Guide to Using Existing Pavement in Place and Achieving Long Life

DETAILS

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AUTHORS

SHRP 2
STRATEGIC HIGHWAY RESEARCH PROGRAM
Accelerating solutions for highway safety, renewal, reliability, and capacity

Guide to Using Existing Pavement in Place and Achieving Long Life

S2-R23-RW-2

TRANSPORTATION RESEARCH BOARD
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THE SECOND STRATEGIC HIGHWAY RESEARCH PROGRAM

Guide to Using Existing Pavement in Place and Achieving Long Life

SHRP 2 Report S2-R23-RW-2

Newton Jackson and Jason Puccinelli
Nichols Consulting Engineers, Chtd.

Joe Mahoney
University of Washington, Seattle, Washington

TRANSPORTATION RESEARCH BOARD

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SUBJECT AREAS

Construction
Highways
Maintenance and Preservation
Pavements

THE SECOND STRATEGIC HIGHWAY RESEARCH PROGRAM

America's highway system is critical to meeting the mobility and economic needs of local communities, regions, and the nation. Developments in research and technology—such as advanced materials, communications technology, new data collection technologies, and human factors science—offer a new opportunity to improve the safety and reliability of this important national resource. Breakthrough resolution of significant transportation problems, however, requires concentrated resources over a short time frame. Reflecting this need, the second Strategic Highway Research Program (SHRP 2) has an intense, large-scale focus, integrates multiple fields of research and technology, and is fundamentally different from the broad, mission-oriented, discipline-based research programs that have been the mainstay of the highway research industry for half a century.

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The research team comprised Newton Jackson and Jason Puccinelli, Nichols Consulting Engineers, Chtd.; Dr. Joe Mahoney and Dr. Steve Muench, University of Washington; Dr. Karim Chatti and Dr. Neeraj Buch, Michigan State University; Dr. Dan Zollinger and Dr. Tom Scullion, Texas A&M; George White, Pavia; Dr. Julie Vandenbossche, University of Pittsburgh; Dr. E. B. Lee, University of California, Berkeley; and Dr. Katie Hall, ARA.

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FOREWORD

James W. Bryant, Jr., PhD, PE
SHRP 2 Senior Program Officer, Renewal

On roadways that have acceptable geometric features, renewal can be greatly accelerated and costs reduced if the existing pavement can be incorporated into the new pavement structure. Transportation agencies need reliable procedures that allow them to identify when an existing pavement can successfully be used in place and how to incorporate it into the new pavement structure to achieve long life. This guide and the accompanying report and web tool provide guidance for selecting, designing, and constructing long-life pavements using existing pavement structure.

The goal of this project was to develop reliable procedures and guidelines for identifying when existing pavements can be used in place and the methods necessary to incorporate the original material into the new pavement structure while achieving long life. "Long life" was defined as 50 years or longer from the time the pavement was renewed or rehabilitated until the next major rehabilitation. (The report and guide for this project do not provide guidance on the use of routine overlays designed for maintenance and preservation, which is included in the report and guide for SHRP 2 Renewal Project R26, Preservation Approaches for High-Traffic-Volume Roadways.)

The report and guide encourage longer-lasting renewed pavement designs; provide realistic, easy-to-use pavement thickness scoping assessments; and guide users through the data-gathering process needed for input into designing and constructing a long-life pavement using the existing pavement structure. The guide includes the following: project assessment manual; best practices for rehabilitation of flexible pavements and rigid pavements; guide specifications; life-cycle cost analysis; and emerging pavement technology. All the guidance has been incorporated into the web-based pavement design scoping tool, which is meant to complement, not replace, a transportation agency's normal processes for design and pavement-type selection. The guide and web tool were developed with the support of several transportation agencies, including the

Illinois Tollway Authority, Michigan DOT, Minnesota DOT, Missouri DOT, Texas DOT, Virginia DOT, and Washington DOT.

As a result of outreach to transportation agencies, a set of enhancements is currently under way and will be included as a future addendum to the report and guide. Those enhancements will include providing guidance on pavement thickness for 30 to 50 years of rehabilitated design life and updating the guidance and design table to incorporate precast concrete pavements and composite pavement as options for the rehabilitation strategy.

CONTENTS

1 PROJECT ASSESSMENT MANUAL

INTRODUCTION

Why This Assessment Manual?

This assessment manual was prepared to aid the process of renewing existing pavements so that long lives can be achieved. To achieve this goal, a systematic collection of relevant pavement-related data is needed. Further, such data need to be organized to maximize the usefulness in the pavement decision-making process. To that end, this manual will help.

The types of data collection contained in this manual range from basic information such as distress surveys to insights on traffic impacts. The last section provides information on life-cycle assessments (environmental accounting). The use of this type of assessment is receiving increased attention and is likely to be widely applied in the future.

How to Use the Manual

The manual is intended to complement the design tools developed by SHRP 2's R23 study. The types of data critical for making pavement-related decisions are described along with methods (analysis tools) for using the information in decision-making applications. It is not assumed that all data categories will be collected or assessed for a specific renewal project. Rather, the manual is designed as a reference document that provides information relevant to all renewal strategies considered in SHRP 2 Renewal Project R23.

Assessment Data Categories

The following 11 categories are described in this manual:

- Pavement distress survey,
- Pavement rut depth and roughness,

- Nondestructive testing via the falling weight deflectometer (FWD),
- Ground-penetrating radar (GPR),
- Pavement cores,
- Dynamic cone penetrometer (DCP),
- Subgrade soil sampling and tests,
- Traffic loads for design,
- Construction productivity and traffic impacts,
- Life-cycle assessment (environmental accounting), and
- Miscellaneous material properties.

Each data category is structured much the same way, into (1) the purpose of collecting the data, (2) applicable standards, definitions, and data organization recommendations, and (3) analysis tools.

Overall Assessment Scheme

The overall assessment scheme performed by the user can range from rather basic information about the existing and proposed pavement structure to substantially more detailed data and analyses. The basic scheme is illustrated in Figure 1.1.

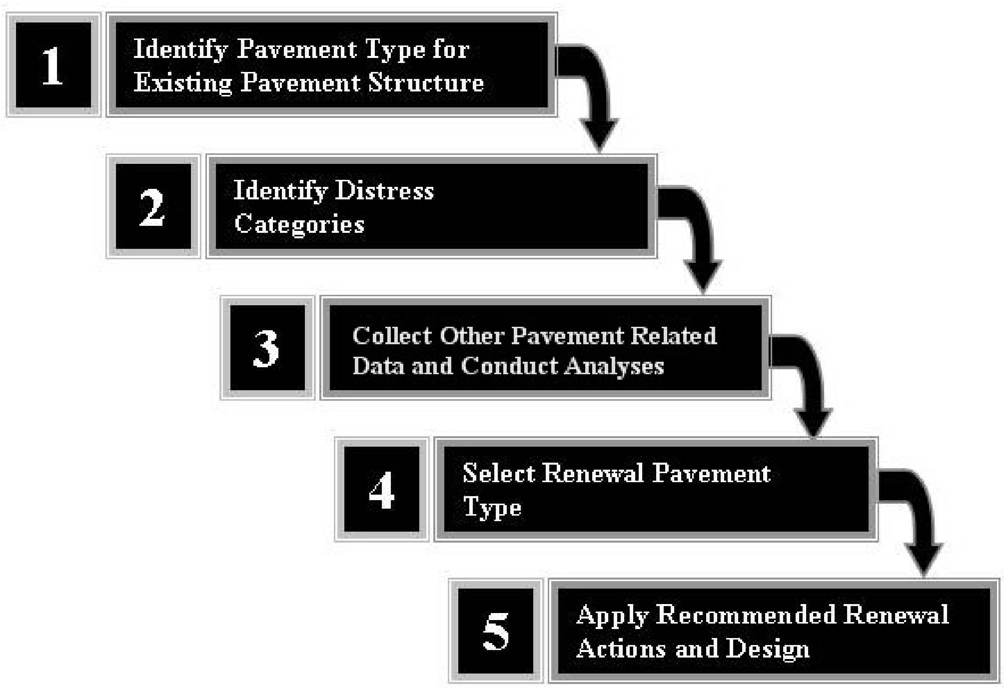


Figure 1.1. *Outline of assessment scheme.*

The first three boxes (1 through 3) shown in Figure 1.1 are addressed in this assessment manual, with that information being applied to the processes shown in the last two boxes (4 and 5).

PAVEMENT DISTRESS SURVEY

Purpose

This section provides an overview of the use of a pavement distress survey to help make pavement assessment decisions.

Measurement Methods

This subsection is used to describe definitions and standards applicable to pavement distresses and provides a way to organize such information.

Pavement Distress Measurements

ASTM D6433-07: Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys.

Distress Identification Manual for the Long-Term Pavement Performance Program

FHWA-RD-03-031, June 2003.

Discussion

Pavement distress data can be used for numerous purposes, but three are noted: (1) establish pavement reconstruction, rehabilitation, and maintenance priorities, (2) determine rehabilitation and maintenance strategies, and (3) predict pavement performance. This type of information is a key element for decision making associated with pavement renewal options.

McCullough (1971) provided a detailed description of three basic pavement distress groups, associated modes, and examples as shown in Table 1.1. The majority of distress survey protocols use a subset of fracture, distortion, and/or disintegration.

Upon closer inspection of Table 1.1 for flexible pavements, two of these—fracture and disintegration—cause most pavement rehabilitation and maintenance actions. More specifically, these can be categorized by fatigue, transverse cracking, and stripping or raveling. Tables 1.2, 1.3, and 1.4 provide templates for flexible pavement distress data collection. It is assumed that cores will be an integral part of the pavement distress examination; hence, locations would logically be organized by mileposts or another appropriate location referencing system. For multilane highways, this information can be collected for the design lane or all lanes in one direction, as per project requirements.

TABLE 1.1. DISTRESS GROUPS

Distress Group	Distress Mode	Examples of Distress Mechanism
Fracture	Cracking	Excessive loading
		Repeated loading (i.e., fatigue)
		Thermal changes
		Moisture changes
		Slippage (horizontal forces)
		Shrinkage
	Spalling	Excessive loading
		Repeated loading (i.e., fatigue)
		Thermal changes
		Moisture changes
Distortion	Permanent deformation	Excessive loading
		Time-dependent deformation (e.g., creep)
		Densification (i.e., compaction)
		Consolidation
		Swelling
		Frost
	Faulting	Excessive loading
		Densification (i.e., compaction)
		Consolidation
		Swelling
Disintegration	Stripping	Adhesion (i.e., loss of bond)
		Chemical reactivity
		Abrasion by traffic
	Raveling and scaling	Adhesion (i.e., loss of bond)
		Chemical reactivity
		Abrasion by traffic
		Degradation of aggregate
		Durability of binder

Source: After McCullough, 1971.

The following distress types should be measured and recorded if present on the existing pavement:

- *Flexible pavement distress* (definitions from or modified after *LTPP Distress Manual*, Miller and Bellinger, 2003)
  1. *Fatigue cracking* occurs in areas subjected to repeated traffic loadings (wheelpaths). It can be a series of interconnected cracks in early stages of development and develops into many-sided, sharp-angled pieces, usually less than 0.3 m on the longest side, characteristically with a chicken wire/alligator pattern in later stages. For illustrations of fatigue-cracking severity levels, see Figure 1.2.

2. *Transverse cracking* involves cracks that are predominantly perpendicular to the pavement centerline. For illustrations of transverse-cracking severity levels, see Figure 1.3.

3. *Stripping or raveling* is the wearing away of the pavement surface caused by the dislodging of aggregate particles and loss of asphalt binder. Raveling ranges from loss of fines to loss of some coarse aggregate and ultimately to a very rough and pitted surface with obvious loss of aggregate (e.g., see Figure 1.4). This study expands the definition to identification of stripping or raveling in the surface layer to include stripping that may be occurring in lower hot-mix asphalt (HMA) layers in the pavement structure. The depth of stripping can be verified by GPR analyses and/or coring as discussed in the sections "Ground-Penetrating Radar (GPR)" and "Pavement Cores" in this chapter.

- *Rigid pavement distress for jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), and continuously reinforced concrete pavement (CRCP)* (definitions from or modified after *LTPP Distress Manual,* Miller and Bellinger, 2003, with the exception of alkali-silica reactivity (ASR) cracking)

1. *Pavement cracking* includes all major types of cracks that can occur in a slab. This can include corner breaks, and longitudinal and transverse cracking as defined by Miller and Bellinger (2003). Corner break cracks intersect the adjacent transverse and longitudinal joints at approximately a 45° angle. Longitudinal cracking and transverse cracking are parallel and transverse to the centerline, respectively. For an example of a portland cement concrete (PCC) slab with multiple cracks, see Figure 1.5.

2. *Joint faulting* is the difference in elevation across a joint or crack. For an example of joint faulting, see Figure 1.6.

3. *Materials-caused distress* includes (1) D-cracking, a closely spaced crescent-shaped hairline cracking pattern that occurs adjacent to joints, cracks, or free edges, with a dark coloring of the cracking pattern and surrounding area, sometimes referred to as durability cracking (see Figure 1.7); and (2) ASR cracking, which is cracking of the PCC that can be easily confused with D-cracking or shrinkage cracking (see Figure 1.8). AASHTO has issued a Provisional Practice (AASHTO Designation PP 65-10) to address ASR. For complete details, please reference Chapter 3 of this Guide, "Rigid Pavements Best Practices."

4. *Pumping* is the ejection of water from beneath the pavement. In some cases, detectable deposits of fine material are left on the pavement surface, which were eroded (pumped) from the support layers and have stained the surface.

5. *Punchouts* are composed of the area enclosed by two closely spaced (usually <0.6 m) transverse cracks, a short longitudinal crack, and the edge of the pavement or a longitudinal joint. Punchouts also include "Y" cracks that exhibit spalling, breakup, or faulting.

Pavement Distress Data Templates

The templates for specific pavement distress types follow.

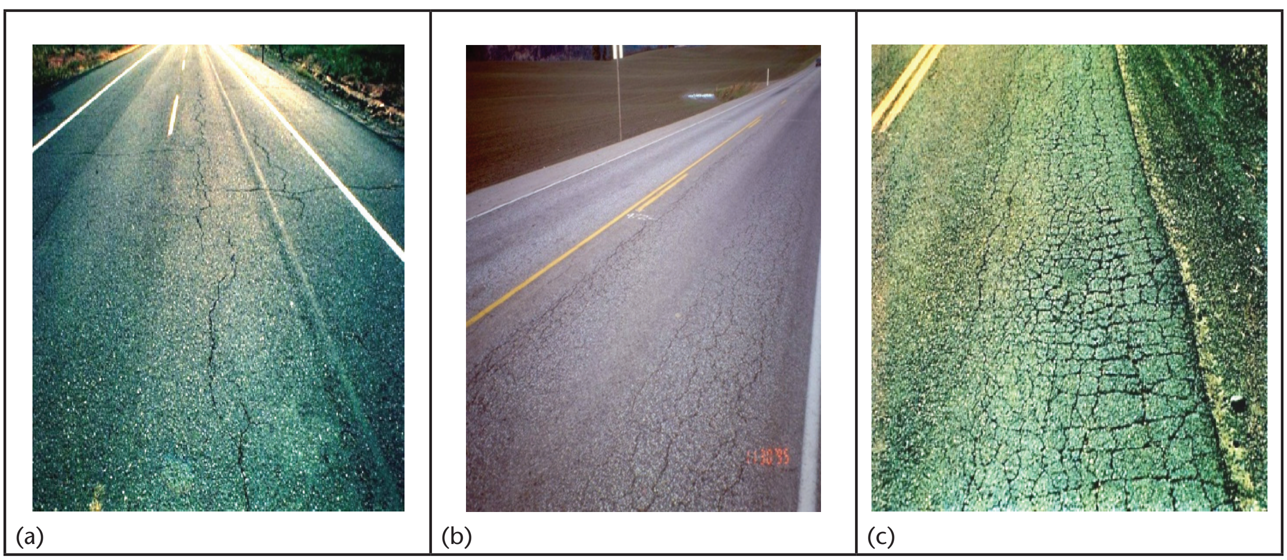
TABLE 1.2. TEMPLATE FOR FLEXIBLE PAVEMENT DISTRESS: FATIGUE CRACKING

	Depth		Distress		
			Fatigue Cracking		
Location (milepost)	HMA (in.)	Base (in.)	Severity[a]	Extent[b]	Depth of Fatigue Cracks (measured from the pavement surface)[c]
			Low		
			Moderate		
			High		

[a] Severity of fatigue cracking is low, medium, or high: (1) low = none or only a few connecting cracks, cracks are not spalled or sealed, and pumping not evident; (2) moderate = interconnected cracks forming a complete pattern, cracks may be slightly spalled, cracks may be sealed, and pumping is not evident; and (3) high = moderately or severely spalled interconnected cracks forming a complete pattern; pieces may move when subjected to traffic, cracks may be sealed, and pumping may be evident. The severity definitions are from the LTPP Distress Identification Manual (Miller and Bellinger, 2003).

[b] Extent of fatigue cracking is based on percent of wheelpath areas. Record extent for each level of severity.

[c] Depth of fatigue cracks can be full-depth or top-down cracking. This should be determined by the use of pavement cores.



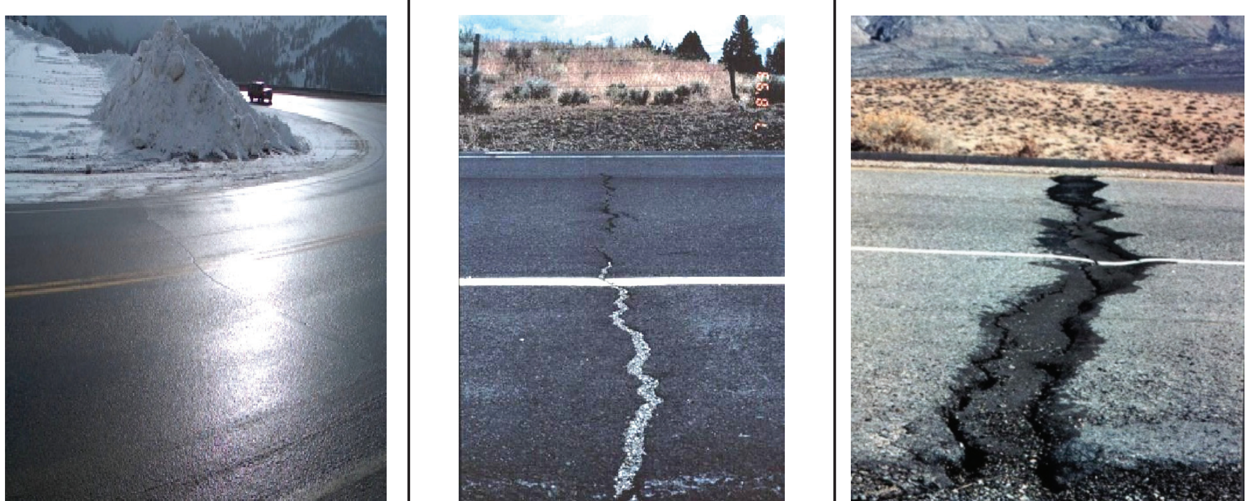
(a) (b) (c)

Figure 1.2. *Illustrations of fatigue-cracking severity levels. (a) Low severity. (b) Moderate severity. (c) High severity.* Sources: (a) Pavement Interactive. (b) N. Jackson. (c) Pavement Interactive.

TABLE 1.3. TEMPLATE FOR FLEXIBLE PAVEMENT DISTRESS: TRANSVERSE CRACKING

	Depth		Distress		
			Transverse Cracking		
Location (milepost)	HMA (in.)	Base (in.)	Severity[a]	Extent[b]	Depth of Transverse Cracks (measured from the pavement surface)[c]
			Low		
			Moderate		
			High		

[a] Severity of transverse cracking is low, medium, or high: (1) low = unsealed cracks with a mean width ≤ 6 mm, sealed cracks with sealant material in good condition and with a width that cannot be determined; (2) moderate = cracks with mean widths > 6 mm and ≤ 19 mm, or any cracks with a mean width ≤ 19 mm and adjacent low-severity random cracking; and (3) high = cracks with a mean width of > 19 mm, or cracks with a mean width ≤ 19 mm and adjacent to moderate- to high-severity random cracking. The severity definitions are from the LTPP Distress Identification Manual (Miller and Bellinger, 2003).
[b] Extent of transverse cracking is based on the number of cracks per 100 ft. Record extent for each level of severity.
[c] Depth of fatigue cracks might be the full depth of the HMA or top-down cracking. This can only be determined by the use of pavement cores.



(a) (b) (c)

Figure 1.3. *Illustrations of transverse-cracking severity levels. (a) Moderate severity. (b) Moderate to high severity. (c) High severity.*
Sources: (a) Pavement Interactive. (b) WSDOT. (c) Pavement Interactive.

TABLE 1.4. TEMPLATE FOR FLEXIBLE PAVEMENT DISTRESS: STRIPPING OR RAVELING

	Depth		Distress	
			Stripping or Raveling	
Location (milepost)	HMA (in.)	Base (in.)	Extent (% of surface area)	Full-depth stripping or raveling or confined to the wearing surface only? Observation must be based on cores.

Note: Severity levels are not applicable for stripping; either it exists or it does not. Coring and/or GPR should be used to verify subsurface moisture damage.

Figure 1.4. *Illustration of raveling.*
Source: WSDOT

Using Table 1.1 again, the most important JPCP distress types that initiate PCC pavement renewal actions are fracture (slab or pavement cracking), distortion (faulting, typically at transverse contraction joints), and disintegration, which includes materials-caused distresses of D-cracking and ASR cracking. These are shown in Tables 1.5 through 1.8. Tables 1.9 and 1.10 apply to CRCP and composite pavements.

TABLE 1.5. TEMPLATE FOR RIGID PAVEMENT DISTRESS (JPCP OR JRCP): PAVEMENT CRACKING

Location (milepost)	Depth			Distress	
		Base		Pavement or Slab Cracking	
	PCC Slab (in.)	Type[a]	Thickness (in.)	% Slabs with Multiple Cracks[b]	Comments

[a] Three types of base underlying PCC: (1) granular base, (2) cement-treated base, and (3) asphalt-treated base.
[b] Percentage of slabs with two or more pavement cracks.

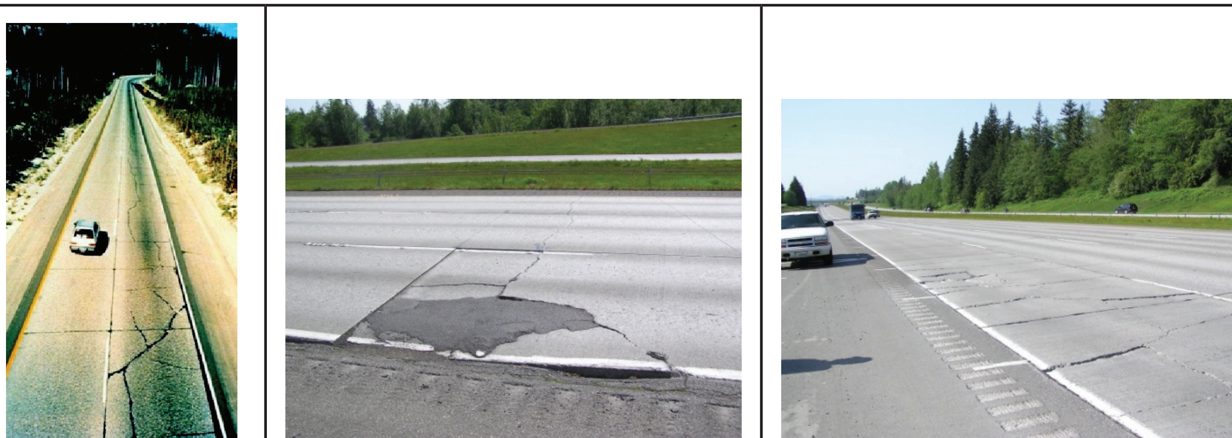


Figure 1.5. *Examples of PCC slabs with multiple cracks.*
Sources: Pavement Interactive and J. Mahoney.

TABLE 1.6. TEMPLATE FOR RIGID PAVEMENT DISTRESS (JPCP OR JRCP): FAULTING

Location (milepost)	Depth			Distress	
		Base		Faulting	
	PCC Slab (in.)	Type[a]	Thickness (in.)	Average Fault Depth (in.)	Comments

[a] Three types of base underlying PCC: (1) granular base, (2) cement-treated base, and (3) asphalt-treated base.

(a) (b)

Figure 1.6. *Examples of various levels of joint faulting. (a) Average fault ~0.25–0.5 in. (b) Average fault ~0.5 in.*
Source: Pavement Interactive.

TABLE 1.7. TEMPLATE FOR RIGID PAVEMENT DISTRESS: D-CRACKING

Location (milepost)	Depth			Distress		
	PCC Slab (in.)	Base		D-Cracking		
		Type[a]	Thickness (in.)	Severity[b]	Extent[c]	Comments
				Low		
				Moderatee		
				High		

[a] Three types of base underlying PCC: (1) granular base, (2) cement-treated base, and (3) asphalt-treated base.
[b] Severity of D-cracking is low, medium (moderate), or high: (1) low = D-cracks are tight, with no loose or missing pieces, and no patching is in the affected area; (2) moderate = D-cracks are well defined, and some small pieces are loose or have been displaced; and (3) high = D-cracking has a well-developed pattern, with a significant amount of loose or missing material. Displaced pieces, up to 0.1 m^2, may have been patched.
[c] Extent is based on the amount of cracks or joints that exhibit D-cracking. This definition of extent is different than used by the LTPP report (Miller and Bellinger, 2003).

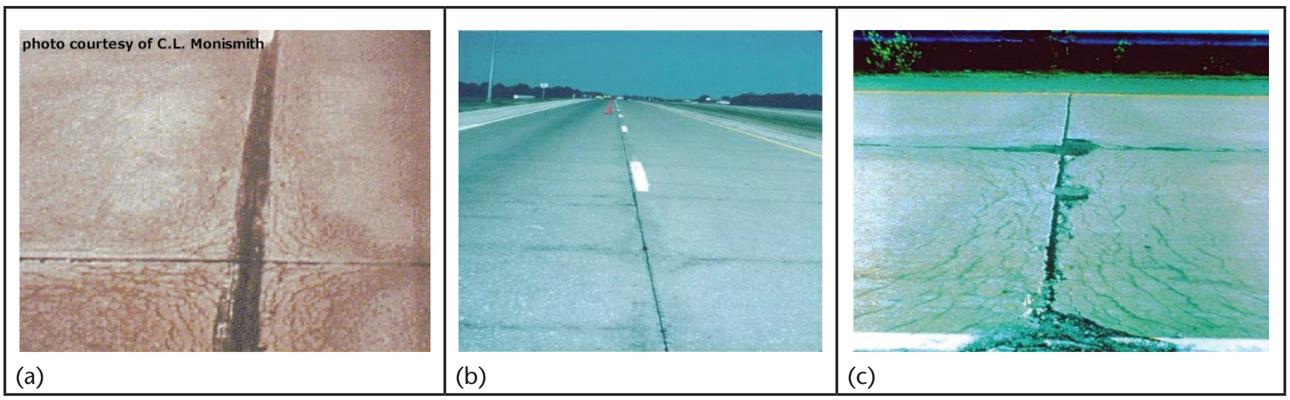
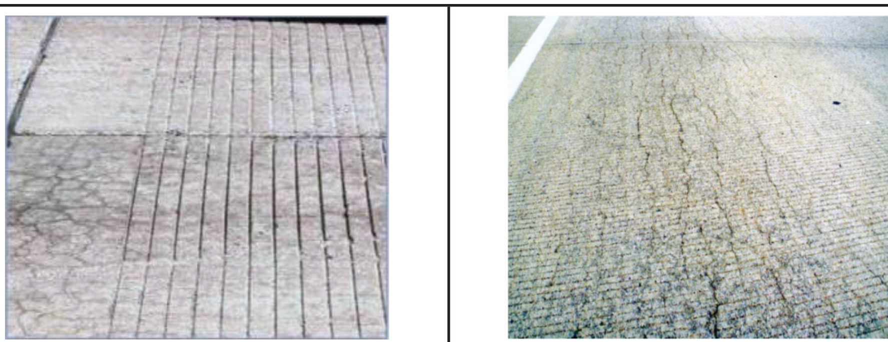


Figure 1.7. *Examples of D-cracking severity levels. (a) Low severity. (b) Low severity. (c) High severity.*
Sources: (a) Pavement Interactive and C. L. Monismith. (b) and (c) N. Jackson.

TABLE 1.8. TEMPLATE FOR RIGID PAVEMENT DISTRESS: ASR CRACKING

Location (milepost)	Depth			Distress	
		Base		ASR-Related Cracking	
	PCC Slab (in.)	Type[a]	Thickness (in.)	Does ASR cracking apply to this pavement? Yes or No[b]	How was ASR detected or measured?

[a] Three types of base underlying PCC: (1) granular base, (2) cement-treated base, and (3) asphalt-treated base.
[b] Severity levels are not applicable for ASR. Either it exists or it does not.



(a) (b)

Figure 1.8. *Illustrations of ASR-cracking severity levels. (a) Early stage of cracking. (b) Advanced stage of cracking.*
Source: N. Jackson.

Table 1.9 applies to CRCP. A critical distress for CRCP is punchouts (which falls under "fracture" in Table 1.1). For an illustration of a CRCP punchout, see Figure 1.9.

TABLE 1.9. TEMPLATE FOR RIGID PAVEMENT DISTRESS (CRCP): PUNCHOUTS

Location (milepost)	Depth			Distress	
		Base		Punchouts	
	PCC Slab (in)	Type[a]	Thick (in)	No./mile	Comments

[a] Three types of base underlying PCC: (1) granular base, (2) cement-treated base, and (3) asphalt-treated base.

Figure 1.9. *Advanced stage for a CRCP punchout.*
Source: FHWA.

TABLE 1.10. COMPOSITE PAVEMENT DISTRESS[a]

Location (milepost)	Depth					Distress[d]	
	HMA Surfacing (in.)	PCC				Describe Condition of Surface Course	Comments
		PCC Type[b]	PCC Slab Thickness (in.)	Base Type[c]	Base Thickness (in.)		
						Poor condition	
						Very poor condition	

[a] Composite pavement definition assumes a flexible (HMA) layer overlies PCC.
[b] Three types of PCC pavement: (1) JPCP, (2) JRCP, and (3) CRCP.
[c] Three types of base underlying PCC: (1) granular base, (2) cement-treated base, and (3) asphalt-treated base.
[d] Distress is broadly defined for composite pavements. The only initial information available to the user is the surface condition, which can include a range of distress types—most likely cracking.

Other PCC pavement distress types can be important and such information can be collected and used; however, the distress types in the preceding tables were judged the most critical for pavement renewal decision making.

Drainage Conditions

An assessment of the existing pavement's subsurface drainage is important in making pavement renewal decisions. The following factors, if observed, suggest that subsurface drainage may be an issue and corrective actions may be needed for the renewal design process:

- Pumping,
- PCC joint or crack faulting,
- Standing water in shallow ditches, and
- Use of cement-stabilized base under PCC.

Analysis Tools

How pavement distress data are specifically used in the renewal decision-making process is covered in Appendices C and D of the R23 report.

PAVEMENT RUT DEPTH AND ROUGHNESS

Purpose

This section overviews the use of pavement rut depths and roughness for aiding pavement assessment decisions.

Measurement Methods

This subsection is used to describe definitions and standards applicable for pavement rut and roughness measurements.

Rut-Depth Measurements

NCHRP Synthesis 334 (McGhee, 2004) notes that 46 state departments of transportation (DOTs) collect automated rut-depth measurements almost always associated with roughness measurements. McGhee (2004) and SHRP (1993) define rut depth as the "longitudinal surface depressions in the wheelpaths."

Figure 1.10 helps to define lateral locations of a typical highway lane (from AASHTO, 2001). Figure 1.11 shows how rut depths are measured with automated equipment.

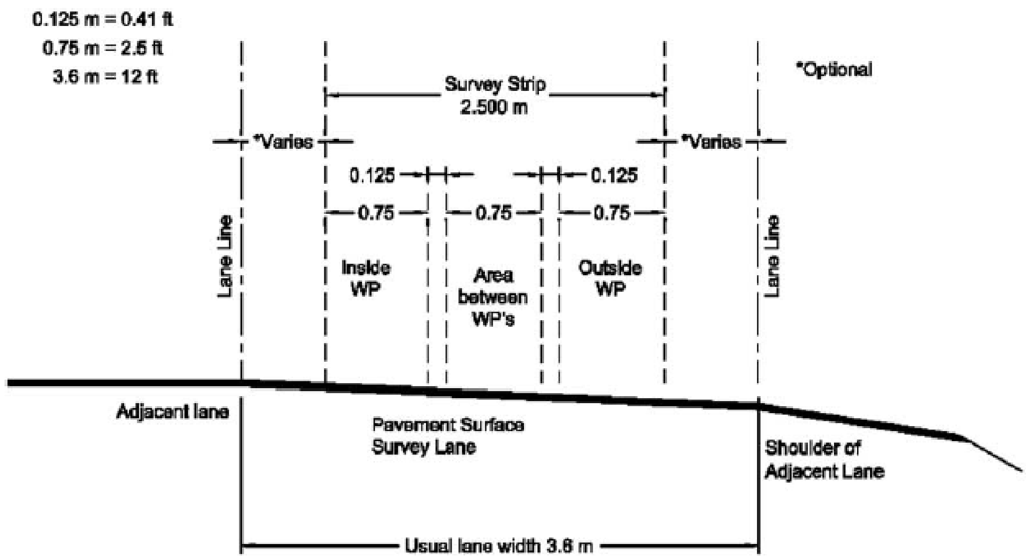


Figure 1.10. *Wheelpaths and area between wheelpaths.*
Source: McGhee, 2004, and AASHTO, 2001.

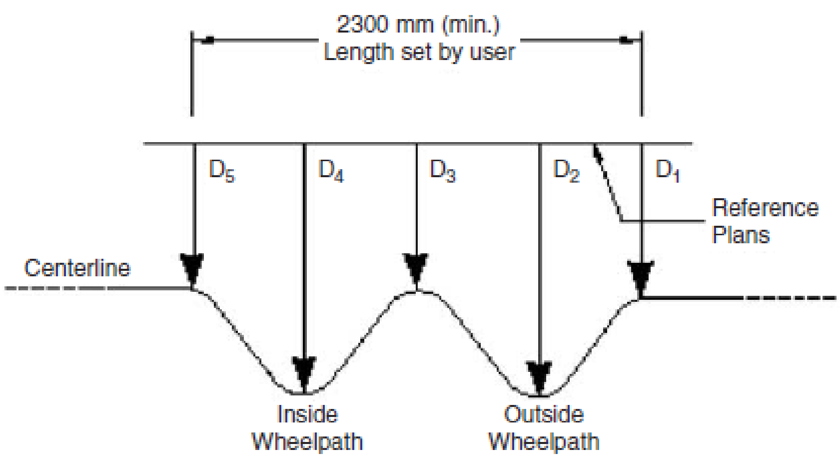


Figure 1.11. *Rut-depth measurements.*
Source: McGhee, 2004, and AASHTO, 2000.

International Roughness Index (IRI) Measurements

McGhee (2004) defines pavement roughness as the "deviation of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics and ride quality." The Standard Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements (ASTM E1926-08) defines the international roughness index (IRI) as the "pavement roughness index computed from a longitudinal profile measurement using a quarter-car simulation at a simulation speed of 80 kph (50 mph)." Further, ASTM E1926 notes that "IRI is reported in either meters per kilometer (m/km) or inches per mile (in/mile)."

Analysis Tools

Some of the analysis tools available include allowable rut depths and recommended IRI levels, which are shown in Tables 1.11 through 1.13.

A study done in Wisconsin found, for state highways with speed limits greater than 45 mph, hydroplaning-related accidents significantly increased when rut depths were 0.3 in. or greater (Start, Jeong, and Berg, 1998). State DOTs such as the Washington State DOT (WSDOT) use a rehabilitation trigger level of 0.4 in. (10 mm). The Texas DOT notes in its Hydraulic Design Manual (2009) that water depths of 0.2 in. or greater, and Fwa (2006) found that a rut depth of 0.5 in. or more, can create the potential for hydroplaning. Thus, a rut depth greater than or equal to 0.5 in. appears to be a reasonable trigger level for rehabilitation decisions.

TABLE 1.11. TYPICAL MAXIMUM RUT DEPTHS

Pavement Type	Maximum Rut Depth, in. (mm)	
Texas DOT [concern about hydroplaning]	>0.2 (>5)	
Wisconsin Hydroplaning Study (Start et al., 1998)	0.3 (7.6)	
Washington State DOT	0.4 (10)	
Fwa (2006) [based on hydroplaning]	0.5 (12.5)	
Shahin (1997) [from the PAVER Asphalt Distress Manual—Pavement Distress Identification Guide for Asphalt-Surfaced Roads and Parking Lots]	Low	0.25–0.5 (6–13)
	Medium	0.5–1.0 (13–25)
	High	>1.0 (>25)

The IRI criteria used by FHWA have evolved as illustrated by review of Tables 1.12 and 1.13. The most detailed breakdown was distributed by FHWA in 1999 and suggests that IRI values of less than 60 in./mile are quite good, whereas those greater than 170 in./mile are poor. Interestingly, many newly paved HMA projects typically have IRI values close to the 60 in./mile value. Eventually, FHWA simplified its criteria as shown in Table 1.12.

A study conducted on Seattle-area urban freeways using driver in-vehicle opinion surveys (Shafizadeh and Mannering, 2003) confirmed that motorists find pavements with IRI values less than 170 in./mile acceptable as to ride quality (85% acceptable). The paper concluded that there was no evidence to change federal IRI guides (in essence those shown in Table 1.13).

TABLE 1.12. FHWA IRI CRITERIA

Ride Quality Terms	All Functional Classifications	
	IRI, in./mi (m/km)	PSR Rating
Good	<95 (<1.5)	Good
Acceptable	≤170 (≤2.7)	Acceptable
Not acceptable	>170 (>2.7)	Not acceptable

Source: FHWA, 2006.

TABLE 1.13. EARLIER FHWA IRI CRITERIA

Ride Quality Terms	PSR Rating	IRI, in./mile (m/km)	National Highway System Ride Quality
Very good	^{3}4.0	<60 (<0.95)	Acceptable, between 0 and 170 in./mile
Good	3.5–3.9	60–94 (0.95–1.48)	
Fair	3.1–3.4	95–119 (1.50–1.88)	
Mediocre	2.6–3.0	120–170 (1.89–2.68)	
Poor	≤2.5	>170 (>2.68)	Less than acceptable, >170 in./mile

Source: FHWA, 1999.

NONDESTRUCTIVE TESTING VIA THE FALLING WEIGHT DEFLECTOMETER (FWD)

Purpose

This section overviews the most commonly used FWD in use and how it can be used to aid pavement assessment decisions.

Measurement Method

This subsection briefly overviews impact (or impulse) pavement loading. FWD devices can obtain measurements rapidly and the impact load is easily varied.

All impact load nondestructive testing (NDT) devices deliver a transient impulse load to the pavement surface. The subsequent pavement response (deflection) is measured. Standard test methods include the following:

- ASTM D4694-96: Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device.
- ASTM D4695-03: Standard Guide for General Pavement Deflection Measurements.

The significant features of ASTM D4694 include the following: (1) the force pulse will approximate a haversine or half-sine wave; (2) the peak force of 11,000 lb must be achievable by the loading device; (3) the force-pulse duration should be within the range of 20 to 60 ms with a rise time in the range of 10 to 30 ms; (4) the loading plate standard sizes are 300 mm (11.8 in.) and 450 mm (18 in.); (5) the deflection transducers, which are used to measure the maximum vertical movement of the pavement, can be seismometers, velocity transducers, or accelerometers; (6) the load measurements must be accurate to at least ±2% or ± 160 N (±36 lb), whichever is greater; (7) the deflection measurements must be accurate to at least ±2% or ±2 μm (±0.08 mil), whichever is greater (note that 0.08 mil = 0.00008 in. and 2 μm = 0.002 mm); and (8) a precision guide in ASTM D4694 notes that, when a device is operated by a single operator in repetitive tests at the same location, the test results are questionable if the difference in the measured center deflection (D_0) between two consecutive tests at the same drop height (or force level) is greater than 5%. For example, if D_0 = 0.254 mm (10 mils) then the next load must result in a D_0 range less than 0.241 mm to 0.267 mm (9.5 to 10.5 mils).

Falling Weight Deflectometer (FWD)

The FWD is widely used in the United States. The device is trailer-mounted and uses deflection sensors that are velocity transducers. By use of different drop *weights* and *heights* this device can vary the impulse load to the pavement structure from about 1,500 to 27,000 lb. The weights are dropped onto a rubber buffer system resulting in a load pulse of 0.025 to 0.030 s. The standard load plate has a 300-mm (11.8-in.) diameter.

Locations for the seven to nine velocity transducers vary. According to ASTM D4694, the number and spacing of the sensors is optional and will depend upon the purpose of the test and the pavement layer characteristics. A sensor spacing of 12 in. is frequently used. A number of state DOTs have used 0, 8, 12, 24, 36, and 48 in. for the distance from the center of the load plate.

SHRP used sensor spacings of 0, 8, 12, 18, 24, 36, and 60 in. from the center of an 11.8-in. load plate.

Analysis Tools

This subsection focuses on straightforward analysis tools that can be applied to FWD deflection results.

Description of Available Analysis Tools for Flexible Pavements

This subsection describes three data-assessment tools: (1) maximum deflection, (2) the area parameter, and (3) a simplified method for calculating the subgrade modulus.

The use of selected indices and algorithms provides a "picture" of the relative conditions found throughout a project. This picture is useful in performing backcalculation and may at times be used by itself on projects with large variations in surfacing layers. Deflections measured at the center of the test load combined with area values and E_{SG} computed from deflections measured at 24 in. from the center of the load plate are shown in the linear plot to provide a visual picture of the conditions found along the length of any project (as illustrated by data from a rural road in Figure 1.12).

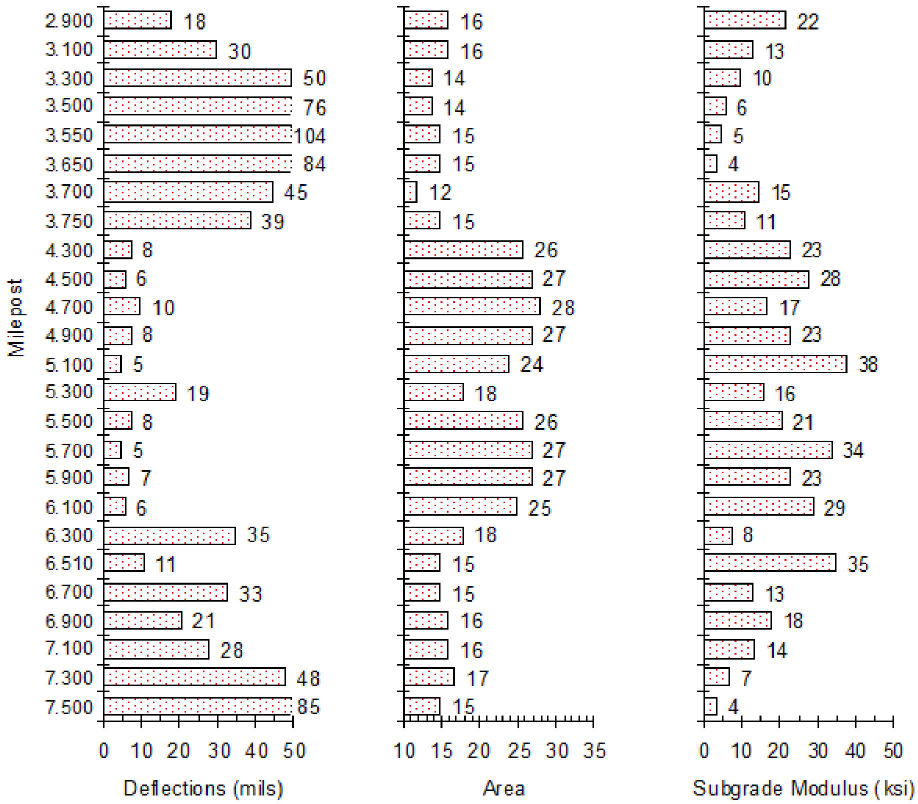


Figure 1.12. *Illustrations of FWD deflection data summarized by the three types of data.*

The deflection data in Figure 1.12 are "normalized" data in that the measured deflections are calculated for a 9,000-lb load. The modulus determination was based on the deflection 24 in. from the center of the load plate.

Table 1.14 provides general information about conclusions that can be drawn from the FWD parameters of area and D_0.

TABLE 1.14. GENERAL INFORMATION ABOUT THE AREA AND D_0

FWD-Based Parameter		
Area	**Maximum Surface Deflection (D_0)**	**Generalized Conclusions**
Low	Low	Weak structure, strong subgrade
Low	High	Weak structure, weak subgrade
High	Low	Strong structure, strong subgrade
High	High	Strong structure, weak subgrade

Maximum Pavement Deflection (D0)

The maximum pavement deflection can vary widely for different pavement structures and throughout the day as the temperature changes. Ranges of D_0 can be grouped into the broad and approximate categories shown in Table 1.15.

