select test methods that result in the lowest amount of variability;
- Eliminate options within test methods to reduce between laboratory variability; and
- Use only QC rather than pooled QC and QA data.

As noted by Hand and Epps, every selection in the QC/QA process has an impact on final acceptance and payment, and as a result it is critical that QC/QA specifications are written with with these items in mind, while anticipating variation in each.

Chapter 7. Pavement Performance Evaluation

The design and construction of AC-PCC, though important issues taken individually, must be considered in terms of the desired performance and service life for a given rehabilitation. Many of the factors that contribute to pavement performance are those considered in the design process, including design thickness, AC mix design, traffic, climate, and the integrity of the existing PCC pavement.

Recommendations for practice based on evaluations of pavement performance are difficult to develop given that agency evaluations can often be very specific to local conditions, including material and construction peculiarities. In this regard, the Long Term Pavement Performance (LTPP) database is extremely valuable, as it provides years of data for an evidence-based evaluation of the effect of various design features and site conditions on pavement performance. As detailed earlier in this document, the LTPP pavement test sections relevant to AC-over-PCC are:

GPS-7. Hot Mix Asphalt (HMA) Overlay of PCC Pavements
SPS-6. Rehabilitation Techniques Using HMA Overlays of PCC Surfaced Pavements

Each of these experiments includes one of four environments (1. Wet, Freeze; 2. Wet, No Freeze; 3. Dry, Freeze; and 4. Dry, No Freeze); traffic levels expressed in ESALs; layer structural thicknesses; and many other variables (Hall et al. 2005).

In general, as expected the LTPP overlays were found to have effectively rehabilitated improved the condition and the roughness of all of the GPS-7 and SPS-6 sections. The performance of these sections was more closely scrutinized by several studies, including but not limited to the NCHRP 20-50 project (Hall et al. 2005) and a recently completed review of AC-PCC performance by the FHWA (Carvalho et al. 2011). While a large number of parameters can account for long-term performance, these studies discussed performance primarily in terms of the parameters shared by the LTPP sections and commonly considered in AC-PCC design and construction, which are:

- Pre-overlay preparation;
- Presence of saw-and-sealing;
- Use of crack-and-seat or rubblization of existing PCC; and
- Overlay thickness.

The following sections follow this precedent in their overview of AC-PCC performance, and each of the sections considers one of the four parameters outlines above and its influence on pavement performance.

7.1 Importance of Preparation Prior to Overlay

The general assessment of pre-overlay preparation on long-term performance, based on the recommendations of the NCHRP 20-50 project (Hall et al. 2005) and a recently conducted FHWA study (Carvalho et al. 2011), is that improvements to pre-overlay preparation (minimal vs. intensive) will generally result in improvements to long-term performance in smoothness for AC overlays of JPCP. The NCHRP 20-50 study divided the preparations prior to placement of the overlay into two categories based on the amount of repair made: minimal pre-treatment preparation and intensive pre-treatment preparation. Minimal preparation included sealing of joints and cracks and partial- and full-depth repairs. Pavements which received load transfer retrofits, undersealing, and drainage treatments (in addition to minimal preparation techniques) were considered to have intensive preparation. No statistically significant difference in the amount of cracking was found between slabs that received minimal preparation and those that received intensive preparation (Hall et al. 2005). Roughness increased more slowly on overlaid sections that received intensive preparation versus those which received minimal preparation (Karamihas and Senn 2010). Finally, the level of preparation before the overlay was not found to affect the amount of rutting experienced by the overlay (Hall et al. 2005).

7.2 Effectiveness of Saw and Seal

Saw and seal is a technique used to mitigate reflective cracking into the asphalt overlay. Joints are saw-cut in the asphalt directly above the joints in the concrete pavement shortly after placement of the overlay. No statistically significant difference was found in IRI between sections with and without saw and seal of the overlay; however sawed and sealed overlays had a slower increase in IRI after treatment than non-sawed and sealed overlays. No statistically significant difference was found in the amount of cracking in sawed and sealed overlays compared with overlays placed on crack-and-seat slabs at low cracking levels. However, at high cracking levels, overlays which were sawed and sealed experienced more reflective cracking than overlays placed on crack-and-seat slabs. Saw and sealing was not found to affect the amount of rutting experienced by the overlay (Hall et al. 2005).

Two recent projects have examined the effectiveness of sawing and sealing in AC-PCC. The first was a study conducted by the Louisiana DOT to evaluate the saw and seal method in terms of pavement performance and cost (Elseifi 2011). Elseifi et al. involved the survey of 15 saw-and-sealed pavements throughout Louisiana over the course of six to 14 years. These saw-and-sealed pavements were then compared with neighboring pavement sections that did not use sawing and sealing for the AC overlay. The study concluded that the use of sawing and sealing extended the pavement service life by an average of four years. Saw and seal was judged a cost-effective alternative for AC overlays. Elseifi et al. also included finite element analysis of an AC-PCC section to determine the mechanics of how saw and seal treatments minimize reflective cracking – this analysis determined that the constructed joints in the AC allow the AC slab to move with the underlying PCC slab as it expands and contracts.

