IS-117

NATIONAL ASPHALT PAVEMENT ASSOCIATION

GUIDELINES FOR USE OF **ASPHALT OVERLAYS** TO REHABILITATE PCC PAVEMENTS

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LIST OF ACRONYMS

AASHTO American Association of State Highway and Transportation Officials
AC Asphalt Concrete
ADT Average Daily Traffic
ADTT Average Daily Truck Traffic
APA Asphalt Pavement Analyzer
ARGG Asphalt Rubber Gap-Graded
BBF Bending Beam Fatigue
B&S Break and Seat
BMD Balanced Mix Design
BRBC Bottom Rich Base Course
BRIC Binder Rich Intermediate Course
CAM Crack Attenuating Mix
CBR California Bearing Ratio
C&S Crack and Seat
CPR Concrete Pavement Restoration
CRCP Continuously Reinforced Concrete Pavement
DCT Disc-Shaped Compact Tension
DOT Department of Transportation
ER Elastic Recovery
FHWA Federal Highway Administration
FWD Falling Weight Deflectometer
GPR Ground Penetrating Radar
HiMA Highly Modified Asphalt
HWTT Hamburg Wheel-Tracking Test
I-FIT Illinois Flexibility Index Test
IRI International Roughness Index
iRLPD Incremental Repeated-Load Permanent Deformation
JPCP Jointed Plain Concrete Pavement
JRCP Jointed Reinforced Concrete Pavement
LCCA Life-Cycle Cost Analysis
LLAP Long-Life Asphalt Pavement
LT-SCB Low-Temperature Semicircular Bend
ME Mechanistic-Empirical
MHB Multiple-Head Breaker
MSCR Multiple Stress Creep Recovery
NAPA National Asphalt Pavement Association
NCHRP National Cooperative Highway Research Program
NMAS Nominal Maximum Aggregate Size
PCC Portland Cement Concrete
PCI Pavement Condition Index
PG Performance Graded
PPA Perpetual Pavement Award
PSI Present Serviceability Index
RAP Reclaimed Asphalt Pavement
RAS Recycled Asphalt Shingles
RFB Resonant Frequency Breaker
RPB Resonant Pavement Breaker
SDI Surface Distress Index
SHRP Strategic Highway Research Program
SMA Stone Matrix Asphalt
SN Structural Number
SPT Standard Penetration Test
SSR Stress Sweep Rutting
TSR Tensile Strength Ratio
VCA Voids in the Coarse Aggregate
VMA Voids in the Mineral Aggregate
WMA Warm Mix Asphalt

REHABILITATION CONSIDERATIONS

1.1 Introduction

Several rehabilitation activities may take place during a pavement's life, each aimed at maintaining or restoring its level of service to provide the driving public with safe, smooth, and functional roadways. Rehabilitation occurs when extensive distresses are present throughout the pavement structure and/or additional structural capacity is needed. Its main objective is to restore the pavement's level of service and extend its service life. However, rehabilitation often requires lane closures, causing traffic disruptions, increasing user delay costs, and demanding agency resources to ensure the safety of construction workers and the traveling public.

1.2 The Rehabilitation Process

For Portland Cement Concrete (PCC) pavements, rehabilitation is typically performed to correct distresses such as joint faulting, slab cracking, joint and crack spalling, or other issues that may affect ride quality (Tayabji et al. 2000). Rehabilitation treatment options generally fall under one of four categories: restoration, resurfacing, recycling, or reconstruction (Hall et al. 2001). Selecting an appropriate rehabilitation strategy requires carrying out a process to understand the causes of the observed distresses, identifying feasible alternatives, and analyzing technical, economic, or other factors to select the preferred solution.

The first phase of the rehabilitation process consists of gathering relevant data and assessing the current condition of the pavement. While pavement distress surveys are the main source of information, other inputs, such as traffic, roadway geometry, materials, drainage, etc., may become driving factors when making a final decision.

Once the pavement condition information has been collected and analyzed, the next phase entails identifying candidate strategies capable of repairing the existing distresses and achieving the desired improvements in structural capacity, functional adequacy, and pavement drainage (Hall et al. 2001). The feasibility of the individual strategies may depend on project constraints identified in the initial stages of the process.

The final phase of the rehabilitation design process culminates with the selection of the preferred solution. Traditionally, the decision is based on a life-cycle cost analysis (LCCA), which compares the lifecycle costs of different rehabilitation options. Recently, more agencies are also employing life cycle assessment (LCA) as part of a Buy Clean policy to evaluate their lifecycle environmental impact. The accuracy of the LCCA and LCA results heavily depends on the agency's ability to predict the performance of different options and quantify costs and environmental impact. Although financial factors are crucial, the final decision also considers non-monetary aspects, such as construction time and traffic disruption. Environmental impact, as assessed through LCA, is increasingly becoming a significant criterion in the selection process.

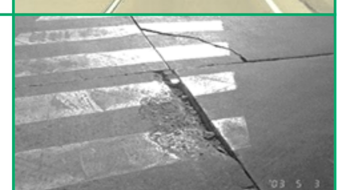
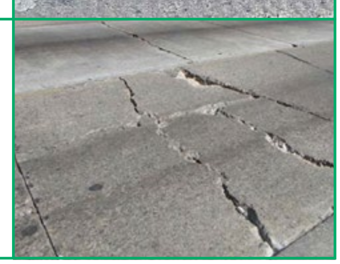
1.3 Rehabilitation of PCC Pavements with Asphalt Overlays

PCC pavement rehabilitation may involve concrete pavement restoration (CPR) techniques or resurfacing with either asphalt or concrete overlays. The use of asphalt concrete (AC) overlays to rehabilitate PCC pavements is a popular practice that involves placing one or more layers of AC directly on the existing PCC or over a broken or rubblized PCC layer (Khazanovich et al. 2012).

The Federal Highway Administration (FHWA) reports that the U.S. has approximately 55,000 lane miles of PCC-surfaced pavements and 91,500 lane miles of composite pavements (FHWA 2020), meaning nearly two-thirds of concrete pavements have been overlaid with asphalt. Many agencies choose asphalt overlays because they are usually more economical and require less construction time than the other alternatives. Asphalt overlays can improve the existing PCC pavement's functional and structural conditions by restoring the surface, improving rideability, and increasing load-carrying capacity. However, the performance of composite pavements can be hindered due to the likelihood of reflection cracking in the asphalt overlay if the PCC pavement is either jointed or distressed.

To prevent or minimize the appearance and/or severity of reflection cracking, the rehabilitation options may include placing a thick AC overlay, sawing and sealing joints in the AC overlay to match the joints in the old PCC pavement, using special materials such as fabrics, stress-relieving interlayers, and specially designed asphalt mixes with enhanced crack-resistant properties, or performing slab fracturing techniques like crack and seat, break and seat, and rubblization. When selecting among these options, best practices consider both the condition and the type of the existing PCC pavement. Table 1.1 describes different condition categories for PCC pavements related to serviceability and observed distresses. Table 1.2 summarizes the appropriate AC rehabilitation options for PCC pavements based on the condition categories described in Table 1.1 for three PCC pavement types, including Jointed Plain Concrete (JPCP), Jointed Reinforced Concrete (JRCP), or Continuously Reinforced Concrete (CRCP). It is important to note that the list of feasible options depends on the project-specific conditions and must always be determined through a detailed technical study. The following chapters discuss the reflection cracking mechanism and provide guidance for designing and constructing AC overlays to rehabilitate PCC pavements.

Table 1.1 Pavement Condition Categories

Condition	Present Serviceability Index (PSI)	Pavement Condition Index (PCI)	International Roughness Index (IRI), in/mi.	Description	Example*
Very Good – Good	> 3.0	> 70	< 60	First signs of surface wear and minor surface defects such as scaling, pop-outs, and map cracking. Isolated transverse and longitudinal cracks, tight or well-sealed. Some open joints.	
Good – Fair	2.5 – 3.0	55 – 70	60 – 94	Minor to moderate corner cracking, joint and crack spalling. First signs of joint or crack faulting. More frequent transverse or longitudinal cracks. Moderate to severe surface scaling. Moderate settlement or frost heave areas.	
Fair – Poor	2.0 – 2.5	40 – 55	95 – 170	Severe polishing, scaling, map cracking, or spalling. More extensive transverse and longitudinal cracks, open joints and cracks with moderate to severe spalling. Moderate to severe faulting that affects ride quality. Extensive patching. Corner cracks with missing pieces. Pavement blowups.	
Poor - Failed	≤ 2.0	≤ 40	> 170	Extensive slab cracking, severely spalled and faulted. Patching in very poor condition. Severe joint deterioration, with spalling and additional cracks parallel to the joint. Severe and extensive settlements or frost heaves. Restricted speed due to poor ride quality.	

*Photos courtesy of Guillermo Thenoux and the Minnesota Department of Transportation.

Table 1.2 Asphalt Concrete Overlay Options by Pavement Condition and PCC Pavement Type (NAPA 1999)

Rehabilitation Option	Condition Category											
	Very Good - Good			Good - Fair			Fair - Poor			Poor - Failed		
	JPCP	JRCP	CRCP	JPCP	JRCP	CRCP	JPCP	JRCP	CRCP	JPCP	JRCP	CRCP
Saw/Seal and/or Overlay	●	●		●	●	●			●			
Crack & Seat w/overlay							●			●		
Break & Seat w/overlay								●			●	
Rubblization w/overlay							●	●	●	●	●	●
Special Materials				●	●	●	●	●	●			

JPCP – Jointed Plain Concrete Pavement
JRCP - Jointed Reinforced Concrete Pavement
CRCP – Continuously Reinforced Concrete Pavement

REFLECTION CRACKING OF HMA OVERLAYS

Asphalt overlays are an effective rehabilitation technique to restore rideability and improve the structural capacity of deteriorated PCC pavements. The long-term functional performance of these overlays depends upon their resistance to potential distresses, including rutting and various modes of cracking, with reflection cracking being the most prevalent. This chapter discusses the causes of reflection cracking and mitigation techniques utilized to control these cracks.

2.1 Causes and Mechanisms of Reflection Cracking

Reflection cracks can occur in an asphalt overlay due to concentrated stresses at the cracks and joints caused by slab movements under environmental and traffic loadings. Most reflection cracking is driven by a combination of the following mechanisms, with the vertical and horizontal slab movements being the most accepted causes (Von Quintus et al. 2009).

2.1.1 Traffic loading

1. If the PCC has poor load transfer between slabs, such as jointed concrete pavements without dowel bars or with deteriorated dowel effectiveness, high vertical deflections can occur when wheel loads move across the joint or crack. This deflection at the joint or crack causes high tensile stresses at the bottom of the asphalt overlay (Figure 2.1a). In addition, differential vertical deflections across the deteriorated joints and cracks under traffic result in shear-stress concentrations in the asphalt overlay (Figure 2.1b). High deflections can develop at joints or cracks due to the gradual reduction of load transfer or the development of voids beneath the PCC at those locations. Thus, reflection cracking driven by traffic loadings is a combination of shear and tensile-fatigue stresses that depend on the magnitude of the vertical deflections across the joint or crack. The important factors include the

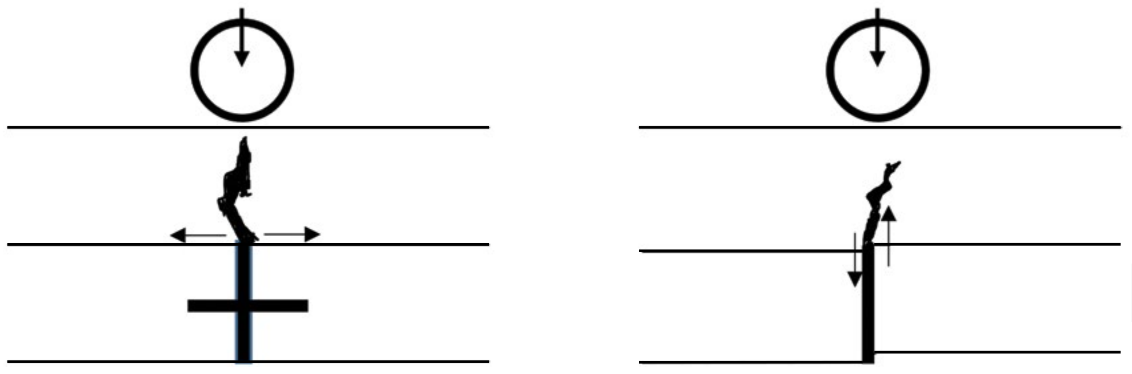


Figure 2.1 Reflection Cracking Induced by Wheel Loads (Von Quintus et al. 2009)

magnitude of the wheel load, the amount of load transfer across the joint or crack, and the subgrade support under the slab at those locations.

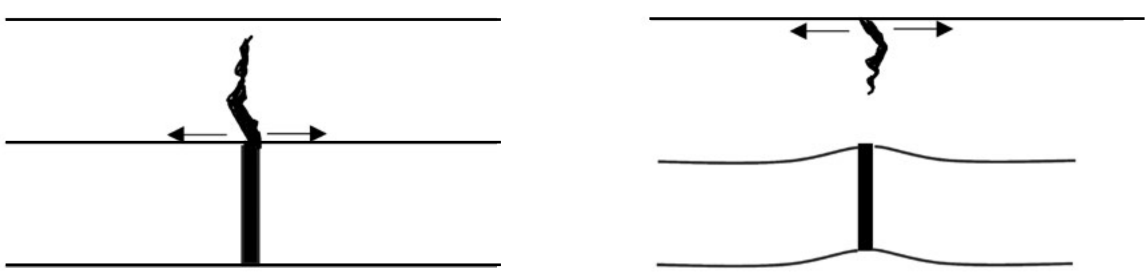
2.1.2 Environmental effects

As shown in Figure 2.2, the following two mechanisms can occur simultaneously.

1. The primary environment-related reflection cracking mechanism starts with horizontal movements concentrated at joints and cracks in the underlying PCC pavement due to expansion and contraction in the PCC slab due to daily and seasonal temperature changes. Slab movements lead to concentrated tensile stresses and strains at the bottom of the asphalt overlay directly above joints and cracks, initiating cracking at the bottom of the asphalt overlay and propagating upward (Figure 2.2a). This mechanism depends upon the magnitude and rate of temperature change, slab geometry, the opening of the joint or crack, and the properties of the asphalt overlay.

2. The secondary environmental-related mechanism is due to the curling of PCC slabs caused by a temperature gradient within the slab. Slabs curl upward when the top of the slab is cooler than the bottom, and they curl downward when the opposite occurs. An upward curl between adjacent slabs results in tensile stresses at the surface of the overlay above the joint, which can cause a crack to initiate at the surface and propagate downward (Figure 2.2b). In winter, the stiffness of the asphalt overlay increases, and it can lose the ability to relax under strain. Over time the surface of the asphalt overlay also becomes more brittle due to aging, resulting in its loss of strain tolerance. This mechanism depends on the magnitude and rate of slab curling and the properties of the asphalt overlay during cold temperatures.