TABLE 1.15. D_0 RANGES

Maximum Surface Deflection (D_0) Level	Generalized Conclusions	Approximate D_0 (in.)
Low deflections	Strong structure	≤0.020
Medium deflections	Medium structure	0.030
High deflections	Weak structure	>0.050

Area Parameter

The area parameter represents the normalized area of a slice taken through any deflection basin between the center of the test load and 3 ft. ”Normalized” means that the area of the slice is divided by the deflection measured at the center of the test load, D_0. Thus, the area parameter is the length of one side of a rectangle where the other side of the rectangle is D_0; hence, the area parameter has units of inches.

The area equation is:

$$A = 6(D_0 + 2D_1 + 2D_2 + D_3)/D_0$$

where

D_0 = surface deflection (mils) at the center of the test load,
D_1 = surface deflection (mils) at 1 ft,
D_2 = surface deflection (mils) at 2 ft, and
D_3 = su.rface deflection (mils) at 3 ft.

The *maximum* value for area is 36.0 and occurs when all four deflection measurements are equal (not likely to actually occur), as follows:

$$\text{If } D_0 = D_1 = D_2 = D_3, \text{ then area} = 6(1 + 2 + 2 + 1)/1 = 36.0 \text{ m} = 36.0 \text{ in.}$$

All four deflection measurements being equal (or nearly equal) indicates an *extremely stiff* pavement system (like PCC slabs or thick, full-depth asphalt concrete).

The *minimum* area value should be no less than 11.1 in. This value can be calculated for a one-layer system, which is analogous to testing (or deflecting) the top of the subgrade (i.e., no pavement structure). Using appropriate equations, the ratios

$$\frac{D_1}{D_0}, \frac{D_2}{D_0}, \frac{D_3}{D_0}$$

always result in 0.26, 0.125, and 0.083, respectively. Putting these ratios in the area equation results in area = 6(1+ 2(0.26) + 2(0.125) + 0.083)/1 = 11.1 in. Further, this area value suggests that the elastic moduli of any pavement system would all be equal (e.g., $E_1 = E_2 = E_3 = \ldots$). This is highly unlikely for actual, in-service pavement structures. Low area values suggest that the pavement structure is not much different from the underlying subgrade material (which is *not* always a bad thing if the subgrade is *extremely* stiff). Typical area values are shown in Table 1.16.

TABLE 1.16. TYPICAL AREA VALUES

Pavement Structure	Area Parameter (in.)
PCC pavement range	24–33
"Sound" PCC	29–32
Thick HMA (~9 in. of HMA)	27+
Medium HMA (~5 in. of HMA)	23
Thin HMA (~2 in. of HMA)	17
Chip-sealed flexible pavement	15–17
Weak chip-sealed flexible pavement	12–15

Subgrade Modulus

An NCHRP study (Darter, Elliott, and Hall, 1991), which revised Part III of the AASHTO Pavement Guide, recommended that the following equation be used to solve for subgrade modulus:

$$M_R = P(1 - \mu^2)/(\mathrm{m})(D_r)(r) \tag{1.1}$$

where

M_R = backcalculated subgrade resilient modulus (psi),
P = applied load (lb) from the FWD,
D_r = pavement surface deflection a distance r from the center of the load plate (in.), and
r = distance from center of load plate to D_r (in.).

Using a Poisson ratio of 0.40, Equation 1.1 reduces to

$$M_R = 0.01114(P/D_2), \tag{1.2}$$

$$M_R = 0.00743(P/D_3), \tag{1.3}$$

$$M_R = 0.00557(P/D_4), \tag{1.4}$$

for sensor spacing of 2 ft (610 mm), 3 ft (914 mm), and 4 ft (1219 mm), respectively.

If a Poisson ratio of 0.45 is used instead for the same sensor spacing, the equations become

$$M_R = 0.01058(P/D_2), \tag{1.5}$$

$$M_R = 0.00705(P/D_3), \tag{1.6}$$

$$M_R = 0.00529(P/D_4). \tag{1.7}$$

Darter et al. (1991) recommended that the deflection used for subgrade modulus determination should be taken at a distance at least 0.7 times r/a_e, where r is the radial distance to the deflection sensor and a_e is the radial dimension of the applied stress bulb at the subgrade "surface." The a_e dimension can be determined from the following:

$$a_e = \sqrt{a^2 + \left(D\sqrt[3]{\frac{E_P}{M_R}} \right)^2}$$

where

a_e = radius of stress bulb at the subgrade–pavement interface,
a = NDT load plate radius (in.),
D = total thickness of the pavement layers (in.),
E_P = effective pavement modulus (psi), and
M_R = backcalculated subgrade resilient modulus.

For "thin" pavements, a_e 15 in., and for "medium" to "thick" pavements, a_e 26 to 33 in. Thus, the minimum r is usually 24 to 36 in. (recall $r \geq 0.7(a_e)$).

Typical subgrade moduli are shown in the Table 1.17 (after Chou, Uzan, and Lytton, 1989).

TABLE 1.17. TYPICAL SUBGRADE MODULI

	Subgrade Moduli and Climate Condition			
			Wet, Freeze	
Material	Dry (psi)	Wet, No Freeze (psi)	Unfrozen (psi)	Frozen (psi)
Clay	15,000	6,000	6,000	50,000
Silt	15,000	10,000	5,000	50,000
Silty or clayey sand	20,000	10,000	5,000	50,000
Sand	25,000	25,000	25,000	50,000
Silty or clayey gravel	40,000	30,000	20,000	50,000
Gravel	50,000	50,000	40,000	50,000

Example Analyses of FWD Deflection Basins for Flexible Pavement

The deflection basins shown in Table 1.18 were obtained with an FWD. The pavement temperature at the time of testing was 46°F (8°C). The deflection basins for the four FWD drops normalized to 9,000 lb are shown in the table.

TABLE 1.18. EXAMPLE FWD DEFLECTION DATA

FWD Load (lb)	Deflection (mils)					
	D_0	$D_{8''}$	$D_{12''}$	$D_{24''}$	$D_{36''}$	$D_{48''}$
16,987	27.07	21.55	18.60	11.27	7.33	5.28
12,070	21.28	16.98	14.62	8.67	5.56	3.98
9,406	17.53	13.95	11.98	7.01	4.45	3.23
6,186	12.33	9.77	8.31	4.65	2.88	2.05
Normalized to 9,000 lb	**16.59**	**13.24**	**11.34**	**6.58**	**4.18**	**2.99**

The pavement structure at the time of FWD testing was as follows:

- HMA: 6.0 in. (and the HMA layer exhibited some fatigue cracking).
- Granular base (sandy gravel): 18.0 in.
- Subgrade: Silt (ML) with a wide seasonal variation in water table depth. The soil is frost susceptible and this area can have substantial ground freezing. The spring thaw occurred about one month prior to the testing.

Requirements

Analyze the available data to characterize the overall structure and estimate the layer properties (moduli) using only the information provided above.

Results

Maximum surface deflection

The maximum surface deflection is 0.01659 in. for a pavement with 6 in. of HMA. This value suggests a "low" pavement deflection.

Subgrade modulus (closed-form equations) from the AASHTO Guide (1993)

$$\begin{aligned} M_R &= P(1-\mu^2)/(\text{p})(D_r)(r) \\ &= 9000(1-0.45^2)/(\text{p})(0.00418)(36) \\ &\cong 15{,}200 \text{ psi.} \end{aligned}$$

Check $r \geq 0.7(a_e)$, OK.

The pavement subgrade modulus for an ML silt is better than average.

Area parameter

$$\begin{aligned}\text{Area} &= 6(D_0 + 2D_{12''} + 2D_{24''} + D_{36''})/D_0\\ &= 6(0.01659 + (2)(0.01134) + (2)(0.00658) + 0.00418)/0.01659\\ &\cong 20.5 \text{ in.}\end{aligned}$$

This area parameter is *low* for this thickness of asphalt concrete (AC). Thus, the area value suggests a weak pavement structure but not extremely so.

More Detailed Project Data Example

Table 1.19 summarizes deflection data that were collected on a portion of an actual project. The project was about 5 mi in length and FWD testing was performed every 250 ft, but only four of the FWD locations are shown (these locations were also coring sites). The average pavement temperature at the time the FWD data were collected was 46°F to 50°F. The timing of the survey was about 1.5 to 2 months after the spring thaw in this area.

TABLE 1.19. FWD DEFLECTIONS, AREA VALUE, AND SUBGRADE MODULUS

Core Location (MP)	Load (lbf)	Deflections (mils)						Area Value (in.)	M_R (psi)
		D_0	$D_{8''}$	$D_{12''}$	$D_{24''}$	$D_{36''}$	$D_{48''}$		
207.85	16,940	31.30	26.18	23.19	13.78	9.09	6.65		
	12,086	24.21	20.31	18.11	10.35	6.81	4.96		
	9,421	19.45	16.38	14.57	8.11	5.28	3.98		
	6,218	13.19	11.26	9.92	5.12	3.39	2.83		
Normalized Values		**18.39**	**15.51**	**13.78**	**7.60**	**5.00**	**3.82**	**21**	**14,358**
208.00	16,987	27.04	21.53	18.58	11.26	7.32	5.28		
	12,070	21.26	16.97	14.61	8.66	5.55	3.98		
	9,405	17.52	13.94	11.97	7.01	4.45	3.23		
	6,186	12.32	9.76	8.31	4.65	2.87	2.05		
Normalized Values		**16.57**	**13.23**	**11.34**	**6.57**	**4.17**	**2.99**	**20**	**16,534**
208.50	16,829	14.92	11.89	10.23	5.91	3.19	2.28		
	12,245	11.65	9.29	7.95	4.49	2.13	1.73		
	9,533	9.61	7.63	6.53	3.62	1.81	1.30		
	6,297	6.73	5.35	4.49	2.40	1.26	0.87		
Normalized Values		**9.01**	**7.17**	**6.10**	**3.39**	**1.69**	**1.26**	**19**	**32,198**
209.00	16,305	59.25	48.58	42.52	21.30	9.53	5.12		
	11,737	46.14	37.52	32.56	15.59	6.69	3.58		
	9,247	36.93	29.80	25.63	11.77	4.96	2.68		
	6,154	25.00	19.88	16.77	7.28	3.03	1.73		
Normalized Values		**35.51**	**28.66**	**24.61**	**11.42**	**4.84**	**2.64**	**19**	**9,572**

As shown in Table 1.19, the normalized D_0 deflections range from about 9 to 36 mils. Deflections less than about 30 mils are considered normal. The HMA thicknesses varied between 4.6 and 5.3 in. with an average of 5.2 in., which constitutes a "medium" thickness of HMA (refer back to Table 1.15).

The area values shown in the table suggest weak HMA, but not necessarily extreme weakness due to stripping. Table 1.20 illustrates typical theoretical area values for various uncracked HMA thicknesses, which aids this type of comparison.

TABLE 1.20. TYPICAL THEORETICAL AREA VALUES FOR UNCRACKED HMA

	Approximate Area Parameter (in.)	
HMA Thickness (in.)	Normal Stiffness	Low Stiffness
2	17	16
3	19	18
4	21	19
5	23	21
6	24	22
7	26	22
8	26	23
9	27	24
10	28	24

A quick, slightly more formal check of the pavement structure is to compare the actual area value to see if it falls within the range (normal to low stiffness), above the range (above normal stiffness), or below the range (below normal stiffness). This comparison is shown in Table 1.21.

TABLE 1.21. COMPARISON OF AREA VALUE AND ACCEPTABLE AREA VALUE RANGE

Core Location	HMA Thickness (in.)	Actual Area (in.)	Above, Below, or Within Range
207.85	5.3	21	Within
208.00	6.0	20	Below
208.50	4.7	19	Below
209.00	4.6	19	Below

Area values provide a very good check on whether the surface cracking observed is full-depth or top-down cracking. If it is top-down cracking, then the area values will be within the expected range. If it is full-depth cracking, then the area values will be well below the expected range. The area values may also be used to provide some assessment of changes in pavement structure or depth, and as such the measure provides a good basis for coring locations. And finally, the area values can provide a very good check against the relative values for the backcalculated modulus for the HMA layer.

Description of Available Analysis Tools for Rigid Pavements

Rehabilitation of PCC pavements is not straightforward. To provide a more consistent analysis process, the load transfer efficiency (LTE) should be checked with FWD-obtained deflection data if the pavement type is JPCP.

Load Transfer Efficiency

When a wheel load is applied at a joint or crack, both the loaded slab and the adjacent unloaded slab deflect. The amount the unloaded slab deflects is directly related to joint performance. If a joint is performing perfectly, the loaded and unloaded slabs deflect equally.

Joint performance can be evaluated by calculating LTE across a joint or crack using measured deflection data. The concept of joint load transfer efficiency is illustrated in Figure 1.13. LTE can be calculated using the following equation:

$$LTE = (\Delta_U / D_L)(100)$$

where

LTE = load transfer efficiency (%),
Δ_U = deflection of the unloaded slab (mils), and
D_L = loaded slab deflection (mils).

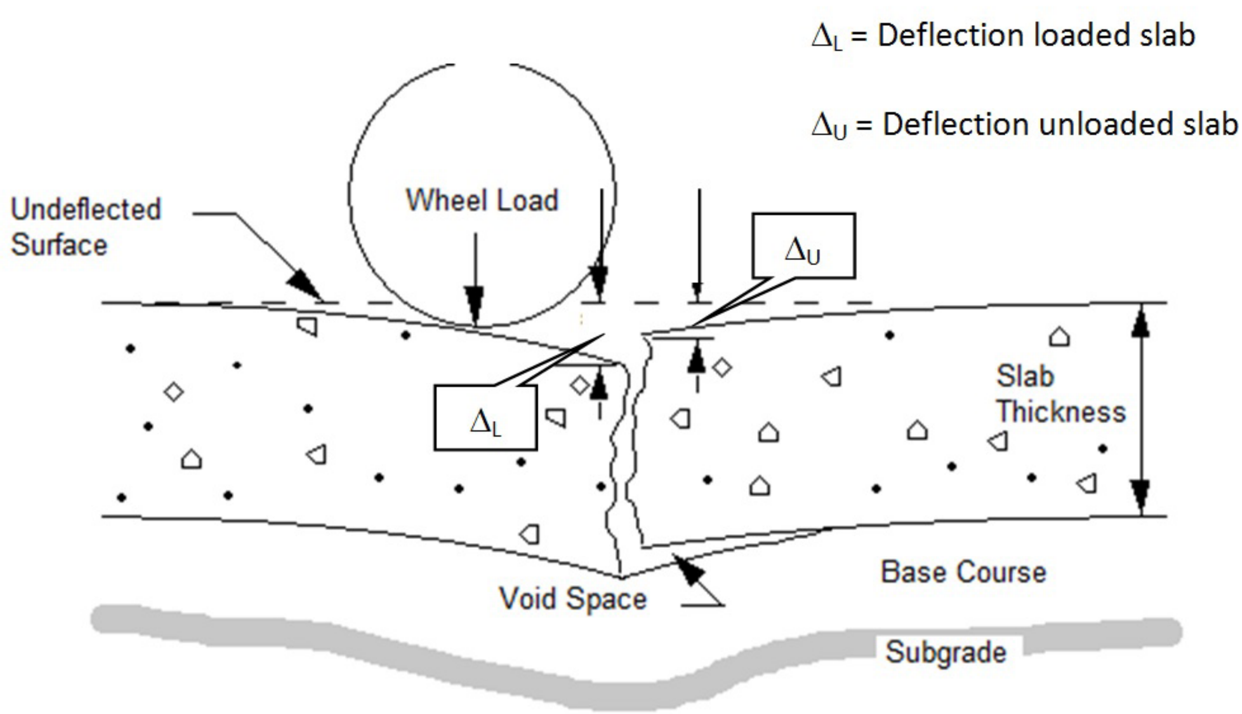


Figure 1.13. *Illustration of joint load transfer efficiency.*
Source: NHI, 2003.

Joint efficiency depends on several factors, including temperature (which affects joint opening), joint spacing, number and magnitude of load applications, foundation support, aggregate particle angularity, and the presence of mechanical load transfer devices.

As mentioned, temperature plays a major role in determining joint effectiveness. In general, the lower the temperature, the lower the load transfer efficiency. Load transfer efficiency is reduced because joints open during cooler weather, reducing contact between faces. Joint load transfer efficiency has also been shown, in both laboratory and field studies, to decrease with increasing load applications. However, this impact is lessened for harder aggregates. The aggregate characteristics play a more significant role after many load applications.

To test the approach side of a joint or crack, the FWD loading plate is placed in front of the joint, with the other velocity transducers located across the joint. The leave side of the joint is tested by placing the loading plate at the joint edge on the leave slab with an extra velocity transducer mounted behind the loading plate across the joint. The concept of slab approach and leave sides and of transverse joint testing are illustrated in Figure 1.14.

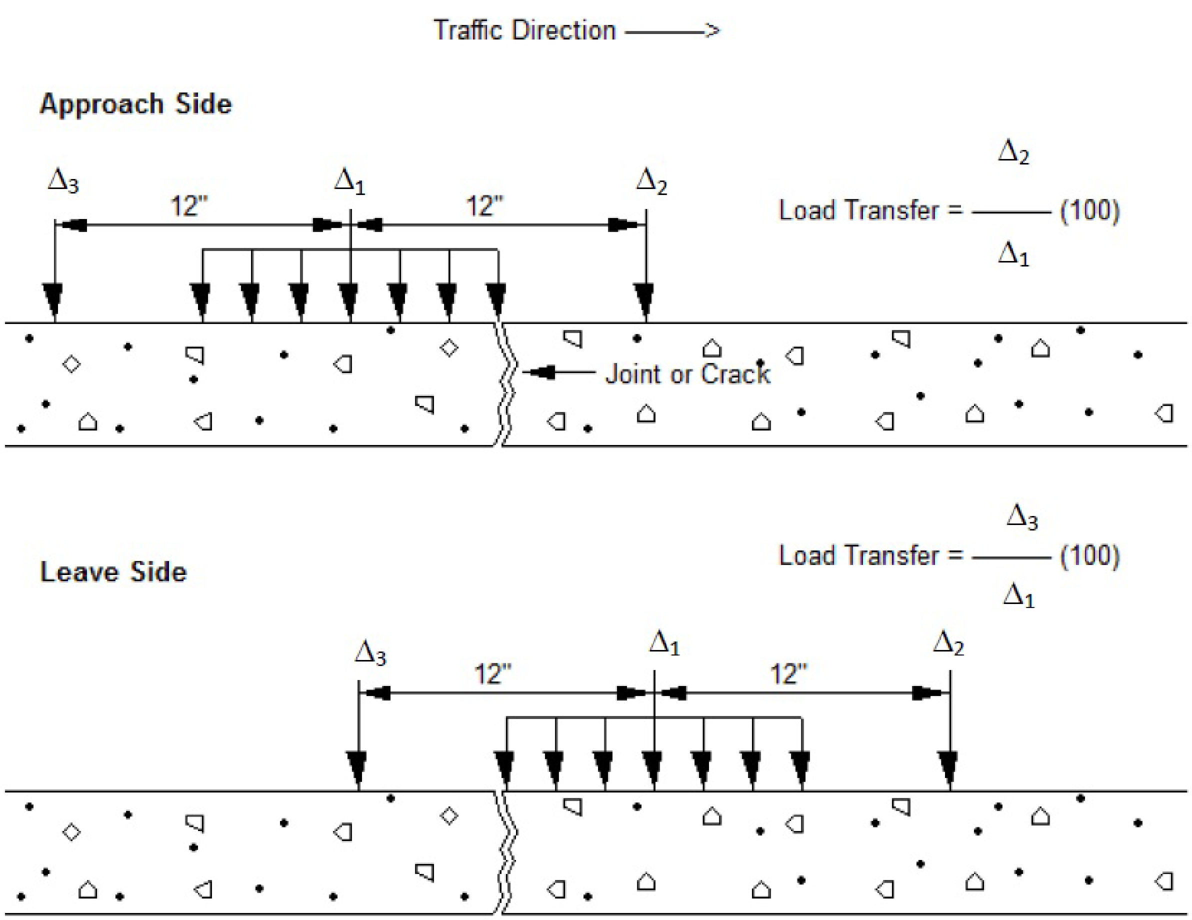


Figure 1.14. *Locations of FWD load plate and deflection sensors for determining load transfer efficiency.*
Source: NHI, 2003.

The percentage of load transfer can vary between almost 100% (excellent) to near 0% (extremely low). AASHTO (1993) notes that load transfer restoration should be considered for transverse joints and cracks with load transfer efficiencies ranging from 0% to 50%. It has been observed for numerous in-service jointed PCC pavements that load transfer efficiencies of 70% or greater generally provide good joint or crack performance.

Backcalculation

Backcalculation is the process by which pavement layer moduli are estimated by matching measured and calculated surface deflection basins. This is done via a computer program and there are a number of these available in the United States. It is likely that within a specific state there is a preferred backcalculation software package to use.

The general guidelines that follow are broad in scope and should be considered "rules of thumb."

Number of Layers

Generally, use no more than three or four layers of unknown moduli in the backcalculation process (preferably, no more than three layers). If a three-layer system is being evaluated, and questionable results are being produced (e.g., extremely weak base moduli), then it is sometimes advantageous to evaluate this pavement structure as a two-layer system. Some experienced users have found that there are times, such as dealing with a highly stress-sensitive subgrade, when it may be beneficial to consider adding layers to reduce compensating error effects. This modification would possibly indicate the base material has been contaminated by the underlying subgrade and is weaker due to the presence of fine material. Alternatively, a stiff layer should be considered if it has not been considered previously (see below). If a pavement structure consists of a stiffer layer between two weak layers, it may be difficult to obtain realistic backcalculated moduli. An example is a pavement structure that consists of deteriorated asphalt concrete over a cement-treated base. A stiff underlying layer, if found, is typically given a modulus value and is treated as a layer of known moduli.

Thickness of Layers

Surfacing. It can be difficult to "accurately" backcalculate HMA or BST moduli for bituminous surface layers less than 3 in. thick. Such backcalculation can be attempted for layers less than 3 in. thick, but caution is suggested.

In theory, it is possible to backcalculate separate layer moduli for various types of bituminous layers within a flexible pavement. Generally, it is not advisable to do this because one can quickly attempt to backcalculate too many unknown layer moduli (i.e., more than three or four). By necessity, one should expect to combine all bituminous layers (seal coats, asphalt concrete, etc.) into "one" layer unless there is evidence (or the potential) of distress, such as stripping, in an HMA layer, or some other such distress that is critical to pavement performance.

Unstabilized base and subbase course. A "thin" base course beneath "thick" surfacing layers (say, HMA or PCC) often results in low base moduli. There are a number of reasons why this can occur. First, a thin base is not a "significant" layer under a stiff, thick layer and, where the measured surface deflections are not significantly affected by the layer, its moduli cannot be backcalculated. Second, the base modulus may be relatively "low" due to the stress sensitivity of granular materials. The use of a stiff layer generally improves the modulus estimate for base and subbase layers.

Subgrade

If unusually high subgrade moduli are calculated, a check should be performed to see if a stiff layer is present. Stiff layers, if unaccounted for in the backcalculation process, will generally result in unrealistically high subgrade moduli accompanied by inappropriately low base-layer moduli due to compensating error effects. This is particularly true if a stiff layer is within a depth of about 20 to 30 ft below the pavement surface.

Stiff Layer

Often, stiff layers are given "fixed" stiffness ranging from 100,000 to 1,000,000 psi with semi-infinite depth. This, in effect, gives the "subgrade" a layer with a "fixed" depth instead of the normally assumed semi-infinite depth. What is not so clear is whether one should always fix the depth to the stiff layer at 20, 30, or 50 ft if no stiff layer is otherwise indicated (i.e., use a semi-infinite depth for the subgrade). The depth to the stiff layer should be verified whenever possible with other NDT data or borings. It should be noted that a number of backcalculation programs include a tool to estimate the depth to the stiff layer.

The stiffness (modulus) of the stiff layer can vary. If the stiff layer occurs because of saturated conditions (e.g., water table), then moduli of about 50,000 psi appear more appropriate. If rock or stiff glacial tills are the source of the stiff layer, then moduli of about 1,000,000 psi appear to be more appropriate.

Layer Moduli

A few comments about layer moduli are appropriate.

Cracked HMA moduli. Generally, fatigue-cracked HMA (about 10% wheelpath cracking) is often observed to have backcalculated moduli of about 100,000 to 250,000 psi. What is most important in the backcalculation process, assuming surface fatigue cracking is present, is to determine whether the cracks are confined to only the immediate wearing course or if they penetrate through the whole depth of the HMA layer. For HMA layers greater than 6 in. thick, cracking only in the wearing course is often observed and the overall HMA layer will have a substantially higher stiffness than noted above (at moderate layer temperatures of 75°F to 80°F).

Base and subbase moduli. Typical base and subbase moduli are shown in Table 1.22.

TABLE 1.22. TYPICAL UNSTABILIZED AND STABILIZED BASE AND SUBBASE MODULI

Material	Compressive Strength (psi)	Typical Modulus (psi)	Modulus Range (psi)
Unstabilized			
Crushed stone or gravel base	—	35,000	10,000–150,000
Crushed stone or gravel subbase	—	30,000	10,000–100,000
Sand base	—	20,000	5,000–80,000
Sand subbase	—	15,000	5.000–80,000
Stabilized			
Lime stabilized	<250	30,000	5,000–100,000
	250–500	50,000	15,000–150,000
	>500	70,000	20,000–200,000
Cement stabilized	<750	400,000	100,000–1,500,000
	750–1,250	1,000,000	200,000–3,000,000
	>1,250	1,500,000	300,000–4,000,000

Subgrade moduli. Typical subgrade moduli were previously shown in Table 1.18.

Backcalculation Summary

Performing backcalculation of pavement layer moduli is part science and part art; thus, experience typically will improve the estimated results. It is advisable to initially work with someone who has solid experience doing backcalculation or to take a short course on the topic, assuming one is available. It will take only a few projects, along with experience from others, to become well informed about this powerful assessment technique.

Reviews of Stubstad et al. (2006), Von Quintus and Simpson (2002), and Yau and Von Quintus (2002) provide additional insight into layer moduli and how to estimate them. These references cover work associated with the LTPP experiments and include both backcalculation and closed-form equations for developing moduli estimates along with laboratory results.

GROUND-PENETRATING RADAR (GPR)

Purpose

This section describes GPR technology and presents an overview of the most common applications of both air-coupled and ground-coupled GPR systems for aiding in pavement assessment decisions.

Measurement Method

This section briefly describes the two types of GPR and the basic principles of operation.

The standard references for GPR applications in highways are:

- AASHTO PP 40-00: Standard Recommended Practice for Application of Ground Penetrating Radar to Highways.
- ASTM D6087-08: Standard Test Method for Evaluating Asphalt Covered Concrete Bridge Decks using Ground Penetrating Radar.
- ASTM D6432-99 (2005): Standard Guide for Using Surface Ground Penetrating Radar Method for Subsurface Investigation.

Air-Coupled GPR Systems

A typical commercially available 2.2-GHz air-coupled GPR unit is shown in Figure 1.15. The radar antenna is attached to a fiberglass boom and suspended about 5 ft from the vehicle and 14 in. above the pavement. This particular GPR unit can operate at highway speeds (70 mph); it transmits and receives 50 pulses per second and can effectively penetrate to a depth of around 20 to 24 in. All GPR systems include a distance-measuring system and many of the new systems also have synchronized or integrated video logging, so the operator can view both surface and subsurface conditions. Global positioning is also included in many new systems for identifying problem locations.

Figure 1.15. *Air-coupled GPR systems for highways.*
Photo: Tom Scullion.

The *advantages* of these systems include the speed of data collection, which does not require any special traffic control. The GPR generates clean signals that without filtering are ideal for quantitative analysis using automated data-processing techniques to compute layer dielectrics and thicknesses. These systems are also excellent for locating near-surface defects in flexible pavements.

The *disadvantages* are that (a) they have a limited depth of penetration, (b) they are not ideal for penetrating thick concrete pavements, and (c) the most popular operating frequency (1 GHz) is now subject to Federal Communications Commission (FCC) restrictions in the United States.

Ground-Coupled GPR systems

As shown in Figure 1.16, a whole range of different operating frequencies is available for ground-coupled GPR systems. The selection of the best frequency for a particular application depends on the required depth of penetration. As the name implies, these antennas have to stay in close contact with the pavement under test.

The *advantage* of these systems is their depth of penetration; several of the lower-frequency systems can penetrate 20 ft under ideal conditions. The higher-frequency systems are superior for many concrete pavement applications such as locating both reinforcing steel and subslab defects such as voids or trapped moisture. The *disadvantage* of these systems is the speed of data collection; when towed behind a vehicle, the maximum speed is around 5 mph. The signals are also noisy, and filtering is required. Substantial training is required to clean up and interpret ground-coupled GPR data.

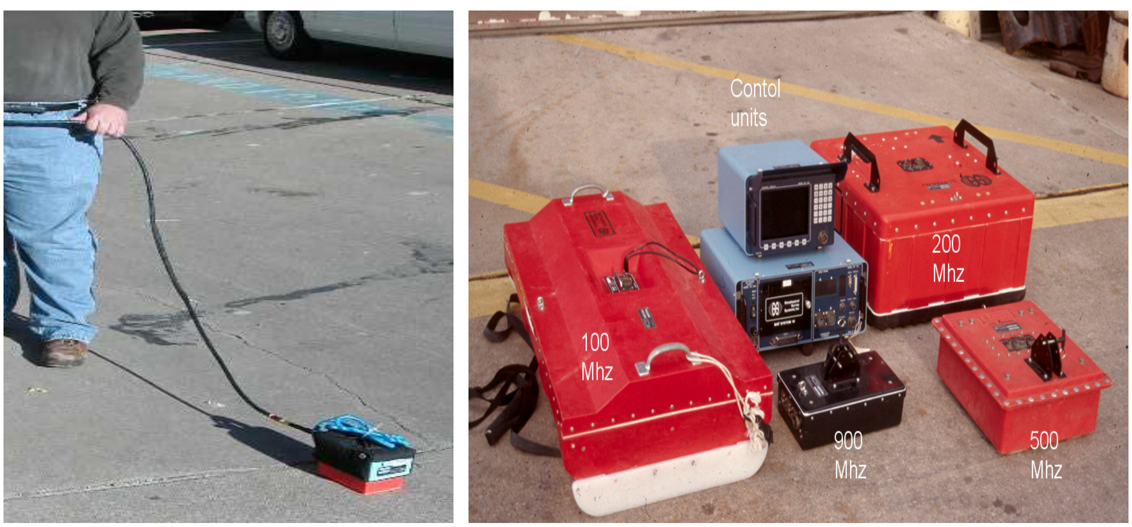


Figure 1.16. *Ground-coupled systems: 1.5 GHz (left); lower-frequency antennas with control unit (right).*
Photos: Tom Scullion.

Analysis Tools

All GPR systems send discrete pulses of radar energy into the pavement and capture the reflections from each layer interface within the structure. Radar is an electromagnetic (EM) wave and therefore obeys the laws governing reflection and transmission of EM waves in layered media. At each interface within a pavement structure, a part of the incident energy will be reflected and a part will be transmitted. It is normal to collect between 30 and 50 GPR return signals per second, which for high-speed air-coupled surveys could mean a trace for every 2 to 3 ft of travel. The captured return signal is often color coded and stacked side-by-side to provide a profile of subsurface conditions; this is analogous to an "x-ray" of the pavement structure. Examples of this process are given later. However, with air-coupled signals as described below, these signals can also be used to automatically calculate the engineering properties of the pavement layers.

Air-Coupled GPR System

A typical plot of captured reflected energy versus time for one pulse of an air-coupled GPR system is shown in Figure 1.17 as a graph of volts versus arrival time in nanoseconds. To understand GPR signals, it is important to understand the significance of this plot.

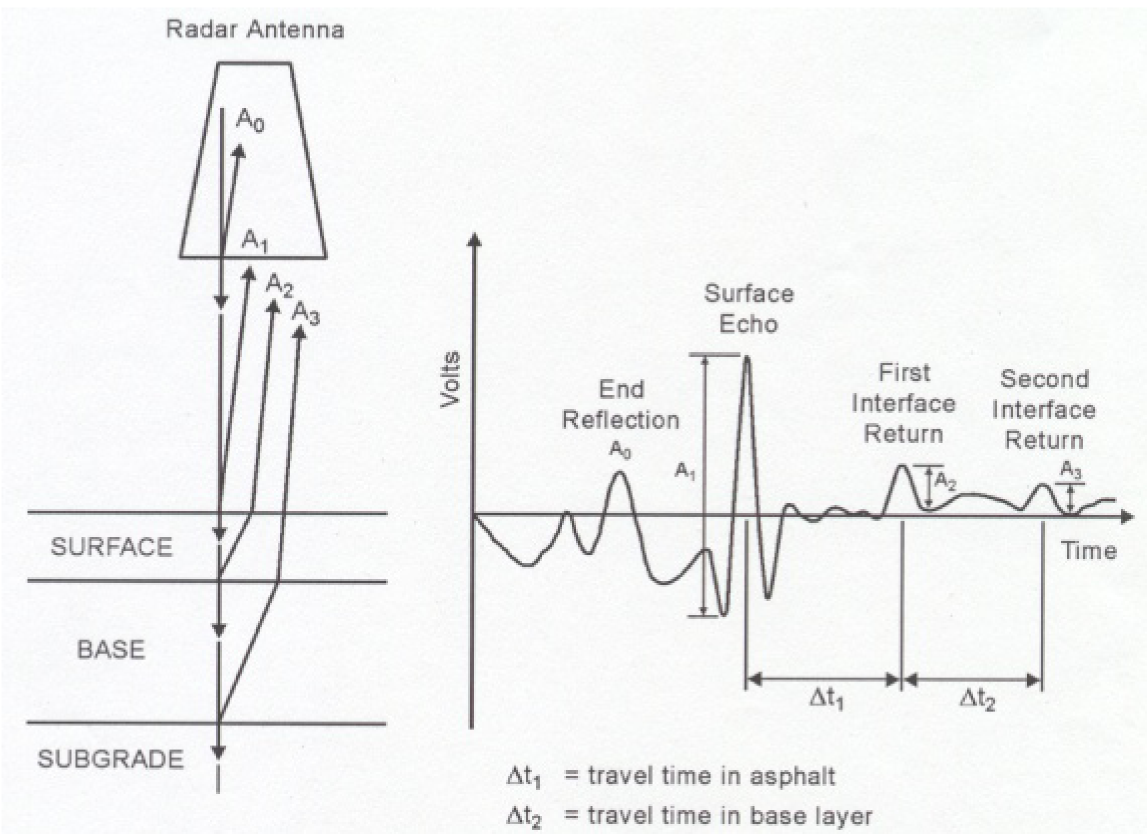


Figure 1.17. *Captured GPR reflections from a typical flexible pavement.*

The reflection, A_0, is known as the end reflection and is internally generated system noise which is present in all captured GPR waves. The more important peaks are those that occur after A_0. The reflection A_1 (in volts) is the energy reflected from the surface of the pavement and A_2 and A_3 are reflections from the top of the base and subgrade, respectively. These are all classified as positive reflections, which indicates an interface with a transition from a low to a high dielectric material (typically low to higher moisture content). These amplitudes of reflection and the time delays between reflections are used to calculate both layer dielectrics and thickness. The dielectric constant of a material is an electrical property that is most influenced by moisture content and density; it also governs the speed at which the GPR wave travels in the layer. An increase in moisture will cause an increase in layer dielectric; in contrast, an increase in air void content will cause a decrease in layer dielectric.

The equations to calculate surface-layer thickness and dielectrics are summarized below:

$$\in_a = \left[\frac{1 + A_1 / A_m}{1 - A_1 / A_m}\right]^2 \tag{1.8}$$

where

$\in_a$ = dielectric of the surface layer,
A_1 = amplitude of surface reflection, in volts (V), and
A_m = amplitude of reflection from a large metal plate (V) (this represents the 100% reflection case; see Figure 1.15 for the metal plate test).

$$h_1 = \frac{cx\Delta t_1}{\sqrt{\in_a}} \tag{1.9}$$

where

h_1 = thickness of the top layer,
c = constant speed of EM wave in air (5.9 in./ns two-way travel), and
Δt_1 = time delay between peaks A_1 and A_2 (ns).

Similar equations are available for calculating the base-layer dielectric and thickness. This calculation process is performed automatically in most operating systems with the end user simply obtaining a table of layer properties.

In most GPR projects, several thousand GPR traces like Figure 1.17 are collected. To conveniently display and interpret this information, color-coding schemes are used to convert the traces into line scans and they are stacked side-by-side so that a subsurface image of the pavement structure can be obtained. This approach is shown in Figure 1.18.

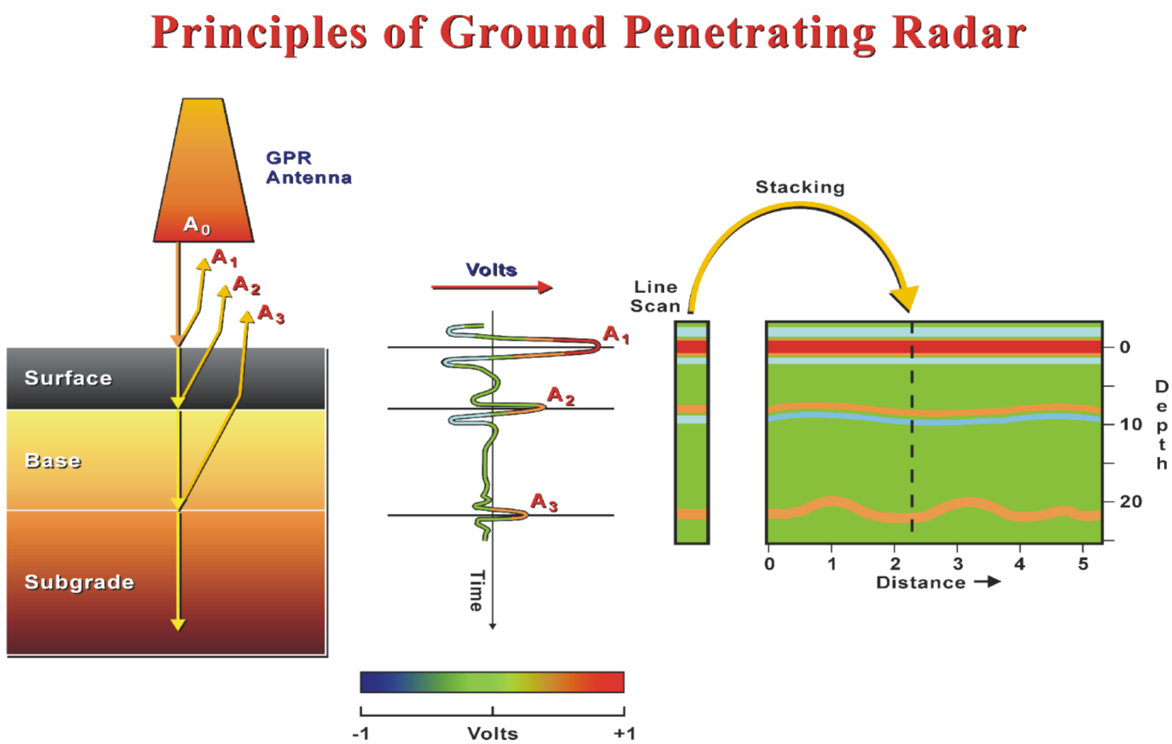


Figure 1.18. *Color coding and stacking individual GPR images.*
Source: Tom Scullion.

The raw GPR image collection is displayed vertically in the middle of Figure 1.18. This image is for one specific location in the pavement. The GPR antenna shoots straight down and the resulting thickness and dielectric estimates are point specific. The single trace generated is color coded into a line scan using the color scheme in the middle of Figure 1.18. In the current scheme, the high positive reflections are colored red and the negatives are colored blue. The green color is used where the reflections are near zero and are of little significance. These individual line scans are stacked so that a display for a length of pavement is developed. Being able to read and interpret these images is critical to effectively using GPR for pavement investigations, to locate section breaks in the pavement structure, and to pinpoint the location of subsurface defects.

An example of a typical GPR display for approximately 3,000 ft by 24 in. deep of a thick flexible pavement is shown in Figure 1.19. This is taken from a section of newly constructed thick asphalt pavement over a thin granular base. In all such displays the *x* axis is distance (in miles and feet) along the section and the *y* axis is a depth scale in inches.

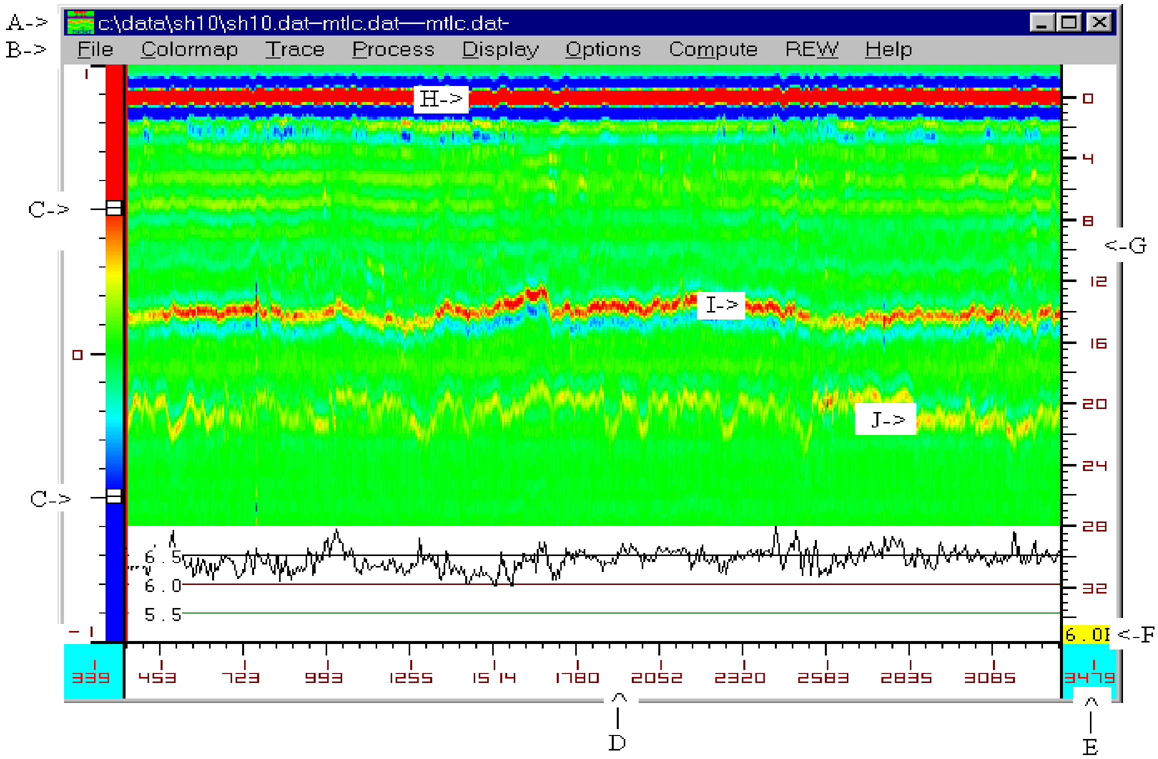


Figure 1.19. *Color-coded GPR traces.*

The labels on Figure 1.19 are as follows: (A) GPR files being used in analysis, (B) main pull-down menu bar, (C) button to define the color-coding scheme, (D) distance scale (miles and feet), (E) end location of data within the GPR file (1 mi and 3,479 ft), (G) depth scale in inches, with the zero (0) being the surface of the pavement, and (F) default dielectric value used to convert the measured time scale into a depth scale. The important features of this figure are the lines marked H, I, and J; these are the reflections from the surface, top, and bottom of the base, respectively. This pavement is homogeneous and the layer interfaces are easy to detect.

When processing GPR data, the first step is to develop displays such as Figure 1.19. From this, it is possible to identify any clear breaks in pavement structure and to identify any significant subsurface defects. The intensity of the subsurface colors is related to the amplitude of reflection; therefore, areas of wet base would be observed as bright red reflections (I).

For many applications, a black-and-white coding scheme is selected. This is widely used during review of data collected with ground-coupled GPR systems. An example of the grayscale image for the pavement shown in Figure 1.20.