The second study was conducted as a portion of the SHRP2 R21 Composite Pavement project; the R21 team conducted a survey of composite pavement project in the European Union (Tompkins et al. 2010). The SHRP2 effort included meeting with pavement experts from various countries and transportation agencies to discuss their paving techniques and experience and to survey their existing composite pavements. This survey included a number of AC-PCC pavements that utilized the saw and seal method, which has been regularly applied to pavements in Germany since the early 1990s.

The SHRP2 team surveyed many sections featuring stone-matrix asphalt (SMA) overlays of PCC pavements along the A93 motorway, south of Munich, in 2008. For one section, illustrated in Figure 78, a 3-cm layer of SMA was overlaid a 26 cm two-layer PCC pavement in 1995-1996. The SMA was a gap-graded mix with a maximum aggregate size of 8 mm. The saw and seal work looked outstanding and of very high quality. The longitudinal joints between PCC slabs had propagated up through the SMA, both between the inner/outer slabs and between inner/shoulder slabs. The only significant maintenance that had been done was to place a few patches at transverse joints where the SMA had debonded from the PCC surface and cracked. These rectangular repairs can be seen in Figure 78.

Figure 78. SMA over PCC along A93 in Germany, rectangular patch repair visible next to red traffic diversion truck (*from Tompkins et al. 2010*)

The R21 research team learned that the use of SMA over JPCP in Germany and other countries has used sawing and sealing for approximately 15 years. In the European experience, reflection cracking was found to be a problem when JPCP is used, and the JPCP joints reflected through unless saw and sealing above transverse and longitudinal joints was used. Furthermore, experts in Germany claimed that the use of sawing and sealing of SMA over CRCP was effective in minimizing reflection cracking. As a result of the tour of European AC-PCC composites, the SHRP2 R21 project recommended saw and seal to handling reflection cracking for an AC overlay, be it SMA, Superpave, rubberized surfacing, etc. (Rao et al. 2010).

7.3 Effectiveness of Crack-and-seating or Rubblization

Though crack-and-seating and rubblization are very different in terms of construction processes and the underlying pavement structure, these techniques are often discussed and contrasted in research that uses AC-PCC data to discuss long-term performance. Hence, this section couples the techniques not to compare or promote one technique over the other, but to review the effectiveness of both of these techniques given the nature of their presentation.

Crack-and-seating is another reflective cracking mitigation method used for asphalt overlays. Rather than placing the overlay over an intact slab, the existing slab is broken at regularly spaced intervals prior to overlay placement (crack-and-seat) or into smaller, irregularly sized pieces (rubblization). The AASHTO layer coefficient for intact and rubblized PCC slabs was found to determine which layer coefficient should be used in the AASHTO design procedure (Galal et al. 1999). Mechanistic-empirical design procedures have been developed by Thompson (1999) to determine the asphalt thickness for overlays placed on rubblized PCC. A summary of best practices and techniques for rubblization and crack-and-seating can be found in Thompson (1989). Best practices are important because the equipment and methods used to break up the concrete have been found to effect the performance of asphalt overlays placed on crack-and-seat pavements (Arudi et al. 1996).

Mixed results have been found in studies of the effectiveness of crack-and-seating. In one study, no statistically significant difference in IRI between sections where the overlay was placed on an intact slab versus one which had been placed over a crack-and-seat slab. However, observationally, crack-and-seat performed better than intact (Hall et al. 2005). Studies have found that crack-and-seat slabs experienced less roughness than those which were sawed and sealed (Karamihas and Senn 2010), as well as less cracking (Hall et al. 2005). A survey of 22 different projects which used crack-and-seat as a reflective cracking mitigation technique found that only two projects experienced reduced reflective cracking after four or five years (Thompson 1989). However, another study of 451 lane miles of crack-and-seat overlaid pavements found that only one section did not perform well with respect to reflective cracking; this section was also found to have been improperly constructed (PCS 1991).

A survey of 38 different state highway departments which use rubblization found that performance of the asphalt overlay was generally satisfactory; however, this poor performance was generally due to a weak subgrade and not to the rubblization technique. When asked to compare their rubblization projects to those which used crack-and-seat, rubblization was found to be more effective at reducing reflective cracking in asphalt overlays than crack-and-seating (Ksaibati et al. 1999). This conclusion was also reached in an independent investigation by PCS/LAW (PCS 1991).

The performance of asphalt overlays of crack-and-seat and rubblized pavements is dependent on the construction techniques used and the extent to which the structural integrity of the pavement structure is preserved (Arudi et al. 1996). One study which investigated the long term structural capacity of rubblized concrete pavements with asphalt overlays through FWD testing found rubblization was found to be particularly effective at preventing reflective cracking for concrete pavements which were very distressed and/or had high levels of patching. This study also found that the structural integrity of these pavement systems did not degrade over time (Thompson 1999). In contrast, a two and half year field study comparing identical sections of roadway with and without crack-and-seating prior to overlay placement found that breaking and seating delayed reflective cracking, but also reduced the structural capacity and flexural strength of the pavement system (Rajagopal et al. 1996). Another study found that the decrease in stiffness of the PCC layer caused by crack-and-seating resulted in an increase in fatigue cracking (Cho et al. 1998).