In summary, asphalt overlays on PCC pavements are subjected to several high strain mechanisms that can occur simultaneously. As illustrated in Figure 2.3a, the combined strain and stress concentrations at the bottom of the asphalt overlay eventually cause cracks at the bottom of the asphalt overlay (Figure 2.3b). Due to cyclic temperature changes and repeated traffic loads, the cracks will grow and reflect upwards to the surface of the asphalt overlay (Figure 2.3c and d). As this process continues, multiple reflection cracks will form, causing portions of the asphalt overlay to deteriorate (Figure 2.3e and f). Figure 2.4 shows a deteriorated reflection crack area in an asphalt overlay over an existing PCC pavement, leading to reduced serviceability and shortened service life of the asphalt overlay. Thus, reflection cracking must be addressed in the rehabilitation selection process, described further in the following section.



a. Horizontal Slab Movements due to Temperature Changes Causing Tension at Bottom of Asphalt Layer

b. Curling of PCC Slab due to Temperature Gradient Creating Tension at Surface of Asphalt Layer

Figure 2.2 Reflection Cracking Induced by Temperature Changes in PCC Slabs (Von Quintus et al. 2009)

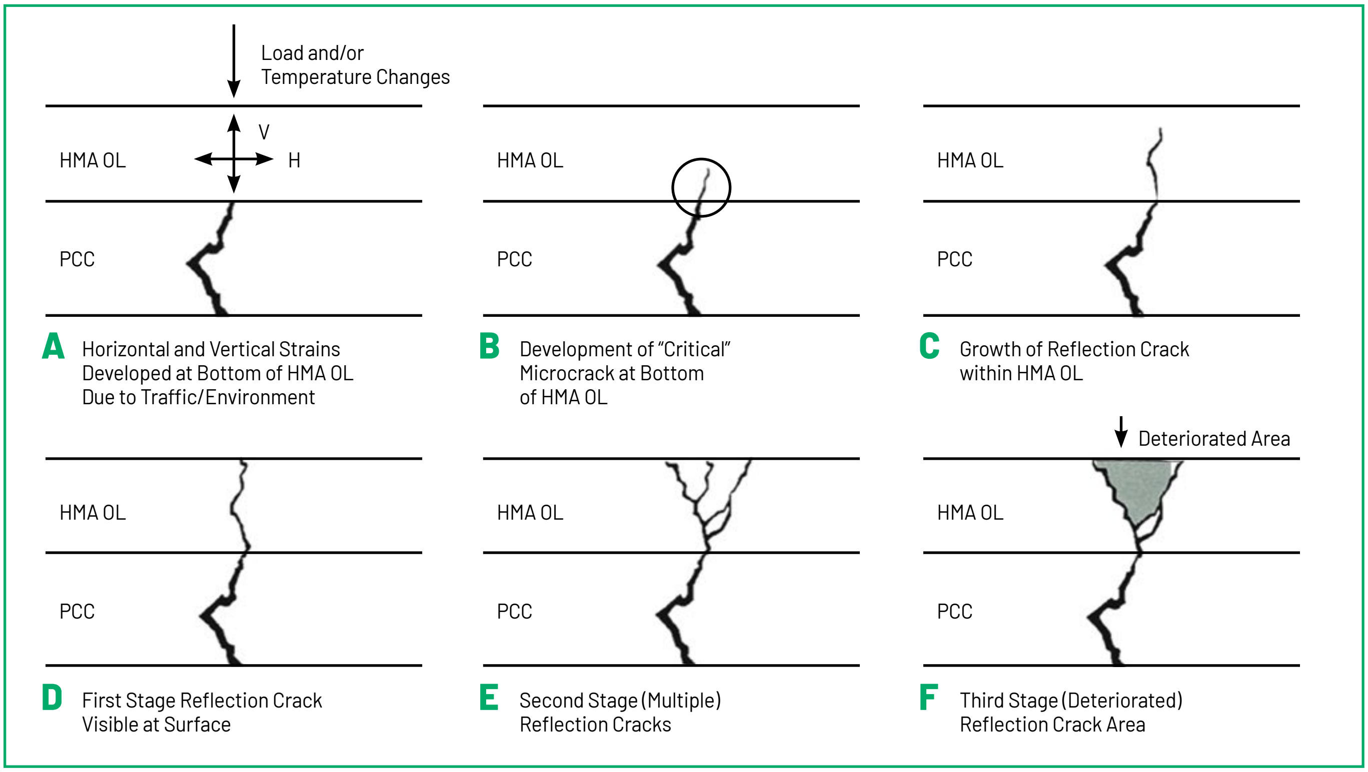


Figure 2.3 Growth Mechanisms Associated with Reflection Cracking (NAPA 1999)

2.2 Mitigation Methods

Several methods have been developed to control or reduce the rate of reflection cracking in asphalt overlays. These methods can be grouped into different categories, as shown in Table 2.1. A previous survey of highway agencies, contractors, and design firms by West et al. (2020) suggests that the most common mitigation method is the fractured slab technique, followed by special interface materials, special overlay asphalt mixtures, and saw and seal overlays. Within the fractured slab technique, the break and seat method is much less frequently used than rubblization and crack and seat methods as approaches to reduce strain concentrations in existing PCC pavements. This document focuses on the fractured slab technique, and the slab fracturing process is discussed in the next chapter.

Figure 2.4 Deteriorated Reflection Crack Area (NAPA 1999)

Table 2.1 Methods for Mitigating Reflection Cracking in Asphalt Overlays over PCC Pavements

Mitigation Method	Purpose
Fractured slab techniques › Rubblization › Crack and Seat › Break and Seat	These techniques reduce or eliminate the effective length of the slab to prevent movement of the concrete layer and, in turn, reflection cracking.
Special interface materials › Stress absorbing membrane interlayer › Geosynthetic/fiberglass interlayer › Crack relief layers (e.g., open graded asphalt or unbound granular base)	These materials act as reinforcement or a stress-energy absorber (i.e., stress relieving layer) to retard crack propagation from the concrete layer to the asphalt overlay.
Special overlay asphalt mixtures › Stone matrix asphalt overlay › Open-graded asphalt overlay › Gap-graded asphalt-rubber overlay › Highly modified stone matrix asphalt overly	These overlay mixtures have better resistance to cracking due to higher binder contents and modified or highly modified binders. Severe distresses in the existing PCC pavement must be repaired before placing these mixtures.
Saw and seal overlays	This method prevents the random propagation of reflection cracking from underlying PCC slab joints to the top of the asphalt overlay.

CHAPTER 3

SLAB FRACTURING PROCESS

3.1 Slab Fracturing Techniques

Reflection cracking in asphalt overlays is caused by the concentration of stresses at existing joints and cracks in the PCC pavement. Since the stress concentration is directly proportional to the spacing of the joints and cracks, reducing the joint and crack spacing through slab fracturing techniques can minimize or eliminate reflection cracking in asphalt overlays. Three slab fracturing techniques are often utilized, including crack and seat (C&S), break and seat (B&S), and rubblization. These techniques involve fracturing and seating the existing PCC pavement but differ in fragment sizes and equipment used.

3.1.1 Crack or Break and Seat Techniques

C&S and B&S involve fracturing PCC slabs into shorter joint spacing and seating the fractured pieces (Von Quintus et al. 2009). C&S is used on JPCP, while B&S is applied to JRCP. Both techniques utilize guillotine or impact hammers to fracture the slabs, followed by heavy pneumatic rollers to seat the fractured slabs onto the base (West et al. 2020).

The two techniques differ in the level of effort applied to fracture the PCC slabs. C&S (Figure 3.1 (a)) is intended to produce tight cracks in JPCP that still allow for load transfer with minimal loss of structural integrity, while B&S (Figure 3.1(b)) requires more effort to break or debond steel reinforcement to properly seat the JRCP slabs. The maximum crack spacing (or resulting fragment size) should be less than 30 inches for C&S projects and less than 12 to 18 inches for B&S projects (Rada 1992; NAPA 1999).

Both techniques involve firmly seating the fractured slabs using a 35 to 50-ton pneumatic roller (Figure 3.2). Two passes of the pneumatic roller are recommended for the C&S process, while five to seven passes are recommended for the B&S process to ensure the steel is fully ruptured. However, care should be taken to not over-roll the slabs to avoid reducing the interlocking of the pieces, especially for C&S, which can negatively impact load transfer and structural integrity. Good construction results in through-slab cracking to produce the desired PCC segment size.

a. Cracking Operation for C&S

b. Breaking Operation for B&S

Figure 3.1 Fractured PCC after Cracking and Breaking Operations (Courtesy of Antigo)

a. Seating after Cracking

b. Seating after Breaking

Figure 3.2 Seating Operations for C&S and B&S (Courtesy of Antigo)

Proper seating of the fractured slabs is important to prevent voids from forming under the shorter slabs, leading to rocking and settlement of pavement sections under repeated traffic loadings. This can cause reflection cracks to appear earlier and increase in severity.

3.1.2 Rubblization Technique

Rubblization is a technique that involves breaking PCC slabs into small pieces, typically 3 to 8 inches, to create a strong aggregate base (NAPA 1999). The process is generally done using specialized equipment such as multiple-head breakers (MHB) or resonant pavement breakers (RPB). Following the rubblization, depressions are filled with coarse aggregate, and the rubblized material is compacted with a steel wheel roller before placing an asphalt concrete overlay. This method can be applied to all types of PCC pavement and can allow for complete separation of steel reinforcement, if present, from the rubblized concrete. While steel reinforcement can remain in the PCC slabs, any steel reinforcement that protrudes above the surface of the rubblized PCC pavement must be removed.

Rubblization is an effective technique for eliminating reflection cracking in PCC slabs, but it requires adequate support from the base and subgrade. If the underlying support is poor, the PCC pavement may not be rubblized to the required extent, affecting the performance of the asphalt overlay. To address this challenge, Antigo Construction, Inc. (Antigo) developed a modified rubblization technique that uses less fracture energy to create a stiffer rubblized concrete layer that supports construction operations and asphalt overlays while effectively eliminating reflection cracking (Buncher et al. 2008). This modified method specifies a maximum particle size of 12 inches at the surface and 15 inches at the bottom of the slab, which is different from the typical 3 to 8 inches specified in IS-117. Figure 3.3 illustrates the difference between traditional rubblization and modified rubblization methods.

a. Traditional Rubblization

b. Modified Rubblization

Figure 3.3 Comparison of Traditional and Modified Rubblization (Courtesy of Antigo)

Both RPB and MHB have been used successfully in the rubblization process, as they deliver sufficient energy to fracture the full depth of the slab and break all existing slab connections. These two types of equipment operate differently to achieve the required rubblization of the PCC pavement. The RPB uses a high-frequency, low-amplitude setting, while the MHB operates in a low-frequency, high-amplitude mode. Both types of equipment can rubblize PCC slabs with thicknesses of up to 26 inches (660 mm) (Buncher et al. 2008). When rubblizing thick PCC slabs with the MHB, a guillotine-type breaker is typically used to prebreak the PCC slabs that are more than 14 inches thick (Buncher et al. 2008). A brief description of each equipment type follows.

Multi-Head Breaker Fracturing and Compacting Processes. The MHB has twelve to sixteen 1,200- to 1,700-lb drop hammers arranged in pairs, half in a forward row and half diagonally offset in a rear row. This design ensures continuous breakage from side to side. Each pair of hammers is attached to a hydraulic lift cylinder and can develop 1,000 to 8,000 ft-lb of energy based on lift height and cycle at a rate of 30 to 35 impacts per minute. The drop height can be adjusted during operation to control breaking energy on PCC pavement. The machine is 8 feet wide with 12 hammers, each 8 inches wide, and can have wings added to each side for a total width of up to 13 ft. With individual control of each lifting cylinder, the breaking width can range from 2.67 ft to 13 ft. The MHB is capable of rubblizing a full lane width of pavement in a single pass. It is equipped with a screen to protect personnel and vehicles from flying chips during fracturing.

After the fracturing process, a smooth steel drum vibratory roller with raised Z-grids on the drum (Z-grid roller) with a gross weight of at least 10 tons is used in the vibratory mode for two passes to further fracture the surface particles and to begin to settle and seat the rubblized pavement. A pneumatic-tire roller with a gross weight of 10 to 25 tons is then used to further settle and seat the rubblized pavement for slab thicknesses of 8 inches (203 mm) or thicker. Finally, a smooth steel drum vibratory roller with a gross weight of at least 10 tons, operated in the vibratory mode, is used to settle the rubblized pavement and provide a smooth surface for the asphalt concrete overlay. Figure 3.4 shows an MHB in operation and three rollers used for compacting the rubblized pavement.

Resonant Frequency Breaker Fracturing and Compacting Processes. The RFB is a self-propelled machine that uses high-frequency, low-amplitude impacts to deliver a force of 2,000 lbs. to the pavement. It features a shoe at the end of a pedestal attached to a beam and counterweight. The force is applied to the pavement by vibrating the large steel beam connected to the shoe, which is moved along the concrete surface at the front of the machine. The breaking principle is low-amplitude and high-frequency resonant energy delivered to the concrete slab, causing high tension at the top. Since concrete has low tensile strength, the slab fractures on a shear plane through the pavement. The machine is designed to allow the adjustment of parameters such as the shoe, beam size, operating frequency, loading pressure, and speed to optimize performance.

a. Multi-Head Breaker

b. Three-Roller Compaction Process

Figure 3.4 Multi-Head Breaker and Compaction Equipment for Rubblization of PCC Pavements (Courtesy of Antigo)

The breaking process usually starts at the pavement's outside edge and continues toward the centerline, as illustrated in Figure 3.5(a). The breaking pattern is around 8 inches wide, typically requiring 18 to 20 passes to break a 12-foot wide lane. To avoid disruptions to the base and damage to underground structures, the RFB is generally operated at a maximum amplitude of one inch. The machine may encroach on an adjacent lane by about 3 to 5 ft to rubblize near the pavement's centerline. Since the RFB has wheel loads of 20,000 lbs. and a total weight of 60,000 to 70,000 lbs., it is important that the fractured pavement, shoulder, and subgrade be able to support multiple passes of the equipment. Proof rolling should be performed on the rubblized areas using a heavy pneumatic roller. If any weak or unstable areas are identified, they must be removed and replaced with full depth asphalt concrete or flexible base material.

After the fracturing process, two to four passes of a smooth 10-ton tandem vibratory roller at a low-amplitude and high-frequency setting are often used to settle the rubblized pavement. A typical asphalt concrete roller can be used. Some agencies may require an 8-to-10-ton pneumatic roller to further seat the fractured pavement before placing the overlay. For example, the Louisiana DOT requires one pass with a pneumatic-tire roller after the initial pass with the vibratory roller. Two additional passes with the vibratory roller are made to settle the rubblized pavement and provide a smooth surface for the asphalt overlay. Figure 3.5 shows an RFB in operation and a smooth steel roller for compaction.

3.2 Best Practices for Slab Fracturing

State highway agencies have established specifications for constructing asphalt overlays over fractured slabs. These specifications generally include requirements for crack patterns or particle sizes, equipment, quality control measures, methods of measurement, and basis of payment. While each project may have specific requirements, the following recommendations can be considered to ensure proper control of the construction process.

3.2.1 For C&S and B&S Processes

- Before starting the fracturing process, removing material from a previous asphalt overlay is important. Failure to do so will significantly decrease the effectiveness of these techniques, as much of the fracture energy will be absorbed by the asphalt overlay.

- A test strip should be constructed at the start of the project to ensure the equipment and slab fracturing process are adequate to achieve the desired cracking pattern for through-slab cracking and acceptable PCC segmentation without causing excessive damage to the existing PCC surface.