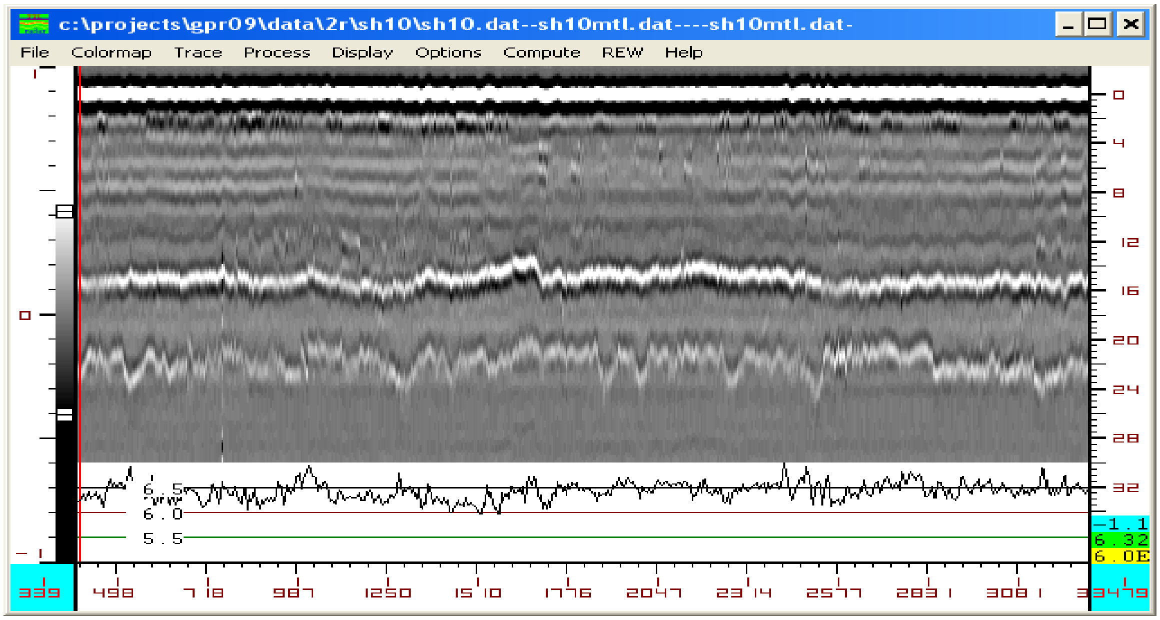


Figure 1.20. *Data similar to Figure 1.19 presented as a grayscale image.*

All the commercially available software packages produce both a color display of the subsurface condition, such as Figures 1.19 and 1.20, together with a table of computed layer thicknesses and dielectrics that is usually exported to Excel. A typical table is shown in Figure 1.21, where Thick1 and E1 are the top-layer thickness and dielectric, respectively.

Trace	Feet	Time1	Time2	Time3	Thick1	Thick2	Thick3	E1	E2	E3
1058	1058	1.6	3.2	0.0	3.8	6.1	0.0	6.2	10.0	11.1
1059	1059	1.5	3.3	0.0	3.7	6.1	0.0	6.2	10.3	11.5
1060	1060	1.5	3.4	0.0	3.6	6.4	0.0	6.2	9.9	10.8
1061	1061	1.4	3.4	0.0	3.4	6.4	0.0	6.3	10.1	10.9
1062	1062	1.4	3.5	0.0	3.5	6.5	0.0	6.2	10.2	11.3
1063	1063	1.4	3.5	0.0	3.4	6.6	0.0	6.2	10.3	11.4
1064	1064	1.4	3.6	0.0	3.4	6.7	0.0	6.2	10.4	11.9
1065	1065	1.4	3.6	0.0	3.3	6.7	0.0	6.2	10.6	11.8
1066	1066	1.4	3.6	0.0	3.4	6.4	0.0	6.3	11.3	12.5
1067	1067	1.4	3.6	0.0	3.5	6.6	0.0	6.2	10.6	12.0
1068	1068	1.4	3.6	0.0	3.5	6.5	0.0	6.3	11.3	12.4
1069	1069	1.5	3.6	0.0	3.5	6.4	0.0	6.1	11.6	12.8
1070	1070	1.5	3.6	0.0	3.6	6.5	0.0	6.1	11.3	12.4
1071	1071	1.5	3.6	0.0	3.6	6.4	0.0	6.0	11.4	12.6

Figure 1.21. *Tabulated thicknesses and dielectric values from GPR data.*

Examples of Analysis of GPR Data for Flexible Pavements

When planning to incorporate the existing pavement as part of a new pavement structure, it is critical to have good information on the existing subsurface layer thicknesses and layer types. A few DOTs maintain good pavement layer databases, but this is not always the case. DOTs often have limited or inaccurate information on existing layer thicknesses. Often, maintenance activities significantly alter the as-constructed pavement structure in localized areas and these activities are often not captured in existing databases.

One popular method of rehabilitating flexible pavements is by the use of full-depth reclamation (FDR) and chemical treatment to incorporate and stabilize the existing pavement to form a solid foundation layer for the new pavement structure. However, because of the failure to account for the variability of the existing pavement in the design phase, several major problems have occurred during construction, or poor pavement performance has resulted. Laboratory designs are based on testing at localized sampling locations, which can miss discrete areas of variable thickness. GPR can help address this issue.

It also must be recalled that processing FWD data, as described in the section "Nondestructive Testing via the Falling Weight Deflectometer (FWD)," requires information about the thickness of the asphalt surface layer. GPR can provide substantial help in analyzing and explaining FWD deflection data.

Three case studies are presented below to demonstrate how GPR can assist in flexible pavement evaluations.

Thickness Profiling for an FDR Application

In many FDR applications, the purpose is to treat the existing pavement to create a stable *uniform* pavement foundation layer for the new pavement structure. In most FDR applications, design samples are taken from the existing pavement and tested in the laboratory to determine the optimal level of either cement or asphalt stabilization to reach a specified target strength. It is therefore important to know that the sampling location selected is representative of the overall project. It is also important to assess if the selected design will be appropriate when variations in layer thicknesses occur.

Figure 1.22 shows variations in asphalt layer thickness for an FDR candidate. At the sample location the structure was 5 in. of asphalt and 10 in. of granular base. Based on laboratory test results, the plan was to recycle to a depth of 10 in., blending 50% asphalt and 50% existing base with 3% cement. However, from a review of Figure 1.22, the average 5 in. of HMA has several noticeable exceptions. The first 800 ft only has 3 in. of asphalt, which is not thought to be a concern. However, for about 2,000 ft, the total HMA thickness is over 12 in. From previous experience, the 3% cement treatment does not work with 100% reclaimed asphalt pavement (RAP). In these locations it was necessary to modify the construction plan, wherein 5 in. of the existing HMA was milled and replaced with 5 in. of new base. In that way, the FDR process can continue and in all locations the as-designed 50/50 blend can be treated with cement.

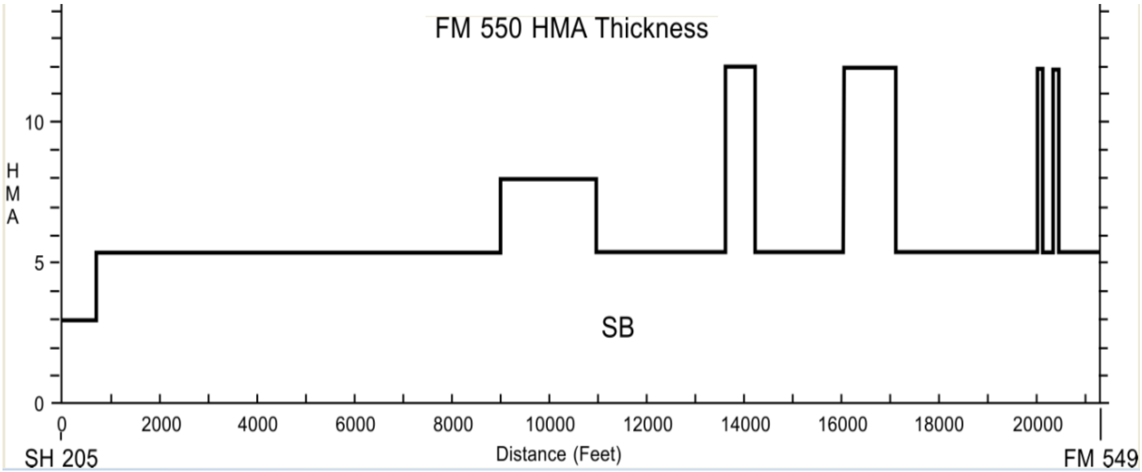


Figure 1.22. *Surface thickness variations from GPR profiling on FM 550.*

Defect Detection Prior to Pavement Rehabilitation

In many cases, long life of the existing flexible pavement can be achieved by simply adding a structural overlay to the existing structure. This process works well provided there are no major defects in the existing HMA layer or flexible base layer. GPR has shown that it can be used to detect stripping problems in HMA layers and areas where the existing base layer is holding moisture. It must be recalled that GPR traces are collected frequently at 2- to 3-ft intervals, so very precise location of defects is possible. The GPR color-coded profile shown in Figure 1.19 is from a thick HMA section with no defects. This should be contrasted with the GPR profile shown in Figure 1.23. This again is a thick HMA section, but in this case there are strong reflections from within

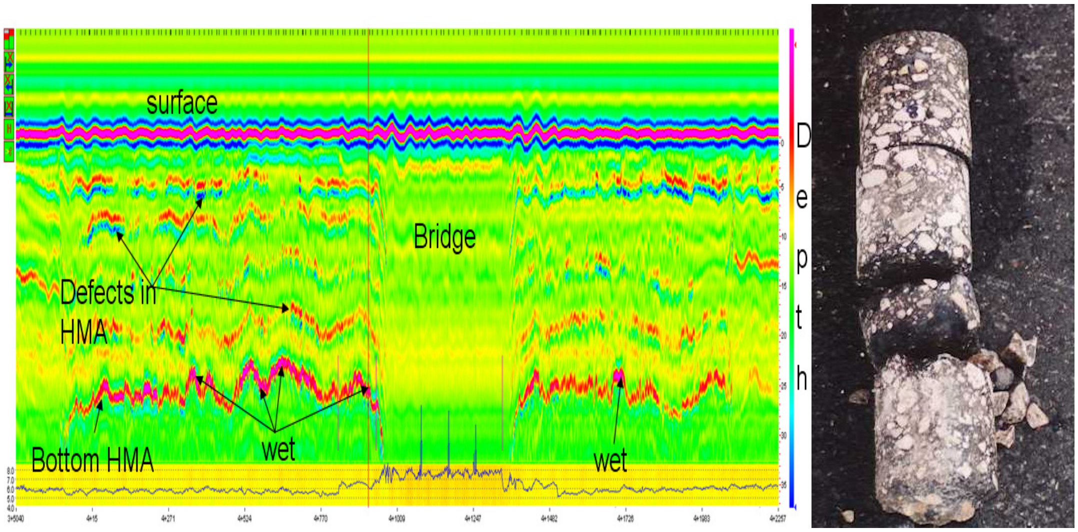


Figure 1.23. *Using GPR to identify defects in surface and base layers.*

the HMA and very strong reflections from the bottom of the layer. The red and blue reflections from within the HMA are associated with deteriorated areas where moisture is trapped. When these deteriorated areas are close to the surface, they can severely affect long-term performance.

Although the presence of defects in either HMA or base layers can be easily detected by GPR, the severity of these defects will need to be confirmed by localized coring. This is valuable input to the pavement designer, who has to make a decision that affects the future anticipated performance of the proposed section. If the defects are very localized, then full-depth milling can be used in these areas.

Section Uniformity

With many older pavements, particularly those involving some form of pavement widening, the existing pavement structure can be very variable. It is important to identify the different structures in order to explain the cause of current conditions and to design future repairs.

Such a case is shown in Figure 1.24. This is a 1.8-mi section and the entire section had received a thin overlay. However, the first part of the section was performing poorly. A GPR survey was undertaken and from the display it was clear that this section had three distinct pavement structures. Structure A was a thin HMA pavement over a flexible base, structure B was thick HMA, and structure C was a road built on top of an existing roadway. This type of subsurface mapping can clearly help designers with their rehabilitation designs.

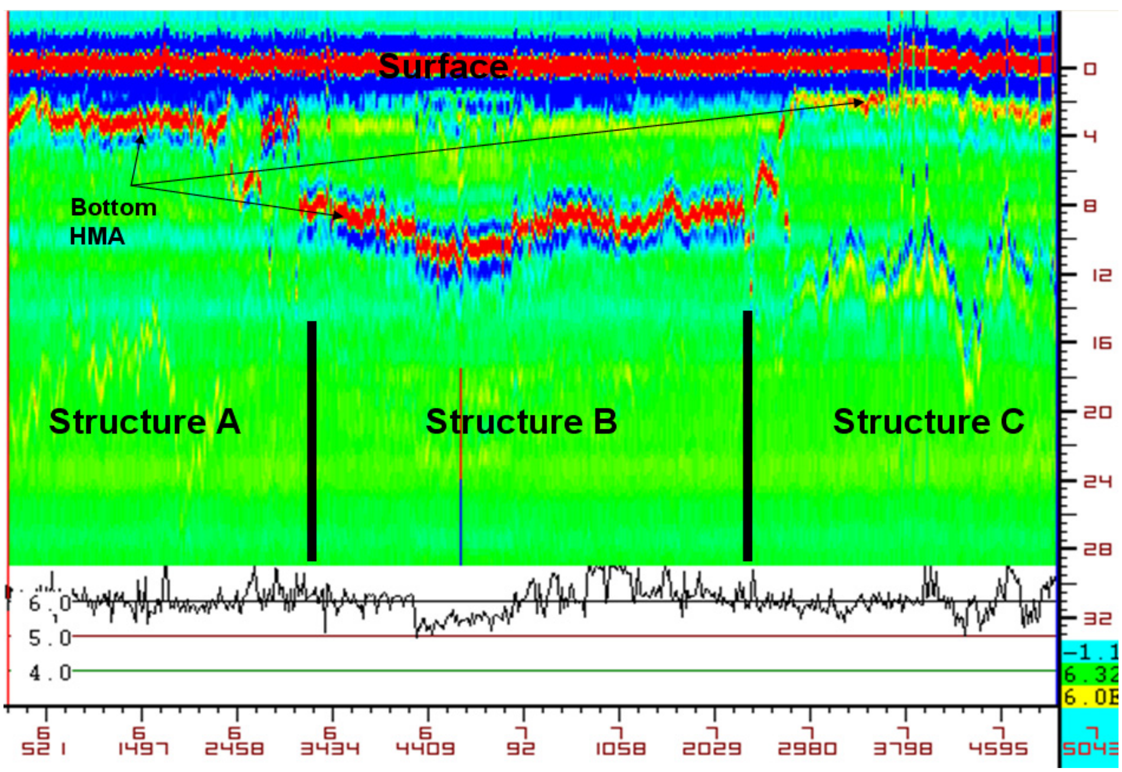


Figure 1.24. *Using GPR to map subsurface variability.*

Examples of Analysis of GPR Data for PCC Pavements

The most popular applications of GPR in evaluating concrete pavements when making pavement rehabilitation decisions are (a) measuring slab thickness, (b) detecting the presence and depth of reinforcing steel, and (c) identifying problems beneath the slab such as voids or trapped moisture. In several instances, especially for steel detection, the ground-coupled systems perform better than the air-coupled systems. The high-frequency ground-coupled systems, such as the 1.5-GHz unit shown in Figure 1.16, can give more focus and better target resolution than air-coupled units. Several case studies are shown below.

Rebar Detection

The Geophysical Survey Systems, Inc. (GSSI) "Handbook for Radar Inspection of Concrete" (GSSI, 2006) has some good examples on rebar detection. Figure 1.25 shows the typical GPR signature obtained over reinforcing steel. There is a hyperbola shape and the top of the hyperbola is the location of the steel. The surface of the concrete is the "direct couple" signature, and the depth between the surface and the top of the hyperbola is the depth of the concrete cover. GSSI also claims that the size of the rebar can be determined by the shape of the hyperbola.

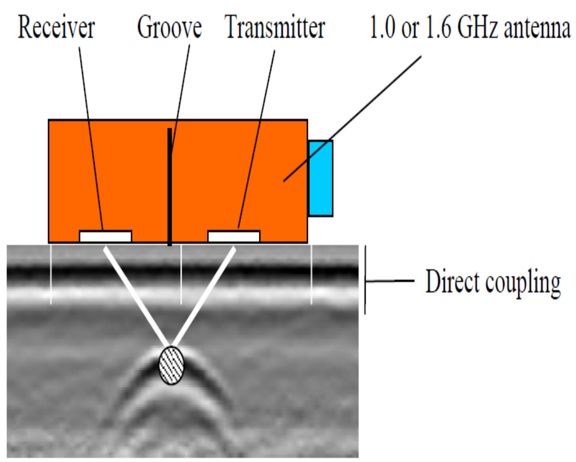


Figure 1.25. *Ground-coupled GPR signals from steel in concrete.*
Source: GSSI.

By moving the GPR antenna slowly across the surface of the concrete, it is possible to map different layers of steel and the bottom of the concrete slab as shown in Figure 1.26.

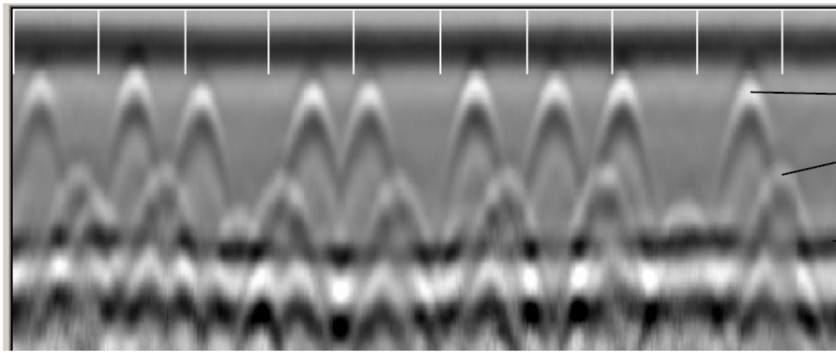


Figure 1.26. *Mapping multiple layers of steel in concrete.*
Source: GSSI.

Void Detection

Detecting thin air voids (which can be very detrimental to slab performance) with air-coupled GPR is often problematic. Controlled studies have found that air voids less than 0.75 in. thick cannot be readily detected with air-coupled GPR. However, if the voids are larger or if they are moisture filled, then they can readily be detected. An example of a GPR color profile for an 8-in. PCC slab with water-filled voids is shown in Figure 1.27. The strong reflections (red areas) indicate locations of trapped water.

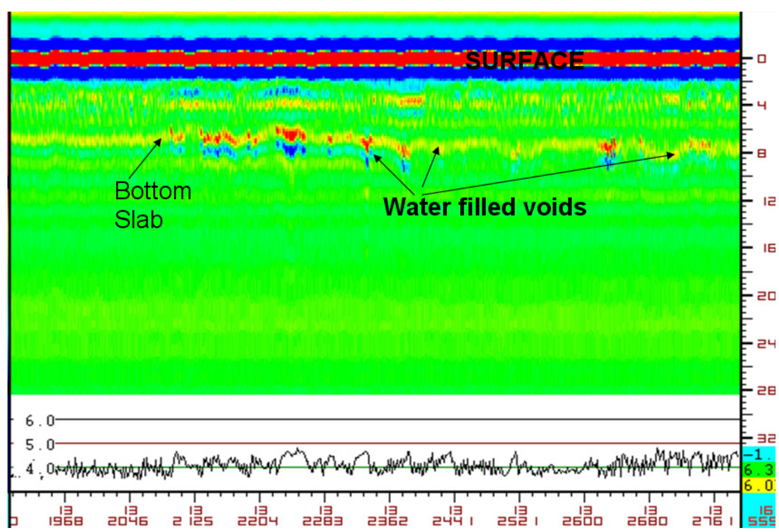


Figure 1.27. *Mapping subslab water-filled voids with GPR.*

Deep Investigations of Subslab Conditions with GPR

Lower-frequency ground-coupled GPR can be used to investigate deep beneath concrete pavements to identify changes in support conditions and possibly to help explain the occurrence of surface distress. Figure 1.28 shows the color profile from a 400-MHz

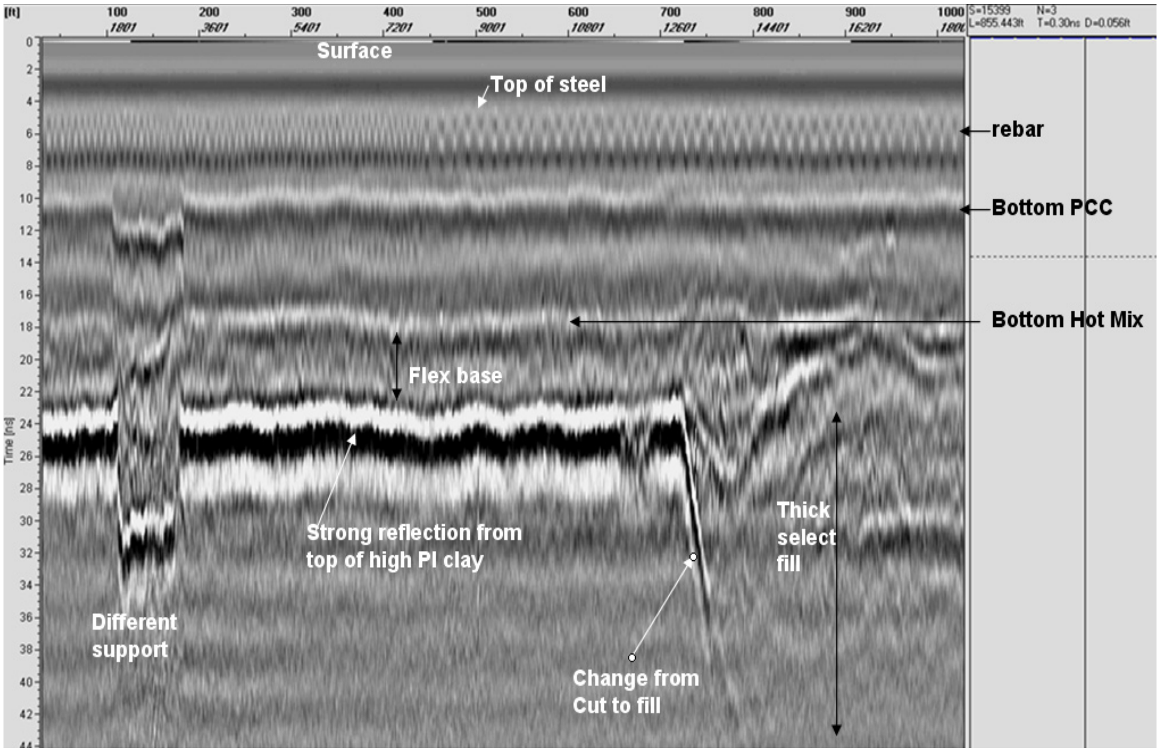


Figure 1.28. *Mapping concrete pavement structure with GPR.*

ground-coupled system. The entire pavement system and changes in pavement support can be observed. The transverse rebar can be seen toward the top of the figure. The steel is more closely spaced in the left of the figure. The anomaly on the left is a culvert. The bottom of the slab is indicated. There is a clear change in subgrade support at the top of the subgrade, showing the transition from a cut to a fill area.

Implementing GPR Technology for Pavement Evaluation

GPR is an excellent technology for inspecting pavements when pavement rehabilitation decisions are being made. Many case studies have been presented over the past two decades, but widespread implementation of the technology has been slow. There are several factors causing this and they are discussed in this section. The main factors are the following:

1. The FCC banned 1-GHz air-coupled systems in 2002 (these units can be purchased in any country worldwide except the United States). For the past decade, most air-coupled GPR systems have been performed with systems built before 2002. Only recently have commercial systems such as GSSI's 2.2-GHz system become available.
2. There is a lack of understanding about what GPR can and cannot do; in many cases the technology was oversold.

3. Current data-processing software are inadequate, and there is a lack of end-user training.

Agencies undertaking GPR implementation should be aware of the following issues, which must be resolved before GPR can be implemented as a routine pavement inspection tool.

1. Need for GPR hardware specifications,
2. Need for data-collection software specifications,
3. Training and specifications for data-collection activities,
4. Specifications and software for processing and interpreting GPR signals,
5. End-user training, and
6. Specifications for output formats and data storage systems.

Several DOTs have implemented GPR technology in house (e.g., the Florida DOT and the Texas DOT), but most agencies get GPR services from consultant companies. Selecting the best vendor can also be a problem.

Obtaining GPR Services

The AASHTO publication has a short section with recommendations for agencies on hiring GPR consultants. In initiating contracts, the agency has to be convinced that

- The consultant has quality equipment. The agency should ask the consultant to run their equipment against the performance specs (which are available).
- The consultant has good data-processing skills. References from existing customers will help here. GPR interpretation should never be done without taking limited field verification cores early in the project. If the project is for layer-thickness determination or for defect detection, it should be simple to set up a verification system early in the project.

Barriers to GPR Implementation

In addition to the FCC requirements, there are also several common misconceptions that must be overcome before any agency will adopt GPR technology. These include the following:

- *GPR is only for layer-thickness determination: My state has good as-built records so we do not need GPR.* As noted throughout this report, GPR is much more than a thickness-measuring tool. It provides information on the quality of existing structures and helps explain the causes of pavement distresses. Distresses are often associated with moisture ingress into pavement layers. GPR signals are highly sensitive to moisture in any layer.
- *GPR systems are too expensive.* A complete air-coupled system described in this section costs around $100,000 for the complete turnkey system, including the vehicle. Ground-coupled systems cost approximately $60,000. Compared to the costs of pavement rehabilitation activities, GPR costs are minimal.

- *GPR is a black box that is impossible to understand.* This is not true; the basics of GPR are simple. The key here is that agency personnel should attend training schools to understand this technology. Even if the plan is to initiate GPR work through consultants, the agency personnel need to have a basic understanding of what this technology can and cannot do.
- *Our first experience with GPR was disappointing.* This is often true. In the early 1990s, a host of companies sold GPR services. They sometimes made extensive claims on GPR's potential and their ability to successfully interpret the signals. Many claimed to be able to find thin voids beneath concrete pavements, often to disappoint the DOT when validation field cores were taken. In some cases, the vendors did not have adequate software or interpretation skills. The key here again is training for end-user agency personnel. The AASHTO publication also is a good source to identify applications that have a high probability of success.
- *When the agency initiates a GPR program, a host of vendors make claims about their capabilities and it is impossible for the agency to judge their merits.* This is often true, but it can be overcome by training of end-user agency personnel prior to initiating a program. Also, as with any new technology, field verification of any predictions must be a critical part of any program. GPR will not eliminate coring, but it will greatly reduce the number of cores.

PAVEMENT CORES

Purpose

This section overviews pavement cores and how they can be used to aid pavement assessment decisions. Much of pavement analysis and understanding stems from knowledge of layer thicknesses, types of materials, and condition.

Measurement Method

This subsection briefly overviews both the frequency of sampling and the organization of data from pavement cores. Pavement coring not only reveals much about the existing pavement structure, but it also allows for use of the DCP. Knowing the HMA layer thickness to within ¼ in. is essential in ensuring a more accurate prediction of layer moduli if a backcalculation procedure is used.

The number of cores obtained will depend on project-specific conditions; however, a reasonable rule of thumb is to obtain a core at every 5th or 10th FWD test location. If the pavement thicknesses are found to vary substantially (not probable, but this can be the case), then cores should be obtained at every FWD test location in those vicinities. FWD area values plotted along the project limits [as discussed in the section "Nondestructive Testing via the Falling Weight Deflectometer (FWD)"] provide good guidance for determining core locations because substantial changes in the pavement structure can be identified. If GPR data are collected, using the layer profiles in conjunction with FWD area values also provides very good guidance for developing coring plans. Calibration cores for GPR data collection can also be used for other assessments.

Typical core diameters are either 4 or 6 in. Coring should also be used to verify the depth of cracking (i.e., determination of top-down versus bottom-up cracking) as well as the presence and severity of stripping in HMA mixtures.

Analysis Tools

This subsection focuses on how to organize pavement core data to aid decision making.

Core data should be organized similarly to the example data shown in Table 1.23. Additionally, the location of each core in the lane should be recorded (such as centerline, left wheelpath, between wheelpath, right wheelpath, and outside pavement edge).

TABLE 1.23. ORGANIZATION OF PAVEMENT CORE DATA

Core Location (milepost)	Depth		Comments (Cores should be taken frequently at cracks, if they exist, to determine if the crack is full depth or partial depth)
	HMA (in.)	Base (in.)	
207.85	5.3	18.0	Core taken at a crack, crack is full depth
208.00	6.0	18.0	Core taken at a crack, core not intact
208.50	4.7	12.0	Core taken at a crack, crack is full depth
209.00	4.6	12.0	Very fatigued, core broke into several pieces

DYNAMIC CONE PENETROMETER (DCP)

Purpose

This section overviews the DCP and how it can be used to aid pavement assessment decisions.

Measurement Method

This subsection describes the DCP device. The standard test method is:

- ASTM D6951-03: Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

From ASTM D6951: “This test method is used to assess in situ strength of undisturbed soil and/or compacted materials. The penetration rate of the 8-kg DCP can be used to estimate in situ CBR (California Bearing Ratio), to identify strata thickness, shear strength of strata, and other material characteristics. The 8-kg DCP is held vertically and therefore is typically used in horizontal construction applications, such as pavements and floor slabs. This instrument is typically used to assess material properties down to a depth of 1000-mm (39-in.) below the surface. The penetration depth can be increased using drive rod extensions. However, if drive rod extensions are used, care should be taken when using correlations to estimate other parameters since these correlations are only appropriate for specific DCP configurations. The mass and inertia of the device will change and skin friction along drive rod extensions will occur.”

"The 8-kg DCP can be used to estimate the strength characteristics of fine- and coarse-grained soils, granular construction materials and weak stabilized or modified materials. The 8-kg DCP cannot be used in highly stabilized or cemented materials or for granular materials containing a large percentage of aggregates greater than 50-mm (2-in.). The 8-kg DCP can be used to estimate the strength of in situ materials underlying a bound or highly stabilized layer by first drilling or coring an access hole."

An illustration of a standard DCP is shown in Figure 1.29.

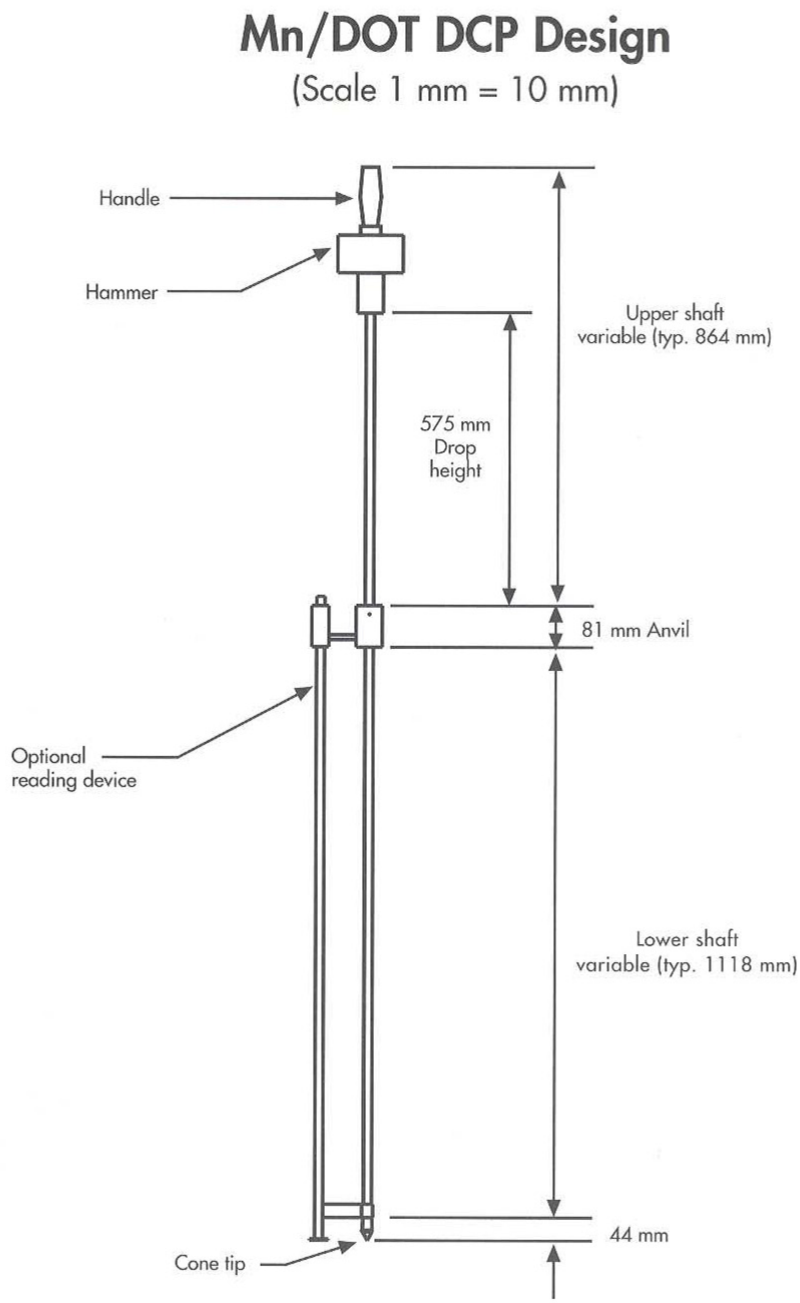


Figure 1.29. *Minnesota DOT DCP.*
Source: Minnesota DOT, 1993.

Analysis Tools

DCP test results are typically expressed in terms of the DCP penetration index (DPI), which is the vertical movement of the DCP cone produced by one drop of the hammer. This is expressed as either mm/hammer blow or in./hammer blow (Minnesota DOT, 1993).

Basic Correlation

A common correlation with DCP data is to estimate the California bearing ratio (CBR) of unstabilized materials in a pavement structure. The following is a correlation developed by the U.S. Army Corps of Engineers (Webster, Grau, and Thomas, 1992):

$$\log CBR = 2.46 - 1.12 \log(DPI) \text{ or } CBR = 292/DPI^{1.12}$$

where DPI = mm/blow.

Table 1.24 shows typical CBR and DPI ranges for three soil types (Minnesota DOT, 1993).

TABLE 1.24. SOILS TYPES, CBR VALUES, AND DPI

Soil Type	CBR Range (%)	DPI Range (mm/blow)
Clay (CL)	~1–14	15–127
Sand (S-W)	14–39	6–15
Gravel (G-W)	47–95	2.7–5

Note: The table was modified by the authors of this document so that the DPI and CBR correlation matched.

Typical Results

Burnham (1997) described an extensive set of DCP measurements on the subgrade soils and base materials used in the various test sections at the MnRoad facility. These are summarized in Table 1.25. Following this work, the following DPI limits were recommended for use by MnDOT personnel when analyzing DCP results for rehabilitation studies: silty/clay materials, DPI <25 mm/blow; select granular materials, DPI <7 mm/blow; and Class 3 special gradation materials, DPI <5 mm/blow.

Subgrade Stability

The Illinois DOT (1982, 2005) has used the DCP to check subgrade stability. The purpose of this is straightforward—they want to know if the subgrade is stable enough to avoid excessive rutting and/or shoving during and following construction activities. The subgrade immediate bearing value (IBV) can be estimated from the DPI. The IBV is similar to the CBR, "except that IBV testing is conducted on a 4-inch molded sample instead of the CBR's 6-inch sample . . . further, the penetration test for determining the IBV is conducted immediately after compaction instead of waiting 96 hours—thus IBV and CBR are similar but not identical" (Illinois DOT, 2005). Figure 1.30 shows the relationship between unsoaked CBR (actually IBV), DPI, and required thickness of

TABLE 1.25. MINNESOTA DCP RESULTS FOLLOWING PLACEMENT OF THE BASE COURSE

Material	DPI Average (mm/blow) (SD) 0–12 in. depth	DPI Average (mm/blow) (SD) 12–24 in. depth	DPI Average (mm/blow) (SD) 24–36 in. depth
Clay/Silt Location 1	11 (3)	21 (7)	21 (7)
Clay/Silt Location 2	14 (6)	18 (5)	16 (5)
Clay/Silt Location 3	12 (5)	20 (7)	15 (7)
Sand	5 (2)	5 (1)	6 (2)
Base Course	4 (2)	3 (1)	3 (<1)

Note: DPI average values were rounded to the nearest whole number.

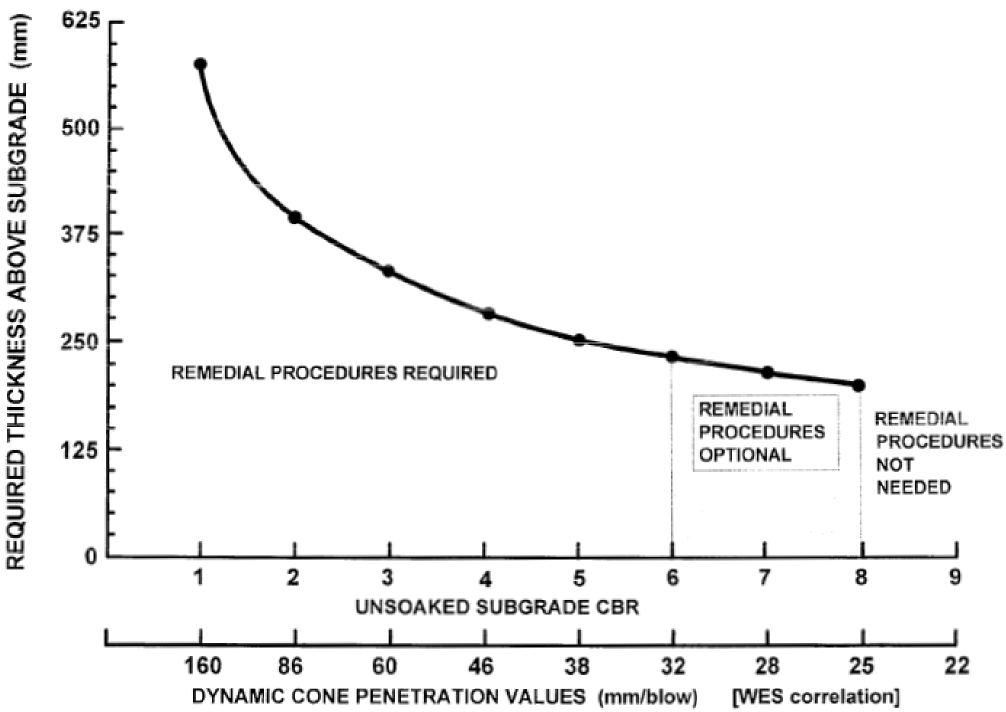


Figure 1.30. *DCP-based thickness design for granular backfill and subgrade modification for the Illinois DOT.*
Source: Burnham, 1997; checked against Illinois DOT, 2005.

remedial measures. Remedial measures can include the addition of granular backfill or subgrade modification such as lime stabilization.

The Illinois DOT DCP results and those from the Minnesota DOT broadly agree in that subgrade DPI values greater than 25 mm/blow are of concern.

Use of DCP Data in Renewal Decisions

The Texas Transportation Institute developed guidelines for the Texas DOT as to conditions suitable for rubblizing existing rigid pavements (Figure 1.31). The "High Risk" portion of the figure implies the pavement is not a good candidate for rubblization because the supporting base and subgrade is excessively weak. Figure 1.31 is similar to, but modified from, similar guidelines developed for Illinois (Figure 1.32). Figure 1.32 is of interest because it includes data obtained by Sebesta and Scullion (2007) for US-83 in Texas plotted by total pavement thickness versus DCP-derived CBR values.

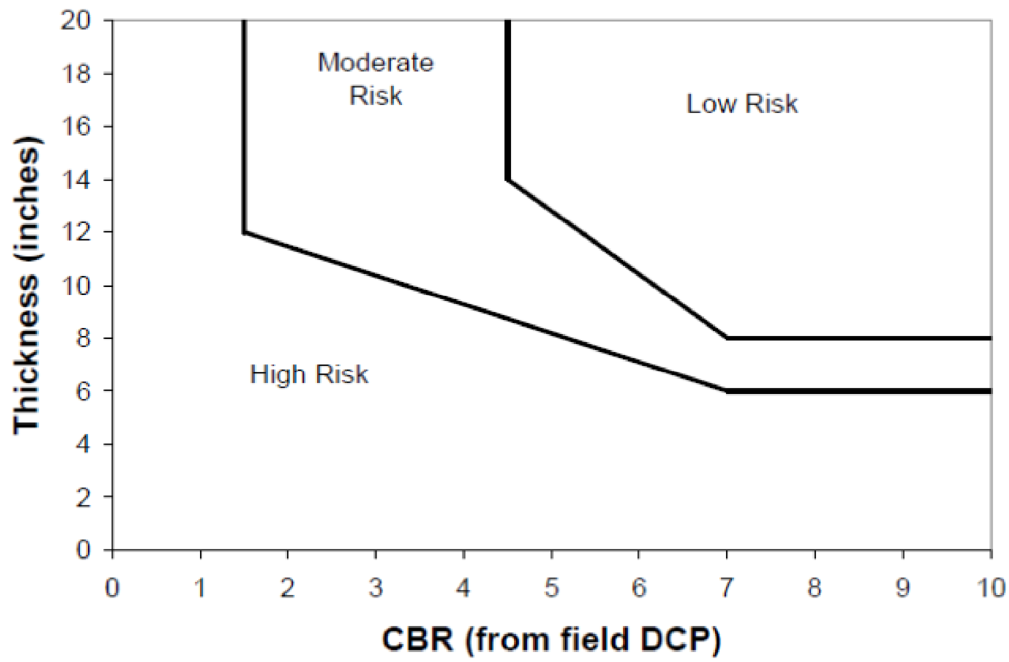


Figure 1.31. *Rubblization selection chart developed by TTI.*
Source: Sebesta and Scullion, 2007.

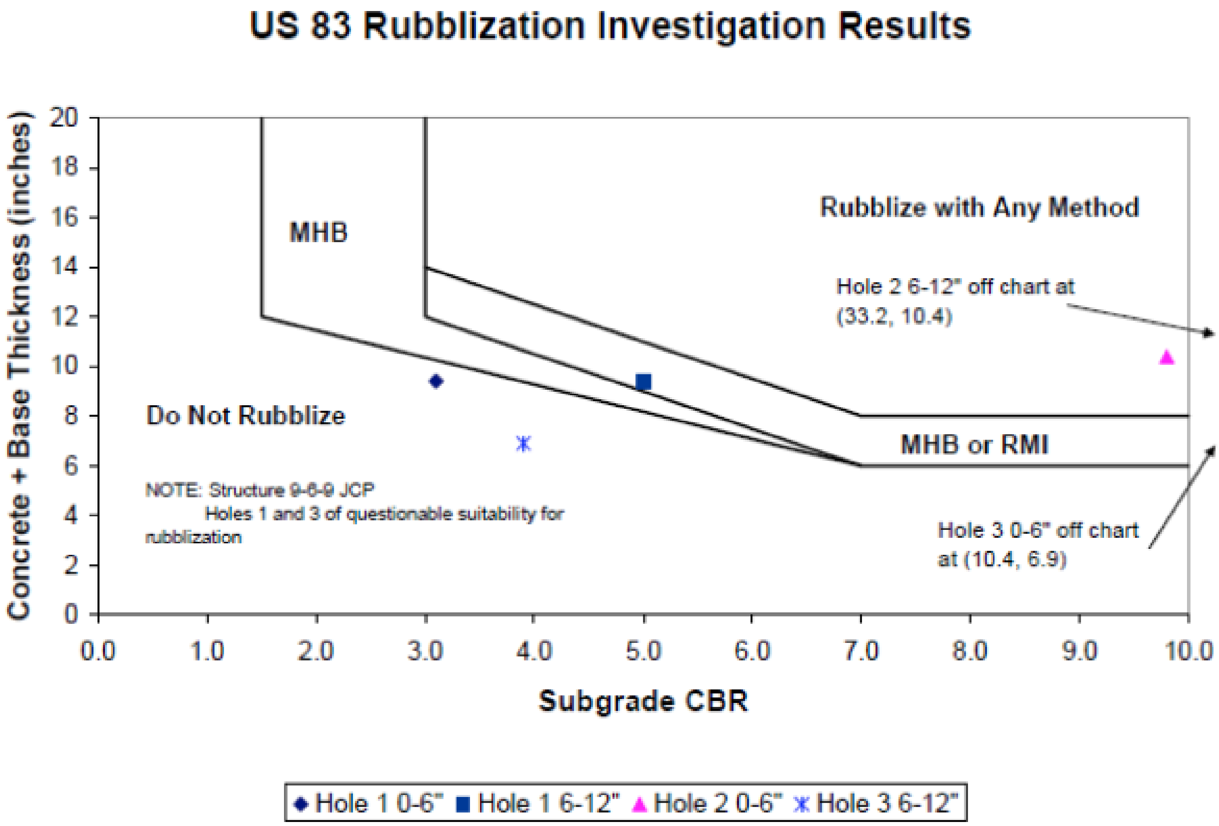


Figure 1.32. *Illinois rubblization selection chart with data from US-83 (Texas).*
Source: Sebesta and Scullion, 2007; original Illinois DOT criteria from Heckel, 2002.

SUBGRADE SOIL SAMPLING AND TESTS

Purpose

This section overviews selected elements associated with subgrade soils and what information is needed to make pavement assessment decisions. Much of pavement analysis and understanding stems from knowledge of layer thicknesses, types of materials, and condition.