Overall, while the success of fractured slab techniques varies from agency to agency, in almost all cases it has been noted that sound construction practices are very important in applying these techniques. Where good construction practices have been applied, fractured slab techniques appear to provide satisfactory results in mitigating reflective cracking. Another factor in the selection of rubblization or crack-and-seat for an individual DOT is cost, local experience, and availability of contractors.

7.4 Importance of Overlay Thicknesses

Based primarily on the work conducted in NCHRP Project 20-50 (Hall et al. 2005) and the recently conducted FHWA report on long-term AC-PCC performance (Carvalho et al. 2011), performance benefits of thicker overlays are increased service life and performance, primarily in ride quality (smoothness) but also possibly in rutting resistance.

For the LTPP sections, the thickness of the overlay was determined during the design process based on the climate, traffic, etc. Recent studies of asphalt overlays of concrete pavements in the LTPP database evaluated the effects of overlay thickness on long-term performance. Only two overlay thicknesses existed in the LTPP sections: four and eight inches. While no statistically significant difference was observed in the amount of cracking between slabs with four and eight inch overlays, the eight inch overlays placed on the crack-and-seat slabs performed significantly better than the four inch overlays in terms of long term roughness (Hall et al. 2005; Carvalho et al. 2011). Similar performance was observed in the analysis of Karamihas and Senn (2010). Furthermore, eight-inch overlays experienced less rutting than four inch overlays in the LTPP study (Hall et al. 2005). It is worth noting that none of the other factors considered in the investigation were found to influence rutting.

For the four inch overlays, all sections performed similarly in terms of rutting and IRI, regardless of whether there was minimal or intensive preparation, if the joints were sawed and sealed or not, or if the slab was intact or subjected to crack-and-seat pre-treatment (Hall et al. 2005). However, one study found that thicker asphalt layers can increase the fatigue life of both the asphalt overlay and the underlying concrete pavement while reducing the potential for rutting in the asphalt layer (Cho et al. 1998).

7.5 Effectiveness of Interlayer

Cleveland et al. (2002) found that although all types of geosynthetic were shown to reduce cracking in the lab, the benefits realized in the field were only marginal when the increase costs associated with using an interlayer were considered. A continuation of this study found that the reflective crack arresting properties of the interlayer were found to decrease over time (Chowdhury et al. 2009). The same trend of decreasing interlayer effectiveness has been noted by Al-Qadi et al. (2009), but was correlated with increased traffic loading rather than time. Hence, in high traffic areas, a fabric interlayer may not be cost effective at reducing reflective cracking.

The general sentiment on the cost of interlayers relative to their reductions in in-field reflective cracking was echoed in the synthesis by Amini (2005) on the effectiveness of using paving fabrics to prevent reflective cracking in AC overlays worldwide and specifically in the state of Mississippi. Amini found that while the fabric generally reduces reflective cracking, in some cases – particularly thin AC overlays – it was not effective.

One way to measure the effectiveness of geosynthetic interlayers is through the use of a "relative life ratio," which is derived from the number of days required to develop large reflective cracks compared to a control section. The life ratio of a control section would be one; life ratios larger than one indicate how much better the performance was than the control section while materials with life ratios less than one shows poor performance. In a field study of several types of geosynthetic interlayers, the life ratio was of a pavement with an interlayer was found to be between 1.06 and 2.625 when compared with a standard 3 inch thick AC overlay. Adding an additional inch of AC gave a life ratio between 0.805 and 1.059 (Chowdhury et al. 2009). This indicates that using geosynthetics in place of thicker AC overlays can provide better resistance to reflective cracking, but, as previously discussed, may not be economical.

Other indices used to measure the effectiveness of interlayers are based on the number of reflective cracks, such as the reflective crack appearance ratio and the transverse crack appearance ratio used by Al-Qadi et al. (2009). Both of these metrics measure cracks in the AC overlay, but the reflective crack appearance ratio quantifies only those cracks which can be attributed to reflective cracking while the transverse crack appearance ratio measures the total number of transverse cracks, regardless of their cause. Both ratios are calculated as the number of cracks per unit length. A weight factor can also be associated with each of the ratios to account for the fact that cracks begin as low severity and worsen over time. By comparing crack appearance ratios and weight factors obtained for various types of interlayer, the performance of the interlayers can be evaluated in terms of a performance benefit ratio which compares the cracking appearance ratios and weight factors of the sections with interlayers to a control section. From these indices, Al-Qadi et al. found that stress absorbing composite interlayer had the best performance while the non-woven fabric interlayer and the self-adhesive strip type interlayer had performance benefit ratios less than one, meaning they performed worse than the control section.

Another type of interlayer is called a reflective crack relief interlayer (RCRI) which is a layer of stress absorbing AC. In a survey of highway agencies, these types of mixtures were found to have better success preventing reflective cracking than other types of interlayer, such as geosyntheics (Bennert and Maher, 2007). As with any pavement system, improperly designed RCRIs do not function as intended. In one case study, the RCRI mixture cracked because joint movements in the PCC section were not considered in material selection. Had the proper RCRI mixture been selected to withstand pavement deflections, laboratory tests on the RCRI mixture showed it was likely the pavement would not have cracked (Bennert et al. 2009). Al-Qadi et al. (2009) found that a high polymer content AC interlayer was found to perform well when compared with a control section.