- The crack spacing for the slab fracturing process can vary depending on the type of existing PCC pavements. The cracking pattern and fractured slab size can be verified by spraying water on the surface of the slabs immediately after the fracturing process. (Figure 3.6).

a. Resonant Frequency Breaker   **b.** Compaction with a Smooth Steel Roller

Figure 3.5 Resonant Frequency Breaker and Smooth Steel Roller for Rubblization (Courtesy of RMI)

- The slabs must be cracked through the entire slab thickness for both fracturing processes. Cores can be taken directly over cracks to confirm the cracks are through the slab thickness (Figure 3.7). Additionally, the B&S process must rupture the reinforcing steel to be effective.

- After the fracturing process, the broken slabs must be firmly seated on the underlying layer to prevent rocking. If any soft spots or local failures are identified during the seating process, they should be removed and replaced.

- Care should be taken to avoid exposing the fractured slab layer to heavy rain, which can saturate and weaken the existing pavement system. As a general guideline, the asphalt overlay should be placed over the properly seated fractured slab within 24 hours of the seating process when rain is in the forecast.

- Before the asphalt overlay is placed, the fractured surface should be swept thoroughly and a tack-coat applied before placing the initial asphalt layer. Limiting traffic on the surface of intermediate lifts is recommended until at least five inches of asphalt mixture has been placed over the fractured PCC surface.

3.2.2 For Rubblization

- As with the other fracturing techniques, any previous asphalt overlay must be removed before starting the rubblization to ensure the equipment can directly contact the PCC pavement. In addition, adjacent utilities and pavements that are not to be rubblized must be isolated from the pavements to be rubblized through a full-depth saw cut or by cutting a relief trench with a wheel saw to prevent impact and vibration damage.

- An adequately designed underdrain system should be installed before rubblization to drain free water from the base or subgrade and remove free water that may enter the rubblized pavement from the surface.

- A test strip is recommended at the start of rubblization to ensure the procedure rubblizes the pavement to the required extent. A test pit can be excavated to verify that the equipment is producing the required rubblized particle sizes throughout the depth of the PCC pavement and adequately debonding any steel reinforcement.

- The rubblized pavements are then rolled and reseated to tighten and smooth the surface in preparation for paving.

- During rubblization and rolling, unstable pavement areas should be replaced with full-depth asphalt patches. Also, leveling courses may be used to adjust the grade and profile since the surface of the rubblized PCC layer cannot be bladed with a motor grader. In some cases, rubblized PCC pavements can be milled to make the final overlay profile match the elevation of other features around the rehabilitated pavement.

- Paving the asphalt overlay on the rubblized surface can be accomplished as with paving on an aggregate base. It is recommended that traffic be kept off the surface until the entire asphalt overlay has been placed.

- Finally, care should be taken to avoid exposing the rubblized pavement to rain, which can saturate and weaken the existing pavement system. As a general guideline, the first asphalt overlay layer should be placed over the properly seated rubblized pavement within 24 hours of the rubblizing process when rain is in the forecast.

Figure 3.6 Cracking Pattern of Cracked and Seated PCC Slabs (Courtesy of Antigo)

Figure 3.7 Coring for Verifying Through-Slab Cracking (Courtesy of Antigo)

STRUCTURAL DESIGN OF ASPHALT OVERLAY

4.1 Project Scoping

Slab fracturing techniques are commonly utilized to minimize reflection cracking in asphalt overlays on existing PCC pavements. C&S and B&S reduce the effective slab length and minimize horizontal slab movements, while rubblization breaks the slabs into smaller pieces, eliminating slab action and reflection cracks. Of the slab fracturing techniques, rubblization is the most effective method for controlling reflection cracking, but it reduces the structural support (i.e., fractured PCC modulus) of the existing concrete layer more than C&S or B&S, requiring a thicker asphalt overlay. Therefore, it is important to select a fracturing technique with the nominal fragment size that yields a critical PCC modulus (E_{PCC}), as illustrated in Figure 4.1, that strikes a balance between the required overlay thickness and the probability of reflection cracking for a PCC pavement rehabilitation project (Decker 2006).

Several factors should be considered when choosing the appropriate slab fracturing technique for a PCC pavement rehabilitation project, as discussed below (Antigo 2023).

- **Type of Existing Concrete Pavement.** While rubblization can be utilized for fracturing all PCC pavement types, C&S only applies to JPCP, and B&S is used for JRCP. Additionally, it is essential to remove any previous asphalt overlay prior to commencing the fracturing process. The asphalt overlay would absorb a significant amount of fracture energy, thereby diminishing the effectiveness of these techniques.

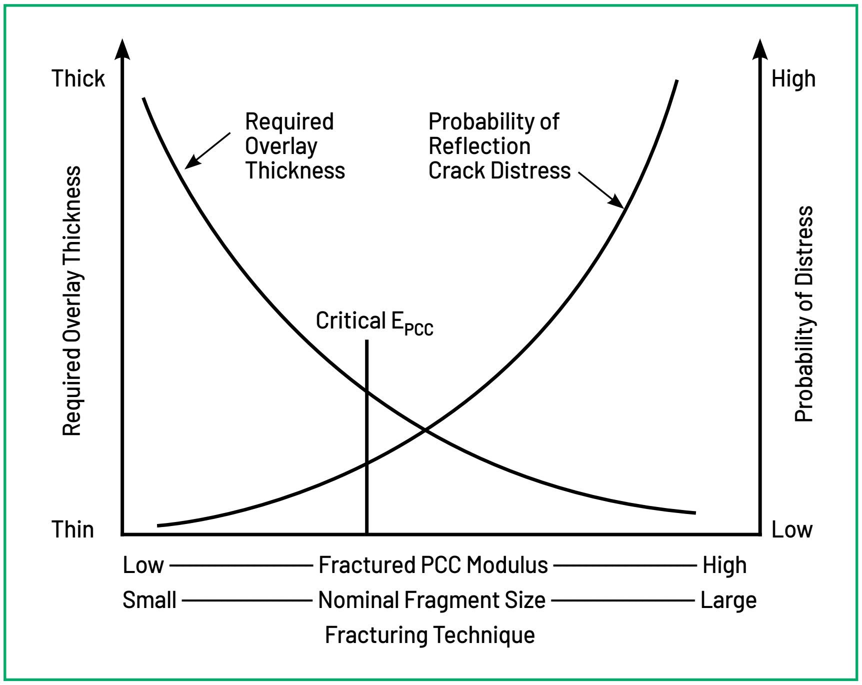


Figure 4.1 Effect of Fractured PCC Modulus on Overlay Thickness and Probability of Reflection Cracking (Decker 2006)

- **Conditions of Existing Concrete Pavement.** C&S and B&S are most appropriate for slabs with limited structural damage, while rubblization is typically better for more severely damaged slabs. For example, the Wisconsin Department of Transportation considers rubblization for PCC pavements meeting one or more of the following conditions (WisDOT 2019):
  - Greater than 20% of the concrete pavement joints need repair.
  - Greater than 20% of the concrete surface has been patched.
  - Greater than 20% of the concrete slabs exhibit “slab breakup” pavement distress.
  - Greater than 20% of the project length exhibits “longitudinal joint distress” over 4 inches wide.

- **Subsurface Conditions.** Fracturing a PCC slab requires adequate support from the underlying layers and subgrade to ensure consistent fracturing and facilitate construction, particularly for rubblization, which involves higher fracture energy. The suitability of the subsurface condition for rubblization can be determined by assessing the relationship between subgrade support, based on CBR determined from testing of recent soil samples, and the total thickness of pavement layers above the subgrade, including subbase, base, and concrete pavement. Figure 4.2 (WisDOT 2019) shows such a relationship, with the area above and to the right of the curve indicating conditions that allow for effective rubblization. For PCC pavements that fall within the shaded "Remedial Action Required" area, it is recommended to utilize a "modified rubblization" process, as described in Chapter 3, or C&S/B&S, which retains more of the structural support of the existing PCC slab to compensate for low subgrade and base support.

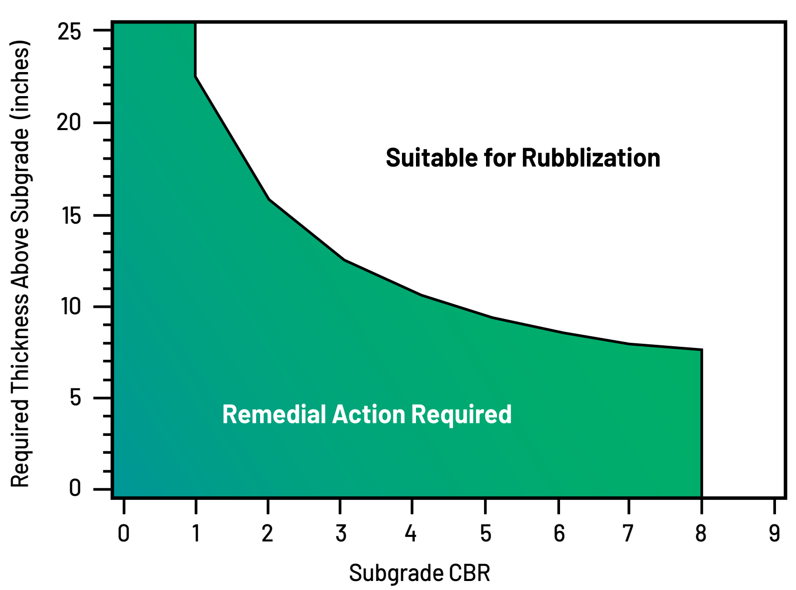


Figure 4.2 Adequacy of Subsurface Layers for Rubblization (WisDOT 2019)

- **Traffic Control and Project Phasing Constraints.** The C&S process can be used if traffic needs to be temporarily returned to the pavement before the asphalt overlay is applied, as the production of tight cracks helps maintain the pavement's structure. In contrast, B&S and rubblization produce surface spalling and fractured particles, respectively, which require an asphalt overlay of sufficient thickness before opening to traffic.

- **Final Surface Elevation.** When a PCC pavement is overlaid with asphalt, the final surface elevation may be constrained by factors such as overpass clearance, side slope, and matching existing curb and gutter. One option is to determine a maximum allowable asphalt overlay thickness that meets all constraints and then evaluate the design overlay thicknesses for different fractured slab techniques to see which ones meet the final surface elevation constraint. A second option may be to use partial-depth milling of the concrete pavement before or after fracturing to lower the final surface elevation. The thickness design of the asphalt overlay can be adjusted to account for the final thickness of the fractured PCC layer after partial-depth milling. This approach effectively maintains minimum clearance heights under overpasses, matches adjacent curbs and gutters, and controls final elevations and slopes on airfield pavements. In cases where overpass clearances are the only elevation constraint, another option may be to entirely remove the existing concrete pavement, and base and subgrade as necessary, for a few hundred feet before and after the bridge, and design a new asphalt pavement structure for that area to provide the necessary clearance.

- **Local Experience.** Finally, it is important to consider the experience a local agency, engineer, or contractor may have with a particular fracturing technique that has worked well in the past for that region. Factors such as existing PCC pavement conditions, subsurface conditions, weather conditions, traffic constraints, and typical asphalt pavement production and placement practices can influence the selection of a fracturing technique for local conditions.

4.2 Asphalt Overlay Thickness Design

After the deteriorated PCC pavement has been fractured and seated properly to prevent or minimize reflection cracking, it needs an asphalt overlay to address structural and functional deficiencies. Several methods can be used to determine the appropriate thickness for the asphalt overlay. As summarized

Table 4.1 Asphalt Overlay Thickness Design Approaches Adopted by State Highway Agencies

Design Approach	Adoption by States	Sources
AASHTO 1972	GA, SC, WI	WisDOT (2019)
AASHTO 1993	AL, AR, CO, FL, IA, KS, LA, MA, MD, MI, MS, ND, NM, NY, OH, OK, OR, PA, VA, WA, WV	AASHTO (1993), Ksaibati et al. (1999), West et al. (2020)
IS-117	MN, NV	PCS (1994), Ksaibati et al. (1999), West et al. (2020)
Pavement ME Design	IN, MO, WY	West et al. (2020)
PerRoad Design	N/A	Decker (2006)
State-Specific ME Design	CA, IL, TX	Ullidtz et al. (2010), Hu et al. (2017), West et al. (2020)

in Table 4.1 (West et al. 2020), most state highway agencies rely on either the AASHTO 1993 Pavement Design Guide or the Mechanistic-Empirical Pavement Design approach to determine the thickness of the asphalt overlay on fractured PCC pavement.

4.3 1993 AASHTO Pavement Design Method

The AASHTO overlay design method utilizes the structural number (SN) concept to determine the overlay thickness required. The SN represents the structural strength of the overall pavement, comprised of each layer's structural contribution. It is expressed as a combination of a layer coefficient and its corresponding thickness. Layer coefficients are material-specific and represent the relative ability of the material to function as a structural component of the pavement.

Figure 4.3 shows a schematic of the structural contribution of a pavement with multiple layers. The required overlay thickness is calculated as a function of the structural capacity required to meet future traffic demands and the structural capacity of the fractured PCC slabs, as given by Equation 3.1:

$$D_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{(SN_f - SN_{eff})}{a_{ol}} \quad (3.1)$$

Where

SN_{ol} = Required overlay structural number
a_{ol} = Structural coefficient for the asphalt concrete overlay
D_{ol} = Required overlay thickness, inches
SN_f = Structural number required to carry future traffic
SN_{eff} = Effective structural number of the existing pavement after fracturing

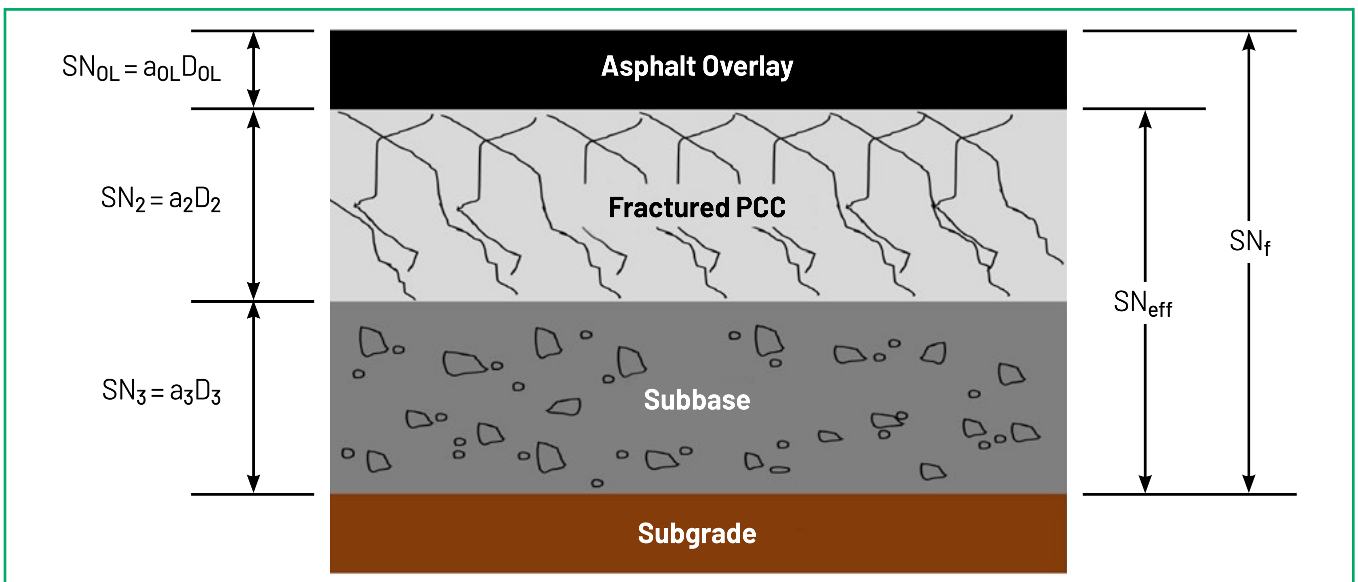


Figure 4.3 Structural contribution of pavement components (Courtesy of Guillermo Thenoux)

Table 4.2 Design parameters for AASHTO method (AASHTO 1993)

Parameter	Description
Estimated future traffic (W_{18})	Future 18-kip equivalent single axle loads (ESALs) in design lane over design period
Effective roadbed soil resilient modulus (M_R)	Adjusted for consistency with flexible pavement model and for seasonal variations. Typical design M_R ranges from 2,000 to 10,000 psi for fine-grained soils, 10,000 to 20,000 psi for coarse-grained soils.
Serviceability loss (ΔPSI)	Difference between the initial design serviceability index and the design terminal serviceability index. Typically 1.2 to 2.5.
Design reliability (R)	Overlay design reliability, 80 to 99 percent.
Standard deviation (S_0)	Overall standard deviation, typically assumed as 0.49.
Future structural capacity (SN_f)	Required structural number for future traffic determined from flexible pavement design equation or nomograph.