Measurement Methods

This subsection shows both the types of tests and the frequency of sampling associated with subgrade soils. A summary of these tests is contained in Table 1.26.

TABLE 1.26. SUMMARY OF TYPICAL SUBGRADE TESTS

Subgrade Test	Standard Test Method	Purpose of Test
Soil classification	ASTM D2487-00, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)	Soil classification is basic information that can be used to estimate various design-related parameters. The required tests for classification can be used for other determinations (gradation, Atterberg limits).
California bearing ratio	ASTM D1883-07e2, Standard Test Method for CBR of Laboratory-Compacted Soils	Straightforward test for determining relative shear strength of the subgrade soils. CBR can be estimated from a laboratory test or through correlations with devices such as the DCP (see section "Dynamic Cone Penetrometer (DCP)" in this chapter). Caution is needed because laboratory- and field-produced CBRs can have quite different moisture conditions and, hence, different results.
Resilient modulus—laboratory	AASHTO T307, Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials	If subgrade soil samples are available, laboratory resilient modulus determinations can be made. Triaxial testing is expensive and the results are a function of sample preparation.
Resilient modulus—NDT	ASTM D4694-96, Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device	The preferred test apparatus for nondestructive testing of pavement structures is the FWD (see section "Nondestructive Testing via the Falling Weight Deflectometer (FWD)" in this chapter). Straightforward methods for estimating M_R are available (same section), or backcalculation procedures allow up to three pavement layers to be estimated.

Analysis Tools

Questions that need to be answered for the project assessment regarding subgrade soils include the following:

- How well do the subgrade soils support the existing pavement structure?
- Are the subgrade soils frost susceptible (if the project is located within a potential freezing zone)?
- Are the subgrade soils subject to expansion and contraction (such as expansive clay soils)?
- Are groundwater issues associated with the project site?

Support for Existing Pavement Structure

The support for the existing pavement structure can be estimated through a combination of laboratory or nondestructive testing—but most likely it will be NDT. A set of FWD deflection basins, pavement coring, and DCP measurements is generally sufficient, along with use of the analysis tools provided in the preceding sections.

Frost Susceptibility

Both sophisticated and very straightforward soils tests are available for estimating the likelihood of subgrade soil frost susceptibility. The basic issue is the potential for the creation of ice lenses under the existing pavement and the resulting loss of support when it all thaws out. When ice lenses form in frost-susceptible soils, large volume changes can occur (just liquid water changing to ice increases the volume by 9%). An illustration of ice lenses in pavements in shown in Figure 1.33.

A basic approach for assessing frost susceptibility is based on gradation, and it has been in use for almost 80 years. Casagrande noted the following in 1932 (taken from Terzaghi and Peck, 1967): "Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than 3% of grains smaller than 0.02 mm. . . . No ice segregation was observed in soils containing less than 1% of grains smaller than 0.02 mm, even if the groundwater level is as high as the frost line."

To determine the percent passing 0.02 mm requires a hydrometer test. A reasonable approximation of 3% passing 0.02 mm is about 7% passing a 0.075 mm (No. 200 sieve).

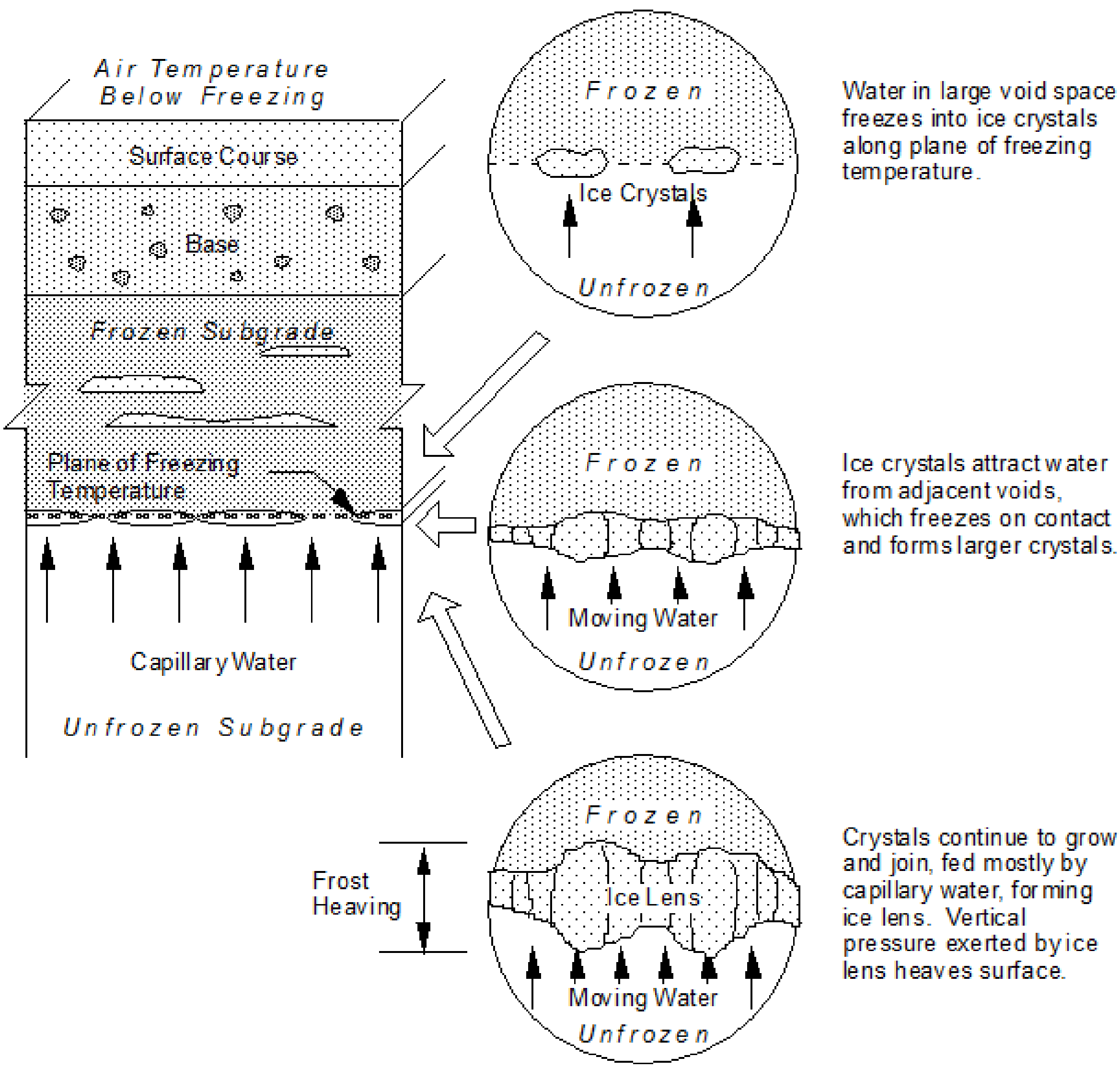


Figure 1.33. *Formation of ice lenses in a pavement structure.*

Another tool that can aid decisions about the potential frost susceptibility of a subgrade soil is to use the U.S. Army Corps of Engineers classification system for frost design, as shown in Table 1.27.

Expansion and Contraction

If these types of soils are present, attempt to answer the following:

- Were the subgrade soils previously treated with materials such as lime?
- Is the profile of the existing pavement stable?

Groundwater Issues

When groundwater issues are apparent, investigation by a geotechnical engineer may be required.

TABLE 1.27. U.S. ARMY CORPS OF ENGINEERS FROST DESIGN SOIL CLASSIFICATION

Frost Group	Soil Type	Percentage Finer than 0.02 mm by Weight (%)	Typical Soil Types under Unified Soil Classification System
Nonfrost susceptible (NFS)	(a) Gravels, including crushed stone and crushed rock	0–1.5	GW, GP
	(b) Sands	0–3	SW, SP
Potentially frost susceptible (PFS)	(a) Gravels Crushed stone Crushed rock	1.5–3	GW, GP
	(b) Sands	3–10	SW, SP
S1	Gravelly soils	3–6	GW, GP, GW-GM, GP-GM
S2	Sandy soils	3–6	SW, SP, SW-SM, SP-SM
F1	Gravelly soils	6–10	GM, GW-GM, GP-GM
F2	(a) Gravelly soils	10–20	GM, GW-GM, GP-GM
	(b) Sands	6–15	SM, SW-SM, SP-SM
F3	(a) Gravelly soils	>20	GM, GC
	(b) Sands, except very fine silty sands	>15	SM, SC
	(c) Clays, PI >12	—	CL, CH
F4	(a) All silts	—	ML, MH
	(b) Very fine silty sands	>15	SM
	(c) Clays, PI <12	—	CL, CL-ML
	(d) Varved clays and other fine-grained, banded sediments	—	CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

Note: Table after U.S. Army, 1990, and NCHRP Synthesis 26, 1974.

TRAFFIC LOADS FOR DESIGN

Purpose

This section overviews the use of basic traffic information to estimate loadings for pavement design. The fundamental parameter to be estimated is the equivalent single axle load (ESAL). More detailed assessments of traffic loading such as load spectra used in the Mechanistic-Empirical Pavement Design Guide (MEPDG) are not needed for use in these guidelines.

Measurement Method

This subsection overviews the kind of traffic information needed to quickly estimate future ESALs.

Tire Loads and Terminology

Typical truck and bus axles are shown in Figure 1.34, which illustrates single and tandem axles with either single or dual tires.

States generally have regulations limiting the allowable load per inch of tire width. This tire load limitation varies from a high of 800 lb/in. to a low of 450 lb/in. The primary impact of such state laws has to do with the use of dual or single tires on a specific axle and steer axles.

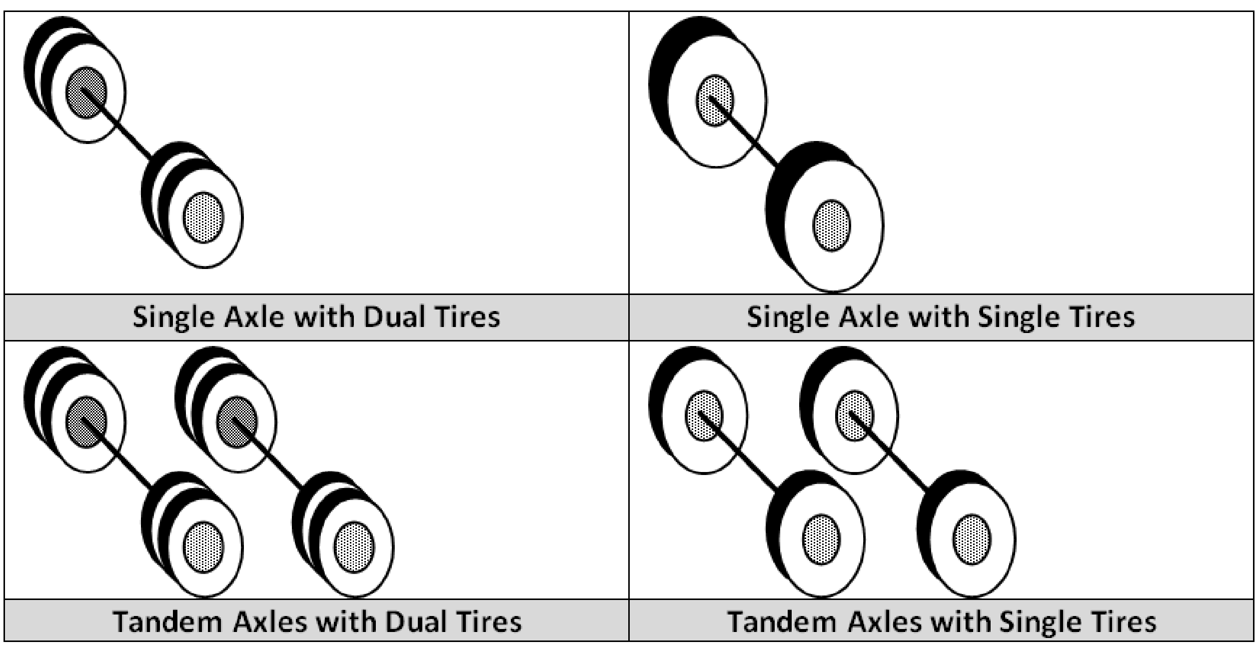


Figure 1.34. *Illustration of typical axle and tire configurations.*

Typical Federal and State Axle Load Limits

Typical federal and state axle load limits are as follows:

- Single axles, 20,000 lb;
- Tandem axles, 34,000 lb; and
- Total truck gross weight, 80,000 lb.

FHWA Bridge Formula

A major additional limitation on U.S. trucks and buses is the FHWA bridge formula. The total gross weight in pounds imposed on the pavement by any group of two or more consecutive axles on a vehicle or combination of vehicles shall not exceed that weight calculated by use of Equation 1.10 below. The bridge formula is needed since an individual set of bridge design computations cannot be performed for every type of truck that may use highways.

Bridge designers use a standard design vehicle for estimating critical stresses, strains, or deflections in a bridge structure. This vehicle is designated HS-20-44 and has been referred to as an umbrella loading. Federal law requires its use in bridge design for the Interstate system.

In effect, the bridge formula helps to ensure bridges are not "overstressed" due to an almost infinite number of truck-axle configurations and weights. The equation is

$$W = 500(NL/(N-1) + 12N + 36) \tag{1.10}$$

where

W = maximum weight on any group of two or more consecutive axles to the nearest 500 lb,

L = distance between the extremes of any group of two or more consecutive axles (ft), and

N = number of axles in the group under consideration.

To illustrate, an example is a five-axle truck with a 51-ft separation from the steer axle to the rear portion of the back tandem. The total vehicle allowable gross weight, via the bridge formula, is then $W = 500(5(51)/(5-1) + 12(5) + 36) = 80{,}000$ lb.

Repetitions of Wheel Loads and Equivalent Single Axle Loads (ESALs)

To compute ESALs, we must be able to convert wheel loads of various magnitudes and repetitions ("mixed traffic") to an equivalent number of "standard" or "equivalent" loads for design purposes. The most commonly used equivalent load is 18,000 lb (80 kN) equivalent single axle loads (normally designated ESAL). The ESAL standard axle load is used in the AASHTO "Guide for Design of Pavement Structures" (AASHTO, 1993).

Wheel load equivalency has been one of the most widely adopted results of the AASHO Road Test (1958 to 1960) and has provided a method to relate relative damage attributed to axles of different type (single and tandem) and weight. Highway design in most states is based on the ESAL traffic input anticipated over a future 10- to 50-year period.

The relationship between repetitions is not arithmetically proportional to the axle loading. Instead, a 10,000-lb single axle needs to be applied to a pavement structure *many more* than 1.8 times the number of repetitions of an 18,000-lb single axle to have the same effect; in fact, it must be applied *more than 12 times*. Similarly, a 22,000-lb single axle needs to be repeated less than half the number of times of an 18,000-lb single axle to have an *equivalent* effect. A sample of ESAL load equivalency factors (LEFs) is shown in Table 1.28.

A basic element in estimating the future ESALs for a specific project is to forecast the truck and bus volumes for the design (and analysis) period. Once this is done, LEFs in various forms can be applied to the forecast volumes and summed.

A complete forecast will include the 13 FHWA vehicle classes (which are not the same vehicle classes as those used by vehicle manufacturers). These classes are shown in Table 1.29.

A somewhat simplified scheme for summarizing the 13 vehicle classes in Table 1.29 is to group all truck and bus traffic into three groups or units as shown in Table 1.30.

TABLE 1.28 SAMPLE OF AASHTO EQUIVALENCY FACTORS

Axle Type (lb)	Axle Load (lb)	ESAL Load Equivalency Factors[a]
Single axle	2,000	0.0003
	10,000	0.118
	14,000	0.399
	18,000	1.000
	20,000	1.4
	30,000	7.9
Tandem axle	2,000	0.0001
	10,000	0.011
	14,000	0.042
	18,000	0.109
	20,000	0.162
	30,000	0.703
	34,000	1.11
	40,000	2.06
	50,000	5.03

[a] Data from AASHTO (1993).

TABLE 1.29. FHWA VEHICLE CLASSES

FHWA Vehicle Class	Vehicle Class Description
Class 1	*Motorcycles* (Optional): All two- or three-wheeled motorized vehicles. Typical vehicles in this category have saddle-type seats and are steered by handlebars rather than wheels. This category includes motorcycles, motor scooters, mopeds, motor-powered bicycles, and three-wheel motorcycles. This vehicle type may be reported at the option of the state.
Class 2	*Passenger Cars*: All sedans, coupes, and station wagons manufactured primarily for the purpose of carrying passengers and including those passenger cars pulling recreational or other light trailers.
Class 3	*Other Two-Axle, Four-Tire Single-Unit Vehicles*: All two-axle, four-tire vehicles, other than passenger cars. Included in this classification are pickups, panels, vans, and other vehicles such as campers, motor homes, ambulances, hearses, and carryalls. Other two-axle, four-tire single-unit vehicles pulling recreational or other light trailers are included in this classification.
Class 4	*Buses*: All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires or three or more axles. This category includes only traditional buses (including school buses) functioning as passenger-carrying vehicles. All two-axle, four-tire single-unit vehicles. Modified buses should be considered to be a truck and be appropriately classified.
Class 5	*Two-Axle, Six-Tire Single-Unit Trucks*: All vehicles on a single frame, including trucks, camping and recreational vehicles, and motor homes, having two axles and dual rear wheels.
Class 6	*Three-Axle Single-Unit Trucks*: All vehicles on a single frame, including trucks, camping and recreational vehicles, motor homes, having three axles.
Class 7	*Four or More Axle Single-Unit Trucks*: All trucks on a single frame with four or more axles.
Class 8	*Four or Fewer Axle Single-Trailer Trucks*: All vehicles with four or fewer axles consisting of two units, one of which is a tractor or straight truck power unit.
Class 9	*Five-Axle Single-Trailer Trucks*: All five-axle vehicles consisting of two units, one of which is a tractor or straight truck power unit.
Class 10	*Six or More Axle Single-Trailer Trucks*: All vehicles with six or more axles consisting of two units, one of which is a tractor or straight truck power unit.
Class 11	*Five or Fewer Axle Multi-Trailer Trucks*: All vehicles with five or fewer axles consisting of three or more units, one of which is a tractor or straight truck power unit.
Class 12	*Six-Axle Multi-Trailer Trucks*: All six-axle vehicles consisting of three or more units, one of which is a tractor or straight truck power unit.
Class 13	*Seven or More Axle Multi-Trailer Trucks*: All vehicles with seven or more axles consisting of three or more units, one of which is a tractor or straight truck power unit.

TABLE 1.30. SIMPLIFIED TRUCK AND BUS GROUPS

Simplified Vehicle Categories	Groupings of FHWA Vehicle Classes
Single Units	(i) Buses (FHWA Class 4) (ii) 2-axle, 6-tire single units (FHWA Class 5) (iii) 3-axle single units (FHWA Class 6) (iv) 4+-axle single units (FHWA Class 7)
Single Trailers	(i) 4-axle single trailer (FHWA Class 8) (ii) 5-axle single trailer (FHWA Class 9) (iii) 6+-axle single trailer (FHWA Class 10)
Multi-Trailers	(i) 5-axle multi-trailer (FHWA Class 11) (ii) 6-axle multi-trailer (FHWA Class 12) (iii) 7+-axle multi-trailer (FHWA Class 13)

Analysis Tools

This subsection focuses on how to organize ESAL data so that an overall ESAL estimate for the design period can be made.

Table 1.31 shows typical ESALs per vehicle according to the groupings in Table 1.30. The ESALs per vehicle were developed by a state DOT and appear to be typical for U.S. truck traffic. They may appear to be low, but the values are averages that include empty backhauls.

TABLE 1.31. ESALS PER VEHICLE FOR SIMPLIFIED VEHICLE GROUPS

Simplified Vehicle Categories	FHWA Classes	Average ESALs per Vehicle
Single-Unit Trucks	4, 5, 6, 7	0.40
Trucks with Single Trailers	8, 9, 10	1.00
Trucks with Multi-Trailers	11, 12, 13	1.75
Buses (half full)	4	1.60

Thus, if you estimated that a specific highway has daily (one-way) 1,000 single-unit trucks, 2,000 trucks with single trailers, and 500 trucks with multi-trailers and no buses, then the daily ESALs would be [1,000(0.4) + 2,000(1.00) + 500(1.75)] = 3,275 ESALs per day or about 1,200,000 ESALs per year. The annual value can be scaled up to the design period with a suitable growth rate (typically 2% to 3%).

CONSTRUCTION PRODUCTIVITY AND TRAFFIC IMPACTS

Purpose

This section overviews the various methods for determining construction productivity and traffic impacts of pavement and roadway construction. Traffic impacts can often make up the largest societal cost associated with a paving project, sometimes being an order of magnitude more than the agency cost to build or rehabilitate the pavement. An early understanding of productivity and potential traffic impacts can assist the project in determining the most advantageous construction timing, project sequencing (staging), and lane-closure scenarios. Often, full roadway closures (in contrast to repeated partial closures) over longer periods of time (e.g., full weekends or multiple days instead of nighttime-only closures) can prove to be the least costly alternative if user costs are properly accounted for in construction planning.

Measurement Methods

Traffic impacts are typically quantified by user delay, with typical metrics being (1) total user delay, (2) total user cost associated with delay, (3) maximum vehicle queue length, and (4) maximum time in vehicle queue. Usually, the goal of minimizing traffic impacts is interpreted to mean minimizing the total user cost attributable to the existence of the project work zone. Other important considerations (e.g., accident and incident minimization, avoidance of certain public event days that generate high traffic) may cause the ultimate traffic impacts to be somewhat greater than the optimal minimum. Nonetheless, it is useful to estimate, as accurately as practical considerations allow, the minimum traffic impact scenario for pavement construction. Generally, this estimate uses the following six basic actions:

1. *Determine construction productivity.* This involves estimating the productivity of basic construction processes associated with the project such as demolition crew speed and efficiency, dump truck number and capacity, paver speed, and materials manufacturing plant productivity. It also involves estimating mobilization and demobilization times, concrete cure time, hot-mix asphalt cooling time, and traffic control setup time. There may be several estimates of each depending on the construction scenarios being investigated.
2. *Measure existing traffic*. Although an actual time history is best (e.g., from loop detector information or manual counts), average daily traffic (ADT) can be used and hourly traffic volumes can be developed by multiplying ADT by typical hourly distribution factors for the type of roadway being analyzed.
3. *Estimate the fraction of traffic that will cancel trips and the fraction of traffic that will use detour routes during the construction*. At best, these will be rough estimates unless extremely sophisticated models are used. These estimates are also highly dependent on the publicity given the roadway work. Values can be obtained from (a) agency experience with similar closures and similar publicity in the past and (b) a general literature review of similar traffic closures.
4. *Develop construction scheduling (staging) alternatives*. This involves determining the number, duration, and sequence of lane closures required to complete the project. As the traffic impact analysis progresses, it is often necessary to refine these alternatives. Strong consideration should be given to scheduling alternatives that result in work-zone traffic capacity greater than traffic demand during the hours of work. Essentially, this results in little or no user cost attributed to the roadway work. However, such scheduling alternatives may not exist or be feasible from a construction productivity and/or constructability standpoint. Any number of lane-closure scenarios can be considered, but it is helpful to at least investigate the following six scenarios.
    a. *Partial night closures* involve closure only during night hours with light traffic where each roadway direction is still open, although with reduced capacity in at least one direction. These closures are often the first considered because they tend to minimize traffic impacts by only closing lanes when traffic is the lightest. However, they may not provide the lowest user costs because mobilization and demobilization can take up a large percentage of total closure time, resulting in low overall productivity. In some scenarios, it may not be possible to make any meaningful progress in a short nighttime closure. Even if partial night closures cannot be used for mainline paving, they are often useful for prepaving work (e.g., PCC panel saw-cutting, restriping lanes, milling HMA).

b. *Full night closures* are the same as above but with at least one roadway direction fully closed. These may involve detouring an entire direction, counterflowing traffic on one side of a highway, or using a pilot car to alternate traffic directions in one lane. Full night closures sometimes require such things as setting up counterflow traffic on one side of a roadway or accomplishing dangerous overhead work such as overpass demolition or placement.

c. *Partial day closures* are closures only during day hours where each roadway direction is still open, although with reduced capacity in at least one direction. These closures are often the first considered for lightly trafficked roadways where user delay is unexpected even with some lanes closed. If traffic delays are minimal, day closures can improve safety by providing better visibility and encountering fewer impaired drivers than night work, and can reduce construction costs by avoiding overtime pay. However, they may not provide the lowest user costs because mobilization and demobilization can take up a large percentage of total closure time, resulting in low overall productivity. In some scenarios, it may not be possible to make any meaningful progress in a partial day closure.

d. *Full day closures* are the same as above but with at least one roadway direction fully closed. These may involve detouring an entire direction, counterflowing traffic on one side of a highway, or using a pilot car to alternate traffic directions in one lane. Full day closures are usually only feasible for lightly trafficked roadways or roadways with large-capacity detour routes that do not add significantly to commute time.

e. *Partial or full weekend continuous closures* start Friday evening after peak hour traffic and end Monday morning before peak hour traffic. The typical scenario is a 55-hour weekend closure starting at 9 or 10 p.m. on Friday night and ending at 4 or 5 a.m. on Monday morning. The long closure time allows for better productivity because mobilization and demobilization takes up a smaller fraction of total closure time and, more importantly, because construction crews generally get better and faster in their work given a longer working window. Weekends are typically preferred because weekend traffic is usually more discretionary (leading to more canceled trips and less total user delay) and often lighter than weekday traffic.

f. *Partial or full week-long continuous closures* are maintained continuously over an entire week (168 hours). Although it may not be known if any closure windows will extend over a week or more, estimating this alternative will generally allow estimation of longer closure windows with reasonable accuracy. For instance, the productivity for a 3-week closure is roughly, but not exactly (due to mobilization and demobilization times), three times the productivity of a 1-week continuous closure.

5. *Model traffic using the tool of choice* (see the section "Analysis Tools"). This modeling will result in an estimate of total user cost for the roadway project. In general, larger projects on major routes warrant more modeling, whereas smaller projects on minor routes can often be estimated sufficiently using spreadsheets.
6. *Apply FHWA Interim Report Life-Cycle Cost Analysis in Pavement Design (Walls and Smith, 1998) standards to estimate user delay cost*. This report provides reasonable values for user time (Table 1.32). The values in this table are in 1996 dollars and should be adjusted to current dollars using the Consumer Price Index. A simple calculator is available from the U.S. Bureau of Labor Statistics at http://www.bls.gov/data/inflation_calculator.htm. Multiplying these values by total delay for each class of vehicle gives an estimate of total work zone user delay cost.

Additionally, FHWA has accumulated additional information that can be found at http://www.fhwa.dot.gov/infrastructure/asstmgmt/lcca.cfm.

TABLE 1.32. RECOMMENDED VALUES OF TIME

Vehicle Class	Value per Vehicle Hour (1996 dollars)	
	Value	Range
Passenger Vehicles	$11.58	$10–$13
Single-Unit Trucks	$18.54	$17–$20
Combination Trucks	$22.31	$21–$24

Note: From Walls and Smith, 1998.

General Guidance

The following general guidance for traffic impacts comes largely from the guidance documents listed in the references for this section.

Closure Scenarios

- Productivity is usually much higher and worker safety is greater with longer, more complete closures (e.g., full closures, weekend closures; FHWA, 2003).
- The public is generally very accepting of full closures or a few longer-duration closures as an alternative to lengthy schedules of night or day closures (FHWA, 2003).
- As a work zone remains in effect for a longer period of time (e.g., over several days or several weekends), the fraction of drivers either canceling their trips or taking the detour route is likely to decrease as drivers become used to the situation or determine that a trip can no longer be put off.
- Detour routes may experience several times their normal traffic volumes (Lee, Lee, and Harvey, 2006; Lee et al., 2001). It may be prudent to improve detour route capacity through additional lanes, a temporarily reversible lane, signal retiming, or other improvements (FHWA, 2003).

- For major highway jobs, the construction of one lane usually requires a second adjacent lane for access. This means either using an existing wide shoulder (e.g., 10-ft shoulder) if one exists or closing a second lane (Lee, 2008).
- For major highway jobs, if the lane under construction has more than one major activity under way on it simultaneously (e.g., demolition and paving), a second access lane will likely be needed to avoid stationary trucks in the adjacent lane (Lee, 2008).
- Avoid creating work zones with live traffic on both sides (e.g., in the middle lanes in one direction). These generally do not leave workers a safe exit from the work zone if it is compromised.
- It may be better to use a simpler lane-closure plan that is more easily understood by the public even if it does not result in the minimum modeled user delay.

Contracting

- Lane rental or time-based bonus/penalty contracts should have a clear clause describing how to address changed conditions or any situation where the owner wishes to add work that impacts productivity (Lee et al., 2007). Often, contractors plan to spend more money than the contract price in order to finish early and receive the bonus. In this scenario, without bonus payments, the contractor will lose money.
- Contracts that contain bonus/penalty amounts for speed and quality should balance these amounts so that it does not become advantageous to sacrifice one bonus to get the other (Muench et al., 2007). For instance, of a maximum quality bonus/penalty is $3,000 but the maximum speed bonus/penalty is $100,000 then in some scenarios it may be logical to sacrifice a small quality bonus for a large speed bonus.

Productivity

- The slowest process in a reconstruction project is often demolition (Lee, Lee, and Ibbs, 2007). If several processes are being done simultaneously, demolition will most often control the overall productivity.
- The rate at which dump trucks can be filled by an excavator or milling machine is relatively consistent from job to job (Lee et al., 2007). Therefore, the best estimate is often what happened on the previous job. If no local information is available, Lee et al. (2007) provide good baseline estimates.
- Production rate is often controlled by access to the construction site and allowances made for traffic (e.g., temporary off-ramps in work zones, separation between work zone and traffic).

Work-Zone Capacity

- Work-zone capacity is highly variable and only moderately predictable. Work-zone capacity can be affected by the number of lanes open, intensity of work, presence of ramps, fraction of heavy vehicles, lane width, lateral clearance, work-zone grade, and more. *Highway Capacity Manual* (HCM) (TRB, 2000) procedures are very rough, but they suggest 1,600 passenger cars per lane per hour (pc/lane/h) be used as a baseline for short-term work zones. Typically this number is adjusted downward based on other factors and can be as low as about half the original value.
- The more a work zone can be physically and visually separated from traffic (e.g., semipermanent barriers like jersey barriers or k-rails instead of traffic cones or barrels), the greater the work-zone traffic capacity.
- Incidents (i.e., accidents, stalled vehicles, etc.) are one of the largest contributors to work zone user delay because there are fewer lanes (if any) that traffic can use to bypass the incident. Dedicating resources (e.g., incident response vehicle, video cameras, variable message boards, traffic management center) to reduce incidents and clear them more quickly can be a cost-effective way to minimize user delay (FHWA, 2004).

Publicity

- Roadway work and closure publicity can be effective in drastically reducing traffic during work-zone closures. Often, several-mile-long queues predicted using normal traffic volumes never materialize because many drivers cancel their trips or alter their routes.
- Even if a local public information campaign is effective, it may still be difficult to get closure information to travelers or freight carriers out of the local area who plan on using the affected roadway.

Analysis Tools

This subsection overviews some of the more popular methods for determining traffic impacts for pavement construction projects and factors that influence the choice of tools. Some key considerations when selecting tools are as follows:

- *How much detail is needed?* Work-zone characteristics, desired outputs, and the stage of planning, design, and construction will influence tool choice. Often a simpler tool, with less detail, is adequate.
- *Is the tool calibrated to the local area?* If not, results may still be useful; however, accuracy may be less than expected or needed.
- *Is the tool stochastic or deterministic?* Construction productivity and traffic can be highly variable and difficult to predict. Although a deterministic model can provide a single number, it is better to provide a reasonable range of answers to capture the variable nature of productivity and traffic.

- *How much detail does the tool produce?* Some tools can only estimate traffic impacts over one 24-hour period while others can estimate over much longer time periods. Some tools can only estimate delay on an hourly basis, whereas others can estimate them in much smaller time increments. Some tools make estimates using one single day's traffic input, whereas others are able to account for daily, weekly, and monthly traffic variations.

Analysis Tools: Construction Productivity

Construction productivity tools discussed are manual methods, standard estimating software, and Construction Analysis for Pavement Rehabilitation Strategies (CA4PRS).

Manual method. Demolition and paving productivity estimates can be made manually by comparing productivities of the constituent processes and identifying the limiting factor. There are a few references to help in paving productivity calculations. The National Asphalt Pavement Association (NAPA) publishes *Balancing Production Rates in Hot Mix Asphalt Operations* (IS 120), which contains a step-by-step guide for determining HMA paving productivity. Several companies also offer custom-printed asphalt productivity slide rules that paving companies can purchase and brand to be given out to potential customers.

Estimating software. Most estimating software (e.g., Bid2Win, HeavyBid) assists users in calculating the productivity of construction processes.

CA4PRS. CA4PRS is a Microsoft Access–based software tool that can be used to analyze highway pavement rehabilitation strategies including productivity, project scheduling, traffic impacts, and initial project costs based on input data and constraints supplied by the user. The goal is to help determine roadway rehabilitation strategies that maximize production and minimize costs without creating unacceptable traffic delays. As of 2009, all state transportation departments have free group licenses for CA4PRS.

First-Order Productivity Estimates

In the early planning stages of a project, it may be useful to quickly determine rough construction productivity based on a few known parameters. This section displays productivity graphs produced using CA4PRS with most inputs being held constant at typical values. The purpose of these graphs is only to give a rough estimate of typical productivity. CA4PRS should be used to produce more accurate numbers based on actual site-specific parameters for use in any project planning. In general, most inputs were fixed except for the trucking rates (i.e., removal of demolition from the site and delivery of paving material to the site). Thus, the 95% confidence intervals seen are mostly dependent on these delivery rates. In all cases, a 10-mi stretch of two lanes was analyzed (20 lane miles total). As with all data input values, this length of highway and total lane miles have some influence on productivity. Tables 1.33 through 1.38 show input parameters used in CA4PRS to generate Figures 1.35 through 1.43. Estimates are given for the following:

TABLE 1.33. CA4PRS INPUT VALUES FOR REMOVE AND REPLACE WITH PCC

Input	Value	Distribution/Comments
Activity Constraints		
Mobilization	1.0 h	None, deterministic
Demobilization	2.0 h	None, deterministic
Base paving	None	NA
Demo-to-PCC paving lag times for sequential method	1.0 h	Triangular (min = 0.5 h, max = 1.5 h)
Demo-to-PCC paving lag times for concurrent method	2.0 h	Triangular (min = 1.0 h, max = 3.0 h)
Resource Profile		
Demolition Hauling Truck		
Rated capacity	18.0 tons	9 yd^3 of a 15-yd^3 truck filled with 2.0 tons/yd^3 material
Trucks/h/team	10 trucks	Triangular (min = 8 trucks, max = 12 trucks)
Packing efficiency	1.0	None, deterministic
Number of teams	1.0 2.0	1 team for screed paving, 2 teams for slipform None, deterministic
Team efficiency	0.90	Triangular (min = 0.85, max = 0.95)
Base Delivery Truck	None	NA (no base material)
Batch Plant		
Capacity	500 yd^3/h	None, deterministic (set high to ensure plant is not the limiting activity)
Number of plants	1	None, deterministic
Concrete Delivery Truck		
Capacity	7.5 yd^3	NA
Trucks per hour	 10/h 13/h	The first rate is for screed paving and the second is for slipform paving Triangular (min = 8/h, max = 12/h) Triangular (min = 15/h, max = 19/h)
Packing efficiency	1.0	None, deterministic
Paver		
Speed	5 ft/min	None, deterministic
Number of pavers	1	None, deterministic
Schedule Analysis		
Construction window	See graphs	
Section profile	See graphs	Note: No base material included in graphs
Change in roadway elevation	No change	
Lane widths	12 ft	
Curing time	12 h	
Working method	See graphs	

TABLE 1.34. CA4PRS INPUT VALUES FOR REMOVE AND REPLACE WITH HMA

Input	Value	Distribution/Comments
Activity Constraints		
Mobilization	1.0 h	None, deterministic
Demobilization	2.0 h	None, deterministic
Base paving	None	NA
Demo-to-HMA paving lag	1.0 h	Triangular (min = 0.5 h, max = 1.5 h)
Half closure traffic switch	0.5 h	Triangular (min = 0.25 h, max = 0.75 h)
Resource Profile		
Demolition Hauling Truck		
Rated capacity	18.0 tons	9 yd^3 of a 15-yd^3 truck filled with 2.0 tons/yd^3 material
Trucks/h/team	10 trucks	Triangular (min = 8 trucks, max = 12 trucks)
Packing efficiency	1.0	None, deterministic
Number of teams	1.0	None, deterministic
Team efficiency	0.90	Triangular (min = 0.85, max = 0.95)
Paver	None	NA (no base material)
Nonpaving speed	15 mph	
Batch Plant		
Capacity	500 yd^3/h	None, deterministic (set high to ensure plant is not the limiting activity)
Number of plants	1	None, deterministic
HMA Delivery Truck		
Capacity	18 tons	NA
Trucks per hour	12/h	Triangular (min = 10/h, max = 14/h)
Packing efficiency	1.0	None, deterministic
Schedule Analysis		
Construction window	See graphs	
Section profile	See graphs	Top two lifts are 2 in. each, all other lifts are 3 in. each; paver moves at 0.6 mph for top two lifts and 0.5 mph for all other lifts
Change in roadway elevation	No change	
Shoulder overlay	Prepaving	Shoulder overlays are not accounted for
Curing time	12 h	
Working method	See graphs	
Cooling time analysis	User specifications	Time calculated in MultiCool and manually entered
Lane Widths		
Number of lanes	2	
Lane widths	12 ft each	

TABLE 1.35. CA4PRS INPUT VALUES FOR MILL AND FILL WITH HMA

Input	Value	Distribution/Comments
Activity Constraints		
Mobilization	1.0 h	None, deterministic
Demobilization	2.0 h	None, deterministic
Mill-to-HMA paving lag	1.0 h	Triangular (min = 0.5 h, max = 1.5 h)
Half closure traffic switch	0.5 h	Triangular (min = 0.25 h, max = 0.75 h)
Resource Profile		
Milling and Hauling		
Number of teams	1.0	None, deterministic
Team efficiency	0.90	Triangular (min = 0.85, max = 0.95)
Milling Machine		
Class	Large	
Material type	AC-Hard	
Efficiency factor	0.90	Triangular (min = 0.85, max = 0.95)
Hauling Truck		
Rated capacity	18.0 tons	9 yd^3 of a 15-yd^3 truck filled with 2.0 tons/yd^3 material
Trucks/h/team	13 trucks	Triangular (min = 11 trucks, max = 15 trucks)
Packing efficiency	1.0	None, deterministic
Batch Plant		
Capacity	500 yd^3/h	None, deterministic (set high to ensure plant is not the limiting activity)
Number of plants	1	None, deterministic
HMA Delivery Truck		
Capacity	18 tons	NA
Trucks per hour	12/h	Triangular (min = 10/h, max = 14/h)
Packing efficiency	1.0	None, deterministic
Paver	None	NA (no base material)
Nonpaving speed	15 mph	
Schedule Analysis		
Construction window	See graphs	
Section profile	See graphs	Lifts are between 1.5 and 3 in.; paver speeds are 0.5–0.6 mph
Change in roadway elevation	No change	
Shoulder overlay	Prepaving	Shoulder overlays are not accounted for
Curing time	12 h	
Working method	See graphs	
Cooling time analysis	User specifications	Time calculated in MultiCool and manually entered
Lane Widths		
Number of lanes	2	
Lane widths	12 ft each	

TABLE 1.36. CA4PRS INPUT VALUES FOR CRACK, SEAT, AND OVERLAY

Input	Value	Distribution/Comments
Activity Constraints		
Mobilization	3.0 h	None, deterministic
Demobilization	2.0 h	None, deterministic
Half closure traffic switch	0.5 h	Triangular (min = 0.25 h, max = 0.75 h)
Resource Profile		
Paver	None	NA (no base material)
Nonpaving speed	15 mph	
Batch Plant		
Capacity	500 yd^3/h	None, deterministic (set high to ensure plant is not the limiting activity)
Number of plants	1	None, deterministic
HMA Delivery Truck		
Capacity	18 tons	NA
Trucks per hour	12/h	Triangular (min = 10/h, max = 14/h)
Packing efficiency	1.0	None, deterministic
Schedule Analysis		
Construction window	See graphs	
Section profile	See graphs	Top two lifts are 2 in. each, all other lifts are 3 in. each; paver moves at 0.6 mph for top two lifts and 0.5 mph for all other lifts
Change in roadway elevation	No change	
Shoulder overlay	Prepaving	Shoulder overlays are not accounted for
Curing time	12 h	
Working method	See graphs	
Cooling time analysis	User specifications	Time calculated in MultiCool and manually entered
Lane Widths		
Number of Lanes	2	
Lane widths	12 ft each	

TABLE 1.37. CA4PRS INPUT VALUES FOR UNBONDED PCC OVERLAY

Input	Value	Distribution/Comments
Activity Constraints		
Mobilization	3.0 h	None, deterministic (longer time accounts for surface preparation)
Demobilization	2.0 h	None, deterministic
Base paving	None	NA
Demo-to-PCC paving lag times for sequential method	0 h	No demolition occurs
Demo-to-PCC paving lag times for concurrent method	0 h	No demolition occurs
Resource Profile		
Demolition Hauling Truck		High numbers are a work-around to make demolition take essentially no time
Rated capacity	100.0 tons	None, deterministic
Trucks/h/team	100 trucks	None, deterministic
Packing efficiency	1.0	None, deterministic
Number of teams	100.0	None, deterministic
Team efficiency	1.00	None, deterministic
Base Delivery Truck	None	NA (no base material)
Batch Plant		
Capacity	500 yd^3/h	None, deterministic (set high to ensure plant is not the limiting activity)
Number of plants	1	None, deterministic
Concrete Delivery Truck		
Capacity	7.5 yd^3	NA
Trucks per hour	10/h	Triangular (min = 8/h, max = 12/h)
Packing efficiency	1.0	None, deterministic
Paver		
Speed	5 ft/min	None, deterministic
Number of pavers	1	None, deterministic
Schedule Analysis		
Construction window	See graphs	
Section profile	See graphs	Note: No base material included in graphs
Change in roadway elevation	No change	
Lane widths	12 ft	
Curing time	12 h	
Working method	See graphs	

TABLE 1.38. MULTICOOL INPUT PARAMETERS FOR HMA OPTIONS

Input	Value
Constant Inputs in All Scenarios	
Start time	1000, 7/15/2010
Environmental Conditions	
Ambient air temperature	60°F
Average wind speed	5 mph
Sky conditions	Clear and dry
Latitude	38° North
Existing Surface	
Material type	Granular base
Moisture content	Dry
State of moisture	Unfrozen
Surface temperature	60°F
Mix Specifications	
Mix type	Dense graded
PG grade	64-22
Delivery temperature	300°F
Stop temperature	140°F
Lift Thicknesses	
3 in. of HMA total	2 lifts of 1.5 in. each
6 in. of HMA total	3 lifts of 2 in. each
9 in. of HMA total	3 lifts of 2 in., 1 lift of 3 in.
12 in. of HMA total	2 lifts of 1.5 in., 3 lifts of 3 in.