7.6 Effectiveness of Reflective Cracking Mitigation Techniques

The previous chapters have detailed a number of strategies used to mitigate reflective cracking, which include the use of interlayer techniques (e.g. geotextiles or paving fabrics); modifications to structural design including excessive overlay thickness; and modifications to AC overlay mix design. In a survey of 26 state DOTs, Bennert (2009) asked state engineers to assess the success of these techniques. This assessment is presented in Figure 79, where abbreviations are paving fabrics and geotextiles (PFG); geogrids (GEO); stress absorbing membrane interlayers (SAMI); reflective crack relief interlayer mixes (RCRI); crack arresting layers (CAL); excessively thick overlays (EOT).

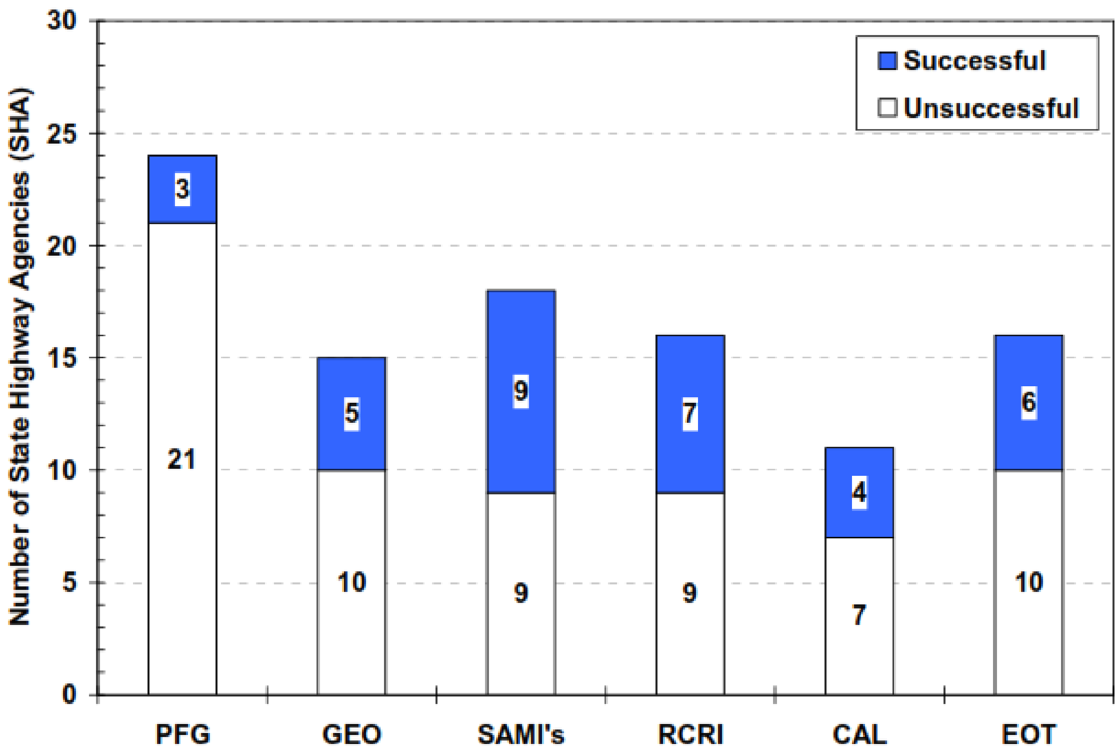

Figure 79. Reflective crack mitigation technique success history, based on a survey of 26 state highway departments (from Bennert 2009)

7.7 Summary Evaluation of AC Overlays of PCC

All of the different types of repairs were successful at improving the road, though some were more successful than others. Over time, the difference in ride quality between the control and the overlaid sections increased – that is, the benefits gained from overlays were observed to become more pronounced as time passed. The most benefit was obtained by applying the treatment sooner, rather than waiting for the pavement to deteriorate further (Hall et al. 2005).

Chapter 8. Case Studies in the Use of AC-over-PCC

Asphalt overlays over PCC pavements have been used extensively in many states. An examination of case studies from selected states highlights real world experiences from a variety of projects built in the field.

8.1 State DOTs Experience with Conventional AC-over-PCC Composite Paving

8.1.1 Texas

In 1998, researchers at the University of Texas conducted a field study of 14 different AC over PCC overlay rehabilitation strategies used on a heavily traveled road in Texas (Cho et al. 1998). Data collected on the Texas test sections included traffic loading (using weight in motion technology), climatic conditions, and structural condition, distress data, and IRI of the pavement before and after overlay placement. The distresses monitored were fatigue cracking, reflective cracking and rutting.

From the Texas field studies, Cho et al. (1998) found that using thicker AC layers does not decrease reflective cracking; the use of an interlayer was found to decrease reflective cracking. However, Cho et al. also observed that sections using an interlayer also experienced to more rutting, thereby complicating an assessment of the effectiveness of interlayers in AC-PCC that continued through Cleveland et al. (2002), Amini (2005), and Button and Lytton (2007) discussed earlier. Cho et al. (1998) observed that the crack-and-seat pre-overlay treatment resulted in more fatigue and reflective cracking. Furthermore, sections constructed with crack-and-seat were associated with increased rutting. The authors attributed this increase to the degradation of the overall stiffness of the pavement system in crack-and-seat cases versus more conventional methods.