SN_f is computed as the required structural number for a new flexible pavement that would be constructed on the subgrade, based on design traffic repetitions, subgrade support, expected pavement terminal serviceability, and design reliability. Table 4.2 provides a description of the design parameters and their typical values.

SN_{eff} is estimated for the existing pavement based on the individual layer components:

$$SN_{eff} = a_2 D_2 m_2 + a_3 D_3 m_3 \qquad (3.2)$$

Where

D_2, D_3 = Thickness of fractured slab and base layers, inches
a_2, a_3 = Corresponding structural layer coefficients
m_2, m_3 = Drainage coefficients for fractured PCC and granular subbase

The 1993 AASHTO Design Guide recommends a default value of 1.0 for the drainage coefficient of the fractured PCC layer (m_2). Table 4.3 provides suggested layer coefficients for the fractured PCC.

The layer coefficient of the fractured PCC layer (a_2) is directly related to the effective slab modulus (E_{PCC}), as shown in Figure 4.4. The relationship between these two parameters is described in Equation 3.3 (Ullidtz 1987):

$$a_2 = (0.27 * \log(E_{PCC}/435ksi) + 0.35) \qquad (3.3)$$

The in-situ E_{PCC} may be obtained from non-destructive deflection testing (i.e., Falling Weight Deflectometer) coupled with layered backcalculation procedures. However, it should be noted that testing of rubblized concrete should be performed after the first asphalt concrete layer is placed.

Table 4.3 Suggested Layer Coefficients for Fractured Slab Pavements (AASHTO 1993)

Material	Slab Condition	Coefficient
Break/Seat JRCP	Pieces greater than one foot with ruptured reinforcement or steel/concrete bond broken	0.20 to 0.35
Crack/Seat JPCP	Pieces one to three feet	0.20 to 0.35
Rubblized PCC (any pavement type)	Completely fractured slab with pieces less than one foot	0.14 to 0.30
Base/subbase granular and stabilized	No evidence of degradation or intrusion of fines Some evidence of degradation or intrusion of fines	0.10 to 0.14 0.00 to 0.10

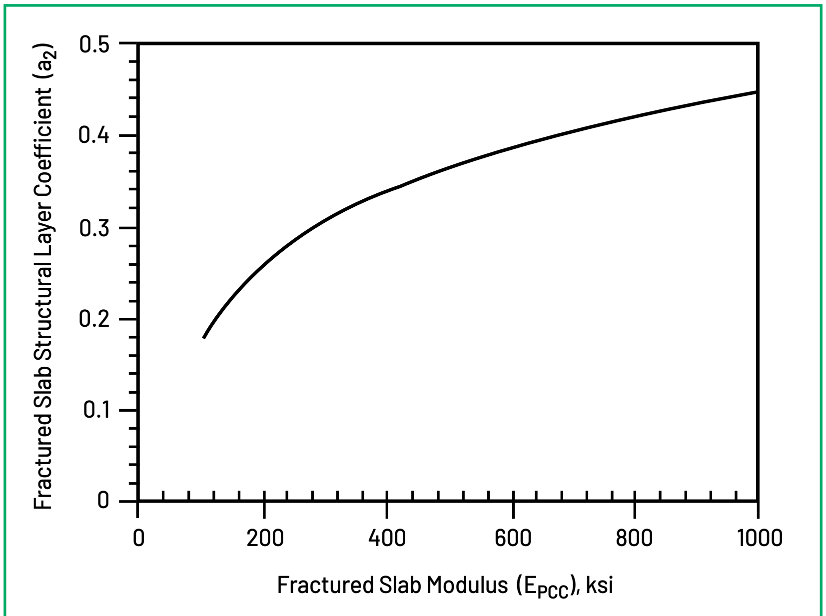


Figure 4.4 Relationship between AASHTO structural layer coefficient and fractured slab modulus (AASHTO 1993)

It is important to note that the layer coefficient for the fractured PCC layer represents its overall structural contribution, which may be influenced by not only the modulus of the layer but also other properties, such as the load transfer capability of the pieces. Additionally, fractured PCC layers often exhibit high modulus variability within a project. To address this, Table 4.2 shows a recommended increased overall standard deviation when designing for a given reliability level.

4.4 AASHTOWare ME Pavement Design Method

The Mechanistic-Empirical (ME) Pavement Design method utilizes mechanistic models to compute pavement responses (i.e., stresses, strains, and deflections) at critical locations in the pavement structure. Based on the calculated pavement responses, pavement performance metrics, such as International Roughness Index (IRI), rutting, and cracking for asphalt pavement, are estimated using the "empirical" transfer functions (i.e., distress models).

This method uses a hierarchical-level input scheme that gives the designer flexibility when determining the input values for most of the material and traffic parameters based on the criticality of the project and the available resources. Level 1 is the highest input level and requires site or project-specific parameters measured directly in situ or in the lab. Level 2 uses parameters obtained from other site-specific data, which may represent regional values. Finally, Level 3 inputs are based on default parameters estimated from national values.

The ME Pavement Design method is an iterative process. The designer first considers site conditions to propose a trial design for the new pavement or rehabilitation strategy. The trial design is then evaluated for the provided input, performance criteria, and reliability values by predicting distresses and smoothness. If the trial design does not satisfy the required criteria, it is revised, and the evaluation process is repeated as necessary.

For asphalt overlays over fractured PCC, the design is similar to the design of a new flexible pavement structure. The primary consideration is the estimation of an appropriate elastic modulus for the fractured PCC layer. The ME Pavement Design Guide (AASHTO 2020) provides guidance for estimating this parameter at different hierarchical input levels. One common method to estimate this value is by performing non-destructive FWD testing and backcalculating the elastic moduli from the deflection basins measured. This is mostly done on similar projects to estimate typical values (Level 2). However, most agencies lack the testing facilities to characterize materials. When testing capabilities are limited or unavailable, agencies may follow the recommended ranges presented in Table 4.4 for input Level 3. The modulus of the fractured PCC layer is a function of the nominal fragment size or crack spacing obtained during the process. Smaller fractured slab pieces result in lower E_{PCC} values, with rubblized layers having an effective moduli similar or higher than that of a high-quality crushed stone base. In addition, default values for other PCC material properties are shown in Table 4.5. Designers may also use a combination of PCC material input Levels 1, 2, and 3 based on their unique needs and testing capabilities.

Table 4.4 Recommended Fractured Slab Design Modulus Values for Input Level 3 (AASHTO 2020)

Fractured PCC Layer Type	Typical Modulus Ranges, psi
Crack and Seat or Break and Seat	150,000 – 1,000,000
Rubblization	50,000 – 150,000

Table 4.5 Material Properties Default Values for Input Level 3 (AASHTO 2020)

Material Property	Default Value
Poisson's Ratio	Crack and Seat or Break and Seat: 0.20 Rubblized: 0.30
Unit weight	150 pcf
Thermal conductivity	1.25 BTU/hr-ft-deg F
Heat capacity	0.28 BTU/lb-deg F

4.5 Perpetual Pavement Design Method

Another method for designing an asphalt overlay over a fractured PCC pavement is the mechanistic-based Perpetual Pavement approach, which aims to create long-lasting flexible pavement structures (Newcomb et al., 2010). The design process can be carried out using the PerRoad software, available as a standalone program on driveasphalt.org or as a design module on PAVEXpress.com.

Following the Perpetual Pavement design approach, an asphalt overlay over a fractured PCC pavement can be designed to last for more than 50 years with only periodic mill and inlay of the top layer to maintain the riding surface. Using PerRoad software, designers can choose layer thicknesses and materials to achieve a perpetual pavement design that meets the thresholds for the tensile strain at the bottom of the asphalt overlay, which prevents bottom-up fatigue cracking, and for the vertical compressive strain deeper in the structure, which prevents rutting of unbound layers. Reflection cracking is not considered in this design, as fracturing techniques are used to eliminate this type of distress.

The following sections provide an overview of the required inputs for PerRoad. Training videos on how to use the software for designing a perpetual pavement can be found on PAVEXpress.com.

4.5.1 Traffic

The traffic inputs for designing an asphalt overlay include traffic volume, growth rate, and load spectra. Load spectra classify the traffic loads based on axle types and weights. The required traffic inputs are similar to those used in the AASHTOWare Pavement ME Design.

4.5.2 Structure

Designers using PerRoad software can select a pavement cross-section of up to five layers, including the subgrade, for the structural inputs. For each layer, designers must provide material properties, thickness, and variability. Material properties include modulus and Poisson's ratio, and the software provides typical values for these properties for various materials. However, designers can also provide user-specified inputs if available. To account for temperature variations throughout the year, material properties for each layer can be adjusted for up to five seasons. The software also allows for the simulation of uncertainties related to material and construction inconsistencies by selecting the distribution type and coefficient of variation for the modulus and thickness of each layer.

PerRoad software uses performance criteria to evaluate pavement responses at critical locations. Designers can specify performance criteria for each layer, but for flexible pavement designs, horizontal tensile strain at the bottom of the asphalt and vertical compressive strain at the top of the subgrade are typically used. For perpetual pavement designs, the performance criteria at these critical locations are limiting strains. A limiting strain is a value or distribution that, if not exceeded, is assumed to result in no damage to the pavement structure. The following limiting strain criteria are recommended for use in PerRoad (Newcomb et al. 2010, Tran et al. 2015).

- A limiting tensile strain of 70 x 10-6 at the 50th percentile (Newcomb et al. 2010) or a limiting strain distribution (Table 4.6) at the bottom of the asphalt layer has been successfully used in perpetual pavement designs to avoid bottom-up fatigue cracking. Designers can adjust the limiting strain distribution for a specific asphalt mixture used in a design by entering the mixture's endurance limit.

Table 4.6 Limiting Strain Distribution for Horizontal Tensile Strain at the Bottom of Asphalt Overlay
(Tran et al. 2015)

Percentile	Microstrain
95th	257
85th	194
75th	158
65th	131
55th	110

- Also, a limiting compressive strain of 200 x 10-6 at the 50th percentile (Newcomb et al. 2010) at the top of the subgrade is recommended to prevent structural rutting.

4.5.3 Analysis

Once the traffic load and pavement structure data have been entered, the PerRoad software can be used to conduct a probabilistic analysis to evaluate the pavement structure. The software randomly selects values within the moduli and thickness variability inputs to develop a range of outputs, which presents a risk assessment of the probability of the pavement's critical responses exceeding the designer's set threshold. An output table, as shown in Figure 4.5, indicates whether the pavement cross-section analyzed is a perpetual pavement design by passing the limiting strain distribution (shown in Table 4.6) for the horizontal strain at the bottom of the asphalt overlay and the limiting strain for the vertical compressive strain at the top of the subgrade. If one or more criteria fail, designers can increase the asphalt overlay thickness or change the asphalt mixture selected until all criteria pass. Designers can also adjust the asphalt overlay thickness to optimize the design, although a minimum thickness of 5 inches is recommended for asphalt overlays over fractured PCC pavements (Decker 2006).

Perpetual Pavement Design Results: Percentile Responses

Layer	Location	Criteria	Units	Target Value	Target Percentile	Actual Percentile	Pass/Fail?
1	Bottom	Tensile Strain	micr...	-257.	95	99.8	Pass
				-194.	85	95.6	Pass
				-158.	75	86.6	Pass
				-131.	65	77.8	Pass
				-110.	55	68.2	Pass
3	Top	Vertical Strain	micr...	200.	50.	51.2	Pass

Figure 4.5 Output Table with New Design Criteria

MIX DESIGN FOR ASPHALT OVERLAYS

Several special asphalt mixture technologies can be used to mitigate reflection cracking, either placed directly over an existing PCC pavement or in combination with slab fracturing techniques. Some of these products have been available for decades, while others are relatively new to the market but have shown great promise. The following examples have been successfully used for rehabilitating PCC pavements with asphalt overlays.

5.1 Stone Matrix Asphalt (SMA)

Stone matrix asphalt (SMA) mixtures are gap-graded mixtures designed to have a high coarse aggregate content and a rich mortar of mineral filler and binder. The coarse aggregate skeleton provides stone-on-stone contact to carry the traffic loads and minimize rutting, while the rich mortar enhances durability and cracking resistance. Figure 5.1 shows a comparison of the aggregate structures of SMA and conventional dense-graded mixtures.

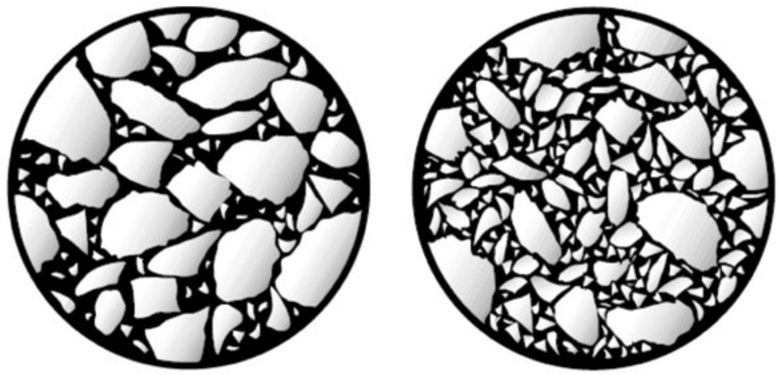


Figure 5.1 Aggregate Structure Comparison of SMA vs. Conventional Dense-Graded Asphalt Concrete (NAPA 2002)

Compared to conventional dense-graded mixtures, SMA mixtures require higher quality materials, such as more cubical, tougher aggregates capable of resisting rutting and breakdown during construction, binders with one or two PG grades higher than recommended for the geographical area (typically polymer modified), and additives to prevent draindown. SMA mixtures are generally more expensive, but provide better field performance than conventional dense-graded mixtures and can achieve life extensions ranging from 1 to 13 years (Yin and West 2018).