- *Remove and replace with PCC.* Remove the existing pavement and replace with the same depth of new PCC pavement. Productivity is estimated for sequential operations (only one major operation—demolition or paving—is occurring on the jobsite at any one time) and concurrent operations (both major operations—demolition and paving—are occurring on the jobsite at once, with the appropriate space in between). One lane is paved at a time. Sequential operations require one additional lane shut down for construction access, whereas concurrent operations require two additional lanes shut down for construction access. Calculations were made for both screed paving (slower) and slipform paving (faster):
  - —Using fixed forms and a screed, *screed paving* is usually slower, assuming the use of 7.5 yd^3 agitating mixers arriving at 10 trucks/h and only one demolition crew.
  - —Using a slipform paver, *slipform paving* is usually faster, assuming the use of 8.5 yd^3 end dump trucks arriving at 17 trucks/h and two demolition crews.
- *Remove and replace with HMA.* Remove the existing pavement and replace with the same depth of new HMA pavement. The roadway lanes being paved are fully shut down, only one paver with a 12-ft-wide screed is used, and HMA is paved in

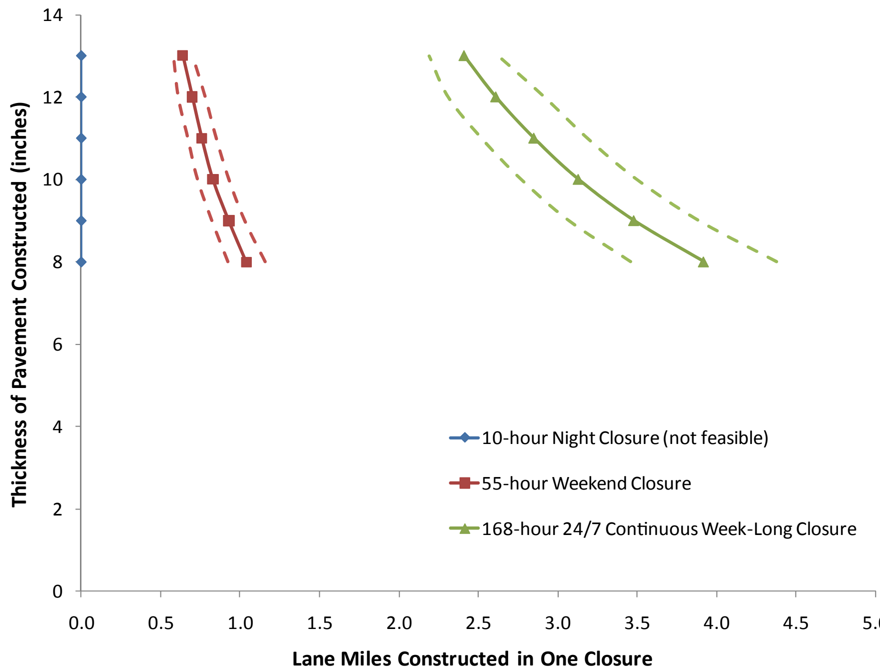


Figure 1.35. *Productivity estimates for remove and replace with PCC (fixed form) using sequential operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. Note: This option is not feasible using 10-h night closures.*

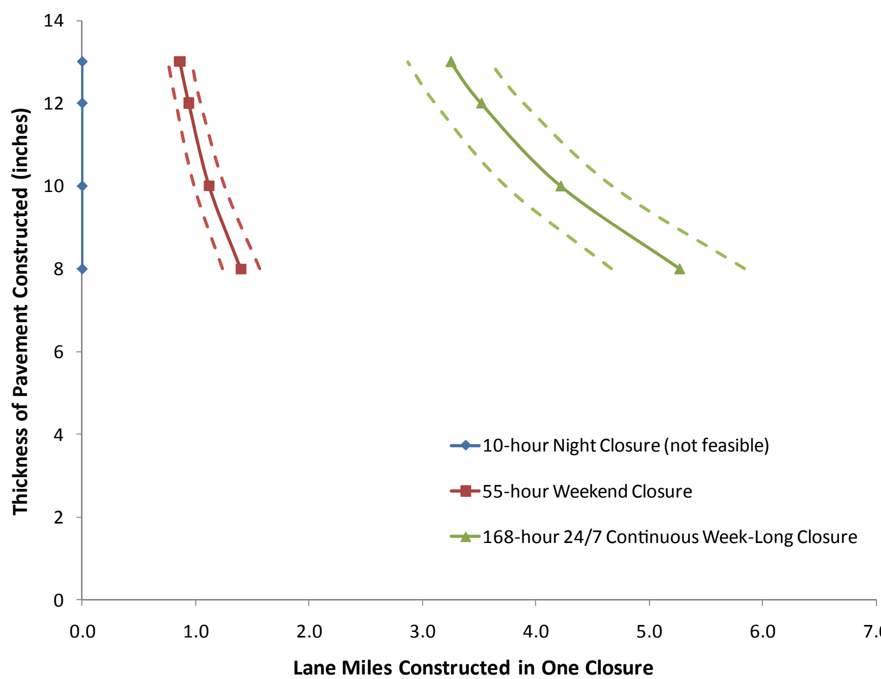


Figure 1.36. *Productivity estimates for remove and replace with PCC (slipform) using sequential operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. Note: This option is not feasible using 10-h night closures.*

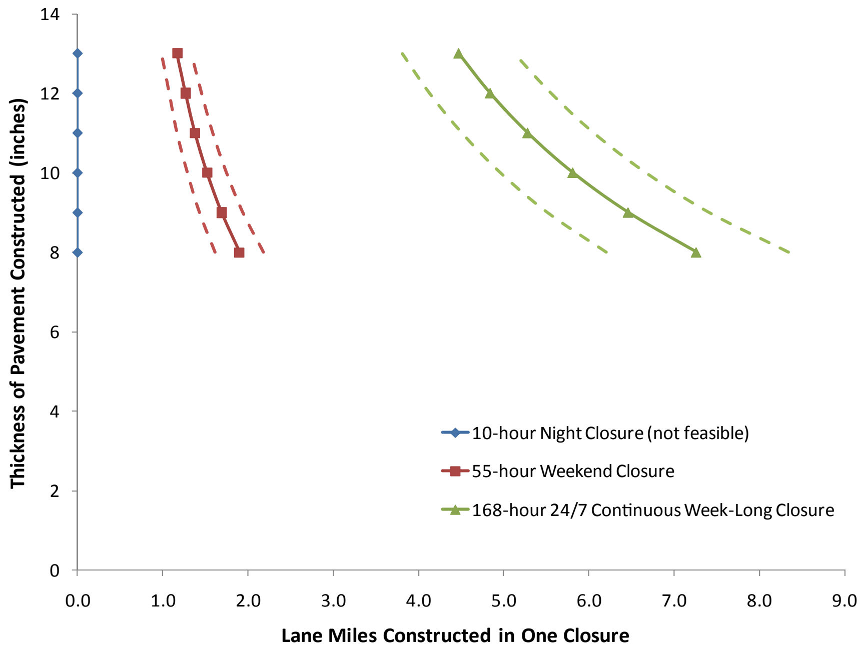


Figure 1.37. *Productivity estimates for remove and replace with PCC (fixed form) using concurrent operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. Note: (1) This option is not feasible using 10-h night closures, and (2) doing demolition and paving concurrently results in significantly higher productivities than doing them sequentially.*

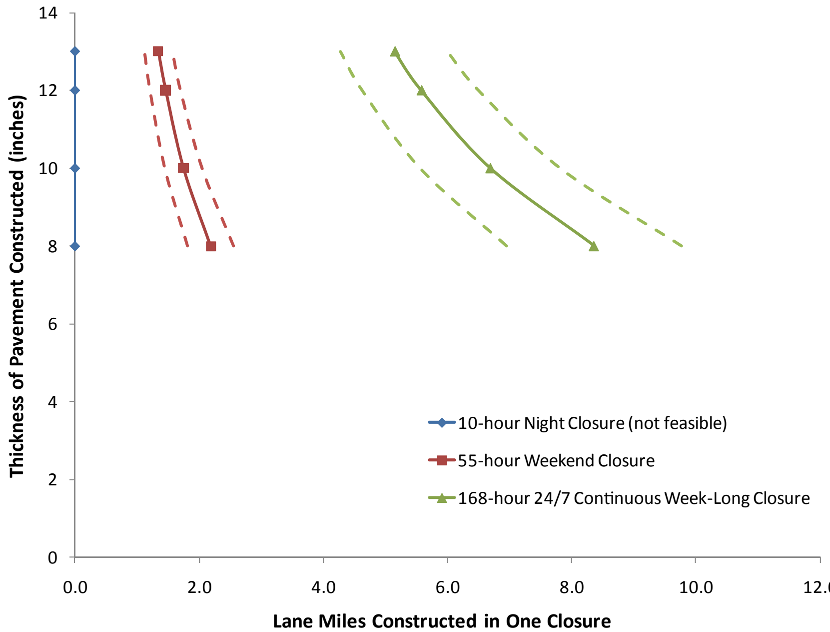


Figure 1.38. *Productivity estimates for remove and replace with PCC (slipform) using concurrent operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. Note: (1) This option is not feasible using 10-h night closures, and (2) doing demolition and paving concurrently results in significantly higher productivities than doing them sequentially.*

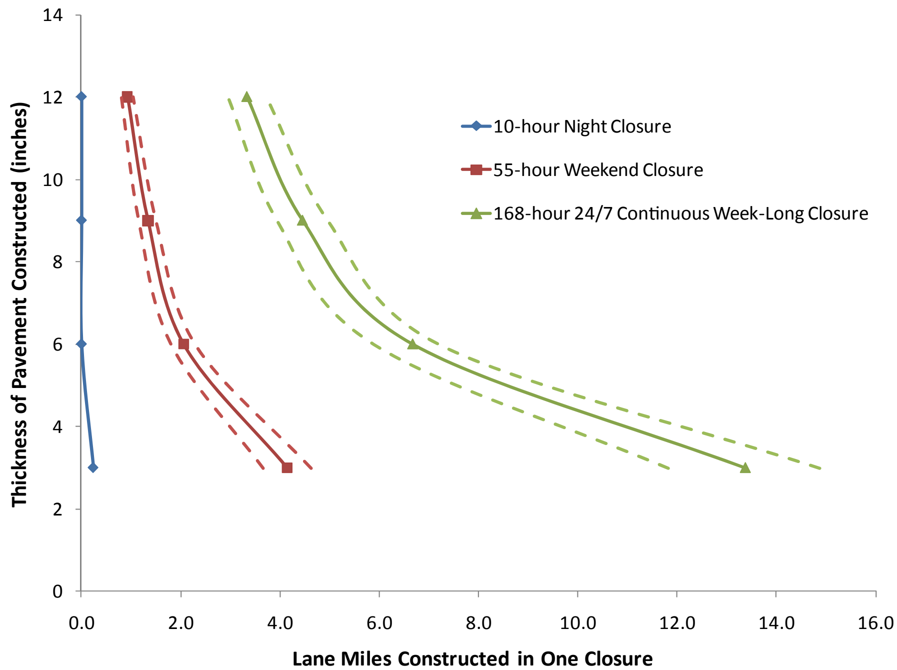


Figure 1.39. *Productivity estimates for remove and replace with HMA. Solid lines indicate averages and dashed lines indicate 95% confidence intervals.*

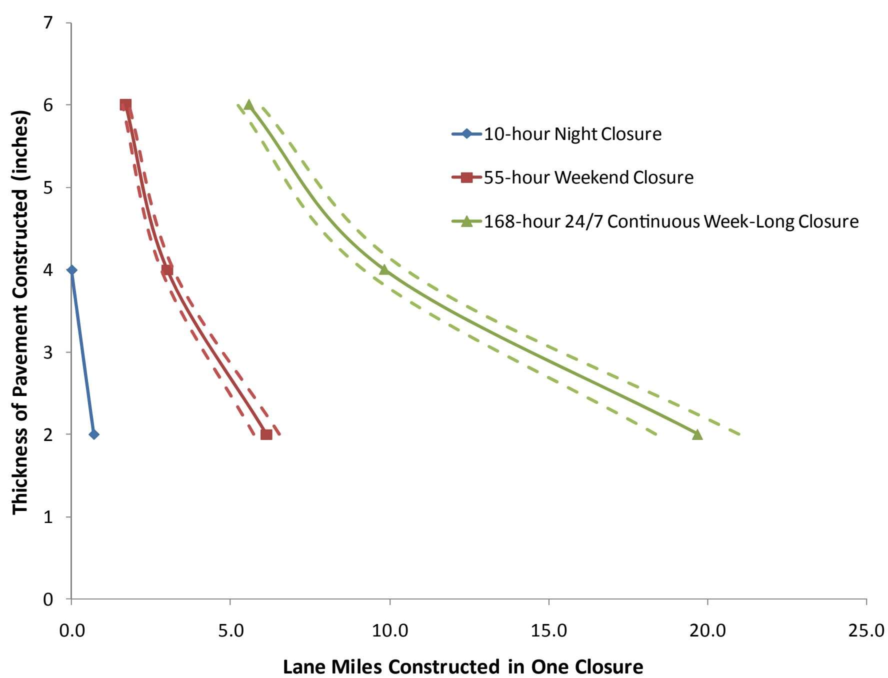


Figure 1.40. *Productivity estimates for mill and fill with HMA. Solid lines indicate averages and dashed lines indicate 95% confidence intervals.*

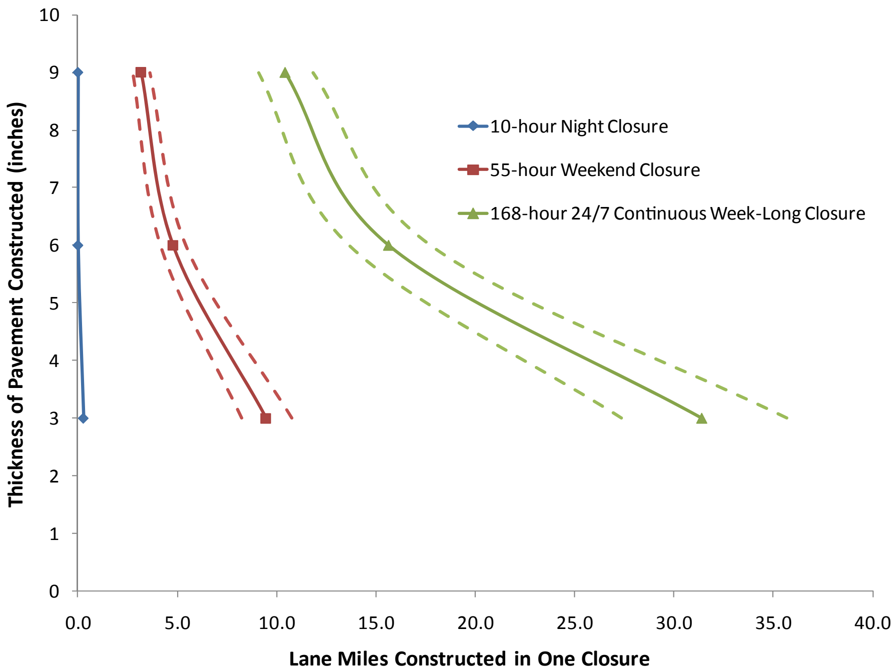


Figure 1.41. *Productivity estimates for crack, seat, and overlay. Solid lines indicate averages and dashed lines indicate 95% confidence intervals.*

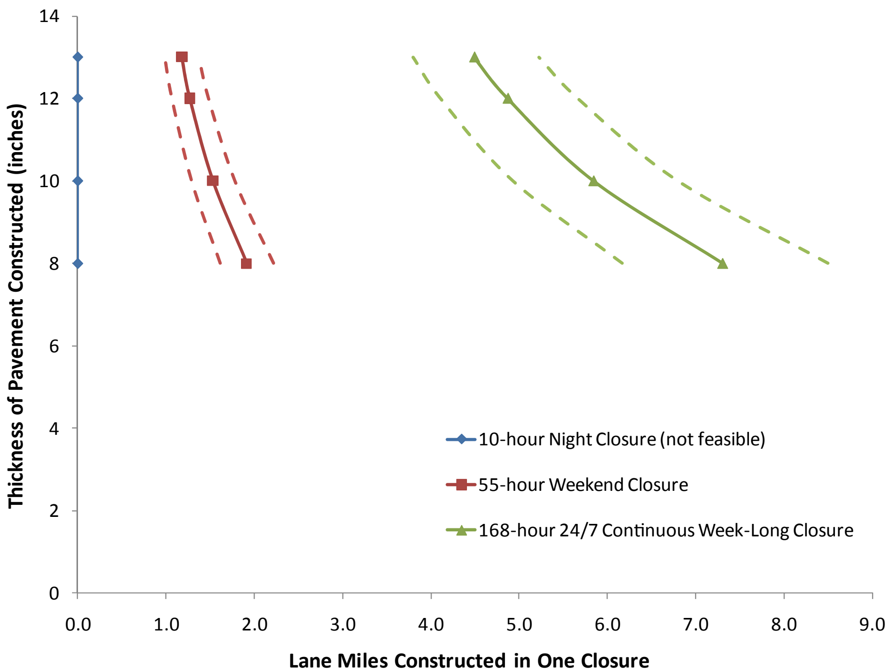


Figure 1.42. *Productivity estimates for PCC unbonded overlay. Solid lines indicate averages and dashed lines indicate 95% confidence intervals.*

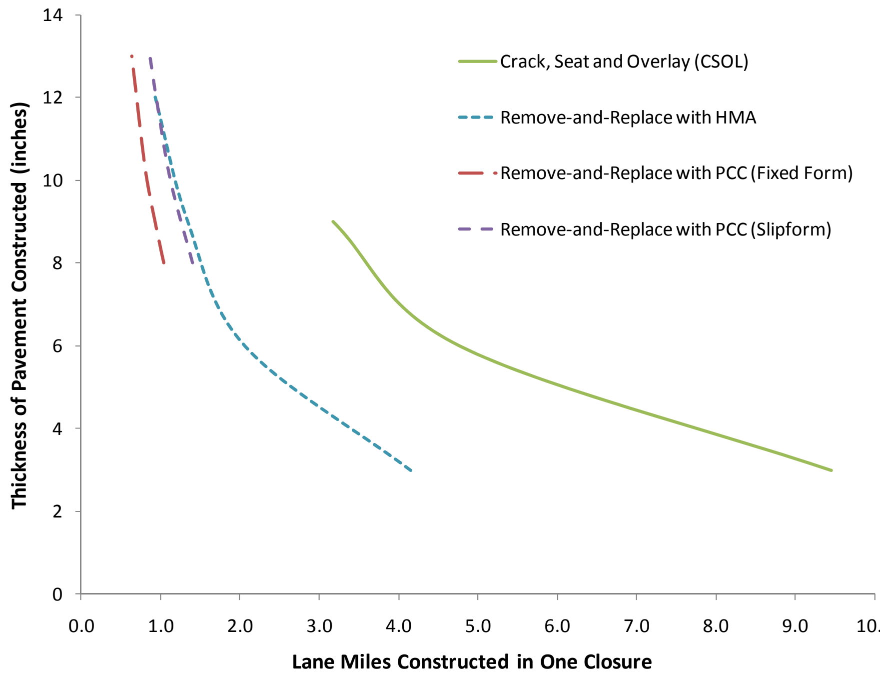


Figure 1.43. *A productivity comparison of PCC remove and replace (both fixed form and slipform), HMA remove and replace, and crack, seat, and overlay (CSOL).*

lifts. Lifts are generally 3 in. thick with the exception of the top two lifts, which are either 2 or 1.5 in. thick. A lift is paved for each lane across before the next lift is paved on any lane.

- *Mill and fill with HMA*. Remove a predetermined thickness from the existing pavement with an HMA milling machine, then replace the same thickness with new HMA. The roadway lanes being paved are fully shut down, only one paver with a 12-ft-wide screed is used, and HMA is paved in lifts. Lifts are generally 3 in. thick with the exception of the top two lifts, which are either 2 or 1.5 in. thick. A lift is paved for each lane across before the next lift is paved on any lane.
- *Crack, seat, and overlay*. Crack and seat the existing PCC pavement, then overlay with HMA. The roadway being paved is fully shut down, only one paver with a 12-ft-wide screed is used, and HMA is paved in lifts. Lifts are generally 3 in. thick with the exception of the top two lifts, which are either 2 or 1.5 in. thick. A lift is paved for each lane across before the next lift is paved on any lane.
- *Use unbonded PCC overlay*. Prepare the surface of the existing PCC pavement, then overlay with PCC that is not bonded to the existing pavement. This is essentially like the "remove and replace with PCC" method without the demolition component.

Analysis Tools: Traffic Impacts

There are a number of analysis tools available to assist in work-zone traffic impacts estimation. FHWA divides these tools into six broad categories (Hardy and Wunderlich, 2008a; summarized in Table 1.39):

1. *Sketch-planning tools* are specialized models designed for work-zone analysis. These models can vary from simple spreadsheet calculations to general delay estimation tools. Typically, models are deterministic and based on simple queuing equations or volume-to-capacity relationships from the HCM. Such simple estimation tools are often adequate for work-zone delay estimation.
2. *Travel demand models* are used to forecast future traffic demand based on current conditions, and to forecast future predictions of household and employment centers (Alexiadis, Jeannotte, and Chandra, 2004). Travel demand models are usually used in large regional planning efforts. In work-zone analysis, they can help predict regionwide impacts of extended roadway closures (e.g., closing a freeway for several months). It is not likely that a travel demand model would be built for the specific purpose of work-zone traffic analysis. Rather, an existing model may be used if available and warranted.
3. *Traffic signal optimization tools* are used to develop signal timing plans. These can be useful if a temporary signal is used or if signals are retimed to accommodate work-zone traffic or increased detour route traffic.
4. *Macroscopic simulation models* are based on the deterministic relationships of traffic speed, flow, and density (Alexiadis, Jeannotte, and Chandra, 2004). These models treat flow as an aggregate quantity in a defined area and do not track individual vehicles. They are useful for modeling larger-area impacts of work zones because of their aggregate nature.

TABLE 1.39. TRAFFIC MODEL TYPES FOR WORK-ZONE TRAFFIC IMPACTS

Model Type	Examples	Strengths	Weaknesses
Sketch planning	HDM, QUEWZ-98, QuickZone, CA4PRS	Low cost, specific to work zones, fast	Limited modeling ability, not well supported
Travel demand	EMME/2, TransCAD, TRANSIMS	Can model large areas	Low detail, cannot model short-term work-zone effects
Signal optimization	PASSER, Synchro	Models signal timing and coordination	Does not model other things
Macroscopic	BTS, KRONOS, METACORE/ METANET, TRANSYT-7F	Can model large areas	Low detail, cannot model short-term work-zone effects
Mesoscopic	CONTRAM, DYNASMART, DYNAMIT, MesoTS	Good compromise between macro- and micromodels	Data intensive
Microscopic	CORSIM, VISSIM, PARAMICS	Can model small details, good communication tool	Data intensive

5. *Mesoscopic simulation models* represent the relative flow of vehicles on a network, but they do not model individual lanes or vehicles. These models are between macroscopic and microscopic models in detail and can simulate both large geographic areas as well as specific corridors. They do not, however, possess the detail to model more modified strategies such as signal timing. These models require large amounts of data.
6. *Microscopic simulation models* simulate the movement of individual vehicles. These models require large amounts of data and can get unwieldy when a user attempts to simulate a large network. Often these models can provide animated output that can clearly communicate to decision makers and the public what the potential traffic impacts of modeled actions will be.

The most appropriate modeling approach depends on the following (Hardy and Wunderlich, 2008b):

- *Work-zone characteristics* affect the expected level of impact a work zone will have on travelers and include the geographic scale of the affected area and the complexity of the road network within this area.
- *Transportation management plan strategies* are the means by which traffic will be managed, including such items as lane closures, full roadway closures, lane shifts, counterflow traffic, night/day work, detours, and weekend work.
- *Data availability and quality* are the type, amount, accuracy, and timeliness of available data.
- *Agency resources* include the owner-agency's funding, technical staff, and schedule.
- *Work-zone performance measures* are selected by the owner-agency to quantify traffic impacts. Typically these measures are some form of delay (in minutes or cost) either in total (total delay/cost) or peak (longest queue, longest wait).

Because the use of modeling tools beyond sketch-planning tools will almost surely require traffic expertise beyond the pavement profession, further discussion is limited to a few sketch-planning tools that may be of use: QuickZone and CA4PRS. Both of these tools can provide meaningful traffic impact estimates for a relatively small monetary and time investment.

QuickZone 2.0. A Microsoft Excel–based tool (requires Excel 97 as a minimum) that estimates work-zone traffic impacts. It allows the user to input a node-and-link network (see Figures 1.44 and 1.45) and then assign traffic counts to that network. It can coarsely simulate traffic variations between days of the week, and months of the year by applying multiples to standard ADT inputs. It can simulate multiple lane closures over time, model traffic over an entire week (Figure 1.46), and display various traffic impact metrics (Figure 1.47). These capabilities are helpful because they allow QuickZone to show differences in traffic impacts between nights and days, weekends and weekdays, and seasons (e.g., summer versus fall work). The user guide explains the algorithm QuickZone uses to estimate delay and user cost, but specific equations are not listed or discussed. QuickZone is inexpensive (about $200) but is getting relatively old (version 2.0 was released in 2005) without any significant upgrade or

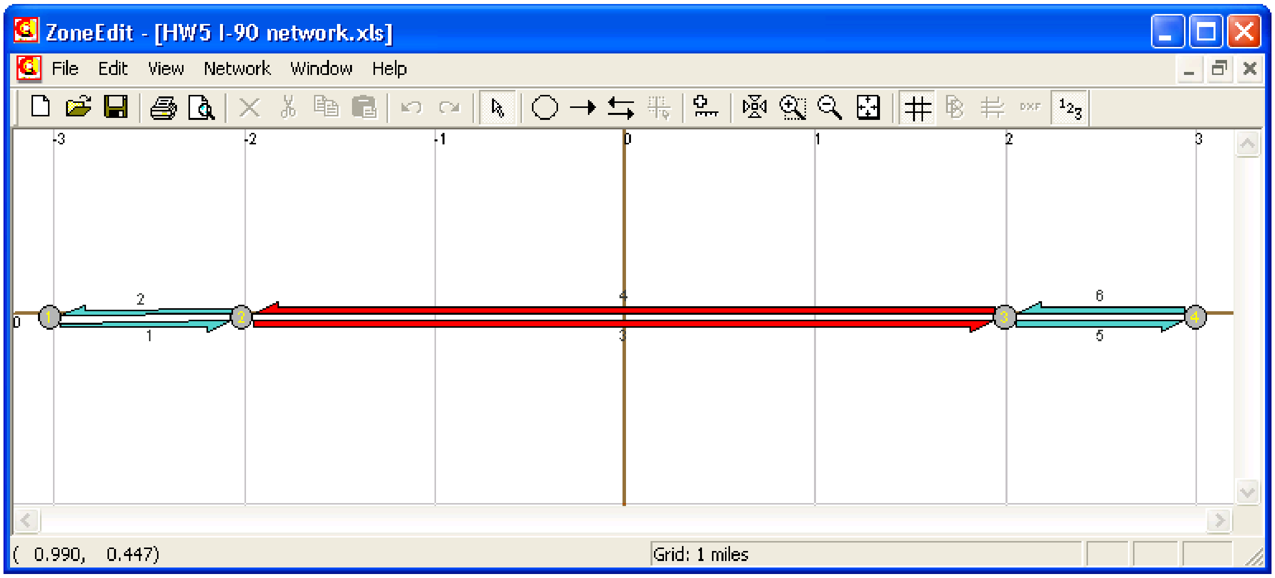


Figure 1.44. *A simple network that works quite well in QuickZone.*

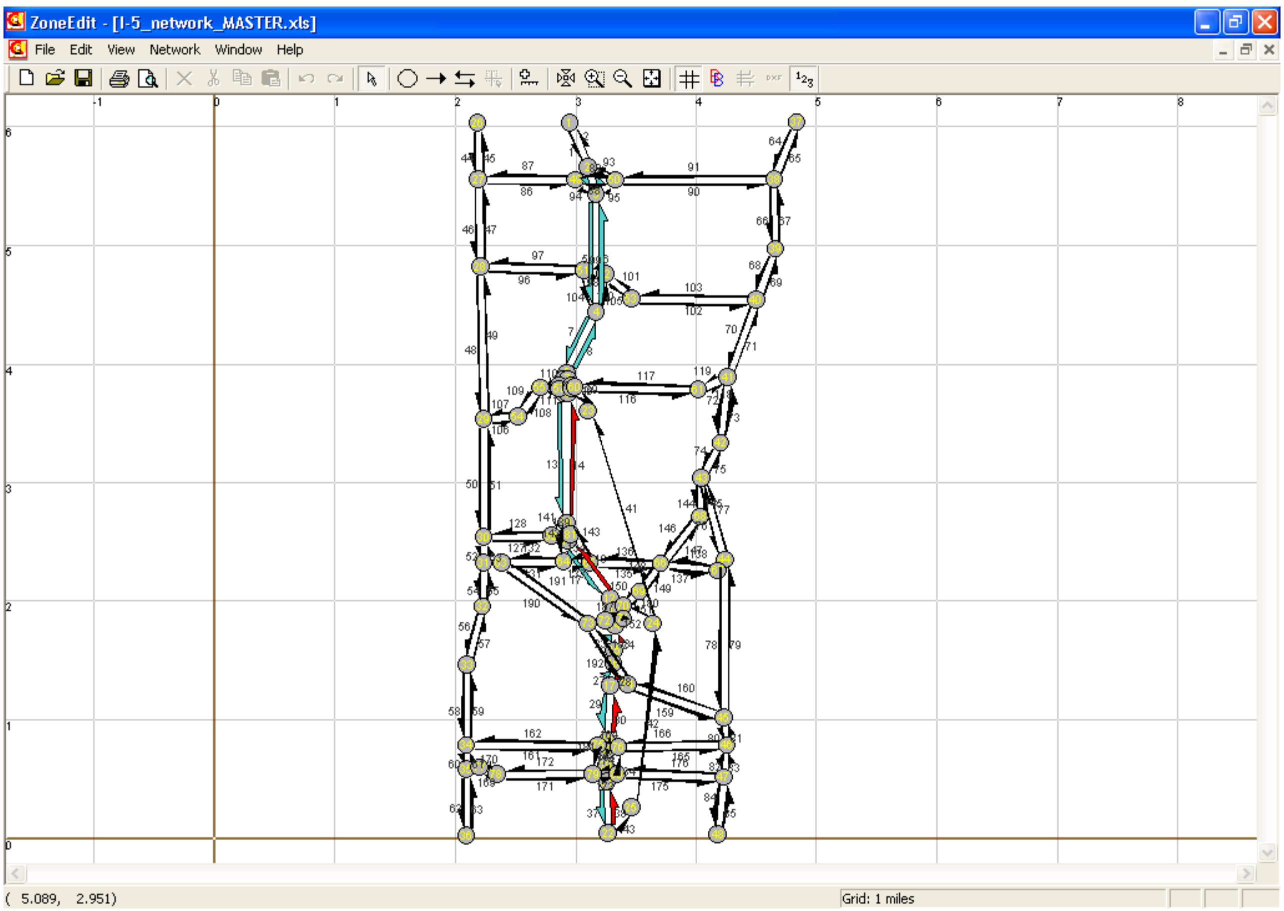


Figure 1.45. *A complex network simulation in QuickZone (I-5 in the Seattle, Washington, area is shown). This network simulation exposed several program bugs, was unwieldy to process, and required tedious troubleshooting to make operational. This level of complexity is not recommended.*

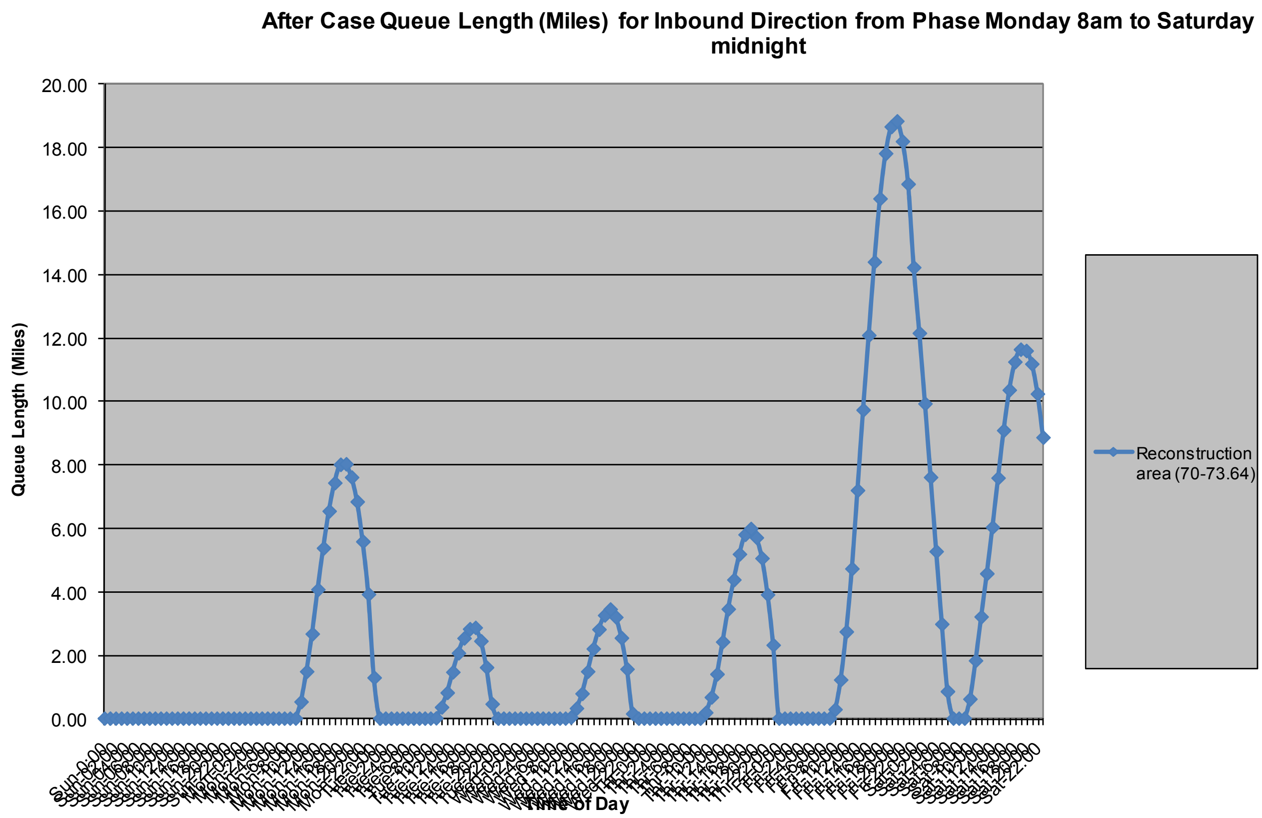


Figure 1.46. *Unedited QuickZone 2.0 simulation output chart for a 1-week time period. Note that the automatic graph labeling on the horizontal axis is unreadable; however, this can be corrected by editing the graph in Excel.*

Period with highest delay in After Case

Phase	midnight to Thursday 8am
Direction	Inbound
Day/Time	Sunday 21:00

			Total Project User Cost ($)				
	Max Queue (Miles)	Max Delay (min)	Passenger Cars	Truck	Detour	Econ/Misc	Total
Baseline	0	0	$0	$0	$0	$0	$0
After	34.13	778.23	$22,036,199	$1,851,536	$0	$0	$23,887,735
Total	34.13	778.23	$22,036,199	$1,851,536	$0	$0	$23,887,735

Figure 1.47. *QuickZone 2.0 summary tables showing available traffic impact metrics.*

support beyond a user guide. Simple scenarios with just a few links and nodes are relatively easy to simulate; however, more complex scenarios become cumbersome due to tedious data entry and difficult input troubleshooting if outputs are suspect.

CA4PRS. A Microsoft Access–based software tool that can be used to analyze highway pavement rehabilitation strategies including productivity, project scheduling, traffic impacts, and initial project costs based on input data and constraints supplied by the user. The traffic impacts analysis portion of CA4PRS (labeled "Work-Zone Analysis" in the software) can simulate 24 h of traffic through a defined work zone. Work zones are defined by the number of lanes closed, the closure duration, and the work-zone capacity (Figure 1.48). Traffic can be entered by hourly count or ADT can be entered and then distributed over 24 h using hourly factors. CA4PRS can simulate a one-lane-closure scenario over a 24-h period. Longer closures are estimated by multiplying the results of one 24-h analysis by the total number of closures. The 24-h simulation limit using only one traffic count makes it difficult to account for longer closures (e.g., over several weeks or months), where traffic flow is likely to change over time (e.g., weekday versus weekend or summer versus fall). Output is similar to that of QuickZone (Figures 1.49 and 1.50).

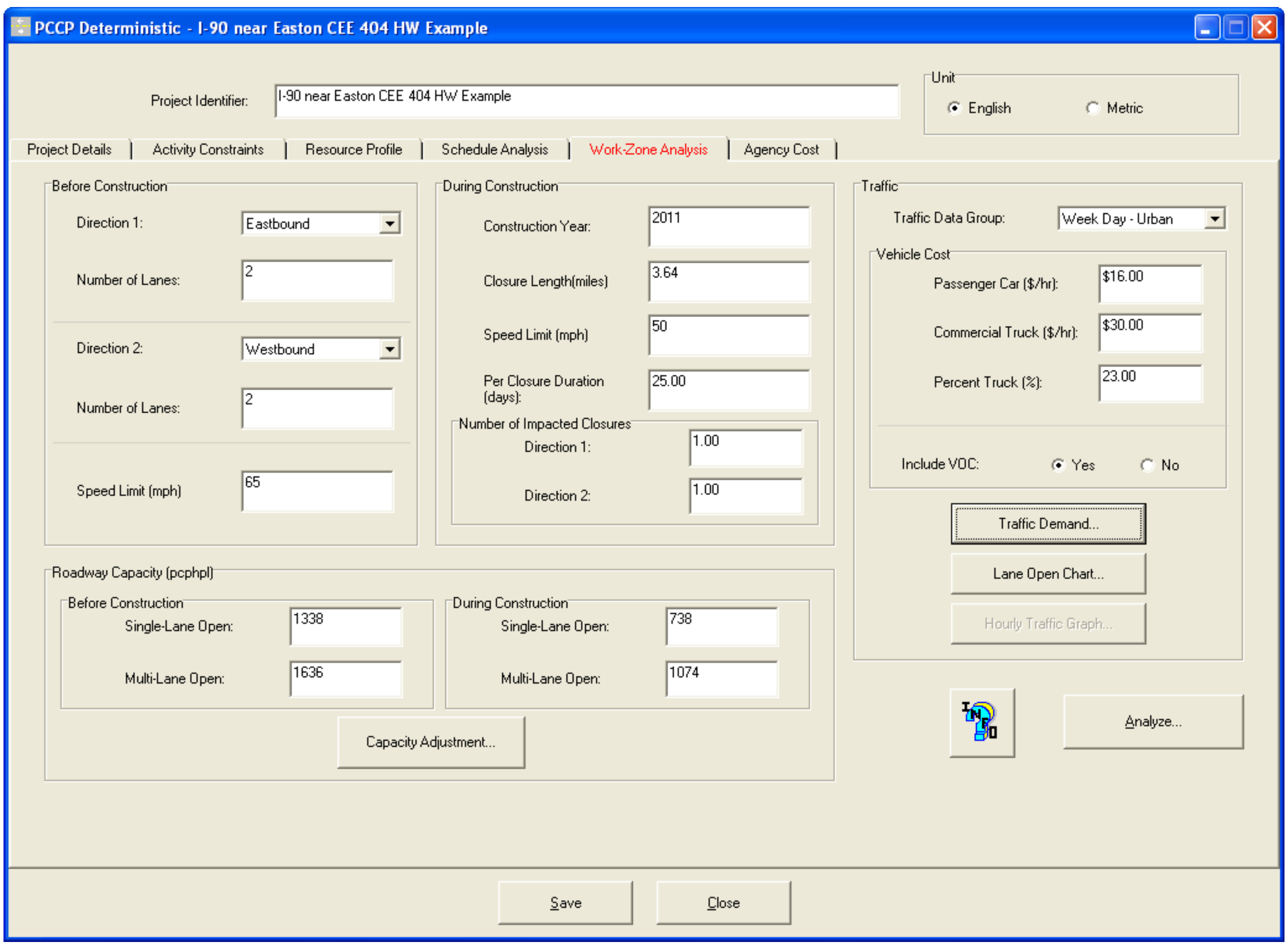
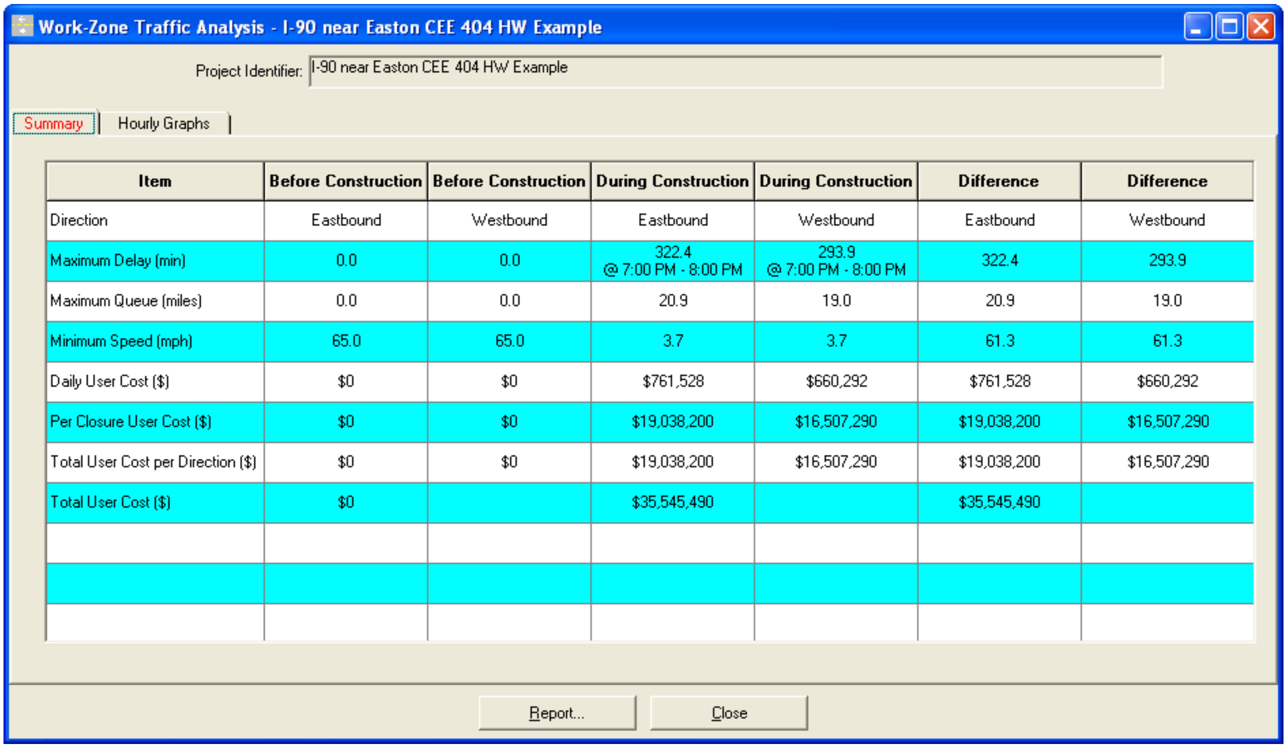


Figure 1.48. *CA4PRS Work-Zone Analysis input screen.*



Item	Before Construction	Before Construction	During Construction	During Construction	Difference	Difference
Direction	Eastbound	Westbound	Eastbound	Westbound	Eastbound	Westbound
Maximum Delay (min)	0.0	0.0	322.4 @ 7:00 PM - 8:00 PM	293.9 @ 7:00 PM - 8:00 PM	322.4	293.9
Maximum Queue (miles)	0.0	0.0	20.9	19.0	20.9	19.0
Minimum Speed (mph)	65.0	65.0	3.7	3.7	61.3	61.3
Daily User Cost ($)	$0	$0	$761,528	$660,292	$761,528	$660,292
Per Closure User Cost ($)	$0	$0	$19,038,200	$16,507,290	$19,038,200	$16,507,290
Total User Cost per Direction ($)	$0	$0	$19,038,200	$16,507,290	$19,038,200	$16,507,290
Total User Cost ($)	$0		$35,545,490		$35,545,490	

Figure 1.49. *CA4PRS Work-Zone Analysis summary results screen showing available traffic impact metrics.*

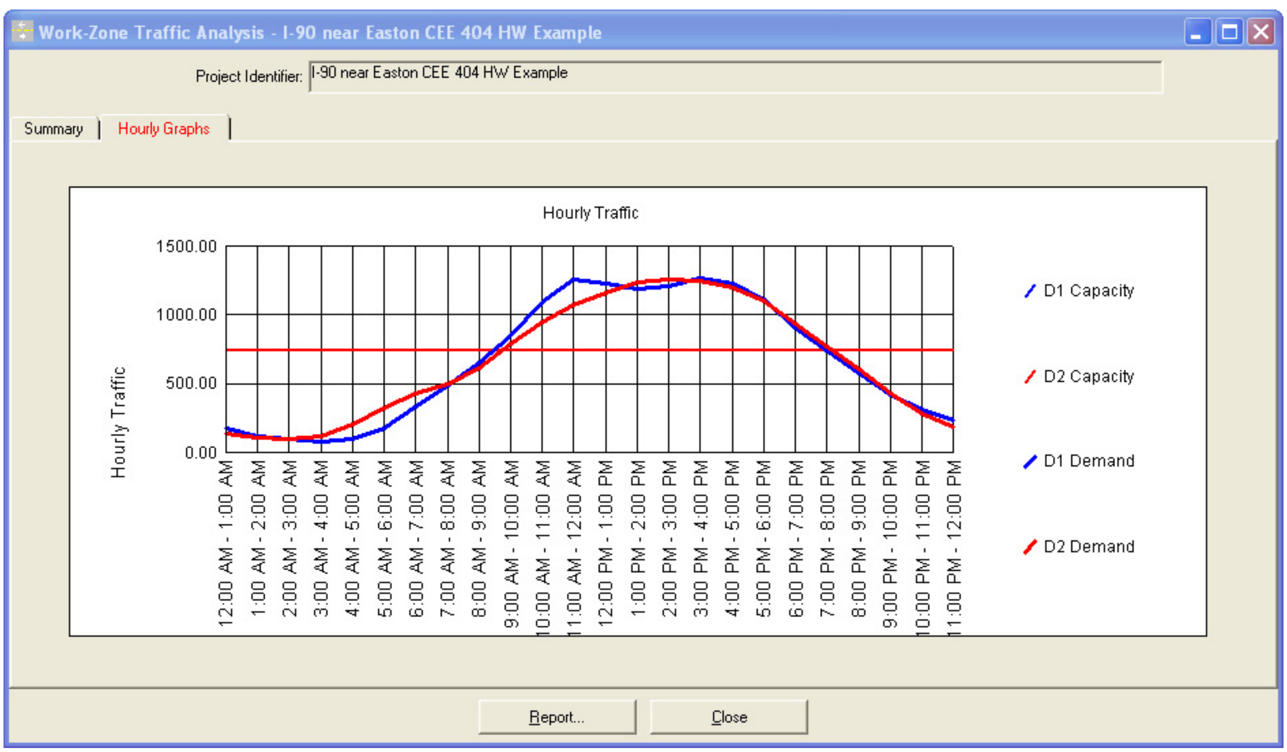


Figure 1.50. *CA4PRS Work-Zone Analysis hourly traffic results graph showing demand versus capacity.*

Currently, the CA4PRS user manual does not explain the delay estimation algorithm it uses. As of 2010, CA4PRS development is ongoing and licenses for state DOTs are free. CA4PRS only models traffic in the work zone and does not model any wider network.