Finally, Cho et al. (1998) developed mechanistic finite element models of each test section to find design equations which could be validated using the empirical results of the field test sections. A sensitivity analysis was also conducted to determine which parameters are most important in design. From this analysis it the following conclusions were drawn:

- Rutting of the subgrade is not a likely failure mode for asphalt overlays of concrete pavements.
- Stiffer interlayers may reduce rutting and fatigue in the asphalt overlay.
- The thickness of the interlayer only affects deflections close to an applied load.
- Tensile stresses in the concrete layer are not reduced by thin, flexible interlayers.
- Flexible base layers do not improve performance and may actually have a negative effect on fatigue resistance.
- Thicker asphalt layers can increase the fatigue life of both the asphalt overlay and the underlying concrete pavement while reducing the potential for rutting in the asphalt layer.

- Dynamic loads should be used in analysis rather than static loads, as they are more representative of actual traffic loading conditions, and dynamic loads result in higher peak stresses.
- The stress distribution in both the asphalt and concrete layers is greatly affected by temperature differentials.

Another interesting study performed in Texas provided a unique opportunity to analyze the first and second generations of asphalt overlays of the same concrete pavement. A case study by McCullough et al. (1996) examined a continuously reinforced concrete pavement (CRCP) which already had a 2 inch asphalt overlay slated for removal and replacement with a new asphalt overlay. The original overlay was placed on the CRCP to counteract the roughness stemming from a swelling clay subgrade.

Assessments of the pavement condition were performed before and after removal of the asphalt original overlay and after placement of the new asphalt overlay. A visual distress survey was conducted, followed by surface profiling and FWD deflection testing. Roughness was calculated using both the serviceability index (SI) and the international roughness index (IRI).

To determine the effectiveness of the original asphalt overlay, the damage accumulated prior to overlay placement was extrapolated out for the life of the overlay. By comparing these estimates to the results of the distress survey, it was determined that the original asphalt overlay had slowed the propagation of distresses and development of new distresses in the underlying concrete pavement. It was found that the overlay helped to slow fatigue cracking in the underlying pavement, but could not prevent it. The roughness of the CRCP pavement was also improved by placement of the original overlay.

A new overlay of the same thickness (2 inches) was placed after removal of the old asphalt overlay. In comparing the original overlay to the new overlay, it was found that the deflections of the two overlays were essentially the same and that the increased stiffness due to age of the old overlay was not a factor in the magnitude of deflections. The two overlays were also found to make a similar contribution to the structural integrity of the entire pavement system.

8.1.2 Illinois

After several changes in policy, practice and procedures governing the construction of asphalt overlays in Illinois, Wolters et al. (2008) conducted a study to see how overlays were performing after these changes. 231 sets of pavement data were examined to compare the service life of the overlay to factors such as the properties and conditions of the original pavement. One of the main goals of this research was to determine how each factor considered affected the service life of the overlay.

It was found that the condition of the original pavement prior to placement of the overlay affected the life of the overlay, but not in the manner expected. It was hypothesized that the life of the overlay would be longer for original pavements in better pre-overlay condition. However, comparing service life to pre-overlay condition showed that the condition of the original pavement was not a good predictor of overlay life. This was attributed to the fact that original pavement condition was quantified on a scale which did not differentiate between types of pavement distress and did not account for the fact that overlays are more appropriate for certain types of pavement distresses. Also, the comparison did not compensate for the effects of overlay thickness on overlay life. Pavements with more distresses generally receive thicker overlays, which could have a longer life.

A similar trend was noted when the effects of D-cracking were investigated. It was found that the presence of D-cracking in the original pavement did not translate into a decrease in the service life of the overlay. Again, however, the fact that thicker overlays were likely to have been placed on pavements which exhibited D-cracking was not accounted for. It was noted that pavements without D-cracking prior to placement of the overlay were found to have up to 30% more load carrying capacity than pavements without D-cracking.

8.1.3 Nevada

Nevada has many miles of concrete pavement, but little experience with rehabilitating those pavements. A study by Bemanian and Sebaaly (1999) investigated the cost effectiveness of different rehabilitation strategies for PCC pavements in Nevada so that the state could begin a rehabilitation of their roads. AC overlays are commonly used to rehabilitate strategy, but many overlays fail via reflective cracking. By examining specifications from other states as well as the field performance of construction projects in Nevada, viable reflective cracking mitigation strategies were identified. Both an economic analysis and a life cycle cost analysis were conducted on the different strategies and recommendations were made as to which techniques were most cost effective.

Both crack-and-seat and rubblization were determined to be viable reflective crack mitigation techniques which merited further investigation. A section of roadway was selected for a field test of both of these methods. The effectiveness of each viable reflective crack mitigation technique was determined by structural and functional evaluations. The functional evaluation measured ride quality in terms of IRI and rut depth while the structural evaluation determined the structural capacity based on FWD testing.