The design procedure of SMA mixtures is outlined in AASHTO R 46 *Standard Practice for Designing Stone Matrix Asphalt (SMA)* and is based on the volumetric properties of the SMA in terms of air voids, voids in the mineral aggregate (VMA), and the presence of stone-on-stone contact. The nominal maximum aggregate size (NMAS) for SMA mixtures may be 9.5, 12.5, or 19.0 mm, depending on their intended use and layer thickness. AASHTO M 325 *Standard Specification Stone Matrix Asphalt (SMA)* specifies the recommended gradation bands and volumetric criteria. The design air void content is usually 4 percent, although some agencies target slightly lower values. Similarly, the minimum VMA requirement is typically 17 percent, but may range among agencies from 16 to 18.5 percent. Stone-on-stone contact in SMA is verified by determining the voids in the coarse aggregate (VCA) and ensuring that the VCA of the SMA mixture (VCA_{MIX}) is less than the VCA of the coarse aggregate in the dry-rodded unit weight test (VCA_{DRC}).

During the design, the moisture susceptibility and draindown of the selected mixture are also evaluated. Draindown sensitivity is especially important in SMA due to the lack of intermediate-size particles and higher asphalt content compared to conventional dense-graded mixes. This could cause the binder to separate and drain out of the mixture while at elevated temperatures (i.e., during production, storage, transport, and placement). To prevent this, additives such as cellulose or mineral fibers are incorporated into the mix at a rate of 0.3 to 0.4 percent based on the total weight of the mix.

SMA mixtures have been found to significantly reduce the propagation rate of reflection cracking thanks to the combination of the gap-graded aggregate structure, higher binder content, and polymer modification. The Wisconsin Department of Transportation was one of the first state agencies to implement the use of SMA in the United States, and has been able to achieve durable and long-lasting pavements when incorporating this material. In the early 1990s, a pilot project was conducted on a section of I-43, where the existing PCC was overlaid with SMA in the mainline while the shoulders were overlaid with a conventional dense-graded mix. The Georgia Department of Transportation has also been a pioneer in the use of SMA. Some of the projects include placing SMA as an intermediate layer combined with an open-graded friction course surface layer to overlay PCC pavements in high-trafficked interstate routes. Results from these experiences have shown improved performance with up to 40 percent reduction in reflection cracking (Watson 2003).

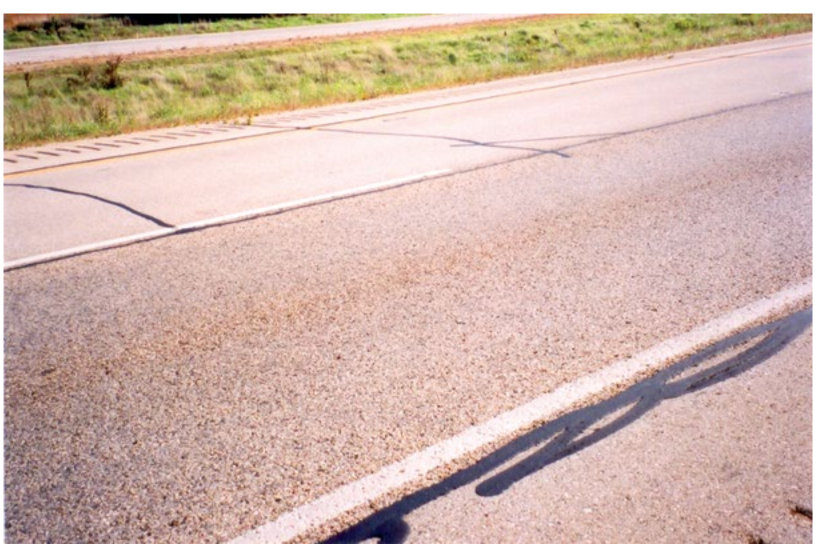


Figure 5.2 Performance of SMA Overlay (near lane) on Concrete Pavement in Wisconsin after 8 Years (Watson and Musselman 2022)

5.2 Asphalt Rubber Gap-Graded (ARGG) Mixtures

Asphalt rubber is a blend of asphalt binder, reclaimed tire rubber, and additives. The rubber component (at least 15 percent by weight of the total blend) reacts in the asphalt binder sufficiently to cause swelling of the rubber particles, adding more flexibility and enhanced crack resistance to the resulting blend (Figure 5.3). Asphalt rubber is often used in gap-graded mixtures.

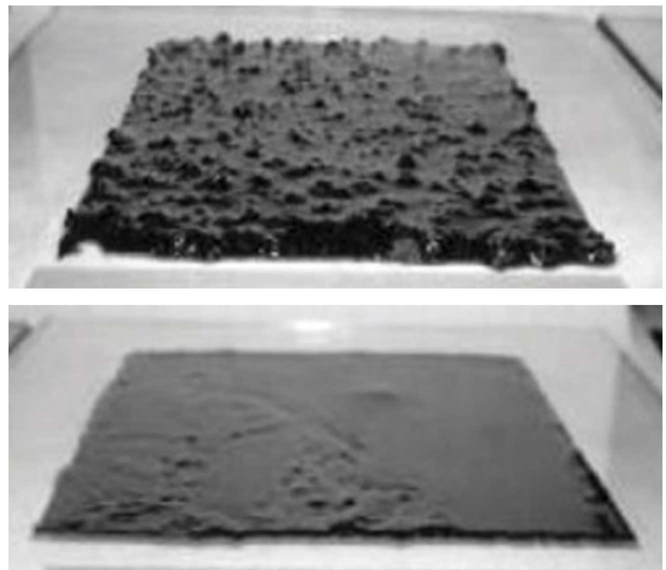


Figure 5.3 Asphalt rubber binder (top) vs. neat or polymer-modified binder (bottom) (Han et al. 2016)

These mixtures provide stone-on-stone contact for excellent rutting resistance and space for the swelled rubber particles but have relatively low percentages of material passing the No. 200 sieve. The NMAS for these mixes is either 9.5 or 12.5 mm, and they are placed at thicknesses ranging from 1.25 to 2.25 inches, depending on the aggregate size used.

ARGG mixtures are commonly designed with the volumetric approach for an air void content ranging from 3 to 5 percent, depending on the agency. Typical VMA requirements range from 18 to 23 percent, and the minimum binder content specified among various agencies is at least 7.5 percent. Performance tests that may be conducted during the design phase include draindown, moisture susceptibility, and rutting; in some cases, permeability, raveling, durability, and cracking tests may be added. It is generally necessary to modify the laboratory sample preparation procedure to provide 5-30 minutes of additional time for specimens to cool in the compaction mold before they are extruded; otherwise, the specimens will expand and possibly fall apart while they are hot.

Production of asphalt rubber requires additional blending equipment, typically a portable unit, where the asphalt binder is pumped from a storage unit into a heating tank. Once the binder reaches the desired temperature (350° to 425°F), it is pumped into a mixing tank to be blended with the rubber and additive components. Coarser-sized rubber particles (#10 to #14 mesh) are most commonly used for asphalt rubber production, but smaller-sized rubber products (#30 mesh to #80 mesh) can be used. After mixing, a "reaction time" of 45 to 60 minutes is necessary to allow the rubber particles to swell and absorb light fractions of the asphalt binder. The reacted asphalt rubber must be continuously mixed to keep the rubber from settling and maintained at an elevated temperature so that it can be pumped to the asphalt plant's binder plumbing line to be introduced into the mixing zone of the plant.

The mobilization costs of the asphalt rubber production equipment, along with other factors like high binder content and the requirement for high-quality aggregates, contribute to making the resulting gap-graded mix 25 to 75 percent more expensive than typical dense-graded asphalt concrete (Antunes et al. 2006). Although ARGG mixes have a higher initial cost, their improved performance makes them cost-effective on a life-cycle basis. Their use has been widely adopted by California and Arizona DOTs as a strategy to retard reflection cracking in flexible and rigid pavements.

The Arizona Department of Transportation (ADOT) has been using asphalt rubber mixes to rehabilitate concrete pavements since the early 1990s. An experimental section built under the Strategic Highway Research Program (SHRP) involved performing crack and seat of the existing jointed concrete pavement, followed by asphalt overlays with conventional dense-graded, asphalt rubber gap-graded, and asphalt rubber open-graded layers. The heavily trafficked section of I-40 in Flagstaff was in poor condition prior to rehabilitation but the asphalt rubber gap-graded mix exceeded performance expectations, providing benefits in terms of construction costs and time compared to the full reconstruction alternative and leading ADOT to adopt widespread use of asphalt rubber mixes throughout Arizona (Way 2000a).

Figure 5.4 I-40 SHRP Test section nine years after construction: conventional asphalt concrete (left) and asphalt rubber concrete (right) (Way 2000b)

5.3 Highly Modified Asphalt (HiMA)

Highly modified asphalt (HiMA) mixtures are produced with asphalt binders with a high polymer content. While conventional polymer-modified binders contain approximately 3 percent polymer by weight of binder, HiMA binders contain more than double this amount, typically 7 to 8 percent.

The increased polymer content causes the binder-polymer structure to change from an asphalt binder with a dispersed swollen-polymer phase to a swollen polymer with a dispersed-asphalt phase (Figure 5.5). This phase reversal enhances the elastic properties of the binder, improving cracking and rutting performance.

Agencies that specify the use of HiMA binders rely on performance grading (PG) and rheological properties to define and accept the highly polymer-modified binders. Agencies that follow AASHTO M 320 *Standard Specification for Performance-Graded Asphalt Binder* specify a minimum percentage for elastic recovery (ER), typically 90 percent. Conversely, agencies that use AASHTO M 332 *Standard Specification for Performance-Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR)* Test require maximum limits for non-recoverable creep compliance ($J_{nr,\,3.2}$) or minimum limits for percent recovery ($R_{3.2}$), or both, in addition to PG grades with an "E" designation (extremely heavy traffic) at the expected pavement temperature for the regional climate (Vargas-Nordcbeck and Musselman 2022).

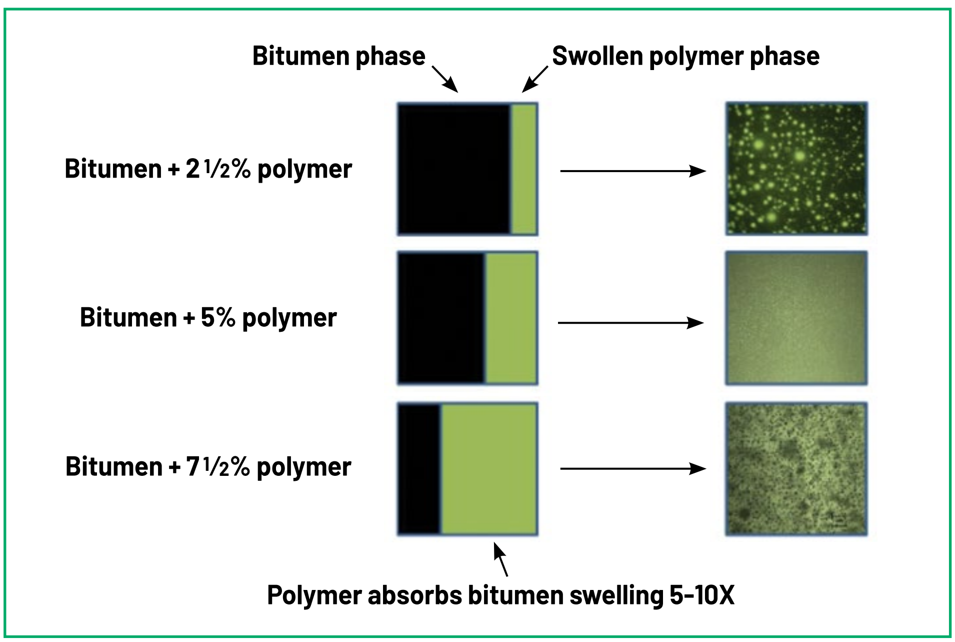


Figure 5.5 Dispersion of polymer in asphalt binder at different proportions (Timm et al. 2012)

In general, the design, production, and placement of HiMA mixtures do not differ significantly from those of conventional polymer-modified mixtures. The main issues when incorporating HiMA binders into asphalt mixtures relate to storage period and temperature. Subjecting the modified binders to high temperatures (above 320°F) will cause the polymer to continue cross-linking, making the resulting blend less workable. In addition, HiMA binders have limited storage time, usually three days to a week, depending on the source. Beyond that time, the material will start to break down, limiting its ability to be pumped out and diminishing its elastic properties.

For pavement design, HiMA mixtures are assumed to have at least the same structural capacity as conventional polymer-modified mixtures, although there is evidence of HiMA pavements being able to achieve the desired performance with a reduced thickness (Willis et al. 2016). A study conducted for the Florida Department of Transportation (FDOT) found an increase of roughly 20 percent in structural capacity for HiMA mixtures compared to conventional polymer-modified mixes (Habbouche et al. 2019).

Mixtures that incorporate HiMA binders are more expensive but the improvement in performance and overall expected pavement life are enough to offset this additional cost. Several state agencies have adopted the use of HiMA mixtures as a strategy to reduce cracking, including reflection cracking in composite pavements. The Virginia Department of Transportation (VDOT) has been using HiMA overlays over existing jointed concrete pavements since 2015 to mitigate reflection cracking, resulting in up to 34 percent extension of performance life compared to conventional polymer-modified mixes (Habbouche et al. 2021). Similarly, the Departments of Transportation of Florida and Oklahoma, among others, use HiMA overlays in heavily trafficked roadways to slow down the progression of reflection cracking and other distresses.

Figure 5.6 HiMA mixture on Oklahoma's I-40 (Vargas-Nordcbeck and Musselman 2021)

5.4 Crack Attenuating Mix (CAM)

Crack Attenuating Mix (CAM), developed by the Texas Department of Transportation (TxDOT), is a fine-graded mixture with a high binder content designed to reduce reflection cracking while also providing high rut resistance. It is typically placed as an interlayer between the existing pavement and a surface layer of asphalt concrete at thicknesses of 0.5 to 1.0 inches, although it has also been used as a thin surface course.

To satisfy the enhanced rutting and cracking performance properties of these mixes, all aggregates used in the CAM should be crushed, high-quality materials. A fine gradation, often referred to as a screenings mix, requires a high binder content (minimum 7 percent) and the use of polymer modifiers to improve cracking resistance and durability. Recycled materials like reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) are not permitted in these mixes. The use of premium materials makes CAM an expensive mix; however, the cost per square yard is less than other mixtures since CAM is applied in thin layers.

The mix design procedure uses the traditional volumetric approach but has high VMA and low air voids requirements. The design density is 98 percent, which minimizes water and air intrusion, consequently minimizing the potential for moisture damage and binder oxidative aging. The high VMA requirement (minimum 17 percent) promotes stone-on-stone contact and reduces permeability. The performance of the mixture is evaluated through the Hamburg Wheel Tracking Device and Overlay Tester.