LIFE-CYCLE ASSESSMENT (ENVIRONMENTAL ACCOUNTING)

Purpose

This section overviews a method for determining the inputs and outputs of a pavement system that are relevant to the environment. This can include, but is not limited to, energy use, water use, emissions, raw materials, and human health impacts. This method, called life-cycle assessment (LCA), is essentially an environmental accounting protocol. LCA results can be used as part of the decision-making process when determining the appropriate pavement rehabilitation or reconstruction strategy. For instance, if an owner-agency must comply with a greenhouse gas (GHG) reduction mandate, options resulting in less GHG may be considered more favorably. Often, but not always, environmental accounting results tend to agree with life-cycle assessment results in pavement construction scenarios.

In the future, it is likely that energy and emissions associated with roadway construction, or any industry, will be scrutinized more carefully. GHG emissions are likely to be subject to a cap-and-trade scheme in the United States and are increasingly being addressed through the National Environmental Policy Act (NEPA) as a recent White House Council on Environmental Quality guidance shows (Sutley, 2010). As this scrutiny increases, there will likely be more tools to help in analysis. It also seems plausible that, once industry has a fair idea what energy, emissions, and other resources are associated with roadway construction, it will begin to adopt (either voluntarily or by regulation) efficiency standards associated with these items similar to what has happened with the automobile industry (i.e., fuel efficiency standards), power generation (i.e., clean energy portfolio requirements), and even toilets (i.e., maximum allowable flow).

Measurement Methods

An LCA attempts to identify inputs and outputs of a system that are relevant to the environment from its inception to its ultimate disposition. This means that an LCA includes everything from gathering raw materials to the point at which those materials are returned to the environment. This collection of all processes from "cradle to grave" allows LCA to provide a cumulative total of inputs and outputs (e.g., energy, emissions, water, use) for a final product and the environmental impacts associated with those inputs and outputs. The resulting environmental impacts of these cumulative inputs and outputs is assessed, and results can be used to compare alternatives and improve the system. The International Standards Organization (ISO) outlines a systematic four-phase approach:

1. *Goal and scope* help define the reasons for carrying out the LCA, the intended audience, geographic and temporal considerations, system functions and boundaries, impact assessment, and interpretation methods.

2. *Inventory assessment* is performed to quantify life-cycle energy use, emissions, and land and water use for technology use in each life-cycle stage.
3. *Impact assessment* is used to estimate the impacts of inventory results.
4. *Interpretation* involves investigation of the contribution of each life-cycle stage and technology use throughout the life cycle, and includes data quality, sensitivity, and uncertainty analyses.

LCA in general, and for pavements in particular, is still in a relatively early stage of development and thus common practices are still developing and available data can be sparse. This presents problems when using LCA as a decision-support tool, especially when comparing alternatives. Results using different data sets, methods, and practices can be an order of magnitude different for the same analyzed pavement section. Common issues with LCA include the following:

- *Data sources.* Often LCA data come from a select few databases, such as the U.S. Life-Cycle Inventory Database (from the National Renewable Energy Laboratory), ecoinvent, and the ELCD database. These are generally reviewed for accuracy or errors and can help standardize information for use in LCAs. However, data usually come from many different sources, ranging from personal observation to national databases, which can lead to problems when comparing one LCA with another. For instance, the CO_2 associated with HMA production is not a universal constant but rather varies depending on plant type, components and manufacturer, aggregate moisture content, fuel type, amount of RAP included, asphalt binder grade, crude oil source, regional electricity mix, and so forth. Although databases of national averages can lead to some consistency in results between LCAs, they often do not provide the detail necessary to distinguish between process changes (e.g., using warm-mix asphalt or not). At the very least, an LCA should clearly identify its data sources.
- *Missing data.* There are many industrial processes where some, if not all, relevant data are not known, recorded, or made available for public use. For instance, the amount of fugitive dust on site associated with pavement construction is not generally known. Or, the exact chemical makeup of an asphalt modifier may be a trade secret that the manufacturer is not willing to divulge.
- *Outdated data.* Sometimes data exist but are outdated. Over time, processes change, equipment improves, raw material sources change, and so forth. For example, one of the more comprehensive sources for asphalt refining comes from Eurobitume and was produced in 2000.
- *Data specificity.* Although general average data may be more readily available or may lead to more consistency between LCAs, these data often do not contain the detail needed to distinguish between two alternatives being considered. For instance, the Environmental Protection Agency's AP-42 document contains average emissions data for asphalt plants; however, it assumes only an average amount of RAP being used at the plant. Therefore, if these data are used it cannot distinguish between a mix using all virgin materials and one using 25% RAP.

- *Setting boundaries.* An LCA that attempts to account for all processes associated with a system can quickly become intractable. For example, one could account for the slipform paver and its energy use and emissions associated with a concrete pavement. One could also account for the energy and emissions associated with manufacturing that slipform paver. However, that leads to potentially considering the energy and emissions associated with the manufacture of the machines that made the paver, and so on. Because of this, every LCA has a defined boundary that details which processes are included and which are not. Inclusions and exclusions are often not consistent between LCAs and can be controversial. For instance, in a pavement LCA one can choose whether to include the effect of material stiffness on the rolling resistance it offers to vehicles that travel upon it. Reduced rolling resistance over the life of a pavement may lead to substantial energy savings when summed over the millions of vehicles that may use the road.
- *Procedural practices.* Most LCAs generally follow ISO 14040 and ISO 14044. However, these standards are still quite generally written and leave much room for interpretation. No set of more precise LCA procedures exists for pavements.

Despite these limitations, LCA can provide meaningful results and aid the project decision process.

LCA Methods

There are two main methods typically used for LCA: the process-based approach and the economic input-output (EIO)-based approach. Both methods are acceptable for performing LCAs, although each has its strengths. Each method is briefly discussed here.

In *process-based LCA*, a selected system is chosen and defined so that it meets a set of desired requirements (e.g., a pavement structure to meet traffic, environmental, and structural requirements). This system is then broken down into separate processes (e.g., aggregate production, cement production, concrete transport) whose energy requirements and emissions can be quantified. Further contributory processes can be defined and analyzed (e.g., manufacture of the aggregate crushers used in aggregate production) but at some reasonable point a “boundary” must be established beyond which no downstream contributory processes are considered. The location of this boundary is an important part of an LCA because it may significantly affect the results. Ultimately, boundary locations are somewhat subjective, which can lead to difficulty in comparing one LCA’s results to another. Process-based LCAs are desirable because they can be done in enough detail so that they include processes that can differentiate between two options (e.g., using warm-mix asphalt or not). They are problematic because of the subjective boundary and difficulty in obtaining data on specific processes.

Economic input-output LCA (EIO-LCA) overcomes the subjective boundary issue and data availability issue by basing processes and their relationships on a national economic input-output model. An EIO model divides the economy of a country into industry-level sectors that represent individual activities in the selected economy and depicts the economic interaction of industries (sectors) in a nation (or a region) by showing how output of each sector is used as input for another. The system boundary is inherently the whole country’s entire economy. Interactions are represented by monetary value in a matrix form, called an economic input-output table (I-O table). The data stored in the table are collected by public agencies (e.g., the Department of Commerce) during a certain time period (usually 5 years). This process conveniently avoids collecting individual process data and sets a consistent boundary (the nation's economy). EIO-LCA can be problematic because it uses aggregate data, which can be inconsistently aggregated or do not contain enough detail to differentiate between two options (e.g., using warm-mix asphalt or not).

Typical Values

There have been a number of documented pavement LCAs in the past decade or so that can provide valuable information on typical values. Muench (2010) reviewed 12 pavement LCA papers and reports that documented 66 assessments of actual or hypothetical roadways and found the following:

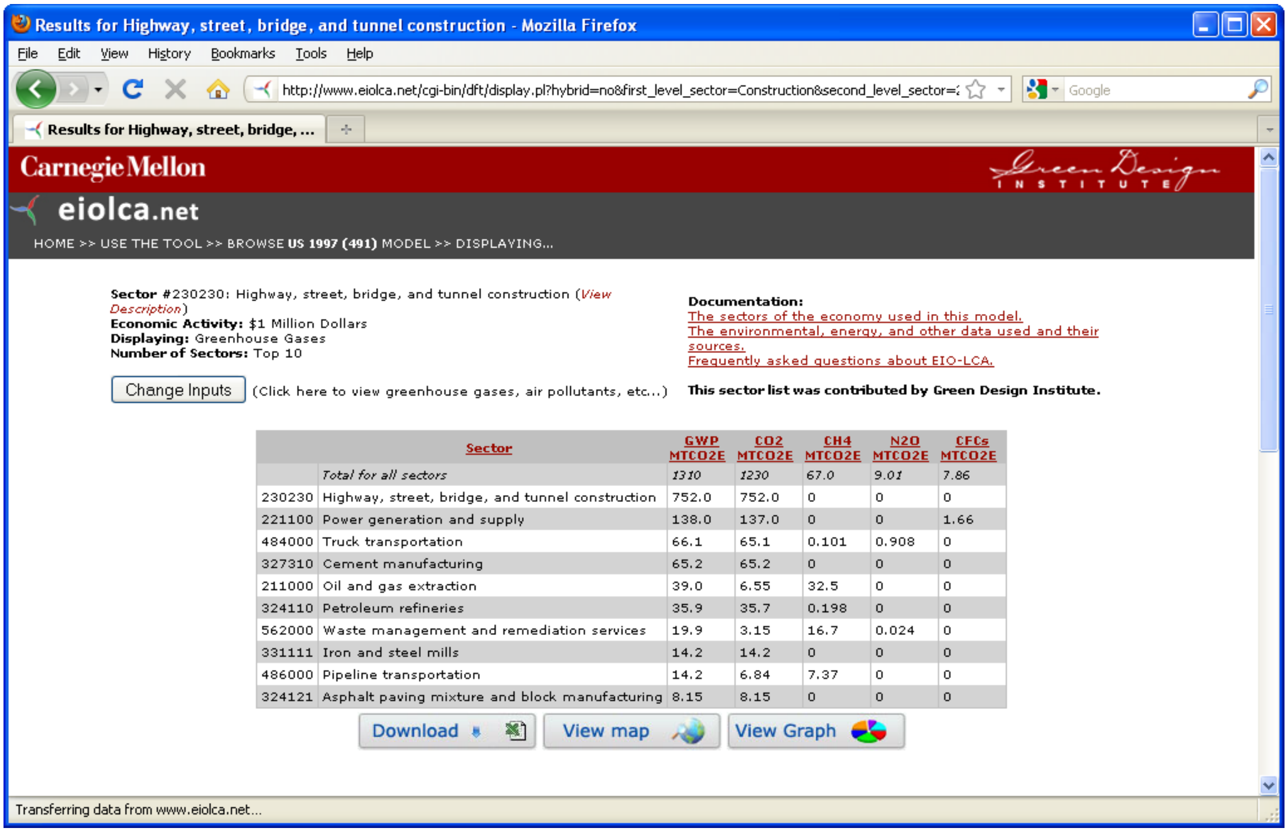
- *System scope.* Most LCAs tend to address the pavement structure only and not include other road features (e.g., striping, guardrails). Analysis periods are usually 40 to 50 years.
- *Relation of roadway construction to traffic use.* A good rule of thumb is that the energy expended in initial construction of a new roadway is roughly equivalent to the energy used by traffic on the facility over 1 to 2 years.
- *Relation of roadway construction to operations*. Operations are defined as those equipment, actions, and operations that happen on a routine basis necessary to ensure proper and safe roadway use. They include items such as lighting, traffic signals, deicing, sanding, drawbridge actions, and toll booths. Construction energy ranges from about 25% to 100% of operations energy.
- *Total energy use.* It can be loosely stated that energy expenditures per lane mile of pavement are typically on the order of 3 to 7 TJ depending on the pavement section, maintenance activities, and LCA scope.
- *CO_2 emissions.* It can be loosely stated that CO_2 emissions per lane mile of pavement are typically on the order of 200 to 600 tons depending on the pavement section, maintenance activities, and LCA scope.
- *Contribution of roadway construction components.* The following general statements are reasonable:
  - —Materials production accounts for about 60% to 80% of energy use and 60% to 90% of CO_2 emissions.
  - —Construction accounts for less than 5% of energy use and CO_2 emissions.
  - —Transportation associated with construction accounts for about 10% to 30% of energy use and about 10 percent of CO_2 emissions.
  - —Maintenance activities account for a broad range of about 5% to 50% of energy and CO_2 emissions.

Analysis Tools

At present, there are few tools available to help the nonspecialist conduct a meaningful pavement LCA; however, several efforts are under way to develop such tools. This section briefly overviews the few existing tools.

EIO-LCA is an online tool from Carnegie Mellon University's Green Design Institute (www.eiolca.net) that uses the EIO method to report U.S. economic sector averages of economic activity, greenhouse gases, energy, toxic releases, and water use for different processes (Figure 1.51). Answers for specific sectors can be obtained quickly; however, there is not enough detail to distinguish between processes within a sector (e.g., using warm-mix asphalt or not).

PaLATE is a Microsoft Excel–based tool from the University of California, Berkeley's Consortium on Green Design and Manufacturing that allows the user to input pavement construction and materials parameters and calculates life-cycle energy use and a number of life-cycle emissions parameters (Figure 1.52). It is primarily built on the EIO-LCA method, but it uses the process approach for a few items. PaLATE contains numerous errors in process data, computation, and physical input parameters. These errors are significant enough to cause results to be incorrect by orders of magnitude



	Sector	GWP MTCO2E	CO2 MTCO2E	CH4 MTCO2E	N2O MTCO2E	CFCs MTCO2E
	Total for all sectors	*1310*	*1230*	*67.0*	*9.01*	*7.86*
230230	Highway, street, bridge, and tunnel construction	752.0	752.0	0	0	0
221100	Power generation and supply	138.0	137.0	0	0	1.66
484000	Truck transportation	66.1	65.1	0.101	0.908	0
327310	Cement manufacturing	65.2	65.2	0	0	0
211000	Oil and gas extraction	39.0	6.55	32.5	0	0
324110	Petroleum refineries	35.9	35.7	0.198	0	0
562000	Waste management and remediation services	19.9	3.15	16.7	0.024	0
331111	Iron and steel mills	14.2	14.2	0	0	0
486000	Pipeline transportation	14.2	6.84	7.37	0	0
324121	Asphalt paving mixture and block manufacturing	8.15	8.15	0	0	0

Figure 1.51. *Output screen of the EIO-LCA online tool showing greenhouse gases associated with $1 million of economic activity in sector 230230 (highway, street, bridge, and tunnel construction) using the 1997 Industry Benchmark Model for producer prices.*

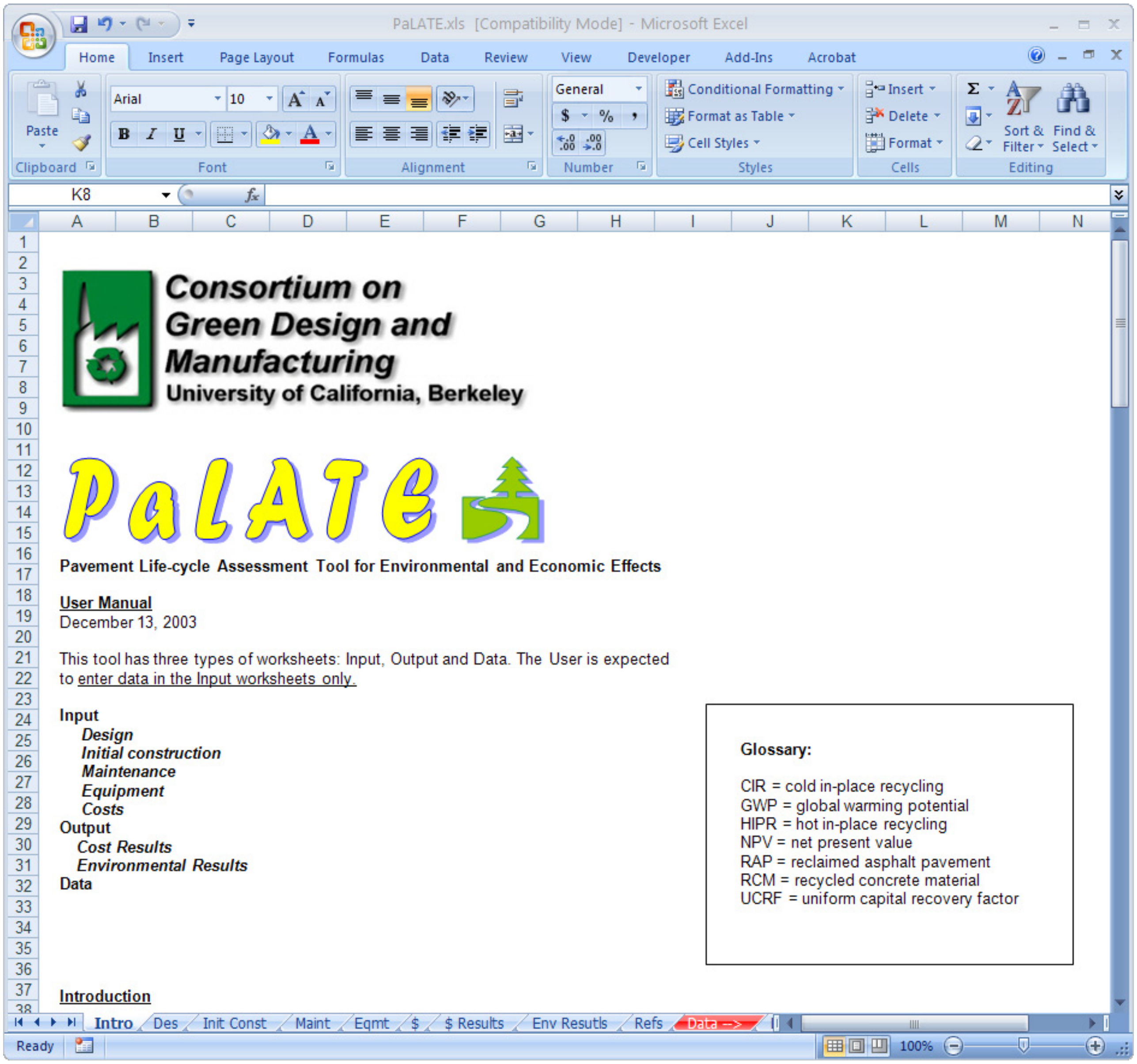


Figure 1.52. *Introduction screen for PaLATE from the Consortium on Green Design and Manufacturing at the University of California, Berkeley.*

in some cases, thus rendering PaLATE essentially useless. Recently, the University of Washington has rebuilt and simplified PaLATE (Figures 1.53 and 1.54) for interim use in their performance metric and has posted a working version on their website (www.greenroads.us). This version has not been validated by any outside party.

CHANGER is a computer software program from the International Road Federation (IRF) that calculates the life-cycle CO_2 emissions associated with pavement construction. It uses a process-based method and has been analyzed and validated by

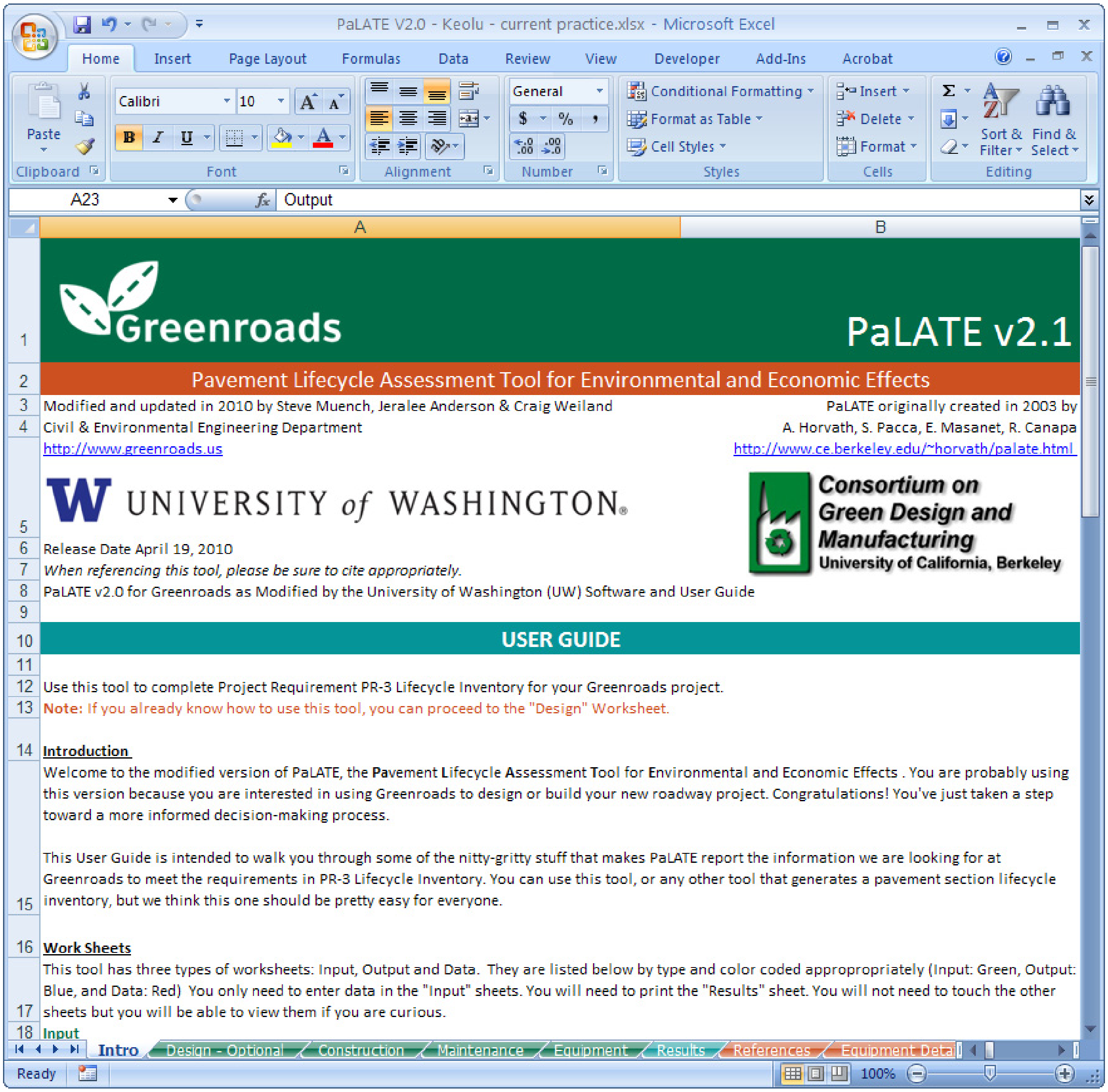
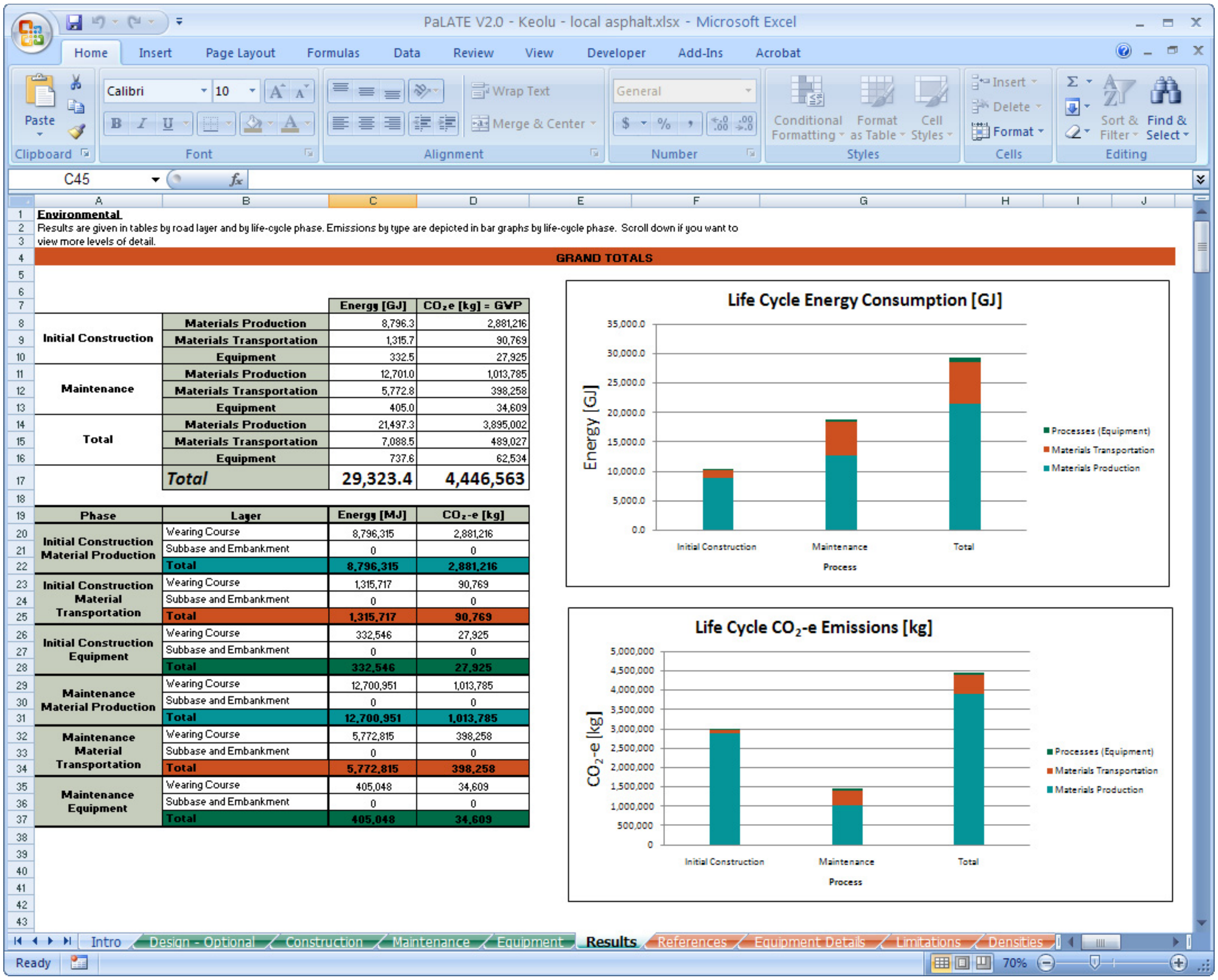


Figure 1.53. *Introduction screen for PaLATE as modified by the University of Washington for use with Greenroads.*

the Traffic Facilities Laboratory (LAVOC) of the Swiss Federal Institute of Technology, or Ecole Polytechnique Fédérale de Lausanne (EPFL). At present it only reports CO_2 emissions. The IRF plans to expand this tool to address the entire roadway (i.e., beyond just the pavement to include signs, striping, guardrail, etc.).



		Energy [GJ]	CO_2e [kg] = GWP
Initial Construction	Materials Production	8,796.3	2,881,216
	Materials Transportation	1,315.7	90,769
	Equipment	332.5	27,925
Maintenance	Materials Production	12,701.0	1,013,785
	Materials Transportation	5,772.8	398,258
	Equipment	405.0	34,609
Total	Materials Production	21,497.3	3,895,002
	Materials Transportation	7,088.5	489,027
	Equipment	737.6	62,534
	Total	29,323.4	4,446,563

Phase	Layer	Energy [MJ]	CO_2-e [kg]
Initial Construction Material Production	Wearing Course	8,796,315	2,881,216
	Subbase and Embankment	0	0
	Total	8,796,315	2,881,216
Initial Construction Material Transportation	Wearing Course	1,315,717	90,769
	Subbase and Embankment	0	0
	Total	1,315,717	90,769
Initial Construction Equipment	Wearing Course	332,546	27,925
	Subbase and Embankment	0	0
	Total	332,546	27,925
Maintenance Material Production	Wearing Course	12,700,951	1,013,785
	Subbase and Embankment	0	0
	Total	12,700,951	1,013,785
Maintenance Material Transportation	Wearing Course	5,772,815	398,258
	Subbase and Embankment	0	0
	Total	5,772,815	398,258
Maintenance Equipment	Wearing Course	405,048	34,609
	Subbase and Embankment	0	0
	Total	405,048	34,609

Figure 1.54. *Screenshot of PaLATE output as modified by the University of Washington for use with Greenroads.*

Example LCA Using PaLATE as Modified by the University of Washington for Greenroads

A local collector road in Kailua, Hawaii, is scheduled for repaving. The work essentially involves removing 6 in. of HMA with a milling machine and replacing it with two layers of HMA: a 4-in. base course and a 2-in. surface course. Initial construction quantities are as follows:

- Surface course: 9,516 tons of HMA.
  - —5.5% asphalt by total weight of mix.
  - —No recycled material in the mix.

- Base course: 18,790 tons of HMA.
  - 5% asphalt content by total weight of mix.
  - 10% glass cullet by total weight of mix.
- Tack coat: 0.15 gallons/yd^2 over 79,386 yd^2 = about 61 yd^3 of asphalt emulsion.
- Milling: 79,386 yd^2 of 6-in.-deep milling.

An LCA is performed using a 50-year analysis period and assuming a 2-in. mill and fill every 10 years (years 10, 20, 30, and 40). Preservation mill-and-fill quantities are as follows:

- Surface course: 7,913 tons of HMA.
  - 5.5% asphalt by total weight of mix.
  - No recycled material in the mix.
- Tack coat: 0.15 gallons/yd^2 over 79,386 yd^2 = about 61 yd^3 of asphalt emulsion.
- Milling: 79,386 yd^2 of 2-in.-deep milling.

Materials for this process come from the following locations:

- Aggregate, HMA, and RAP from a local quarry 6 mi from the job site.
- Asphalt from a local asphalt terminal 30 mi from the job site.

Results from this analysis are as follows (Figure 1.54):

- 29,323.4 GJ of life-cycle energy consumption.
- 4,446,563 kg of life-cycle CO_2 equivalent emissions.

Using this baseline scenario, several other options were investigated to determine LCA impacts:

- Remove the glass cullet from the base course (No Glass Cullet).
- Include 10% RAP in the surface and base courses (10% RAP).
- Include 20% RAP in the surface and base courses (20% RAP).
- Include 20% RAP in the surface course and 40% RAP in the base course (20% surface/40% base RAP).
- Include 30% RAP in the surface course and 40% RAP in the base course (30% surface/40% base RAP).
- Use warm-mix asphalt assuming a 20% reduction in energy and CO_2 emissions from the HMA manufacturing process only (WMA).
- Use a stone-matrix asphalt (SMA) surface course at 6.5% asphalt by total weight of mix that allows a surface life of 15 years. This results in resurfacing at years 15 and 30 only.
- Use an ultimate combination of a SMA surface course, no glass in the base course, 40% RAP in the base course, and warm-mix asphalt for both courses (Ultimate).

Figures 1.55 through 1.57 show the percentage change from the baseline practice in terms of energy consumption.

Some general conclusions that can be reached using this example are the following:

- Extending service life can be the biggest single influence in energy used and CO_2 emitted by the pavement.
- Often, a combination of options can produce an even greater savings in energy used and CO_2 emitted by the pavement.
- The inclusion or exclusion of the glass cullet (inclusion is a state requirement) makes very little difference in energy used and CO_2 emitted by the pavement.

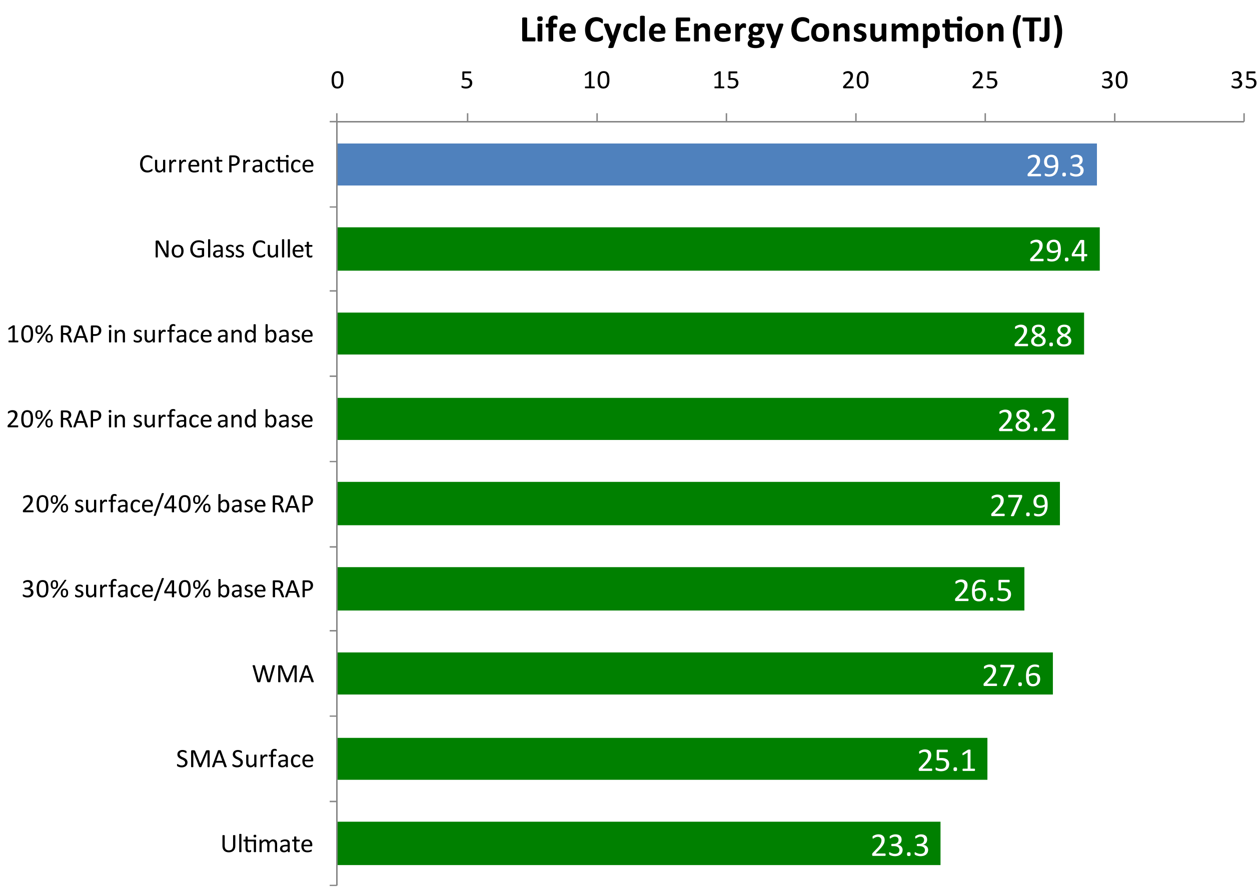


Figure 1.55. *Life-cycle energy consumption for the current practice and eight alternate scenarios for the example LCA.*

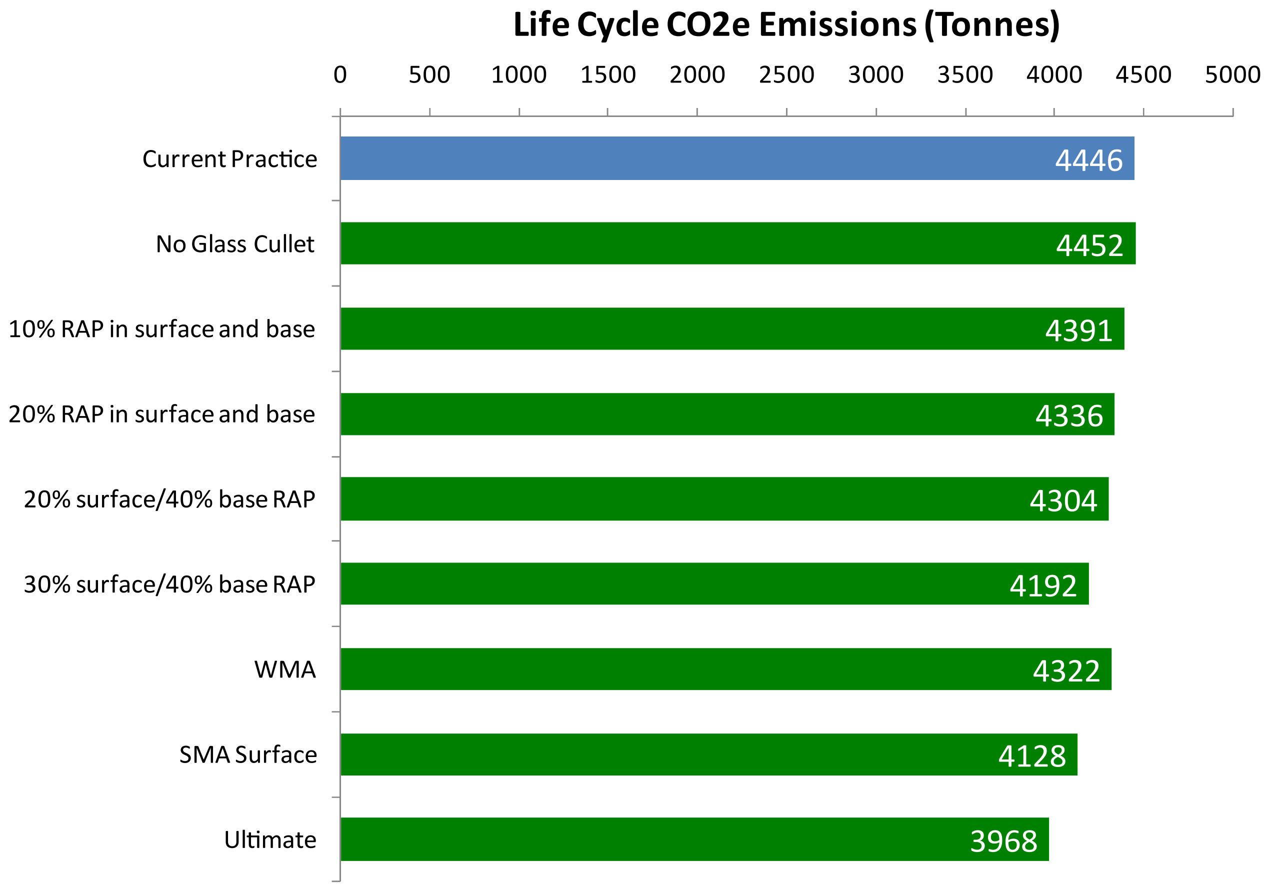


Figure 1.56. *Life-cycle CO_2 equivalent emissions for the current practice and eight alternate scenarios for the example LCA.*

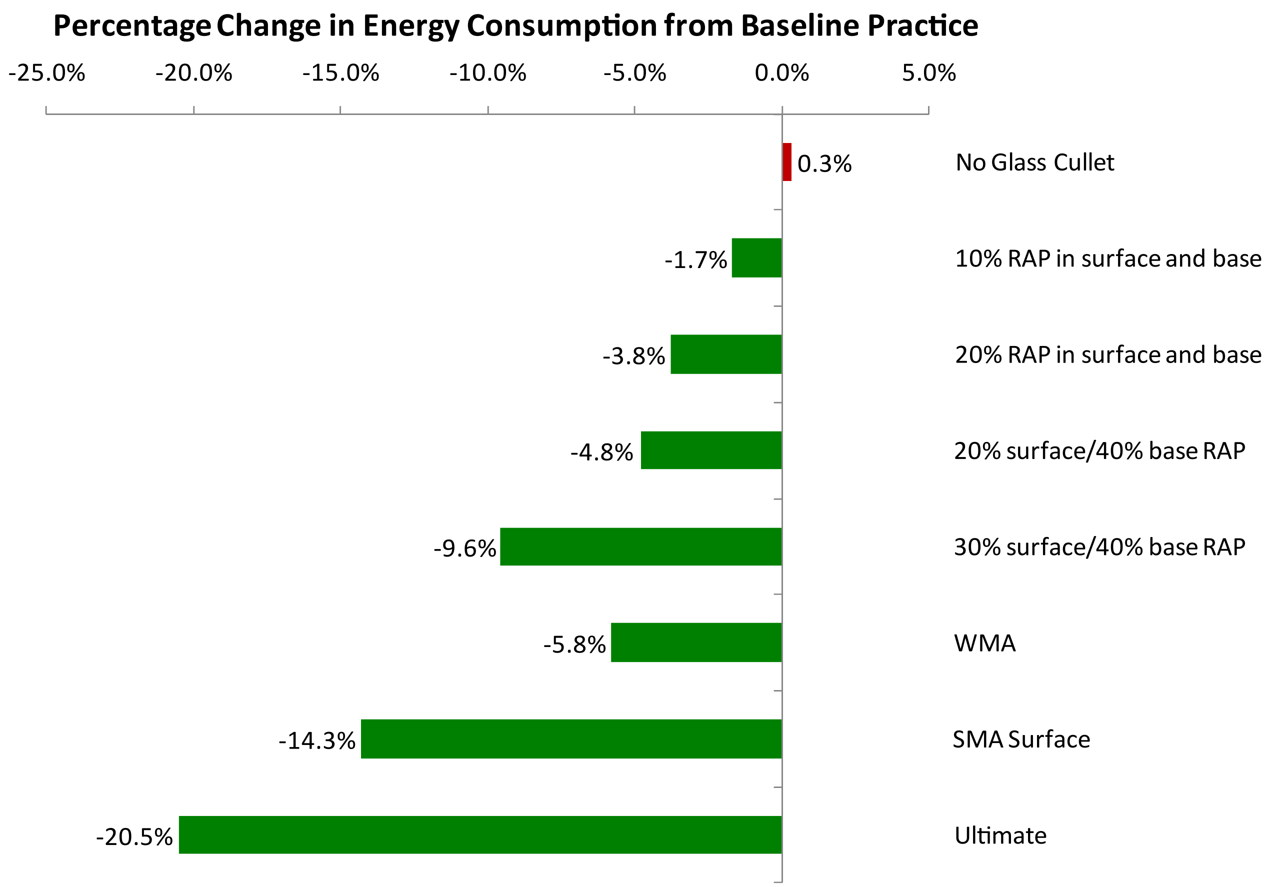


Figure 1.57. *The percentage change from the baseline value of energy consumption for a number of alternate scenarios for the example LCA.*

Further Reading on Pavement LCAs

Athena Institute. "A Life-Cycle Perspective on Concrete and Asphalt Roadways: Embodied Primary Energy and Global Warming Potential," Report to the Cement Association of Canada, 2006.

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MISCELLANEOUS MATERIAL PROPERTIES

Purpose

This section provides summaries of material properties that are relevant in designing pavement renewal options.

Material Properties

Table 1.40 shows typical layer moduli for several material conditions. Table 1.41 shows information about rubblized PCC and Table 1.42 about crack-and-seat renewal.

TABLE 1.40. HMA PAVEMENT TYPICAL MODULI AND RANGES OF MODULI

Material	Modulus Range (psi)
HMA (temperature dependent)	50,000–4,000,000
Cracked HMA range	50,000–500,000
Cracked HMA (10% of wheelpath—slight to moderate fatigue cracks)	100,000–250,000
Pulverized HMA	40,000

TABLE 1.41. PCC PAVEMENT RUBBLIZATION TYPICAL MODULI AND RANGES OF MODULI

Material	Value or Property
Ratio of rubblized PCC elastic modulus to original PCC slab elastic modulus	0.05
Slab modulus range before rubblization	Range: 3,000,000–7,000,000 psi
Typical slab modulus	4,000,000 psi
Rubblized PCC modulus	Range: 40,000–700,000 psi
Typical rubblized PCC modulus	150,000 psi

TABLE 1.42. CRACK-AND-SEAT AND BREAK-AND-SEAT RENEWAL

Material	Value or Property
Typical modulus of crack-and-seated PCC pavement	200,000 psi
Modulus of crack-and-seated PCC pavement	Range: 200,000–800,000 psi
Modulus of break-and-seated PCC pavement	Range: 250,000–2,000,000 psi

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2

FLEXIBLE PAVEMENT BEST PRACTICES

INTRODUCTION

For purposes of this study, long-life pavement is defined as pavement sections designed and built to last 50 years or longer without requiring major structural rehabilitation or reconstruction. Only periodic surface renewal in response to distresses confined to the top of the pavement would be required. This document was developed by the study team with input from state departments of transportation (DOTs) and hot-mix asphalt (HMA) paving contractors.