A life cycle cost analysis was conducted to determine which strategy was the most cost effective over the lifetime of the pavement. From this analysis, it was determined that the life cycle cost of crack-and-seat and rubblization was approximately the same, though crack-and-seat has a lower initial cost. However, this assessment does not consider performance as a criterion in deciding the total project cost. Because this project did not evaluate the long term field performance, no determination can be made as to which technique will perform better.

In the field test, both the crack-and-seat section and the rubblized section had both structural and functional ratings of good to excellent after four years. Neither strategy appeared to be performing better than the other. It should be noted however, that the structural layer coefficient of the rubblized section was found to increase over time. This means that the load carrying capacity of the rubblized section increased with age.

8.1.4 Michigan

Michigan has a fairly extensive history with using rubblization as a reflective cracking mitigation technique for AC overlays of concrete pavements. After 10 years of using resonant frequency breakers to rubblize the existing concrete pavement, the state began using multi-headed hammer breakers (MHB) on some projects. A study by Wolters et al. (2007) evaluated the performance of AC over PCC sections in Michigan to determine the cause of some premature failures of rubblized sections, specifically of those built using the MHB.

Twenty-one different AC overlays of rubblized PCC pavements were investigated. The distress index (DI), international roughness index (IRI) and ride quality index (RQI) of each pavement was determined by the Michigan Department of Transportation (MDOT). DI is determined through a video survey by quantifying distresses per unit length. A low DI is preferable: zero indicates no distresses in the pavement while a DI of 50 corresponds to a pavement with essentially no remaining service life if it is not rehabilitated. RQI is a metric similar to IRI, and is used to quantify the opinion of a highway user as to the ride quality of the pavement. Again, a higher value is better. Construction documentation and distress surveys were used to determine if improper construction played a role in then DI or RQI score of the pavement sections. In particular, the quality of the rubblization itself was noted.

Of the 21 sections investigated, 20 were ranked as good based on their DI, and all were ranked as good or excellent based on their RQI and IRI. However, even sections ranked as good can exhibit some distresses. These distresses were attributed to lack of drainage or problems related to the overlay, either its construction or the asphalt mixture itself. Rubblization was not found to be a cause of pavement distresses.

8.1.5 Arizona

Given the extent of Arizona DOT's experience with composite pavements, the following full section is dedicated to describing this example of DOT experience with AC-PCC.

8.2 ARFC-over-JPCP and CRCP in Arizona (I-10 and US-93)

While the Arizona Department of Transportation has a history of using asphalt rubber friction course (ARFC) overlays extending back to 1990, in 2003 Arizona DOT became much more aggressive in its use of ARFC overlays to rehabilitate its concrete paving through the initiation of its Quiet Pavement Pilot Program (QPPP). QPPP involves the overlay of concrete pavements with a thin AFRC lift. These pavements are constructed in urban areas specifically with this surface course to reduce tire/pavement noise levels while retaining the structural benefits of a thick concrete pavement (Scofield and Donovan 2003).

An asphalt rubber friction course uses a binder that consists of 80% AC and 20% waste tire rubber, which is ground into crumbs to be incorporated into the binder at high temperatures. Typically the ground tire is added to the hot asphalt and heated to a temperature of about 190°C, then mixed for at least one hour. After reaction the asphalt rubber is kept at a temperature of about 175°C until it is introduced into the mix at the mixing plant (Kaloush et al. 2009).

Asphalt rubber friction course layers are typically placed in Arizona using thicknesses between 1.5 and 2.0 inches. These layers are overlaid on existing or newly constructed jointed plain and continuously reinforced concrete pavements. For more detail on specific applications in Arizona, the reader is encouraged to consult Scofield and Donovan and Khaloush et al. Much of the high volume Phoenix metropolitan roadway system has been rehabilitated with a single thin lift of ARFC, and this design has performed well for Arizona DOT both in terms of noise reduction and friction.

As reported in Kaloush et al. (2009), the use of ARFC is approximately 25 to 50 percent more expensive than conventional asphalt overlay. The Arizona DOT and researchers in asphalt rubber maintain that this additional expense is lessened over the service life of the pavement given that ARFC-over-PCC pavements can potentially reduce cracking and the need for maintenance costs typically scheduled for conventional AC overlays. The higher cost of ARFC should be viewed in light of its improvements to important road characteristics, such as both noise and friction.

8.2.1 Noise

A study by Scofield (2003) on the use of different asphalt wearing courses found that AR-ACFC mixes had the greatest amount of noise reduction compared with other systems tested. The other materials tested included Permeable European Mixture (PEM), Stone Matrix Asphalt (SMA), Polymer Modified Open Graded Friction Course (P-ACFC), and ADOT's Standard Open Graded Friction Course (ACFC). Figure 80 shows the results of this test and another similar test by Kaloush et al. (1999).