It is necessary to select an appropriate surface mix in order for the CAM interlayer to perform well. When the surface mix is much stiffer than the interlayer, cracks tend to "skip" the CAM interlayer and reflect through the surface layer. To address this issue, TxDOT commonly

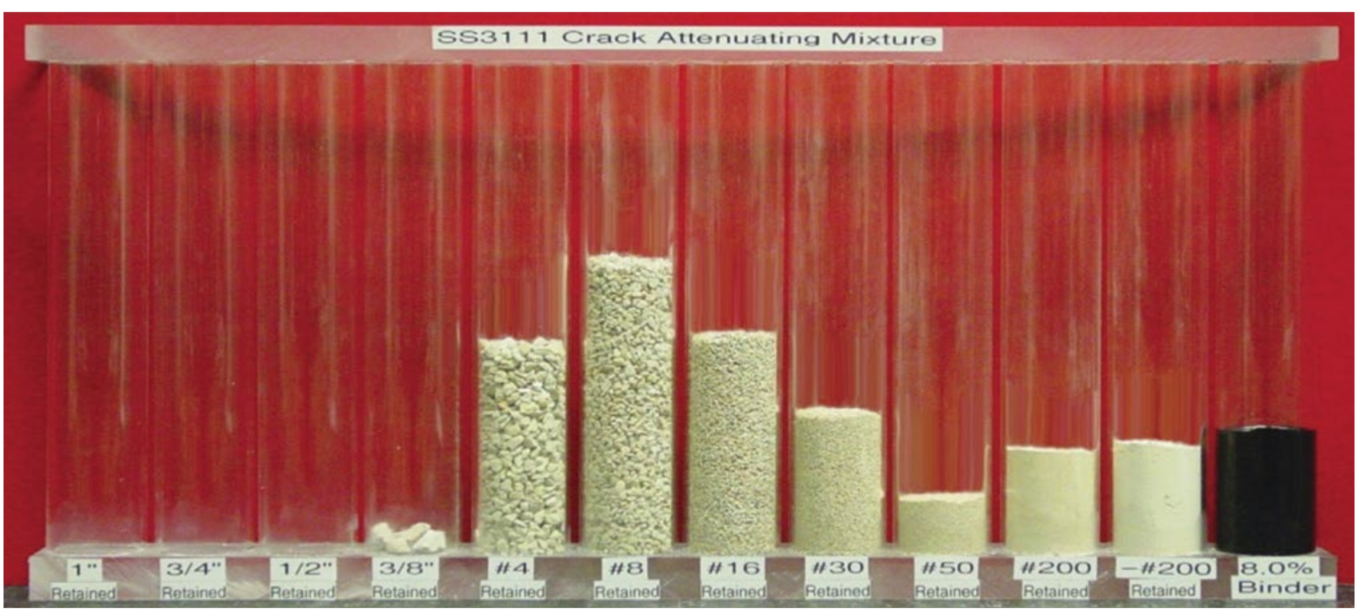


Figure 5.7 Typical composition of crack attenuating mixtures (Source: TxDOT)

Table 5.1 Mix Design Properties for CAM and TOM-C Mixes (TxDOT)

Sieve Size	CAM	TOM-C
1/2″	–	100
3/8″	98 – 100	95 – 100
#4	70 – 90	40 – 60
#8	40 – 65	17 – 27
#16	20 – 45	5 – 27
#30	10 – 30	5 – 27
#50	10 – 20	5 – 27
#200	2 – 10	5 – 9
Asphalt Binder Content, % Min	7.0	6.0
Design VMA, % Min	17.0	16.0
Design Gyrations (N_{design})	50	50
Target Laboratory-Molded Density, %	98.0	97.5
Tensile Strength (dry), psi	85 – 200	85 – 200
Dust/Asphalt Ratio	1.4 Max	NA
Hamburg Wheel Test, minimum # of passes @ 0.5″ rut depth tested @ 122°F	PG 64 or lower: 10,000 PG 70: 15,000 PG 76 or higher: 20,000	PG 70: 15,000 PG 76: 20,000
Overlay Tester, minimum # of cycles	750	300
Draindown, % Max	NA	0.20

uses a thin overlay mix (TOM-C) as the surface layer over the CAM in the overlay design. TOM-C is a coarser mixture with some crack attenuating properties; it has a more open gradation and a lower minimum asphalt content than CAM but provides good skid and rut resistance. TOM-C mixes are also tested with the Overlay Tester but have a lower requirement than CAMs.

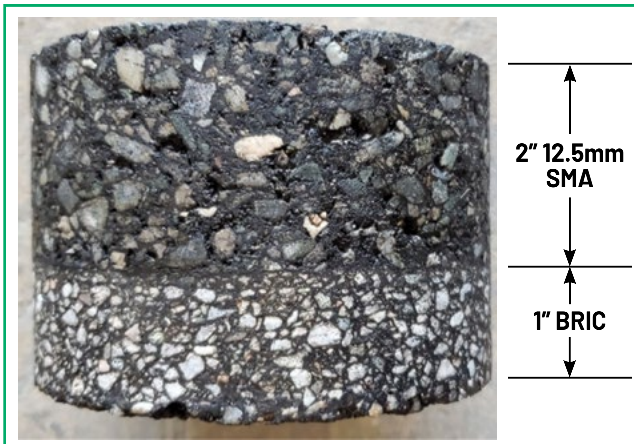


Figure 5.8 BRIC-SMA overlay system for reflection cracking mitigation (Photo courtesy of T. Bennert)

The specified mix design properties for CAM and TOM-C mixes are shown in Table 5.1

The CAM overlay system used by TxDOT has found success as a strategy to mitigate reflection cracking when rehabilitating deteriorated jointed plain concrete pavements (JPCP) and continuously reinforced concrete pavements (CRCP). The Houston District reports that CAM overlay systems are expected to more than double the service life of the conventional 2-inch overlays previously used (Gilliand et al. 2022).

5.5 Binder Rich Intermediate Course (BRIC)

The Binder Rich Intermediate Course (BRIC) is another interlayer product developed by the New Jersey Department of Transportation (NJDOT). Similar to TxDOT's CAM, this mixture is designed to minimize reflection cracking and is mainly placed over existing PCC and at the bottom of an asphalt concrete overlay. The overlay is typically an SMA mix (Figure 5.8), which is still flexible enough to help ensure that the surface

Table 5.2 Mix Design Properties for BRIC Mixes (NJDOT)

Sieve Size	BRIC
3/8"	100
#4	90 – 100
#8	55 – 90
#30	20 – 55
#200	4 – 10
Asphalt Binder Content, % Min	7.4
Design VMA, % Min	18.0
Design Gyrations (N_{design})	50
Target Laboratory-Molded Density, %	97.5
Dust/Asphalt Ratio	0.6 – 1.2
Asphalt Pavement Analyzer maximum rut depth @ 8,000 loading cycles, mm	6
Overlay Tester, minimum # of cycles	700
Draindown, % Max	0.1

layer can withstand the residual vertical and horizontal movement. The specified maximum thickness for the BRIC interlayer is 1.5 inches.

The BRIC is a 4.75 NMAS mixture with high binder content (minimum 7 percent). The asphalt binder used must be polymer modified and specially formulated for meeting the mix performance criteria specified by NJDOT (Table 5.2). BRIC mix designs are evaluated for rutting resistance using the Asphalt Pavement Analyzer (APA) and for cracking resistance using the Overlay Tester.

An analysis of the NJDOT pavement management database showed that BRIC improves the projected life of the pavement, but the benefit obtained is largely influenced by the surface course material used. The BRIC-SMA overlay system has been the most successful, providing 10 more years of service life than dense-graded asphalt mixtures (Bennert 2017).

5.6 Balanced Mix Design (BMD)

As new products and technologies are introduced to the market, the balanced mix design (BMD) approach offers a methodology for designing asphalt concrete mixtures as a function of performance. With BMD, mixtures are designed using appropriately selected performance tests that address multiple modes of distress. AASHTO PP 105 *Standard Practice for Balanced Mix Design of Asphalt Mixtures* describes four alternate approaches that allow agencies to select how much of the existing volumetric criteria are retained and the potential to utilize innovative and sustainable materials (Yin and West 2021).

Approach A consists of a volumetric properties based mix design with performance verification. It starts with a volumetric design to determine the optimum binder content and then uses selected performance tests to assess the mixture's resistance to cracking, rutting, and moisture damage. The approach requires full compliance with the existing volumetric requirements and additional performance test requirements, making it the most conservative and with the lowest innovation potential.

Approach B, volumetric design with performance optimization, also starts with a volumetric design to determine the optimum binder content followed by performance tests, but it is slightly more flexible as it allows moderate changes in the binder content to optimize mixture performance.

Approach C is referred to as a performance-modified volumetric mix design. In this approach, the volumetric design is used to guide initial component material properties, proportions, and binder content. However, it is a less conservative approach since it allows some of the volumetric requirements to be relaxed or eliminated as long as the performance requirements are met.

Finally, Approach D is purely performance-driven. The volumetric requirements are eliminated and the design relies solely on mixture performance test results for mix design optimization. It is the least conservative and has the highest potential for innovation.

There are various laboratory performance tests available to select and evaluate materials for balanced mix design of asphalt mixtures. AASHTO MP 46 *Standard Specification for Balanced Mix Design* lists the different tests that may be selected (Table 5.3) and specifies minimum performance testing requirements for each. Before implementing a given test method, it is recommended that agencies conduct an objective review of the method being considered, including establishing relationships between the results and field performance (Yin and West 2021). Other factors that might influence an agency's selection include equipment cost, specimen fabrication, and testing time.

5.7 Geosynthetics

Geosynthetics used in roadway construction are products made from polymeric materials and are generally classified as geotextiles and geodrids. Geotextiles are permeable geosynthetics made of textile materials and can be manufactured using traditional weaving methods (woven geotextiles)

Table 5.3 Summary of Mixture Performance Tests Available for BMD (from AASHTO MP 46)

Name	Test Method
Rutting Tests	
Asphalt Pavement Analyzer	AASHTO T 340
Flow Number Test	AASHTO T 378
Hamburg Wheel-Tracking Test	AASHTO T 324
Hveem Stability Test	AASHTO T 246
Superpave Sheer Tester	AASHTO T 320
Incremental Repeated-Load Permanent Deformation (iRLPD)	AASHTO TP 116
Stress Sweep Rutting (SSR) Test Using the AMPT	AASHTO TP 134
Cracking Tests	
BBR Mixture Bending Test	AASHTO TP 125
Direct Tension Cyclic Fatigue Test	AASHTO T 400
Disc-Shaped Compact Tension Test	ATM D7313
Flexural Bending Beam Fatigue Test	AASHTO T 321
Illinois Flexibility Index Test	AASHTO T 393
IDEAL Cracking Test	ASTM D8225
Indirect Tensile Creep Compliance and Strength Test	AASHTO T 322
Energy Ratio Test	NA
Overlay Test	Tex-248-F and NJDOT B-10
Semi-Circular Bend Test at Intermediate Temperature	ASTM D8044
Abrasion Loss of Asphalt Mixture Specimens	AASHTO T 401
Small Specimen Geometry Cyclic Fatigue Test	AASHTO TP 133
N_{flex} Factor Test	AASHTO TP 141
Moisture Damage Tests	
Hamburg Wheel-Tracking Test	AASHTO T 324
Tensile Strength Ratio (TSR)	AASHTO T 283
Moisture Induced Stress Tester	ASTM D7870/D7870M

or by placing and orienting the filaments onto a conveyor belt and subsequently bonding them by needle punching or melt bonding (non-woven geotextiles). Geogrids are flexible mesh-like products with uniformly distributed apertures and can be uniaxial, biaxial, or triaxial, to distribute stresses in one or more directions.

These products are used to provide one or more major functions: separation, filtration, reinforcement, stiffening, drainage, protection, and acting as a hydraulic barrier. For the purpose of reflection cracking mitigation, geosynthetics act through one or a combination of several of these functions at the interface between the existing pavement and the new overlay.

Reinforcement is the primary function that acts against reflection cracking by developing tensile forces in the vicinity of the existing cracks and reducing strains in the asphalt concrete overlay (Figure 5.9). This function is mainly achieved by using geogrids, and only occurs if the geosynthetic has a higher modulus than the asphalt concrete and sufficient cross-sectional area to substantially strengthen the overlay (Khodaii et al. 2009).

The separation function can provide stress-relief by allowing the geosynthetic interlayer to absorb some of the horizontal movement in the old pavement, protecting the asphalt concrete overlay from stress-related cracking. This mechanism can be characterized as controlled debonding and often involves bitumen-impregnated non-woven geotextiles (Zornberg 2017).

Finally, the hydraulic barrier function reduces the permeability of the underlying pavement layers by one to three orders of magnitude (Nithin et al. 2015), reducing the impact of deterioration mechanisms that can be accelerated by water intrusion, including reflection cracking. This function is also often performed by bitumen-impregnated non-woven geotextiles (Zornberg 2017).

The effectiveness of geosynthetics as crack mitigating treatments depends on many factors, such as material properties, condition of the existing pavement, traffic and climate conditions, installation procedure, overlay thickness, etc.

5.8 Summary

Even after applying slab fracturing techniques, particularly C&S and B&S, conventional asphalt overlays placed over concrete pavements may still be susceptible to reflection cracking. Selecting appropriate materials and mixture designs for the overlay can help slow down the progression of reflection cracks and overall achieve longer service lives in a cost-effective manner. There are several options available to serve this purpose, some with a long history of successful application and others that have been developed more recently but show promising results. Agencies should consider these alternatives when designing rehabilitation projects to maximize the performance of the new pavement.

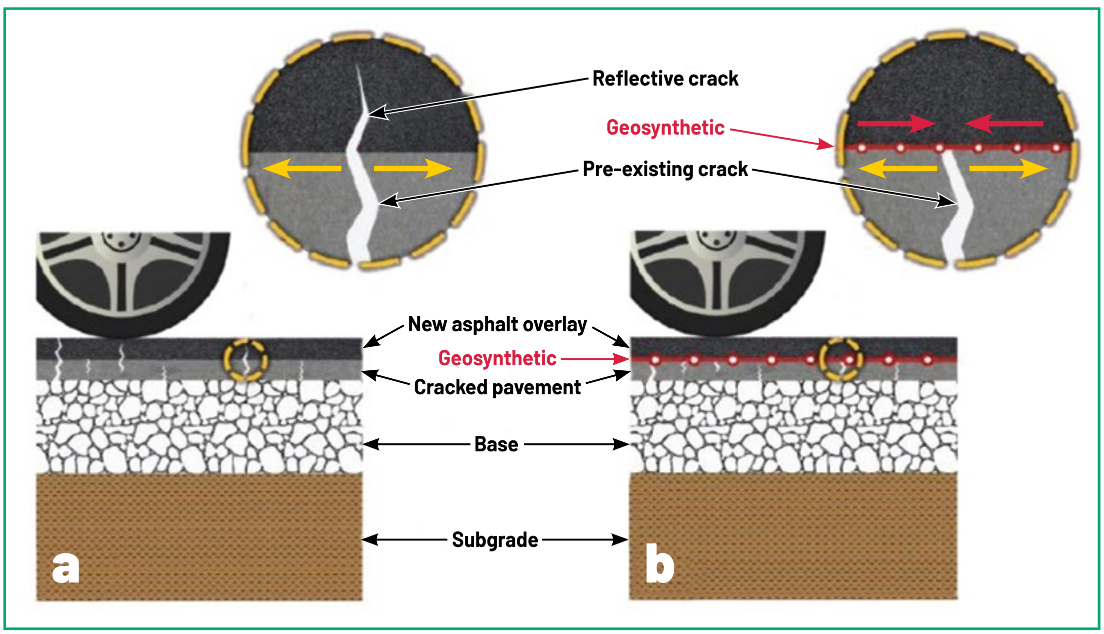


Figure 5.9 Use of geosynthetics in mitigation of reflection cracking in asphalt overlays a) with and b) without geosynthetic (Zornberg 2017)

CASE STUDIES

There are multiple examples of slab fracturing techniques being used to rehabilitate deteriorated PCC pavements. Projects have been successfully conducted across the country with different techniques, materials, and design method combinations. This chapter describes two case studies of jointed reinforced concrete pavements rehabilitated with slab fracturing techniques: a rubblized JRCP in New Jersey designed as a perpetual pavement, and a broken and seated JRCP in Pennsylvania designed with the 1993 AASHTO method and verified with the Pavement ME Design. While the two agencies used different approaches, in both instances the results led to excellent performance of the rehabilitated pavements with significant time and cost savings.