The intent of the long-life pavement concept is to significantly extend current pavement design life by restricting distress, such as cracking and rutting, to the pavement surface. Common distress mechanisms such as bottom-up fatigue cracking and rutting in the unbound layers should, in principle, be completely eliminated. However, surface-initiated (top-down) cracking will still be possible. This type of cracking is caused by a complex combination of pavement structure, load spectra, and environmental and material characteristics. Although its causes are still not fully resolved, this deterioration mechanism involves a fatigue-like response in the upper layers of the pavement. In addition to fatigue cracking and rutting, in cold climates, low-temperature cracking and frost heave must also be taken into account. Another deterioration mechanism that should be accounted for is aging. Aging mainly affects the top asphalt layers and is manifested by increased stiffness and decreased flexibility over time. A common denominator of the distress mechanisms mentioned above is they are difficult to model using current mechanistic-empirical methods. In the case of top-down cracking and permanent deformations in the asphalt-bound layers, new and improved design methods may address this in the future.

When using existing pavements, the inhibition of reflective cracking is crucial. Reflective cracking is caused by repetitive shearing—for example, when a new asphalt layer is laid upon an already cracked layer. With time, the crack will propagate through the new layer. This is true no matter the existing pavement type [i.e., distressed HMA or portland cement concrete (PCC)], although experience shows that reflective cracking can be more predominant when the existing pavement is a PCC. Reflection cracking can occur in an HMA overlay over any joint or crack in the PCC pavement. The current state of the art does not provide accurate methods to predict the occurrence and growth of the reflection crack. However, a number of approaches have been shown to minimize or eliminate these occurrences. These approaches are discussed in the following sections along with a discussion of those features and construction processes that are considered critical to produce long-life pavements.

HOT-MIX ASPHALT (HMA) RENEWAL STRATEGIES

The most promising renewal strategies for long life using existing pavements are the following:

- HMA over HMA renewal methods
  - HMA over existing HMA pavement
  - HMA over reclaimed HMA (recycling)
- HMA-over-PCC renewal methods
  - HMA over existing HMA-surfaced composite pavements
  - HMA over crack-and-seated jointed plain concrete (JPC) pavements
  - HMA over saw, crack, and seat jointed reinforced concrete (JRC) pavements
  - HMA over rubblized JPC pavements
  - HMA over existing continuously reinforced concrete (CRC) pavements

Each strategy will be described in this document.

GENERAL GUIDING PRINCIPLES

The following are guiding principles for any renewal solution to achieve good-performing long-life pavements:

- Keep the renewal solution as simple as possible, but not too simple so as to not address critical underlying problems.
- The quality of construction is essential in achieving long-life pavements.
- Pavements are supposed to act as one layer; therefore, the bond between layers should never be compromised, and a few thick layers are always better than multiple thin layers.
- All joints are weaknesses; therefore, they need to be treated as such.

- Good, continuous, and sustainable drainage is essential to long-life pavement; therefore, no matter how thick the renewal solution is, it can fail if drainage is not provided.
- Foundation uniformity is essential to reduce and/or eliminate stress concentrations, which can cause future cracking.
- A solid foundation allows good compaction; unsupported edges can never be properly compacted.
- Thermal movements of the existing pavement are the underlying cause for much reflective cracking; therefore, they must be eliminated (by fracturing the existing pavement).
- Good-performing asphalt mixtures should have high binder content and low air voids (to have high durability), and smaller nominal size (to avoid segregation).

The following sections provide best practices (guidelines) for each rapid renewal strategy to achieve long-life pavements based on relevant literature and agency information.

HMA OVERLAYS OVER EXISTING HMA PAVEMENTS

Criteria for Long-Life Potential

This renewal solution is viable as long as the following critical features are met:

- The surface condition is good and the structural capacity of the existing asphalt concrete (AC) pavement is adequate for a potential long-life pavement.
- There is no evidence of stripping in any of the existing HMA layers [determined through coring and/or ground-penetrating radar (GPR) testing].
- Proper repair and surface preparation is provided for the existing surface layer, and a good tack/bond coat is provided.
- The existing drainage system is in good working condition, or adequate drainage is provided.

If there is no visible distress in the existing HMA pavement other than in isolated areas, the existing pavement can be directly overlaid as long as it is structurally sound, level, clean, and capable of bonding to the overlay. Small areas of localized distresses in the existing pavement should be repaired or replaced to provide the required structural support. Milling before placing an overlay significantly aids the bond between the old and new HMA.

When there is visible surface distress and it is determined that cracking is only present near the surface (through coring), the first step in the resurfacing process is the removal of the existing surface to the depth of the cracking. This could vary between 1 and 4 in. of milled depth. The milled material would be replaced, and an additional thickness would be paved to ensure that limiting-strain criteria are met. This layer would need to have the same characteristics as the original surface (i.e., rut resistance, durability, thermal cracking resistance, and wear resistance). Figure 2.1 shows a typical milling operation.

Figure 2.1. *Typical milling operation of existing HMA layer.*
Source: WSDOT, 2010.

After a pavement has been milled, the surface should be cleaned by sweeping or washing before any overlay is placed; otherwise, the dirt and dust will decrease the bond between the new overlay and the existing pavement. When sweeping, more than one pass is typically needed to remove all the dirt and dust. If the milled surface is washed, the pavement must be allowed to dry before paving.

It is essential that bonding between the new wearing course and the existing pavement be ensured to achieve long-life performance of the resurfaced pavement. A tack/bond coat is needed to ensure this bond. A tack coat should be applied uniformly across the entire pavement surface and result in about 90% surface coverage (by ensuring double or triple coverage during spraying). Sufficient time should be allowed for the emulsion to break and dry before the next layer of HMA is applied. Figure 2.2 shows examples of good and poor tack-coat application. Milling the existing surface before an overlay significantly aids the bond between the two layers.

Construction (longitudinal and transverse) joints should be minimized to the extent possible. Joints should be staggered between successive layers, to prevent a potential direct path for water, and sealed. Care should be taken to maximize the compaction (reduce the air voids) near joints, although it is difficult to achieve the same level of compaction as the main mat. The difference in air voids near joints should not be more than 2% relative to the density of the main mat. Furthermore, no joints should be allowed within the area of the wheelpaths. Consideration should be given to sealing the longitudinal joints in addition to the emphasis on joint density.

It is assumed that the existing pavement structure is competent enough to provide 50 years of service with the addition of sufficient overlay thickness. This condition will only be met by an existing pavement that is structurally sound and thick enough to satisfy limiting-strain criteria. It is also assumed that this approach would be included in a project where additional lanes are constructed and the existing pavement is utilized to the extent possible.


(a)


(b)

Figure 2.2. *(a) Good tack coat. (b) Poor tack coat (left portion of photo).*
Source: WSDOT, 2010.

The main limitation of this renewal solution is that reconstruction (i.e., removal of the existing pavement structure) is necessary if the condition of the existing base or subbase and/or subgrade is poor, or if the existing pavement is not structurally sound.

HMA over Existing HMA and Specifications

A selection of significant practices associated with paving HMA over existing HMA were chosen and are included in Table 2.1. The table includes a brief explanation of why the issue is of special interest along with examples from the recommendations in the Guide Specifications (Chapter 4). Three major practices are featured: (1) milling of existing HMA, (2) tack coat between HMA lifts, and (3) longitudinal and transverse joints.

HMA OVER RECLAIMED HMA PAVEMENT

Criteria for Long-Life Potential

This renewal solution is necessary if the surface condition of the existing HMA layer is poor and the depth of the distress (cracking) is deeper in the pavement section. To enable use of the existing pavement, this solution entails the pulverization of the existing HMA layer. However, by definition, once this solution is adopted, the reclaimed HMA material is considered a base layer and its thickness should not be included in the total thickness that is used to calculate the limiting tensile strain at the bottom of the new HMA layer.

Similar to using existing HMA pavement, the partial-depth and full-depth reclamation (FDR) renewal solution is viable only if the following critical features are met:

- Proper surface preparation is provided for the reclaimed HMA layer, and a good tack/bond coat is provided between the reclaimed base and the new HMA overlay.

TABLE 2.1. SUMMARY OF BEST PRACTICES AND SPECIFICATIONS FOR HMA OVER EXISTING HMA PAVEMENT

Best Practice	Why This Practice?	Typical Specification Requirements
Milling of existing HMA	Existing cracks in the wearing course must be removed before HMA overlay to reduce the potential for reflection cracks in the new HMA layer. Milling is considered superior to crack sealing before placing an HMA overlay and also aids the bond between the existing and new HMA.	Equipment must consistently remove the HMA surface, in one or more passes, to the required grade and cross section, producing a uniformly textured surface. Machines must be equipped with all of the following: • Automatically controlled and activated cutting drums. • Grade reference and transverse slope control capabilities. • An approved grade referencing attachment, not less than 30 ft in length. An alternate grade referencing attachment may be used if approved by the engineer before use.[a]
Tack coat between HMA lifts	It is essential that bonding between the new HMA layers courses and lower layers (such as the existing pavement) be achieved to ensure long-life performance. If this is not done, then excessive tensile strains occur resulting in fatigue cracking. This is critical for the wearing course. Keep traffic off the fresh tack to the extent possible.	• Apply the bond coat to each layer of HMA and to the vertical edge of the adjacent pavement before placing subsequent layers. • Apply a thin, uniform tack coat to all contact surfaces of curbs, structures, and all joints. • Apply undiluted tack at a rate ranging from 0.05 to 0.10 gal/yd^2. • Consider the use of a hot tack (traditional paving-grade asphalt cement)—reduces wheel tracking and provides a consistent tack coat that is less susceptible to run-off during a rain event.[b]
Longitudinal and transverse joints	There are two major issues: (1) achieve proper joint density, and (2) stagger the joints. If the joint density is low, then high air voids are the result—a typical restriction is no more than 2% higher voids in the joint than the middle of the HMA mat. If this type of criterion is violated, this leads to early joint raveling and cracking. Staggering the joints helps to prevent a direct path for water entering the pavement structure. Consider sealing longitudinal joints.	• Stagger joints according to AASHTO Guide Specification 401. An exception to the use of staggered joints can be made for achieving crown lines. • The minimum density of all traveled-way pavement within 6 in. of a longitudinal joint, including the pavement on the traveled-way side of the shoulder joint, shall not be less than 2.0% below the specified density when unconfined.[c]

[a] For more details, refer to Elements for AASHTO Specification 409 in Chapter 4.
[b] For more details, refer to Elements for AASHTO Specification 404 in Chapter 4.
[c] For more details, refer to Elements for AASHTO Specification 401 in Chapter 4.

- The foundation (subgrade) support is good (e.g., the back-calculated subgrade modulus is adequate for the planned section).
- Drainage is adequately addressed.

The main limitation of this renewal solution is that the performance of partial- and full-depth reclamation with cement or asphalt emulsion has not been substantiated for a long life (>50 years); therefore, their use in the context of long-life pavements has not yet been fully proven in the field. Records on performance are highly variable as there has not been a common definition applied to judge the comparative performance levels. Causes commonly noted for poor performance using cold in-place recycling (CIPR) include the following (Hall et al., 2001): (1) use of an excessive amount of recycling agent, (2) premature application of a surface seal, (3) recycling only to the depth of an asphalt layer, resulting in delamination from the underlying layer, and/or (4) allowing a project to remain open for too long into the winter season. In addition, excessive processing can result in higher fines content, leading to rutting due to low stability.

Construction Operations

In the FDR process, a reclaimer pulverizes the existing pavement and its base 4 to 10 in. deep and mixes in asphalt emulsion. Portland cement, lime, and/or other materials can also be added as required to achieve desired mix quality, although the potential for shrinkage cracking that will reflect through the HMA layers is possible when dealing with cementitious materials. When only asphalt emulsion or foamed asphalt is used, it is directly blended within the reclaimer unit. When other cementing agents are added (e.g., dry lime, fly ash, or cement), they are spread with a vane spreader before blending. The mixed material is next compacted with a pad foot compactor, then bladed to level the surface. The level surface is then compacted with rubber tire rollers, followed by blade and steel face roller, without vibration, to shape. Finally, the new HMA base, wearing, and surface courses are added to satisfy long-life criteria. Figure 2.3 shows pictures of FDR construction with different stabilizing agents.

Partial-depth reclamation by CIPR is limited to correcting only those distresses that are surface problems in the asphalt layer (Hall et al., 2001). Typically, this involves recycling of the asphalt-bound layers to a depth of 3 to 4 in. The finished product is considered as a base only; therefore, new HMA base, wearing, and surface courses should be added to satisfy long-life criteria.

CIPR is accomplished by a self-contained, continuous train operation that uses a milling machine to remove the existing surface layers to a given depth (up to about 4 in.). The material is sized with the oversized material crushed and rescreened. The material is then mixed in a pug mill, with asphalt cement or special asphalt-derived products (cationic, anionic, and polymer-modified emulsions or foamed asphalt, rejuvenators, and recycling agents developed especially for CIPR processes). Virgin aggregate might be added to complete the mix. The resulting mix is then laid using a reclaim/paver unit. After about 30 min of curing and drying, the material is compacted with a large rubber-tired roller, followed by a vibratory steel drum roller. Curing of about 2 weeks during favorable weather conditions (preferably at temperatures at or in excess of 60°F) is needed before the new HMA overlay is applied (Federal Highway Administration, 1997). The addition of quick lime has been used to significantly reduce the cure time. Figure 2.4 shows typical CIPR train operations.

Figure 2.3. *FDR construction with different stabilizing agents. (a) FDR with asphalt emulsion. (b) FDR with cement/fly ash stabilizer. (c) FDR with asphalt emulsion and dry lime. (d) FDR with foamed asphalt.*
Source: Bang et al., 2010.

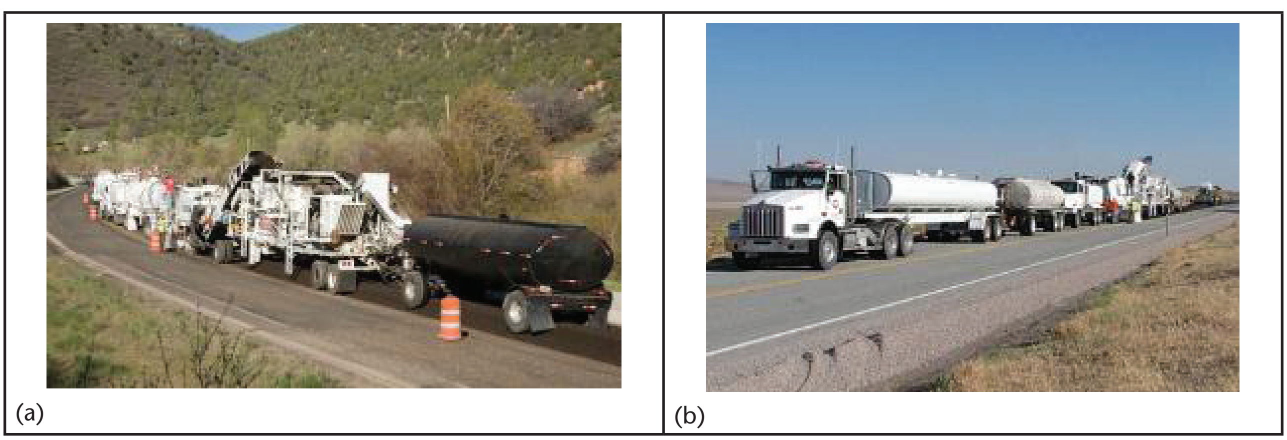

Figure 2.4. *Typical CIPR train operation. (a) CIPR train with engineered asphalt emulsion. (b) CIPR train with addition of lime slurry or cement in slurry.*
Source: Cold In-Place Recycling, 2010.

Quality Control

The crucial initial step in the quality control of CIPR mixes is in the pavement-type selection process. Pavements with rutting, heavy patching, or chip seals are not good candidates for CIPR projects. Core specimens should be taken from the existing HMA and examined for variations in pavement layers including delaminations and evidence of saturated material.

The quality control of the reclaimed asphalt pavement (RAP) material itself is essential to ensure the success of a CIPR mix. This should involve taking random samples of the recycled material to analyze for aggregate gradation, asphalt content, and moisture content. Care should be taken to ensure that the RAP is consistent in size and appearance and is free of contaminants.

Field quality control measures during CIPR operations should include monitoring the depth of scarification, the coating of the aggregate by the emulsion, the proper curing of the emulsion, the visual appearance and possible segregation of the recycled material, the compaction procedure, and appearance of the recycled pavement surface after compaction. The recycled mix should be monitored for gradation, emulsion content, moisture content, and in-place density. Compaction of CIPR paving mixtures is normally accomplished at a moisture content of less than 2% at a minimum of 97% of laboratory maximum density (Federal Highway Administration, 1997).

HMA OVER RECLAIMED HMA PAVEMENT AND SPECIFICATIONS

A significant practice associated with the gradation of the pulverized material was selected and included in Table 2.2. The table includes a brief explanation of why the issue is of special interest, along with examples from the recommendations in the Guide Specifications (Chapter 4). One major practice is featured, which is the gradation of the pulverized material.

TABLE 2.2. BEST PRACTICES AND SPECIFICATIONS FOR HMA OVER RECLAIMED HMA PAVEMENT

Best Practice	Why This Practice?	Typical Specification Requirement[a]
Gradation of pulverized material	The existing pavement to be remixed with binder must have a gradation, and specifically the maximum particle size, small enough that the mixing process achieves well-coated particles.	• The gradation of the pulverized material must achieve 100% passing the 2-in. sieve and 90%–100% passing the 1.5-in. sieve. • Subgrade materials that can contaminate the pulverized asphalt pavement should be rejected.

[a] For more details, refer to Elements for AASHTO Specification 411 in Chapter 4 and AASHTO Guide Specification 411.

HMA OVERLAYS OVER EXISTING HMA-SURFACED COMPOSITE PAVEMENTS

A viable long-life HMA renewal solution for HMA-over-concrete pavement is to mill the old HMA overlay and consider the HMA-over-PCC renewal methods described below (crack-and-seat JPC pavements; saw-cut, crack, and seat JRC pavements; or rubblize PCC pavement). Figure 2.5 shows a photo of an exposed concrete pavement after removal of the HMA overlay.

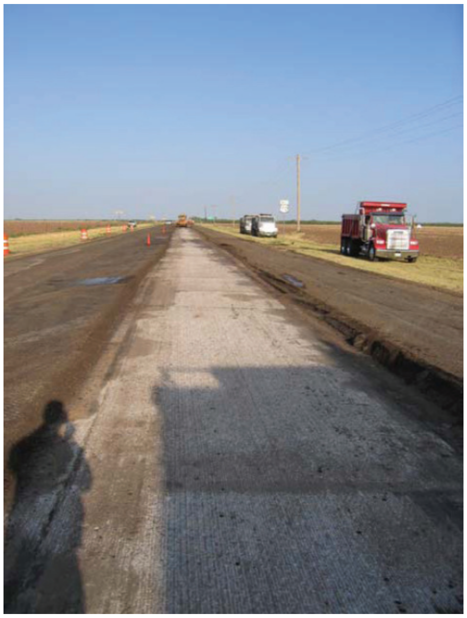

Figure 2.5. *Existing concrete pavement exposed after removal of HMA layer.*
Source: Sebesta and Scullion, 2007.

HMA OVER CRACK-AND-SEAT JOINTED PLAIN CONCRETE (JPC) PAVEMENTS

Criteria for Long-Life Potential

This renewal solution is only suitable for plain (unreinforced) concrete pavements. The rationale behind the crack-and-seat technique is to shorten the effective slab length between the transverse joints or cracks in the existing concrete pavement before placing the HMA overlay. This will distribute the horizontal strains resulting from thermal movements of the concrete more evenly over the existing pavement, thus reducing the risk of causing reflective transverse cracks in the overlay. Care must be taken during cracking operations such that the induced concrete cracks are kept vertical and fine (tight). Generally, the cracking of the PCC slabs is in the transverse direction; however, the addition of longitudinal cracking between wheelpaths has shown good performance by Caltrans. Verification coring should follow to ensure that fine, full-depth vertical cracks are achieved (see Figure 2.6).

The renewal solution of HMA overlay over crack-and-seat concrete is viable as long as the following critical features are met:

- There is no evidence of pumping underneath the existing slabs.
- The foundation support is good (i.e., there are no voids between the concrete slab and the underlying base or subbase).
- The existing drainage system is in good working condition.

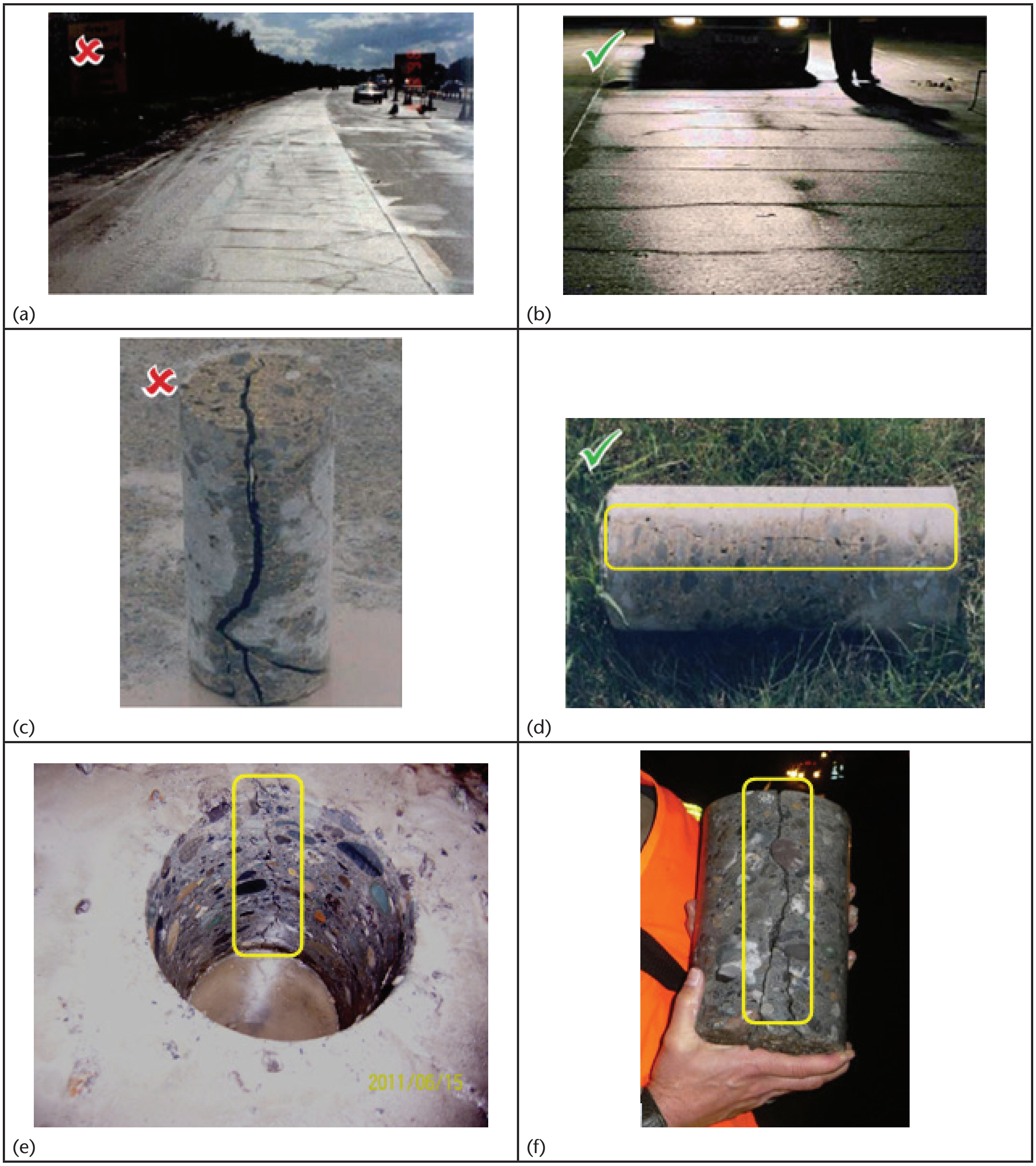

Figure 2.6. *Poor and good practices of crack and seat. (a) Example of excessive longitudinal cracking. (b) Example showing good transverse cracking. (c) Noncompliant core: overcracked. (d) Compliant core: fine, full-depth vertical crack. (e) Compliant crack illustrated by core hole. (f) Compliant crack illustrated by reassembled core.*
Sources: (a) through (d), Jordan et al., 2008. (e) and (f), WSDOT, 2010.

However, the following limitations and additional cautions are warranted:

- The performance of HMA overlays on crack-and-seat concrete pavements has been variable in the United States; therefore, it is unclear whether their efficacy is 50 years or longer. This could be tied to the quality of the cracking operation. If construction guidelines are put in place to ensure the realization of closely spaced, tight, full-depth vertical cracks, then potential for long life should be achievable. Experience in the United Kingdom has been excellent, but with a strict quality control process and HMA overlay thickness in excess of 7 in. Thinner overlays like those commonly used in the United States were not found to work as well in test sections in the United Kingdom (Coley and Carswell, 2006). The need for informed inspectors on the jobsite during cracking operations cannot be overemphasized.
- If the foundation underneath the existing concrete is not sufficiently strong, the crack-and-seat operation may cause excessive structural damage to the existing pavement.

Caltrans (2004) has extensive experience with crack and seating of PCC slabs followed by an HMA overlay. The agency applies this treatment wherever the PCC pavement has an unacceptable ride and extensive slab cracking. The typical crack spacing is about 4 ft by 6 ft followed by seating with five passes of a pneumatic-tired roller of at least 15 tons (Caltrans, 2008). For a number of years (1980s through the 1990s), the overlay thickness associated with the crack-and-seat process ranged from a minimum of 4 in. up to about 6 in. Service-life expectation was a minimum of 10 years with these thicknesses [or about 10 to 20 million equivalent single-axle loads (ESALs)]. Starting in 2003 with the Interstate 710 rehabilitation of existing 8-in.-thick PCC slabs near Long Beach, California (Monismith et al., 2009a, 2009b), the crack-and-seat process has been followed by HMA overlays totaling 9 in. thick. The design ESAL levels for these sections of I-710 have ranged between 200 and 300 million. This renewable strategy adopted by Caltrans implies a long life of at least 40 years.

A report by Rahim and Fiegel (2011) overviews the latest examination of crack, seat, and overlay (CSOL) performance in California. The information generally shows very limited longitudinal, transverse, and alligator cracking for a range of pavement sections located in various climate regions in the state. No attempt was made to determine if the origin of the cracking was bottom up or top down. A reasonable conclusion is that the recent California data do not suggest any major issues for CSOL even with HMA overlay thicknesses of about 4.0 to 6.0 in.

HMA over Crack-and-Seat PCC and Specifications

A significant practice associated with cracking operations that precede paving HMA over crack-and-seated PCC pavement was selected and included in Table 2.3. The table includes a brief explanation of why the issue is of special interest along with examples from the recommendations in the Guide Specifications (Chapter 4).

TABLE 2.3. BEST PRACTICES AND SPECIFICATIONS FOR HMA OVER CRACK-AND-SEATED PCC PAVEMENT

Best Practice	Why This Practice?	Typical Specification Requirement[a]
Cracking operations	The crack-and-seat technique shortens the effective slab length between the transverse joints or cracks in the existing concrete pavement before the HMA overlay is placed. This distributes the horizontal strains resulting from thermal movements of the existing PCC more evenly, thus reducing the risk of causing reflective cracks in the AC overlay.	• AASHTO 567 recommends a cracking pattern that results in PCC pieces of 1.2 to 1.8 ft^2 in area. Other state experience, such as that of Caltrans, suggests that a larger cracking pattern can work well for jointed plain concrete pavement (JPCP) such as 6 ft by 5 ft. (For a 12-ft-wide lane with 15-ft contraction joint spacing, this results in a lane cracked in half and approximately at the third points.) • The study team recommends the minimum distance from a contraction joint to initiate cracking be 3 ft. This should ensure that the cracked areas be dimensioned with a 2-to-1 ratio or less. This assumes the slab is longitudinally cracked down the middle. • Produce cracks that are continuous without extensive spalling along the crack. Verify that the cracking extends fully through the slab by use of cores (not an AASHTO guide specification requirement).

[a] For more details, refer to Elements for AASHTO Specification 567 in Chapter 4 and AASHTO Guide Specification 567.

HMA OVER SAW, CRACK, AND SEAT JOINTED REINFORCED CONCRETE (JRC) PAVEMENTS

Criteria for Long-Life Potential

It has been established that the crack-and-seat technique of fracturing joint reinforced concrete pavements (JRCPs) has not been successful because of the inability to either break the bond between the reinforcing steel and concrete or shear the steel along the plane of the crack. The bonded reinforcing steel results in thermal contraction concentrated at the existing transverse joints, thus leading to reflective cracks through the new HMA layer.

An alternative solution is to saw narrow transverse cuts into the concrete deep enough to cut through the longitudinal steel reinforcement, then crack the pavement at the locations of the sawed cuts using the same crack-and-seat procedure described above (Merrill, 2005); see Figure 2.7. The same precautions as noted for crack-and-seat construction apply. The depth of the cut can be determined from coring and/or GPR testing. The use of a strike plate is recommended to prevent spalling during the cracking operations. Verification coring should follow to ensure that fine, full-depth vertical cracks are achieved (see Figure 2.8). The spacing of saw-cuts should be similar to the cracking pattern used in the crack-and-seat procedures. The U.K. Department of Transport Road Note 41 (Jordan et al., 2008) recommends a spacing of 3 to 6 ft. Under these conditions, the critical features and limitations are the same as for the crack-and-seat approach.

Figure 2.7. *Sawing concrete slabs.*
Source: Jordan et al., 2008.

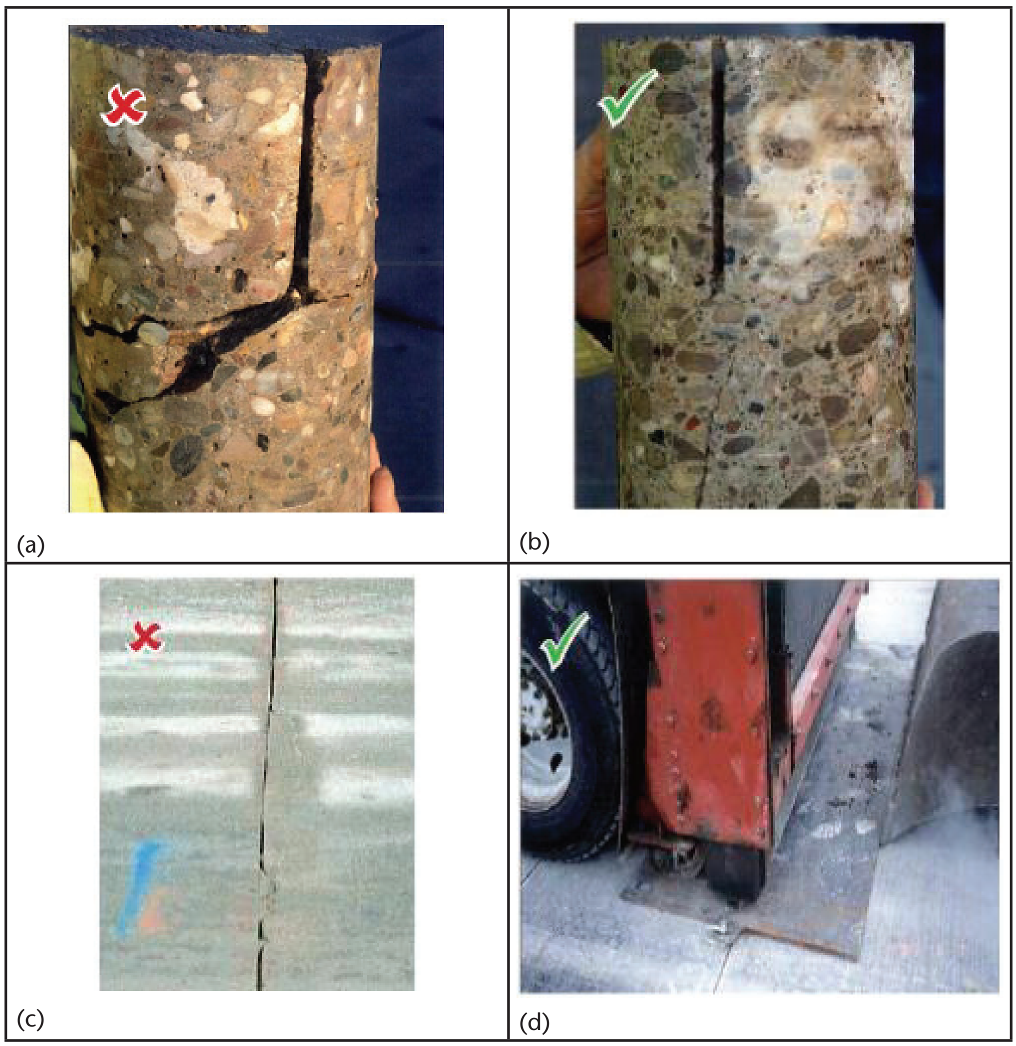

Figure 2.8. *Poor and good practices of saw-cut, crack, and seat. (a) Noncompliant core: steel reinforcement not severed. (b) Compliant core: fine, full-depth, vertical crack. (c) Spalling of saw-cut because no strike plate use. (d) Strike plate in use.*
Source: Jordan et al., 2008.

Because cracks are not visible in this process, more extensive coring is required to confirm that the pavement has been cracked. The U.K. Department of Transport (2010) also requires cores to verify that the steel reinforcing has been cut and the slab is fully cracked. In addition, it requires falling weight deflectometer (FWD) deflection testing and back-calculation to verify a minimum modulus (termed effective stiffness modulus) of the PCC layer following cutting, cracking, and seating.

Following cutting and cracking, the U.K. Department of Transport (2010) requires seating the PCC with a pneumatic roller with a total weight ≥20 tonnes.

Similar to crack and seating, thicker overlays were found to perform much better than thinner overlays in test sections in the United Kingdom (Coley and Carswell, 2006).

HMA over Saw, Crack, and Seat PCC and Specifications

A significant practice regarding precutting existing reinforcing steel before paving HMA over saw, crack, and seat PCC was selected and included in Table 2.4. The table includes a brief explanation of why sawing the existing reinforcing steel is of special interest, along with examples from the recommendations in the Guide Specifications (Chapter 4).

TABLE 2.4. BEST PRACTICES AND SPECIFICATIONS FOR HMA OVER SAW, CRACK, AND SEAT JOINTED REINFORCED PCC

Best Practice	Why This Practice?	Typical Specification Requirement[a]
Depth of saw-cut	The reinforcing steel in JRP must be fully severed so that the bond between the PCC and the steel is released. This significantly reduces the thermal stresses at the preexisting joints to be reduced to manageable levels. This saw cutting precedes the crack-and-seat operation.	1. Preparatory work: Before sawing, the following work must be complete: a. If required, construct pavement drainage systems at least two weeks before saw cutting and cracking and seating. b. Any existing material overlaying the concrete pavement must be removed. 2. Sawing: Transverse saw-cuts will be made at a 4 to 5 ft. spacing along the centerline of the pavement to the depth required to cut the reinforcing steel found in the jointed reinforced concrete pavement. 3. Cracking and seating: Cracking and seating shall proceed in accordance with the guide specifications for Cracking and Seating with the additional requirement that the equipment used to crack the pavement will include a protective plate that eliminates any spalling of the saw-cut during the cracking operation.

[a] For more details, refer to the R23 Guide Specifications for Saw, Crack, and Seat Elements in Chapter 4.

HMA OVER RUBBLIZED CONCRETE PAVEMENTS

Criteria for Long-Life Potential

In principle, rubblization effectively eliminates the problem of reflection cracking, because the technique is supposed to completely disintegrate the existing concrete slab. However, it also reduces the strength of the existing concrete pavement substantially because it renders the concrete into broken fragments resembling an unbound base course, although with "aggregate" sizes much larger than a regular crushed aggregate base layer. Thus, it is the only approach that utilizes the existing concrete pavement and fully addresses slab movement responsible for reflective cracking; however, crack-and-seat processing is generally preferred to rubblization because the former keeps more of the existing PCC slab material intact.

This renewal solution is viable as long as the following critical features are met:

- There is no evidence of pumping underneath the existing slabs.
- The foundation support is good (i.e., there are no voids between the concrete slab and the underlying base or subbase).
- The subgrade strength is acceptable.
- The existing drainage system is in good working condition, or provisions can be made for installing a drainage system before rubblizing the concrete pavement.

However, the following limitations and additional cautions are warranted:

- The performance of this solution is tied to the quality of the rubblization operation. If construction guidelines are put in place to ensure that (1) concrete below the reinforcement is broken; (2) the size distribution of the rubblized concrete pieces is as uniform as possible, although this will vary with depth; (3) the maximum size of the rubblized concrete pieces in the bottom half is kept within the specification limits; and (4) the steel reinforcement—where present—is debonded from the concrete, then long life may be achievable.
- If the foundation underneath the existing concrete is not sufficiently strong, the rubblization operation may damage the base or subbase and/or the existing subgrade and produce an unstable base layer.
- Moisture problems, soft spots, and voids underneath the slab should be addressed before rubblization for enhanced performance.

It is noted that the rubblization process leads to the largest HMA overlay thicknesses among all flexible pavement renewal solutions of concrete pavements, because the rubblization process transforms the PCC layer into an untreated aggregate base layer.

Construction Operations

Rubblizing involves breaking the existing concrete pavement into pieces, thereby destroying any slab action, and overlaying with HMA. The sizes of the broken pieces usually range from 2 to 6 in. (Asphalt Pavement Alliance, 2002). The technique is suitable for both JPC and JRC pavements. It has also been used on severely deteriorated CRC pavements, although the heavy reinforcement in the CRC pavement presents challenges and requires extra care in quality assurance/quality control (QA/QC) procedures.

A rubblized PCC pavement should behave, at a minimum, like a high-quality granular base layer, and, if so, the loss of structure must be accounted for in the HMA overlay design thickness. A study by the National Asphalt Pavement Association (NAPA) indicated that the strength of the rubblized layer is 1.5 to 3 times greater than a high-quality dense-graded crushed-stone base (NAPA, 1994). Somewhat higher moduli for rubblized PCC were reported by Buncher et al. (2008) in terms of slab thicknesses (the recommendations were for airfield pavements but much of the data used came from highway projects):

- For slabs 6 to 8 in. thick, E_{rub} ranges from 100 to 135 ksi.
- For slabs 8 to 14 in. thick, E_{rub} ranges from 135 to 235 ksi.
- For slabs greater than 14 in. thick, E_{rub} ranges from 200 to 400 ksi.

Buncher et al. (2008) also reported data from field sections that resulted in average retained moduli values (E_{rub}/E_{PCC}) of about 6.0%. Furthermore, thicker slabs exhibited higher retained moduli values than thinner PCC slabs.

A summary of measured field moduli for rubblized PCC provided in the R23 Project Assessment Manual (Chapter 1) suggests a possible range of 40,000 to 700,000 psi with a typical value of 150,000 psi. These values largely support those by Buncher et al. (2008).

Rubblization is considered to be a viable, rapid, and cost-effective rehabilitation option for deteriorated PCC pavements. Good performance of rubblized pavements requires a high-quality process of rubblization, effective rubblizing equipment, and a maintained strong base and/or subgrade soil. Poor performance can occur when the underlying soils are saturated. Installation of edge drains before rubblization has proven to be successful for this type of condition. If the existing concrete pavement is deteriorated because of poor subgrade support, then rubblization is unlikely a viable option. Two types of equipment are used in the rubblization process: (1) resonant breaker and (2) multiple-head breaker.

The resonant breaker (Figure 2.9) is composed of a sonic shoe (hammer) located at the end of a pedestal, which is attached to a beam—whose dimensions vary from one machine to another—and a counterweight situated on top of the beam. The principle on which the resonant breaker operates is that a low-amplitude (about 0.5-in.) high-frequency resonant energy is delivered to the concrete slab, which causes high tension at the top. This causes the slab to fracture on a shear plane inclined at about 35° from the pavement surface. Several equipment variables affect the quality of the rubblization process, including shoe size, beam width, operating frequency, loading pressure, velocity of the rubblizer, and the degree of overlapping of the various passes. The rate of production depends on the type of base or subbase material and is approximately 1.0 to 1.5 lane miles/day.


(a)


(b)

Figure 2.9. *Resonant frequency pavement breaker. (a) Resonant breaker machine. (b) Close-up of the sonic shoe.*
Source: Baladi, Niederquell, and Chatti, 2000.

During its operation, a resonant rubblizer encounters difficulty in the vicinity of pavement discontinuities such as joints or cracks. At a discontinuity, the microprocessor controller increases the rubblizer speed, causing a decrease in the energy delivered to the concrete or even a shutdown. Bituminous patches or unmilled overlays can also be problematic, because the shoe penetrates the asphalt, causing a large loss in the energy delivered to the concrete. Finally, the type of base or subbase material, the roadbed and/or subgrade soil, and the condition of the concrete pavement being rubblized all affect the quality of the rubblized product. For example, if the base or subbase materials are softer than the roadbed soil, shear failure may result. If excessive moisture is present, the vibrations from the rubblizer may cause "quick" conditions resulting in a significant loss in bearing capacity of either the base aggregate or the subgrade soil.

For the process of rubblization, it is recommended to begin at a free edge or previously broken edge and work transversely toward the other edge. In the event the rubblizer causes excessive deformation of the pavement, the engineer may require high flotation tires with tire pressures less than 60 psi. Then any particle greater than 6 in. in its largest dimension remaining on the pavement surface should be reduced to an acceptable size or removed, and the area filled with granular base. Then any projecting reinforcing steel below the rubblized surface should be cut off and disposed of. Then compaction can be performed by seating rubblized pavement with the following rolling pattern:

- One pass from a vibratory roller, followed by at least one pass with the pneumatic roller, and
- At least two more passes with the vibratory roller.

The rolling pattern may be changed as directed.

The multihead breaker operation includes multiple drop hammers arranged in two rows on a self-propelled unit and a vibratory grid roller (Figure 2.10). The bottom of the hammer is shaped to strike the pavement on 1.5-in.-wide and 8-in.-long loading strips. The hammers in the first row strike the pavement at an angle of 30° from the transverse direction. The hammers in the second row strike the pavement parallel to the transverse direction. The sequence of hammer drops is irregular because each cylinder is set on its own timer and frequency system. By disabling some cylinders, the width of the rubblized area can be varied from 3 to 13 ft. The vibratory grid roller (10 tons) follows the multihead breaker to reduce the size of the broken concrete. The rate of production of the multihead breaker depends on the type of base or subbase material and is about 0.75 to 1 lane-mile per 10-h shift. Several variables affect the rubblization process, including speed, height, weight, and frequency of the drop hammers. The multihead breaker encounters difficulties on weak or saturated subbase and/or roadbed soil, which fail in shear, causing large concrete pieces to rotate and/or penetrate the underlying material. Such failure would result in poor pavement performance.

It is recommended to rubblize the entire lane width in one pass. The user should provide a screen to protect vehicles from flying particles. Any particle greater than 6 in. in its largest dimension remaining on the pavement surface should be reduced to an acceptable size or removed, and the area should be filled with granular base. Any projecting reinforcing steel below the rubblized surface should be cut off and disposed of. Then compaction can be performed by seating the pavement with the following rolling pattern:

- A minimum of four passes with the Z-grid vibratory roller,
- Four passes with a vibratory roller, and
- At least two passes from a medium-weight pneumatic roller.

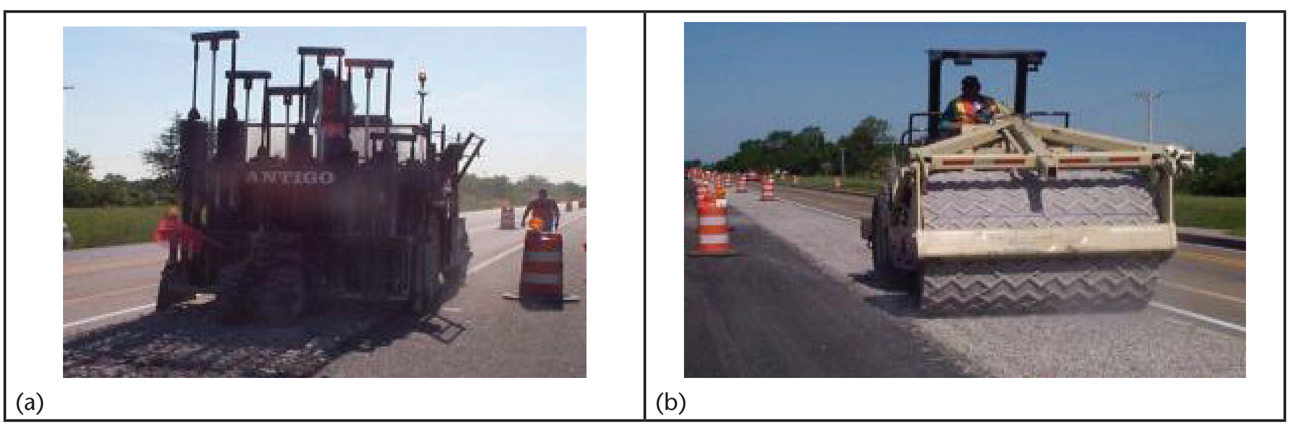

Figure 2.10. *(a) Multihead breaker. (b) Grid roller.*
Source: Baladi et al., 2000.