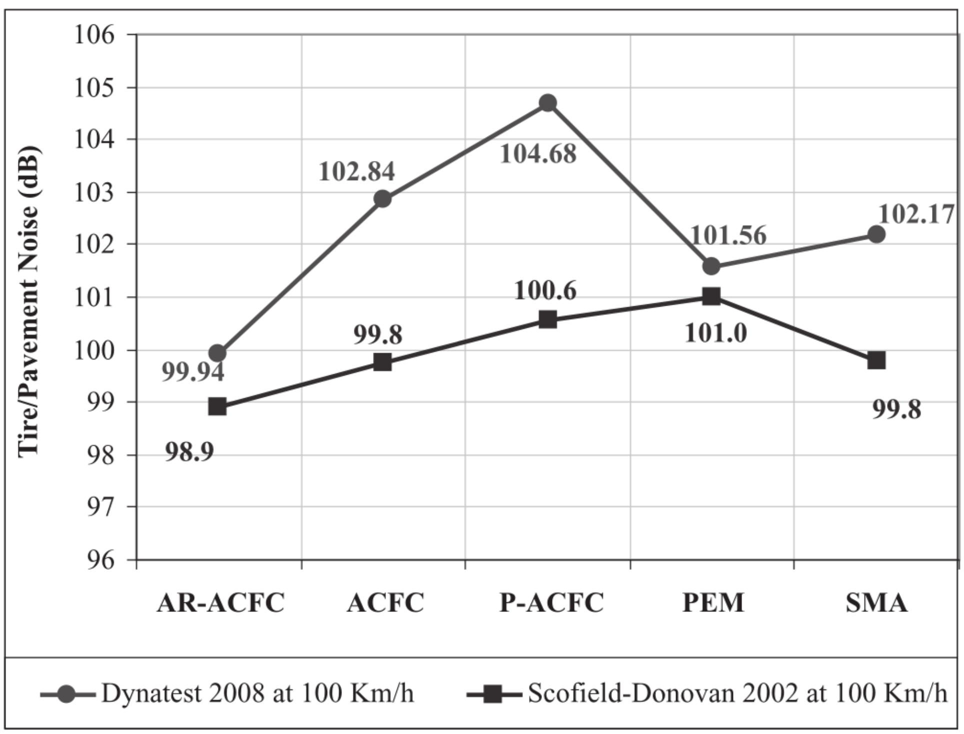

Figure 80. Noise associated with different asphalt wearing course mixes [From Kaloush et al. 2009]

It is believed that the noise reduction experienced by AR-ACFC mixes is due to a combination of effects. The first is acoustic absorption stemming from the viscoelastic nature of asphalt itself and the sound absorption of the rubber, both of which cause damping. Secondly, because the mix has a high voids content, air is pushed through the pavement and dissipated, rather than being compressed by the tires. Finally, there is noise reduction due to the smooth quality of the ride and smaller aggregates, which cause less tire deformation (Kaloush et al. 1999).

8.2.2 Friction

Surface friction is quantified as a skid number, with a higher number indicating more friction, and is measured with a MU meter. ADOT considers intervention necessary when the skid number is 0.34 or less. The measured skid numbers of several pavement sections before and after an ARFC overlay are shown in Figure 81. From this figure, it can be seen that the overlay did improve the surface friction for the pavement in all but one case. Additionally, it should be noted that the values of surface friction are much more uniform across different pavement sections after placement of the overlay. This can be attributed to the fact that any polished surface problems on the original concrete pavement surface have been corrected by the overlay.

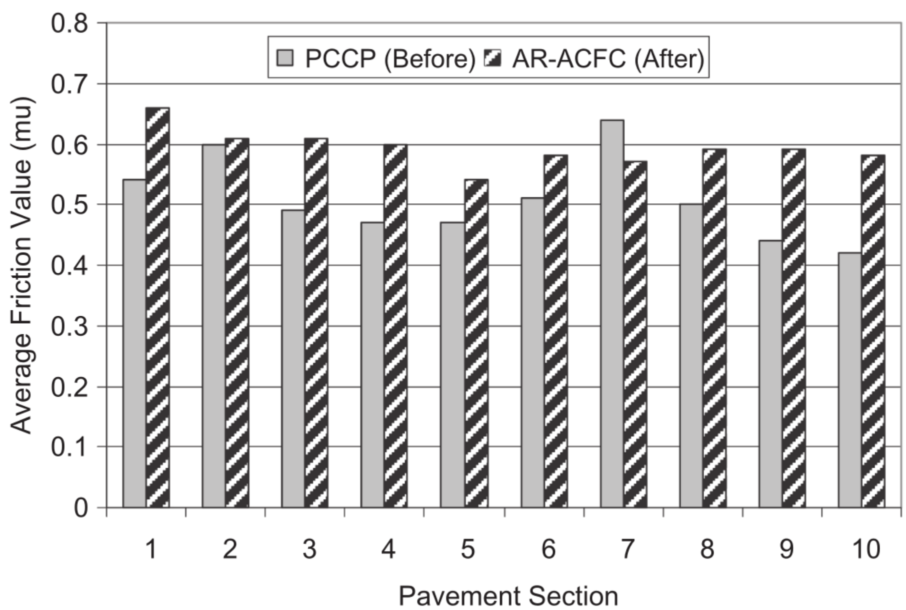

Figure 81. Surface friction measured as skid number before and after placement of an overlay [*From Kaloush et al. 2009*]

8.2.3 Thermal gradient

The ability of ARFC to effectively insulate the underlying concrete pavement was investigated and reported in Kaloush et al. (2009). It was found pavement systems with and without ARFC overlays experienced the same thermal gradient through the system on average (see Table 9).