6.1 Case Study 1: Perpetual Asphalt Pavement over Rubblized JRCP on I-295 in New Jersey

In 2022, the New Jersey Department of Transportation (NJDOT) received the Perpetual Pavement Award (PPA): By Conversion. This award recognized NJDOT's 11.8-mile perpetual pavement design over the rubblized JRCP from milepost 45 to 56.8 on Interstate 295 (I-295) in Burlington County. The project, designed by the NJDOT Division of Highway and Traffic Design, Roadway Design Group 1, was constructed by Haines & Kibblehouse Inc. and Intercounty Paving Co. It was opened to traffic in 2010.

6.1.1 Deteriorated JRCP

Constructed between 1972 and 1974, the original six-lane JRCP section featured 9-inch slabs with doweled expansion joints. Despite two rehabilitative efforts in 1989 and 1997, the NJDOT Maintenance and Operation Unit still frequently repaired this pavement. By 2002, the JRCP had reached its terminal serviceability, exhibiting significant distresses (Figure 6.1). These included joint spalling, mid-slab spalling, alkali-silica reactions, dowel bar failures, and faulting, resulting in a notably high International Roughness Index (IRI) value.

NJDOT uses the Surface Distress Index (SDI), a composite index that incorporates surface distresses, such as cracking, rutting, faulting, and joint deterioration, to initiate pavement projects for resurfacing, rehabilitation, or reconstruction, with an SDI of 5 for new pavement and an SDI of 0 for completely failed, severely distressed pavement. Prior to rehabilitation, the SDI for the JRCP section fell as low as 1.60, with the trigger value for rehabilitation or reconstruction set at 2.40.

Figure 6.1 Surface Conditions of I-295 JRCP Section from MP 45 to 56.8 in Burlington County (Courtesy of NJDOT)

6.1.2 Pavement Evaluation and Rehabilitation Strategy

Following a 2007 pavement condition assessment involving visual surveys, Falling Weight Deflectometer (FWD), Ground Penetrating Radar (GPR), and coring, the NJDOT developed various rehabilitation strategies. A life-cycle cost analysis (LCCA) was conducted, identifying rubblization with asphalt overlays as the most cost-effective rehabilitation approach. This strategy optimized on-site concrete recycling, reducing both project cost and duration. It required 20 undercut locations to avoid altering 21 structures. Traffic disruptions were minimized with full road closures limited to 59 summer days. This tight schedule required a daily supply of 10,000 tons of asphalt mixture, sourced from a minimum of three plants.

6.1.3 Initial Pavement Design

The 1993 AASHTO Pavement Design initially recommended a 12-inch asphalt overlay over rubblized JRCP. However, this design would require the removal of a minimum of 2,400 linear feet of PCC and box outs at each structure, possibly longer in some locations due to the close proximity of some structures and constructability, potentially leading to construction challenges.

6.1.4 Perpetual Pavement Design

The NJDOT reevaluated the initial design and determined a potential to reduce the overlay thickness from 12 to 8 inches by adopting the perpetual pavement design approach, which could minimize construction time and cost associated with box outs and undercuts. This design approach aims to limit both tensile strain at the bottom of the asphalt layer and compressive strain at the top of the subgrade. These limits help prevent bottom-up fatigue cracking and structural rutting, thereby enabling surface-level interventions for pavement distresses.

The cross-section for the 8-inch asphalt overlay consisted of:

- A 2-inch SMA surface.
- A 3-inch 19M76 intermediate layer (19 mm NMAS, 75 gyrations, PG 76-22).
- A 3-inch bottom rich base course (BRBC).

The maximum tensile strain at the bottom of the asphalt overlay, determined using JULEA—the same

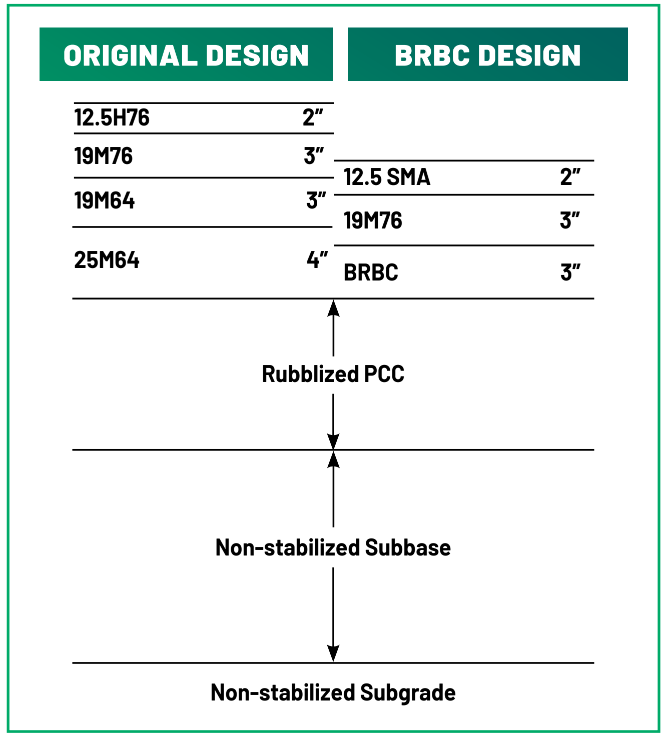


Figure 6.2 Comparison of 1993 AASHTO Pavement Design and Perpetual Pavement Design with BRBC Mixture
(Courtesy of NJDOT)

software used in the AASHTO Pavement ME Design—was 82 micro-strains. To prevent bottom-up fatigue cracking, the BRBC mixture for this project was specially designed to have an endurance limit exceeding the maximum tensile strain of 82 micro-strains. Figure 6.2 compares two pavement cross sections based on the 1993 AASHTO Pavement Design and the Perpetual Pavement Design with the BRBC mixture.

6.1.5 Mix Design and Performance Testing of BRBC Mixture

Laboratory testing to support the perpetual pavement design was conducted at Rutgers University. Based on the test results, a specification for the BRBC mixture was established. This specification included the following requirements:

- A 19-mm NMAS gradation.
- 3.5% design air voids at 50 gyrations.
- A minimum VMA of 13.5 percent
- A dust to P_{be} ratio between 0.6 and 1.2.
- A maximum draindown of 0.1 percent.

The mixture should have a minimum binder content of 5 percent, utilizing a PG 76-28 binder with a minimum elastic recovery of 60 percent per AASHTO T301.

The design gradation would have no RAP or natural sand. Additionally, the mixture is required to meet specific performance requirements, including:

- A maximum APA rut depth of 5 mm at 8,000 cycles and 64°C per AASHTO T340.
- A minimum endurance limit of 100 micro-strains at 100 million cycles.

The endurance limit was determined following the methodology outlined in NCHRP Report 646 (Prowell et al. 2010). The determination was based on the Bending Beam Fatigue (BBF) test results conducted at 400 and 800 micro-strains and 15°C under sinusoidal loading per AASHTO T321.

6.1.6 Construction

Construction began in the Spring of 2010 with the installation of a geocomposite edge drain (Figure 6.3a). This was followed by the rubblization of the JRCP using resonant breakers (Figure 6.3b). The breaking pattern was approximately 8 inches wide, requiring 18 passes to break a 12-foot lane width. To achieve the project's production targets, multiple breakers were deployed. The rubblized JRCP was subsequently compacted using vibratory rollers (Figure 6.3c).

Before paving, the BRBC mixture was designed based on the volumetric mix design. Loose mix samples were provided to Rutgers University for APA and BBF testing. Upon obtaining satisfactory performance test results, a test strip was paved outside the project site, and plant loose mix samples were taken for APA and BBF testing. Once the off-site test strip met the requirements, the contractor continued with the paving production for the project (Figure 6.4).

During production, the acceptance criteria included lab air voids (ranging between 2.5 and 4.5 percent), VMA, Dust to P_{be} ratio, and percentage of draindown.

Figure 6.4 Construction of BRBC Layer over Rubblized JRCP (Courtesy of NJDOT)

The performance benchmarks for acceptance encompassed a maximum APA rut depth of 5 mm and a minimum of 30,000 cycles to failure when tested at 800 micro-strains. This BBF test condition was chosen to expedite the testing time. The underlying assumption was that samples that satisfied this BBF criterion would also meet the endurance limit specification.

6.1.7 Savings from Perpetual Pavement Design with BRBC Mixture

By opting for the perpetual pavement design with the BRBC mixture over rubblized JRCP, NJDOT achieved significant savings. This design reduced 170,000 tons of asphalt mixture and decreased the PCC removal and replacement by 3 miles, resulting in a total saving of at least $7 million. This design also enabled the completion of all rubblization and asphalt courses within one season, ahead of schedule, from May 2010 to October 2010.

6.1.8 Post-Construction Evaluation and Recognition

In 2018, an assessment of the rubblized pavement section was undertaken using FWD and GPR techniques. GPR results showed that the average

Figure 6.3 Rubblization of JRCP: a) Installation of geocomposite edge drain, b) Rubblization of JRCP, and c) Compaction of rubblized JRCP (Courtesy of NJDOT)

Figure 6.5 **I-295 Perpetual Pavement after 10 Years of Excellent Performance** (Courtesy of NJDOT)

as-built asphalt overlay thickness was 8.7 inches for the northbound direction and 10.0 inches for the southbound. Consequently, there were differing rates of IRI increase between the two directions: 2.1 in/mi per year northbound and 0.4 in/mi per year southbound. Nonetheless, both directions exhibited low deflections, with the rubblized concrete having a layer coefficient of 0.24. Further analysis indicated that the asphalt overlays over the rubblized JRCP in both directions functioned as perpetual pavements. These are expected to have a lifespan exceeding 50 years, which is significantly longer than the 30-year design life. The excellence in design and construction resulted in exceptional performance (Figure 6.5). This achievement was recognized with the prestigious PPA: By Conversion award in 2022.

6.2 Case Study 2: Asphalt Overlay over Broken and Seated JRCP on SR-28 in Pennsylvania

The Pennsylvania Department of Transportation (PennDOT) has successfully rehabilitated over 300 miles of deteriorated PCC pavements using a fracturing technique, followed by an asphalt overlay. A notable example of this rehabilitation strategy was the project completed in 2018 on SR-28, Section A55, in Allegheny County, a main expressway into Pittsburg, Pennsylvania. PennDOT's District 11 Design Division was responsible for the PCC pavement rehabilitation design, and Lindy Paving Inc. undertook the construction work.

This project spanned approximately seven miles, starting from the Creighton Interchange (Exit 13) in East Deer Township and extending to the Butler County line just north of Exit 16. As part of the project's scope, five miles of jointed reinforced concrete pavement were broken, seated, and subsequently overlaid with asphalt.

6.2.1 Deteriorated Jointed Reinforced Concrete Pavement

The original roadway featured a four-lane JRCP, with each lane being 12 feet wide and 10 inches thick on top of 12 inches of subbase. Additionally, the roadway was designed with 4-foot Jointed Plain Concrete Pavement (JPCP) inside shoulders and 10-foot JPCP outside shoulders. This pavement section was constructed in 1984 and underwent diamond grinding in 2004 and concrete pavement restoration in 2009. Despite these two repair efforts, the pavement section continued to deteriorate significantly.

On February 27, 2013, PennDOT District 11 conducted a scoping field review of the deteriorated pavement section with attendees from several PennDOT divisions and the Federal Highway Administration (FHWA). Following the field review, a separate pavement scoping meeting was held on March 20, 2013. During this meeting, the PennDOT and FHWA pavement engineers agreed that the existing JRCP had reached its terminal service life, as shown in Figure 6.6, and could not be effectively repaired with a standard patch and overlay project. When evaluating the viability of concrete pavement restoration, PennDOT determined the uncertainty associated with scope and budget were too great to pursue further patching of the existing JRCP. Specifically, they noted it can be very difficult to fully assess the extent of the damage to the JRCP and effectively estimate the number of patches required and the amount of slab stabilization that would be necessary without starting the restoration process. Such uncertainty, in combination with existing performance, was cited as reasons for pursuing alternative rehabilitation techniques. Consequently, they recommended two rehabilitation alternatives: an asphalt overlay over the fractured JRCP or an unbonded concrete overlay over the existing JRCP.

6.2.2 Structural Pavement Design

In late 2016, PennDOT's Design Division carried out a traffic count and geotechnical investigation to provide the information needed for the structural pavement design and analysis. The average daily traffic (ADT) was around 22,800 vehicles, with an average daily truck traffic (ADTT) of approximately 1,200 vehicles. Based on the traffic count data and guidelines in PennDOT's

Figure 6.6 Surface Conditions of SR-28, Section A55 in Allegheny County in 2018 Before Rehabilitation (Courtesy of PennDOT)

Publication 242 – Pavement Policy Manual (PennDOT 2015), the 20-year design traffic was projected to be 7.8 million equivalent single axle loads (MESALs) for flexible pavement design and 11.3 MESALs for rigid pavement design.

The geotechnical investigation encompassed boring to extract samples for California Bearing Ratio (CBR) testing and soil classification and conducting the in-situ Standard Penetration Test (SPT). Analysis of the collected data showed that the in-situ density of the subgrade soil was similar to the value determined in the lab for the CBR test. Accordingly, following Publication 242, Section 6.2.A, the laboratory determined CBR value was multiplied by 1,500 to calculate the resilient modulus for pavement design, resulting in a value of 7,500 psi (i.e., CBR 5 x 1,500). The geotechnical report also indicated a JRCP thickness variation between 9 to 10 inches.

The 20-year structural pavement design was then conducted using the 1993 AASHTO Pavement Design Guide and the Pavement ME Design Guide. The 1993 AASHTO Pavement Design Guide suggested an 8.5-inch asphalt overlay over the fractured JRCP for flexible pavement design, whereas the rigid pavement design necessitated a 4.0-inch unbonded concrete overlay over the existing JRCP. A layer coefficient of 0.35 for the break and seat (B&S) JRCP and a JRCP thickness of 9.0 inches were part of the inputs for the flexible pavement design. However, according to the Pavement ME Design Guide, the flexible pavement design required only a 5.0-inch asphalt overlay over B&S JRCP, while the rigid design called for a 6.0-inch unbonded concrete overlay over the existing JRCP. Two conservative pavement structures were selected for life cycle cost analysis, as follows:

- 8.5 inches of asphalt overlay over the fractured JRCP; and
- 6.0 inches of unbonded concrete overlay over the existing JRCP.

Subsequently, the asphalt overlay design over B&S JRCP was chosen as the final design. The cross-section for the 8.5-inch asphalt overlay comprised:

- A 1.5-inch 9.5-mm SMA, PG 76-22, surface course
- A 2.5-inch 19-mm Superpave, PG76-22, WMA binder course
- A 4.5-inch 25-mm Superpave, PG64-22, WMA base course

The final design represented a significant departure from PennDOT's traditional approach of full pavement reconstruction. Traditionally, such a design would necessitate the complete removal of deteriorated concrete pavement and require 16 inches of asphalt concrete atop the existing 12-inch subbase at an estimated engineering cost of $50 million. However, an alternative approach was considered based on a prior award-winning rehabilitation project on I-79 in District 11. In the prior project, the deteriorated 10-inch concrete pavement was kept in place and subjected to fracturing and seating, which not only provided robust foundational support for construction but also eliminated the need for exposing and potentially repairing the underlying subbase and subgrade layers, thereby averting unforeseen construction delays and change orders. Consequently, overlay designs over the fractured or existing JRCP were considered. The final 8.5-inch asphalt overlay design over B&S JRCP for the SR 28 A55 project was estimated to cost $39 million, with actual construction costs of approximately $35 million, resulting in a net saving of $15 million.