This effect may have been due in part to the additional heat absorbed at the surface, given the low albedo of the ARFC relative to that of the exposed concrete.

Table 9. Thermal gradients in the pavement system with and without an overlay [*From Kaloush et al. 2009*]

Overlay/Traffic case	Max ΔT °C	Min ΔT °C	Range of ΔT °C
With AR-ACFC: Traffic vs. No Traffic	3.5	–2.5	6.0
Without AR-ACFC: Traffic vs. No Traffic	5.0	–1.0	6.0
With Traffic: AR-ACFC vs. No AR-ACFC	4.0	–3.5	7.5
Without Traffic: AR-ACFC vs. No AR-ACFC	4.0	0.5	4.5

However, it was found that ARFC overlaid pavements have a lower urban heat island effect at night due to a lower thermal mass and the combined effects of increased porosity and the aerating quality of traffic loading. Thermal gradients during the day and at night are approximately 25% and 8% higher respectively for pavements without the ARFC overlay. This is important because higher thermal gradients are associated with more damage for concrete pavements. Using ARFC overlays to reduce the thermal gradients experienced by the concrete pavement can increase the service life of the pavement system.

8.3 SMA-over-JPCP in Germany (A93)

The SHRP2 R21 project included a review of composite pavements constructed in Europe. Some of these pavements included AC overlays of existing JPCP pavements the A93 motorway in Germany (Tompkins et al. 2010). The A93 pavement was originally two-lift concrete pavement that experienced unexpected failures within a few years of construction in the upper lift (wearing course) concrete. To quickly rehabilitate these sections, some were overlaid with a thin SMA surfacing. This included a section featuring a 3 cm layer of SMA over the original total 26 cm two-layer PCC pavement. The SMA was gap-graded, with maximum aggregate size of 8 mm (aggregate type was reported as a locally available granite). The SMA was sawed and sealed after construction.

The SMA layer was placed in 1996, and the SHRP2 R21 survey found that many sections performed very well after 12 years of heavy commercial traffic along A93. Other SMA-over-JPCP sections along A93 that had been overlaid in 1996 had experienced one of two failures after 8 years of service: reflective cracking in the SMA and/or debonding between the SMA and PCC layers, resulting in spalling-type effects as thin sheets of the SMA layer debonded and lifted from the surface.

Overall, the thin SMA layers used to rehabilitate JPCP pavements along A93 performed very well given their age and economy. An example of these pavements is illustrated in Figure 78. Germany had decades of success using these overlays in a cost efficient manner. This success relied on high quality materials and established construction techniques such as sawing-and-sealing.

Chapter 9. Conclusions

The majority of rigid pavements will deteriorate in ride quality far before they lose their structural integrity; in a growing number of such cases, asphalt overlays are a popular mode of rehabilitation. This reporting, while not comprehensive, provides the user a broad appreciation for the evaluation, design, construction, and performance of asphalt overlays of concrete pavements. As AC overlays become even more popular, an understanding its design, construction, and performance will continue to be of importance for pavement engineers as existing infrastructure ages.

Each chapter of the reporting detailed a specific issue in the evaluation, design, construction, and performance of AC-PCC. These chapters followed the natural process of rehabilitation:

- **Chapter 2**. The condition of the existing pavement is determined on the basis of popular structural and functional evaluation methods. This evaluation allows the engineer to determine the appropriate measure to prolong the pavement life.

- **Chapter 3**. If an AC overlay is selected to rehabilitate the existing pavement, the engineer must identify and repair distresses in the existing pavement. This may also include measures taken to eliminate distress causes, such as faulty drainage or base/subgrade support.

- **Chapter 4**. Given traffic and climate considerations, a mix design must be selected for the AC overlay. This design should emphasize important performance issues for the agency and road users, issues such as resistance to deformation in rutting or noise suppression.

- **Chapter 5**. The engineer must design the pavement structure given available traffic, climate, and existing pavement conditions, as well as selected material properties of the AC overlay. Mechanistic-empirical design tools such as the MEPDG ensure that the engineer can optimize the overlay such that the desired performance is achieved using the most economical design.

- **Chapter 6**. The selected AC overlay mix and structural designs are used to place the overlay using the state-of-the-practice in overlay construction.

- **Chapter 7**. Prior to construction, the design and construction of the overlay can be adjusted given the results and observations of major studies on the long-term performance of AC-PCC pavements, including those constructed and regularly monitored under the LTPP.

- **Chapter 8**. Additional case studies, with conditions similar to those of the engineer’s project, can be consulted to further refine the AC overlay design and construction methods.

Finally, this reporting is an indirect product of the FHWA Pooled Fund Project TPF-5(149) Thermally Insulated Concrete Pavement project, which focused on the design, cost analysis, construction, and analysis of AC overlays of newly constructed PCC pavements. Although this project dealt with new construction, many of its research products, including models and software for the design and analysis of thermally insulated concrete pavements, are as applicable to AC-PCC as they are overlays of newly constructed PCC pavements. The reader is referred to the TPF-5(149) project documentation for additional details on its advancements in the design and analysis of AC-PCC.

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