Key engineering considerations for the final design included:

- PennDOT Publication 242 requires considering the loss of serviceability due to frost heave when determining the Terminal Serviceability value. To determine the loss of serviceability, three parameters are needed, including frost heave rate, maximum potential serviceability loss due to frost heave, and frost heave probability. While the first two are specified in Publication 242, the latter can vary from 25 to 75 percent, and is determined by the District Geotechnical Engineer based on the soils report. For this design, a frost heave probability of 50 percent was selected.
- Publication 242 typically recommends a CBR-to-resilient modulus conversion multiplier of 1,000 for calculating the Structural Number (SN) for Future Traffic. However, a multiplier of 1,500 was applied to the lab CBR value to calculate the resilient modulus for this design, as explained earlier.
- Based on Publication 242, the existing 12-inch subbase was not included in Effective Existing SN calculations due to its potential poor condition.
- Although Publication 242 recommends a layer coefficient of 0.25 for fractured concrete pavements, a coefficient of 0.35 was selected for this project. This decision was influenced by FWD test results from prior projects involving asphalt overlays on fractured concrete pavements. Further, the chosen coefficient closely aligns with the 0.34 value suggested later by Ramirez and Morian (2020) for B&S JRCP in Pennsylvania.

6.2.3 Fracturing Deteriorated PCC Pavement

The project was constructed in 2018, beginning with fracturing the outside shoulders. This was followed by the outside lanes, then the inside lanes, and finally, the inside shoulders.

Before the fracturing operations commenced, breaking and cracking patterns were proposed for the mainline travel lanes and for the shoulders, respectively. B&S was specified for the travel lanes due to reinforcing mesh in the JRCP. Crack and seat (C&S) fracturing was specified for the jointed plain concrete pavement (JPCP) shoulders. A T8600 Badger Breaker, as illustrated in Figure 6.7, was used for B&S fracturing of the mainline lanes and for C&S fracturing of the 10-foot outside shoulders. For the 4-foot inside shoulders, the MHBT Badger Breaker (i.e., multi-head breaker) was employed for C&S fracturing due to its capability to selectively disable certain hammer sets to crack the narrow shoulder.

For B&S fracturing, the pattern involved a 30-inch hammer lift height and a 24-inch hammer strike spacing. In contrast, the C&S pattern for the 10-foot outside shoulder was set at an 8-inch hammer lift height with a 24-inch hammer strike spacing. The 4-foot inside shoulder had a variable C&S pattern, ranging from 20 to 30 inches in hammer lift height and 24 to 30 inches in hammer strike spacing. Notably, the total hammer weight of the MHBT Badger Breaker hammers utilized was considerably lighter than the 12,000-lb hammer of the T8600.

Figure 6.7 T8600 Badger Breaker (Courtesy of PennDOT)

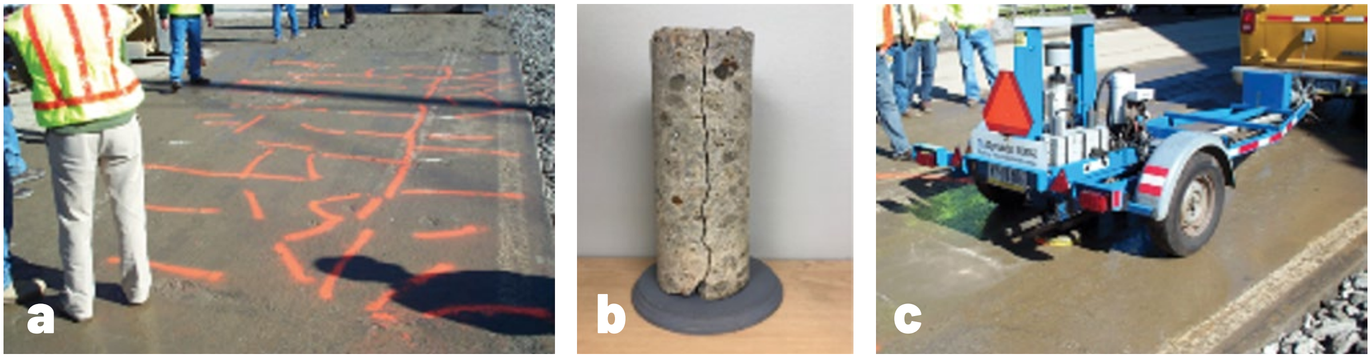


Figure 6.8 Establishing Fracturing Patterns: a) Verifying Cracking Pattern, b) Verifying Crack Depth, and c) Verifying Elastic Modulus using FWD (Courtesy of PennDOT)

To confirm the efficacy of the established patterns, a 250-foot test section of the mainline lane and a 380-foot test section of the shoulder were fractured. The resulting fracture sizes varied between 18 and 24 inches (shown in Figure 6.8a) for the breaking pattern and 18 to 36 inches for the cracking pattern. Cores were also extracted to ensure fractures through the full slab depth (Figure 6.8b). In addition, FWD testing using a 9,000-lb load (Figure 6.8c) was carried out to validate the fracturing patterns on June 8 and 11, 2018. The average backcalculated elastic modulus was 674 ksi for the mainline lane and 656 ksi for the shoulder. Both values passed the required range of 400 to 900 ksi, as specified in the Special Provisions.

Once the patterns were verified, the fracturing operation got underway. Saw-cutting was done full lane width at transverse joints and at one-third points to a depth sufficient to sever mesh reinforcing steel. The spacing of existing joints and saw-cuts was approximately 20 feet. The project encompassed 18 lane miles of B&S fracturing for the mainline travel lanes and 3.75 miles of C&S fracturing for the shoulders.

The broken pavement was then seated by two to four passes of a 50-ton pneumatic tire roller (Figure 6.9). Proper seating was achieved when the vertical deflection beneath the roller was an inch or less. Sections not meeting this requirement were extracted and repaired.

6.2.4 Paving with Asphalt Overlays

Upon completion of the final seating, the fractured mainline traveling lanes and shoulders were surfaced with an 8.5-inch asphalt overlay. This overlay on the mainline traveling lanes comprised a 4.5-inch 25-mm Superpave WMA base course, a 2.5-inch 19-mm Superpave WMA binder course, and a final 1.5-inch 9.5-mm SMA surface. A PG 64-22 binder was designated for the base course, while the binder and surface courses utilized a PG 76-22 binder. In addition, the binder course and SMA mixtures were designed following the Long-Life Asphalt Pavement (LLAP) specifications to ensure extended longevity. The overlay for the shoulders was similar to that of the mainline traveling lanes, except for the wearing surface, which was a 1.5-inch 9.5-mm Superpave WMA mixture.

Figure 6.9 Seating JRCP with 50-ton Pneumatic Tire Proof Rollers (Courtesy of PennDOT)

Following these specifications, a series of performance tests were conducted during the mix design. These included the Hamburg Wheel-Tracking Test (HWTT, AASHTO T324), Disc-Shaped Compact Tension Test (DCT, ASTM D7313), Low-Temperature Semicircular Bend Test (LT-SCB, AASHTO TP105), Illinois Flexibility Index Test (I-FIT, AASHTO TP124), and Overlay Test (OT, TEX 248-F). Furthermore, the HWTT, DCT, and I-FIT were carried out on field cores taken from each lot during construction. The requirement for the HWTT rut depth at 50°C was 12.5 mm after 20,000 passes, and the DCT fracture energy value at -12°C was required to be a minimum average of 460 J/m². Results from other tests were reported for information only.

The overlay design strategy over the fractured JRCP facilitated a single lane closure concurrently in both the northbound and southbound directions, limited to a three-mile stretch. There were no restrictions for traffic moving from northbound to SR 366 and from SR 366 to the southbound. This lane closure plan reduced the number of weekends required for detours and lessened traffic buildup at the SR 366 junction, which typically sees around 50,000 vehicles daily while providing the safety of the work zone for drivers, workers, and inspection staff.

Operating under single-lane restrictions, the contractor could work continually to complete break and seat operations and most of the asphalt base and binder placements in both directions (Figure 6.10). Only two full closures and detours were needed to pave the SMA wearing course on both northbound and southbound lanes. The asphalt mixtures were produced at Lindy Paving's facility in New Kensington, PA, with an average production rate of 325 tons per hour. The haul to the project site was about 20 minutes. The average paver speed was around 35 feet per minute, depending on the material being placed. The project was completed in December 2018.

6.2.5 Savings from Asphalt Overlays over B&S JRCP Design

Opting for asphalt overlays on the broken and seated JRCP proved to be a cost-effective decision for PennDOT. The entire project was completed at a cost of $34.34 million, significantly lower than the projected $50 million required for a full reconstruction. This design led to tangible savings of over $15 million.

Additionally, the rehabilitation strategy enabled the completion of the project in one construction season, even though it was originally scheduled to take two construction seasons. This shorter timeline reduced the duration of work zone congestion, which in turn lessened greenhouse gas (GHG) emissions from vehicles idling in traffic. Further reductions in GHG emissions were achieved by eliminating the need to remove the existing PCC pavement, thereby conserving resources.

Furthermore, the rehabilitation offered traffic management and safety advantages. On an average day, there are 50,000 vehicles a day traveling SR 28 at the southern end of the project, and 21,000 at the Butler County Line. The design facilitated a smoother flow of vehicles during the construction phase. The strategy negated the requirement to redirect a large traffic volume through several municipalities. This reduced the possibility of traffic congestion, accidents, and enhanced the safety of the work zone, benefiting

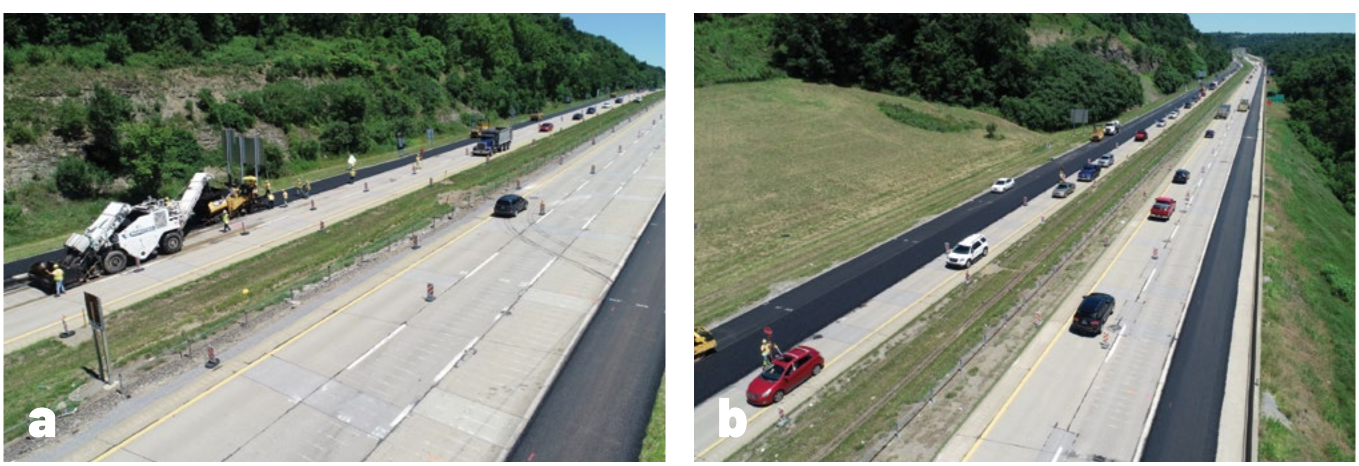


Figure 6.10 Construction of Asphalt Overlay over B&S JRCP: a) Paving Exit Ramp, and b) Paving Main Lane (Courtesy of PennDOT)

drivers, on-site workers, and inspection teams. Twelve weekend closures were initially planned, but ten were efficiently replaced with two extended single-lane restrictions, spanning only a few days.

6.2.6 Post-Construction Evaluation

The asphalt overlay atop the B&S JRCP has exhibited excellent performance since its opening to the traveling public. A comprehensive surface condition survey by PennDOT on April 30, 2022, showed an IRI measurement of 56 inches/mile or lower. The only exception was a section showing an IRI of 91 inches/mile, but it showed no cracks or surface wear and tear, as shown in Figure 6.11. This rehabilitation strategy has outperformed the two earlier repair attempts in 2004 and 2009, where the JRCP showed accelerated deterioration.

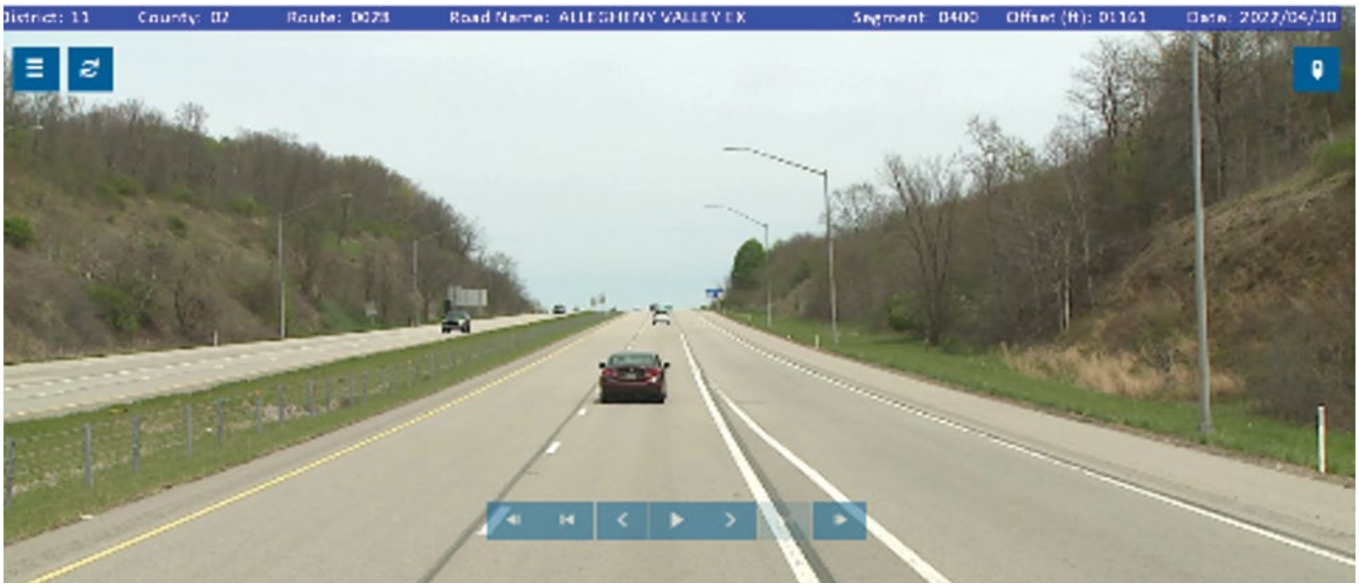


Figure 6.11 SR-28 After Over 4 Years of Excellent Performance (Courtesy of PennDOT)

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