The rolling pattern may be changed as directed.

Figure 2.11 shows examples of good and poor rubblization outcomes.

Rubblized Concrete Size Requirements

Construction-related problems with nonuniform particle size distribution throughout the PCC slab thickness will lead to underperforming pavements. Also, pavement sections that have been "overrubblized" (i.e., with rubblized pieces less than 2 in. in size) have a higher probability of cracking prematurely. Table 2.5 summarizes size requirements by various state highway agencies (SHAs) in the United States. In addition, recent rubblization particle size information was summarized for the Wisconsin DOT (WisDOT; Wisconsin Department of Transportation, 2010). The results available in Table 2.5 and those from WisDOT differ somewhat; thus, the information shown must be used with significant judgment.

Figure 2.11. *Examples of rubblized concrete pavements. (a) Rubblized layer from multihead breaker. (b) Rubblized layer from resonant breaker. (c) Partial debonding of temperature steel. (d) Partial destruction of the joint integrity.*

Sources: Sebesta and Scullion, 2007; Baladi et al., 2000.

TABLE 2.5. SIZE REQUIREMENTS BY VARIOUS STATE HIGHWAY AGENCIES

Agency	No Reinforcement	Top Half of Slab (above reinforcement)	Bottom Half of Slab (below reinforcement)
Michigan	$d < 8$ in.	2 in. $< d < 5$ in.	$d \leq 8$ in.
Arkansas	$d < 6$ in. 100% at $d \leq 8$ in. 51% at 1 in. $< d < 3$ in.	$d < 6$ in. 100% at $d \leq 8$ in. 51% at 1 in. $< d < 3$ in.	$d < 6$ in. 100% at $d \leq 8$ in. 51% at 1 in. $< d < 3$ in.
Illinois	See next columns	75% at $d \leq 3$ in. 100% at $d \leq 9$ in.	75% at $d \leq 9$ in. 100% at $d \leq 12$ in.
Ohio	NA	100 % at $d < 6$ in. 100% at 1 in. $< d < 2$ in.	100 % at $d < 6$ in. 51% at 1 in. $< d < 2$ in.
Pennsylvania	$d < 6$ in. 100% at $d \leq 8$ in. 51% at $d \leq 4$ in.	$d < 6$ in. 100% at $d \leq 8$ in. 51% at $d \leq 4$ in.	$d < 6$ in. 100% at $d \leq 8$ in. 51% at $d \leq 4$ in.
Indiana	$d < 6$ in. 51% at 1 in. $< d < 2$ in.	$d < 6$ in. 100% at 1 in. $< d < 2$ in.	$d < 6$ in. 51% at 1 in. $< d < 2$ in.
Texas	60% at $d \leq 3$ in. 100% at $d \leq 6$ in.	60% at $d \leq 3$ in. 100% at $d \leq 6$ in.	75% at $d \leq 9$ in. 100% at $d \leq 12$ in.
FAA	75% at $d \leq 3$ in. $d \leq 1.25\ D$	75% at $d \leq 3$ in. $d \leq 1.25\ D$	75% at $d \leq 12$ in. 100% at $d \leq 15$ in.

Note: d = dimension of rubblized concrete pieces; D = depth of existing concrete.

Suitability for Rubblization

The collection of the pavement evaluation data allows the agency to analyze the project for its suitability for rubblization. Performing the following steps enables making this determination (Sebesta and Scullion, 2007):

- Evaluate the dynamic cone penetrometer (DCP) data using a modified version of the Illinois DOT rubblization selection chart (shown in Figure 2.12) as follows:
  - —Plot the concrete thickness versus the California bearing ratio (CBR) of the base. These data are used to gauge whether the concrete will rubblize, because sufficient support beneath the slab is crucial for satisfactory breakage.
  - —Plot the combined thickness of the concrete and base versus the CBR of the subgrade. Use a "dummy" base layer of 6 in. if the DCP data do not distinguish a base layer. These data are used to evaluate whether the subgrade can support construction traffic after rubblization.

  High risk for rubblization should translate to moderate risk for crack and seat, and moderate risk for rubblization should translate to low risk for crack and seat (and saw-cut, crack, and seat).

- If all the data points fall in the zones that indicate rubblization is feasible, the project should be suitable for rubblization.

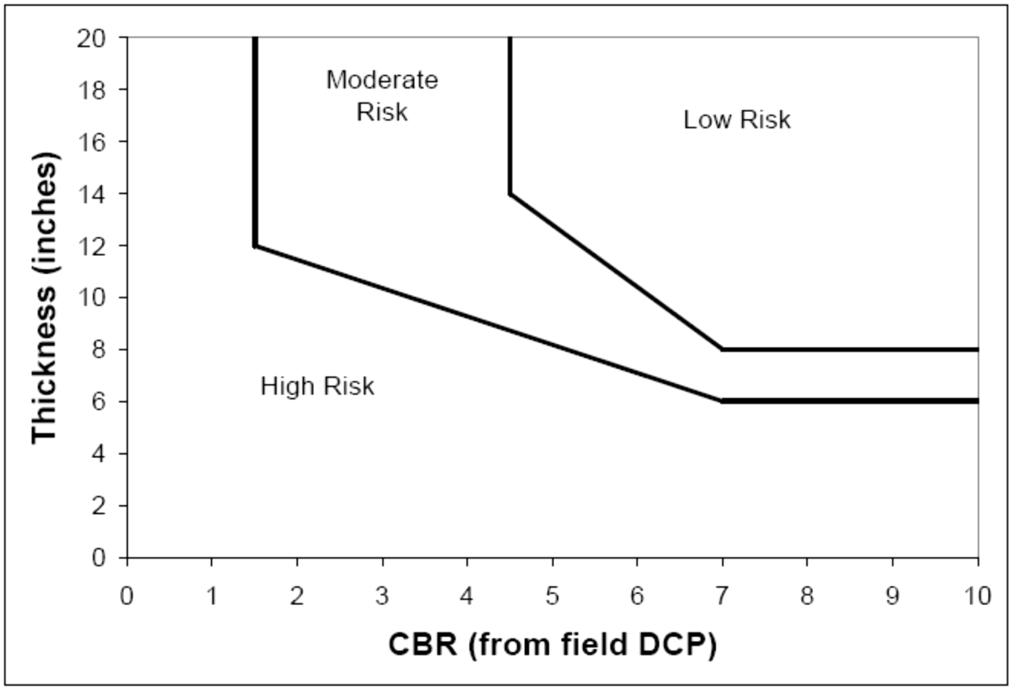

Figure 2.12. *Modified Illinois DOT rubblization selection chart as proposed by the Texas Transportation Institute–Texas DOT.*
Source: Sebesta and Scullion, 2007.

- If all the data points fall in the high-risk zone of the chart, rehabilitation options other than rubblization (crack and seat for JPCP, saw-cut and crack and seat for JRCP) should be considered.
- If some, but not all, of the data points fall in the high-risk zone, certain portions of the project may not be suitable for rubblization. More analysis, interpretation, and judgment are required. Typically these instances are encountered in older concrete pavements where there is no or insufficient base support. Additional analysis should be done as follows:
  - —Determine the average CBR of the first 12 in. beneath the concrete.
  - —From the rubblization selection chart, determine the minimum CBR necessary to support rubblization for the known concrete thickness at the project. Do this by starting on the *y*-axis at the known concrete thickness, then project horizontally until intersecting the boundary where rubblization is feasible. At this intersection, project down to the *x*-axis, and read the minimum subgrade CBR required.
  - —Form a relationship between the subgrade modulus and CBR by graphing the average CBR of the first 12 in. beneath the concrete versus the subgrade modulus. Input the minimum CBR necessary into this relationship to determine the anticipated minimum subgrade modulus needed. Typically this modulus value ranges between 10 and 15 ksi.

— Graph the subgrade modulus with distance for the project. Where the modulus does not exceed the minimum subgrade modulus needed, a risk exists that the project may not rubblize. At this point the data must be reviewed on a case-by-case basis and a judgment made as to where, if at all, rubblization should be attempted. Rehabilitation options other than rubblization (crack and seat for JPCP, saw-cut and crack and seat for JRCP) should be considered.

HMA over Rubblized PCC Pavement and Specifications

A selection of significant practices associated with paving HMA over existing rubblized PCC pavement is included in Table 2.6. The table includes a brief explanation why the issue is of special interest, along with examples from the recommendations

TABLE 2.6. BEST PRACTICES AND SPECIFICATIONS FOR HMA OVER RUBBLIZED JOINTED PLAIN PCC PAVEMENT

Best Practice	Why This Practice?	Typical Specification Requirement[a]
Work before rubblization	The rubblization of the preexisting PCCP is a process that reduces the PCC to aggregate. Damage to adjacent facilities, such as storm drains, is likely if connecting steel is not severed.	• Before rubblizing a section, cut full-depth saw-cut joints at any locations shown on the plans to protect facilities that will remain in place.
Rubblization and compaction	For reinforced PCC pavement, it is required that all reinforcing steel be removed during the rubblization process. This allows the rubblized material to behave in a consistent manner and precludes any further corrosion of the existing steel. The second item governs the end-result PCC particle sizes. The practice described largely comes from projects that have performed well.	• Reinforcing steel exposed and projecting from the surface after rubblization or compaction shall be cut off below the surface and removed. • Completely debond any reinforcing steel and rubblize the existing concrete pavement. Above the reinforcing steel or upper one-half of the pavement (if unreinforced), the equipment shall produce at least 75% of broken pieces less than 3 in. in size. At the surface of the rubblized layer, all pieces shall be less than 6 in. Below the reinforcing steel or in the lower half of the pavement, the maximum particle size shall be 9 in.
Verification of rubblization	The end-result PCC particle sizes must be verified. The way to do this is to describe in the specifications a test section and select a test-pit location. The PCC material will be sampled and checked for sizing.	Before full production begins, the engineer will select approximately 200 linear feet of one lane width to verify the rubblization operation. The contractor shall rubblize the test section, using the section to adjust equipment. From within this test section, the engineer and contractor shall agree upon a test-pit location. At the test-pit area, excavate a 4-ft. square test pit. The engineer shall test the material to verify that the specified particle size distribution has been achieved through the entire depth of the pavement.
Traffic	Allowing public traffic on a rubblized PCC layer is not advisable for several reasons—the major one being that the rubblized layer cannot carry heavy traffic and there is the potential for degradation of the PCC particles.	Public traffic shall not be allowed on the rubblized pavement and the contractor shall avoid unnecessary trafficking of the rubblized pavement with construction equipment.

[a] For more details, refer to the Rubblization Guide Specification in Chapter 4.

in the Guide Specifications (Chapter 4). Four major practices are featured: (1) work needed before rubblization, (2) the rubblization process and associated compaction, (3) verification of rubblization, and (4) traffic control.

HMA OVER CONTINUOUSLY REINFORCED CONCRETE (CRC) PAVEMENTS

Criteria for Long-Life Potential

The combination of a CRC pavement and an HMA overlay has significant potential to provide long-life pavement. This is because a CRC pavement eliminates moving joints within the concrete slab as it develops narrow transverse cracks at a regular spacing. If these cracks remain tight, then no reflection cracking should appear in the overlay as long as the surface of the existing CRC pavement is in good condition and a good bond between the HMA overlay and the CRC pavement is achieved. Also, in principle, this solution should lead to thinner overlays compared to HMA over existing jointed concrete pavements.

This renewal solution is viable as long as the following critical features are met:

- The surface condition of the CRC pavement is good (i.e., the deflection is low and there are no major defects such as spalling, punchouts, depressions, or broken reinforcement).
- There is no evidence of pumping underneath the existing slabs.
- The foundation support is good (i.e., there are no voids between the concrete slab and the underlying base or subbase).
- The existing drainage system is in good working condition or a drainage system can be put in place.

The main limitation of this renewal strategy is that any untreated or improperly treated defect in the existing CRCP that is left untreated or improperly treated can develop into a major repair in the future. Therefore, this approach would only apply to CRCP in very good condition, which limits its application. Also, if bonding is not properly ensured, water caught between the HMA overlay and the existing CRCP can lead to severe stripping of the HMA. The performance of HMA overlays on CRC pavements has been variable in the United States based on information provided by the states in Phase 1 of this study. Therefore, the performance of HMA overlays using this solution has not been substantiated for a long life (>50 years), and their use in the context of long-life pavements, while possible, is still unproven.

Surface Preparation and Repair and Overlay Depths

For HMA over CRC pavements, the following surface preparations and/or repairs are recommended by the U.K. Transport Research Laboratory (TRL) in Road Note 41 (Jordan et al., 2008), depending on the condition of the existing CRC pavement:

- HMA overlay ≤1.6 in. thick can be used for the following conditions:
  - —If the existing CRC pavement is in good condition with no structural problems, no repairs are necessary. Good condition translates to regularly spaced

transverse cracks of up to 0.5 mm in width, but with no longitudinal cracks (see Figure 2.13).

- If the existing CRC pavement has minor spalled cracks in the wheelpath that do not affect the structural integrity of the CRCP, clean and fill or seal the cracks before overlay (see Figure 2.13).

- HMA overlay >1.6 in. to <4.0 in. thick can be used for the following conditions:
  - If the existing CRC pavement has large crack widths (between 0.5 and 1.5 mm) (see Figure 2.14), full-depth repairs are required at locations where the cracks propagate through the total thickness of the concrete.
  - If the existing CRC pavement has surface spalling and scaling, the top of the concrete should be milled. Full-depth repair is required in areas where spalling has led to large pieces of concrete breaking away from the surface.
- HMA overlay >4.0 in. thick can be used for the following condition:
  - If the existing CRC pavement has structural defects such as "punchouts" (see Figure 2.15), settlement, faulted cracks, and severe spalls, all distressed areas should be repaired with concrete before the HMA overlay.

Partial-depth repair should be done with cementitious material. Full-depth repairs must include reinstating reinforcement and tying it to the existing bars.

HMA over Existing CRC Pavement and Specifications

A significant issue associated with paving HMA over existing CRC pavement was selected and included in Table 2.7: full-depth patching. The table includes a brief explanation of why the issue is of special interest along with examples from the recommendations in the Guide Specifications (Chapter 4).

ADDED LANES AND APPROACHES FOR ADJACENT STRUCTURES

There is little guidance found in the literature on integrating the new or rehabilitated pavements into adjacent pavements and features. This section addresses adding lanes to an existing pavement structure as well as accommodating existing features such as bridge abutments and vertical clearance restrictions within the limits of a pavement renewal project. These issues are paramount when using the existing pavement in place as part of long-life renewal because there is typically a significant elevation change associated with each renewal alternative. The following recommendations are based on discussions with the SHAs surveyed in Phase 1 and those agencies who participated in Phase 2.

Approaches to Undercrossing Structures, Bridges, and Overcrossing Structures

All of the agencies that participated in the study indicated that a completely new roadway section was constructed as a transition between the in-place renewal cross section and the existing feature. New pavement sections were constructed either approaching an overcrossing or bridge structure abutment or before passing under a structure

(a)

(b)

Figure 2.13. *Examples of minor cracks in CRC pavement. (a) Closely spaced tight transverse cracks. (b) Tight bifurcated cracks.*
Source: Jordan et al., 2008.

(a)

(b)

Figure 2.14. *Examples of major crack defects in CRC pavement. (a) Spalled cracks. (b) Intersected crack pattern.*
Source: Jordan et al., 2008.

(a)

(b)

Figure 2.15. *Examples of "punchouts" in CRC pavement. (a) Punchout strip. (b) Severe punchout block.*
Source: Jordan et al., 2008.

TABLE 2.7. BEST PRACTICES AND SPECIFICATIONS FOR HMA OVER EXISTING CONTINUOUSLY REINFORCED PCC

Best Practice	Why This Practice?	Typical Specification Requirement[a]
Full-depth patching process	The described steps are a systematic process for making any needed patches in the CRCP before resurfacing the existing pavement. The use of polyethylene sheets as a bond breaker is to reduce the amount of shrinkage-related cracks.	• Saw-cut full depth through the concrete around the perimeter of the repair area before removal. • Remove or repair loose or damaged base material, and replace or repair it with approved base material to the original top of the base grade. Place a polyethylene sheet at least 4 mils thick as a bond breaker at the interface of the base and new pavement. Allow concrete used as base material to attain sufficient strength to prevent displacement during further construction. • Broom finish the concrete surface unless otherwise shown on the plans.

[a] For more details, refer to Elements for AASHTO Specification 558 in Chapter 4.

where there is not sufficient clearance to meet standards. The length of this transition section depended on the elevation difference but was usually in the range of 200 to 400 ft before and after the structure.

Consideration of the longitudinal drainage is required in design of the transition section. Where possible, the existing subgrade elevation and grade should be maintained in the longitudinal direction as well as in the transverse direction. Because the new roadway section is generally not as thick as the renewal approach using the existing pavement, the elevation difference is usually made up with untreated granular base material. The elevation difference can often be accomplished by varying the thickness of that base layer. However, there are cases where there may be an advantage to replacing the existing PCC with HMA and only using one material to construct the transition for ease of staging, as shown in Figures 2.16 and 2.17.

In some cases, agencies reported that they were able to raise an overcrossing rather than reconstruct the roadway for less cost and reduced impact on traffic. That option may be considered where possible, particularly in more rural areas where there is little cross traffic on the overcrossing.

Added Lanes or Widening

A project that calls for additional lanes or widening often facilitates the staging of the traffic through the project, but it usually produces a mismatch in pavement sections in the transverse direction. The elevation and grade line of the subgrade should be maintained so that water flowing along the contact between the base and the subgrade does not get trapped in the transverse direction. There is a risk of reflection cracking between the existing pavement and the new pavement section, particularly when the existing pavement is a PCC pavement. Also of concern is the need for stabilizing the subgrade soil if required for widening. Subgrade stabilization will increase the stability of the roadway section, accelerate pavement construction, and help to reduce some of the settlement or differential vertical deflection that causes reflection cracking along the contact with the

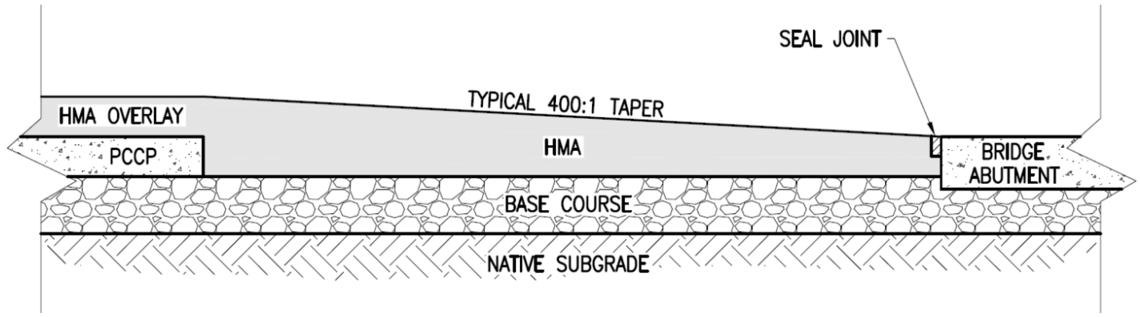

Figure 2.16. *Diagram of transition to bridge approach.*

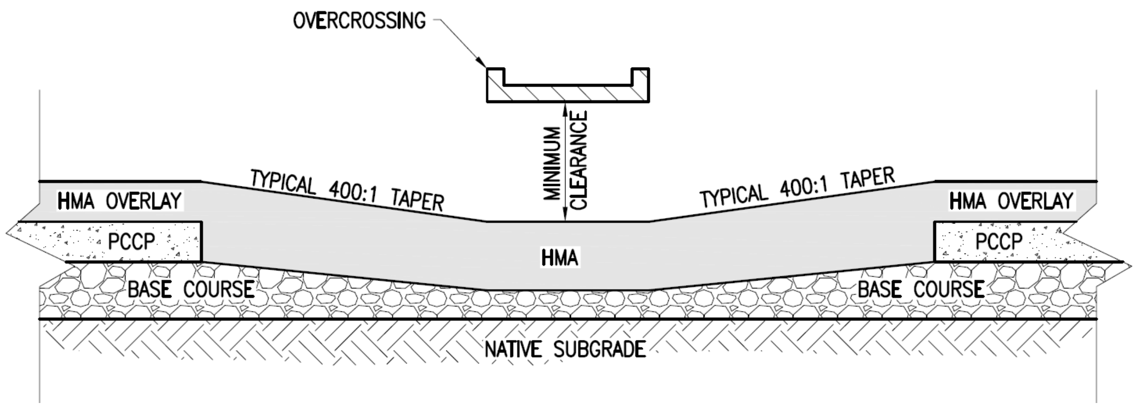

Figure 2.17. *Diagram of transition beneath structure.*

old PCC pavement. Specifically, the SHRP 2 R02 project guidance for geotechnical solutions for transportation infrastructure and its recommendations for stabilization of the pavement working platform should be considered.

Widening Next to Rubblized PCC Pavement

Since the rubblized PCC pavement is basically turned back into a form of gravel, there has been little in the way of complications widening these pavement sections. Where the shoulder is not full-depth gravel to the subgrade contact (as shown in Figure 2.18), it is recommended that the shoulder be removed to the subgrade contact and the section next to the rubblized PCC pavement be replaced with untreated granular base. This will ensure that water flowing transversely along the base–subgrade interface will not get trapped under the pavement structure. If the subgrade soils need to be stabilized, then that should take place before backfilling with untreated granular base; however, where soils are weak and wet enough to require stabilization, they may not be stable enough to allow rubblization.

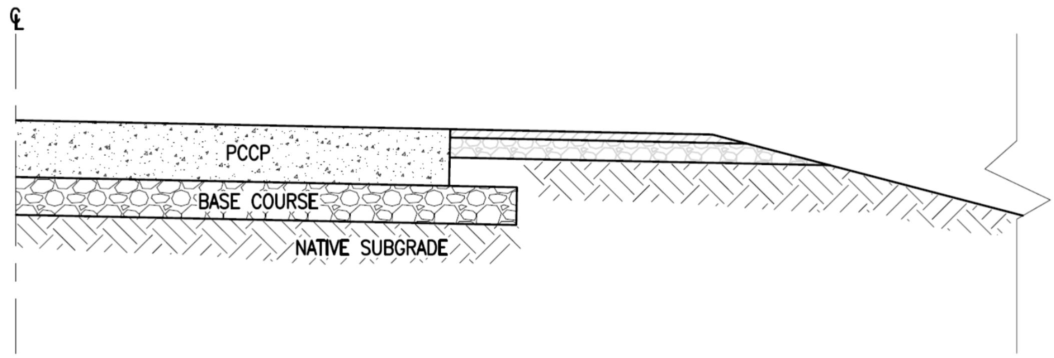

Figure 2.18. *Diagram showing existing PCC pavement.*

Depending on the widening needs, there may be cases where the shoulder is reconstructed and used to carry traffic while the existing PCC pavement is being rubblized. In cases where the HMA is placed next to the PCC pavement before rubblization, the lateral restraint aids rubblization. The thickness of the HMA placed next to the existing PCC pavement depends on the traffic loading during staging and the amount of construction traffic that would use the widened lane before the final overlays are placed.

Figure 2.19 shows the design roadway section with free-draining granular base extending either to the in slope of the ditch or the fill slope (i.e., "daylighting") to provide drainage. An agency may elect to use internal drainage where longitudinal drains are installed just outside of the traveled lane. Either drainage approach is acceptable as long as some form of drainage is provided.

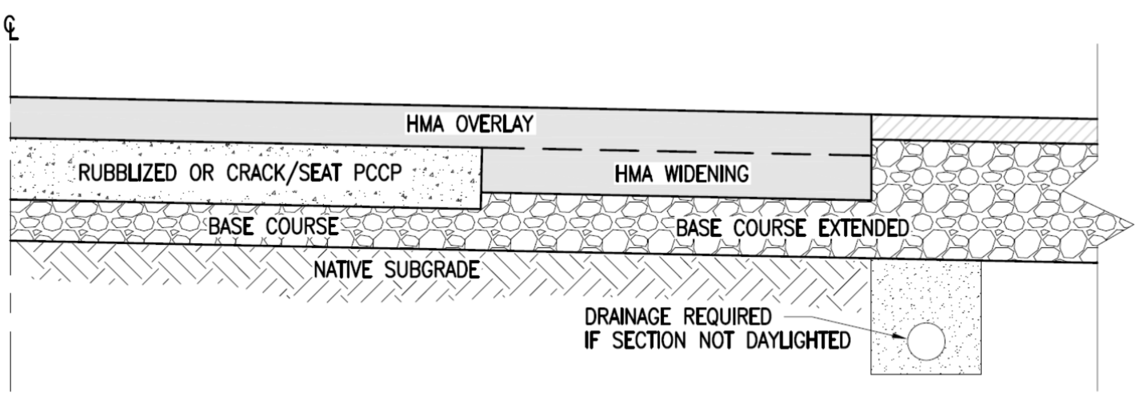

Figure 2.19. *Diagram of widening the shoulder with daylighting or drainage installed.*

Widening Next to Cracked-and-Seated or Sawed, Cracked, and Seated PCC Pavement

Widening next to cracked-and-seated PCC pavement is treated much the same as described for rubblized PCC, except there is a risk that a longitudinal reflection crack may form along the edge of the existing PCC pavement. This is most likely caused by the differential vertical deflection found between the rigid pavement and the more flexible adjacent pavement. The deflection difference can be reduced by a number of options. The first consideration would be to stabilize the subgrade soil in the widened area. Even where stabilization is marginally indicated, it may be advisable to stabilize the subgrade to facilitate construction and reduce the differential deflection between the two pavement sections.

When overlaying cracked-and-seated PCC pavement with HMA, most states interviewed have used HMA in the widening for economic reasons. Again, the thickness of the HMA placed next to the existing PCC pavement will depend on the amount of traffic loading expected during the staging. The final thickness of the HMA in the widened lane will depend on the total thickness design for the traffic in that lane or a combination of that required to accommodate traffic before the overlay and the thickness of the overlay, whichever is greater. In some cases, the use of an interlayer stress-absorbing composite (ISAC) may reduce the amount of reflection cracking along the longitudinal joint between the existing PCC pavement and the HMA widening (Hoerner et al., 2001).

STRUCTURAL DESIGN CRITERIA TO ACHIEVE LONG LIFE

Basic Approach

The most accepted approach to designing HMA long-life pavements is to use mechanistic-empirical (ME) concepts as described by Monismith (1992). The basis of this approach is that pavement distresses with deep structural origins could be avoided if pavement responses such as stresses, strains, and deflections could be kept below the thresholds (endurance limits) where the distresses begin to occur. Thus, an asphalt pavement could be designed for an "indefinite" structural life if it is designed for the heaviest vehicles without being overly conservative (Thompson and Carpenter, 2004; Timm and Newcomb, 2006). The basic concept of a long-life HMA pavement is illustrated in Figure 2.20 (Newcomb, Willis, and Timm, 2010). This approach can be extended to HMA renewal solutions.

Endurance Limits

Suggested values for the horizontal tensile strain at the bottom of the HMA layer and vertical compressive strain at the top of the subgrade are 60 microstrains and 200 microstrains, respectively (Monismith and Long, 1999a). The value for the endurance limit of the tensile strain at the bottom of the HMA layer is still debated. Original work by Monismith and others suggests a value of 60 microstrains, but currently accepted values range from 70 to 100 microstrains (Thompson and Carpenter, 2004). Research at the National Center for Asphalt Technology (NCAT) suggests even higher fatigue endurance limits could be possible (Willis et al., 2009).

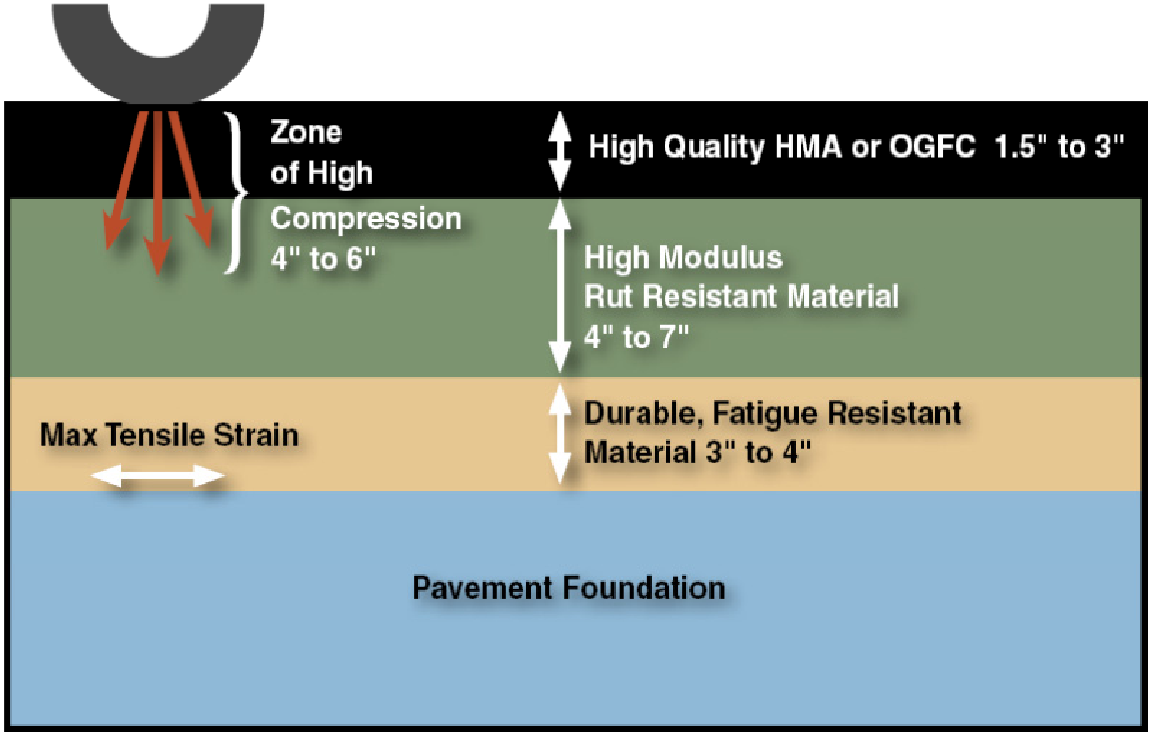

Figure 2.20. *Long-life HMA pavement design concept.*
Source: Newcomb et al., 2010.

Pavement Design Software

In principle, adopting the limiting-strain criteria for design allows for using any layered elastic analysis computer program, because the main output needed is the strain value at specific depths. However, a program that was developed specifically for the purpose of designing long-life HMA pavements is the PerRoad software (Timm, 2008). The program uses the basic ME design philosophy and couples layered elastic analysis with a statistical analysis procedure (Monte Carlo simulation) to predict stresses and strains within a pavement (Timm and Newcomb, 2006). The Monte Carlo simulation allows for incorporating variability into the analysis to more realistically characterize the pavement performance. PerRoad requires the following inputs:

- Seasonal pavement moduli and annual coefficient of variation (COV),
- Seasonal resilient moduli of unbound materials and annual COV,
- Thickness of bound materials and COV,
- Thickness of unbound materials,
- Load spectrum for traffic (or ESAL equivalents),
- Location for pavement response analysis,
- Magnitude of limiting pavement responses, and
- Transfer functions for pavement responses.

The output for PerRoad consists of an evaluation of the percentage of load repetitions lower than the limiting pavement responses specified in the input, an estimate of the amount of damage incurred per single-axle load, and a projected time to when the accumulated damage is equal to 0.1 (where D = 1.0 is considered failure). In high-volume pavements, the critical parameter is the percentage of load repetitions below the limiting strains. It is generally recommended that the designer strive for a value of 90% or more on high-volume roads.

PerRoad 3.5 (Timm, 2008) may also be used to design asphalt pavements over fractured concrete pavements. This only requires that the second layer be specified as rubblized, cracked-and-seated, or broken-and-seated concrete pavement. Beyond that, it follows the same mechanistic design process for a long-life HMA pavement as described above.

The AASHTO "Mechanistic-Empirical Pavement Design Guide" (MEPDG) (AASHTO, 2008b) can be used for long-life pavement design if the agency uses the option of selecting a fatigue endurance limit ranging between 75 and 250 microstrains. Willis and Timm (2009) found good agreement between PerRoad and the MEPDG in terms of thickness requirements when the fatigue endurance limit was used. (During June 2011, the MEPDG was released by AASHTO as Darwin-ME.)

In the MEPDG software, the elastic modulus of the rubblized PCC is assigned a modulus of 150 ksi for Level 3 design (the simplest approach, requiring the fewest and simplest user inputs). For Level 1 design (the most sophisticated approach, requiring the most numerous and precise user inputs), however, the rubblized PCC modulus may be assigned a value from 300 to 600 ksi, depending on the expected level of control on the breaking process and the anticipated coefficient of variation of the fractured-slab modulus.

Example Designs

The following long-life examples are cited in the synthesis by Newcomb et al. (2010).

HMA "Mill-and-Fill" Overlay over Existing HMA Pavement

The rehabilitation of I-287 in New Jersey is an excellent example of the process for evaluation and design of an overlay to an existing pavement. The 26-year-old pavement structure was a 10-in.-thick asphalt pavement that had received a minimum of maintenance. The New Jersey DOT investigation of distresses that developed on the surface showed fatigue cracking, longitudinal cracking in the wheelpaths, and ruts deeper than 1 in. (Fee, 2001). A detailed examination of the pavement structure showed that none of the distresses extended more than 3 in. deep into the HMA. The pavement subsequently had the top 3 in. milled and replaced with 4 in. of HMA surfacing. This work was done in 1994, and a pavement survey done in 2001 showed no signs of cracking or rutting (Rowe et al., 2001).

HMA Overlay over Fractured PCC Pavement

HMA over Crack-and-Seat PCC. Most of the I-710 freeway project in California consisted of a 9-in.-thick asphalt overlay (8 in. of dense-graded HMA capped with a 1-in. open-graded wearing course) on a cracked-and-seated concrete pavement (Monismith and Long, 1999b; Monismith et al., 2009a, 2009b). The HMA overlay does not have a more fatigue-resistant bottom layer (often referred to as a "rich bottom" layer), because the cracked-and-seated concrete provides a stiff foundation for the asphalt and prevents the excessive bending associated with bottom-up fatigue cracking. An asphalt-saturated fabric was placed over a 1-in. leveling course on top of the concrete to resist reflective cracking.

HMA over Rubblized JPCP. Von Quintus and Tam (2001) developed a procedure for designing long-life asphalt pavements over rubblized concrete for Michigan that followed the same approach they used for asphalt pavements. The thicknesses for these asphalt pavements varied depending on design period and traffic levels, with mill-and-fill rehabilitation assumed at years 20 and 32. Table 2.8 shows the total HMA thickness along with the HMA mix type recommended for the surface course.

TABLE 2.8. MICHIGAN DESIGN CATALOG FOR LONG-LIFE HMA PAVEMENTS OVER RUBBLIZED CONCRETE

Design Period (years)	Total HMA Thickness (in.) and Type of Surface Mix (as a function of 20-year ESALs)			
	3 million	10 million	20 million	30 million
20	6.0	8.5	10.6	11.4
	Superpave	Superpave	SMA	SMA
30	7.0	10.0	12.0	13.0
	Superpave	Superpave	SMA	SMA
40	8.5	10.6	13.0	14.6
	Superpave	Superpave	SMA	SMA

Note: SMA = stone matrix asphalt.
Source: After Asphalt Pavement Alliance, 2002, and Von Quintus and Tam, 2001.

HMA over Rubblized CRCP. A portion of the I-5 experimental project in Oregon consists of a 12-in.-thick HMA layer over an 8-in.-thick rubblized CRCP and a JRCP (Renteria and Hunt, 2008; Sholz et al., 2006). The test site located on the JRCP is instrumented to monitor pavement responses and environmental conditions.

Minimum HMA Thicknesses

TRL Road Note 41 (Jordan et al., 2008) recommends the following minimum HMA overlay thicknesses for the various HMA-over-concrete pavement renewal approaches:

- For HMA over cracked-and-seated (or sawed, cracked, and seated) concrete pavements, TRL recommends a minimum HMA overlay thickness of 6 in.
- For HMA over rubblized concrete pavements, TRL recommends a minimum HMA overlay thickness of 8 in., but with the expectation that overlays for rubblized PCC will be significantly higher than that for cracked-and-seated pavements. HMA thicknesses over rubblized PCC range up to 17 in. thick based on TRL Road Note 41.

- For HMA over CRC pavements (as noted previously), TRL recommends the following HMA overlay thicknesses, depending on the condition of the existing CRC pavement, and with the proper repairs made to distressed areas before overlaying (see CRCP section above):
  - — A thin overlay (about 2 in. or less) can be used when
    - o The existing CRC pavement is in good condition with no structural problems but may have an unacceptable level of skid resistance and/or surface noise characteristics.
    - o The existing CRC pavement has minor spalled cracks in the wheelpath that do not affect the structural integrity of the CRCP.
  - — A medium overlay (about 2 to 4 in.) can be used when
    - o The existing CRC pavement has large crack widths (between 0.5 and 1.5 mm).
    - o The existing CRC pavement has surface spalling and scaling.
  - — A thick overlay (greater than 4 in.) should be used when
    - o The existing CRC pavement has localized deformation and settlement due to poor subgrade condition.
    - o The existing CRC pavement has structural defects such as "punchouts," settlement, faulted cracks, and severe spalls.
    - o The existing CRC pavement needs strengthening to accommodate higher traffic loading levels.

Broadly, for HMA overlays over processed PCC, thicknesses will typically be in the range of 8 to 10 in. for long-life pavements. Many agencies will find this level of thickness costly; however, the issue is whether to spend more initially, minimizing future costs, or to enter into an endless cycle of rehabilitation and marginal pavement performance.

HMA MIX DESIGN CRITERIA TO ACHIEVE LONG LIFE

Achieving long-life HMA pavement solutions requires the combination of a rut- and wear-resistant top layer with a rut-resistant intermediate layer and a fatigue-resistant base layer. A high-quality HMA wearing surface or an open-graded friction course, a thick, stiff dense-graded intermediate layer, and a flexible (asphalt-rich) bottom layer are recommended. However, the experience from the states would indicate that the rich bottom layer is not required as long as there is sufficient HMA depth and a strong enough foundation to satisfy the limiting-strain criteria.

Surface Course

The surface-course layer should be able to withstand high traffic and environment-induced stresses without surface cracking or rutting. It should also possess a texture that ensures adequate skid resistance and low tire–pavement noise emission, and a structure that would allow for mitigation of splash and spray. No single material can provide all the desired characteristics, because these tend to compete against each other (e.g., open-graded mixtures are excellent for drainage but are generally not durable, especially in wet-freeze environments). Possible solutions include stone matrix asphalt (SMA), an appropriate Superpave dense-graded mixture, or open-graded friction course. Guidance on mix type selection can be found in Newcomb and Hansen (2006), as shown in Figure 2.21.

For heavily trafficked roads, the need for rutting resistance, durability, impermeability, and wear resistance would dictate the use of SMA (European Asphalt Paving Association, 2007; Michael, Burke, and Schwartz, 2005). This might be especially true in urban areas with high truck traffic volumes. When properly designed and constructed, an SMA mix will provide a stone skeleton for the primary load-carrying

Pavement Layer	Mix Type	NMAS, mm (in.)	Lift Thickness Range, mm (in.)[1]	Traffic Level, MESAL[2,3]		
				<0.3	0.3-10	>10
Base	Dense, Fine	37.5 (1-1/2)	110-150 (4.5-6)	√√	√√	√√
		25 (1)	75-100 (3-4)	√√	√√	√√
		19 (3/4)	60-75 (2.5-3)	√√	√√	√√
	Dense, Coarse	37.5 (1-1/2)	150-190 (6-7.5)	√√	√√	√√
		25 (1)	100-125 (4-5)	√√	√√	√√
		19 (3/4)	75-100 (3-4)	√√	√√	√√
	ATPB	37.5 (1-1/2)	75-100 (3-4)			√√
		25 (1)	50-100 (2-4)			√√
		19 (3/4)	40-75 (1.5-3)			√√
Intermediate	Dense, Fine	25 (1)	75-100 (3-4)	√√	√√	√√
		19 (3/4)	60-75 (2.5-3)	√√	√√	√√
	Dense, Coarse	25 (1)	100-125 (4-5)	√√	√√	√√
		19 (3/4)	75-100 (3-4)	√√	√√	√√
Surface	Dense, Fine	19 (3/4)	60-75 (2.5-3)	√√	√√	√
		12.5 (1/2)	40-60 (1.5-2.5)	√√	√√	√
		9.5 (3/8)	25-40 (1-1.5)	√√	√√	√
		4.75 (1/4)	15-20 (0.5-0.75)	√√	√√	√
	Dense, Coarse	19 (3/4)	75-100 (3-4)			√√
		12.5 (1/2)	50-60 (2-2.5)			√√
		9.5 (3/8)	40-50 (1.5-2)			√√
	SMA	19 (3/4)	50-60 (2-2.5)		√	√√
		12.5 (1/2)	40-50 (1.5-2)		√	√√
		9.5 (3/8)	25-40 (1-1.5)		√	√√
	OGFC	12.5 (1/2)	25-40 (1-1.5)			√√
		9.5 (3/8)	20-25(0.75-1)			√√

Notes: 1. Lift thickness conversion is approximate for practical design.
2. MESAL – Millions of Equivalent Single Axle Loads
3. (√) indicates "Recommended," (√√) indicates "Strongly Recommended."

Figure 2.21. *Mix type selection guide for long-life HMA pavements.*
Source: Newcomb and Hansen, 2006.

capacity, and the matrix (combination of binder and filler) gives the mix additional stiffness. European experience has shown that SMA tends to exhibit the best performance (high durability, good skid resistance, and low noise emission) compared to a range of hot-mix types. A study from the European Asphalt Pavement Association (2007), found SMA mixtures to have an average life of 20 years, whereas traditional hot mixes averaged 15 years. Similar performance trends were noted by those agencies that regularly use SMA in their paving program. Methods for SMA mix design are given in NCHRP Report 425 (Brown and Cooley, 1999). The matrix in an SMA can be obtained by using polymer-modified asphalt, fibers, or specific mineral fillers. The use of fibers is beneficial to preclude drain-down. Care should be taken in controlling the aggregate gradation, especially on the 4.75- and 0.75-mm sieves (Brown and Cooley, 1999).

For lower truck traffic levels, the use of a well-designed, dense-graded Superpave mixture could be warranted. Similarly to SMA, these mixes should be designed against rutting, permeability, weathering, and wear. The Asphalt Institute (1996b) provides guidance on the volumetric proportioning of Superpave mixtures.

It is recommended that a performance test of dense-graded mixtures, whether SMA or Superpave, be done during mixture design. At a minimum, a rut test should be conducted (Brown, Kandhal, and Zhang, 2001). The two most common HMA rut tests are the Hamburg Wheel Track Test (AASHTO T324) and the Asphalt Pavement Analyzer (AASHTO TP63). Later in this chapter ("HMA Stripping: Causes, Assessment, and Solutions"), the Hamburg test is discussed in additional detail (note Figure 2.26 within that section).

In western and southern regions of the United States, open-graded friction courses (OGFCs) are used to improve wet-weather friction. Some northern states such as Massachusetts, New Jersey, and Wyoming use OGFCs as well. These mixes are designed to have voids that allow water to drain from the roadway surface. Void contents as high as 18% to 22% can provide good long-term performance (Huber, 2000). Fibers can be used to help resist drain-down of the asphalt during construction, and polymer-modified asphalt will help in providing long-term performance (Huber, 2000). The mix design for OGFCs can be done using the method that has been developed by Kandhal and Mallick (1999). Kandhal (2001) also gives guidance on the construction and maintenance of OGFC surfaces. This type of mix enhances safety, but it is likely to require more frequent rehabilitation than dense-graded HMA mixes, in part due to clogging of the voids.

The PG grade used in the asphalt mix should be appropriate for the climate and traffic in a given area, consistent with Superpave practice. The LTPPBind software should be used to provide guidance on the proper grade of asphalt if local guidance is not available (Long-Term Pavement Performance, 2010). Normally, 95% or 99% reliability should be used, depending on availability and cost.

Other notable HMA mix issues that should be considered for long-life performance include the following:

- Nominal maximum aggregate size (NMAS) SMA gradations of 4.75 or 9.5 mm are a viable option for thin overlays. These mixes are rut resistant and exhibit low permeability (Cooley and Brown, 2003; Newcomb, 2009). Thin overlays could be considered for the periodic resurfacing that is needed for HMA wearing courses.
- The permeability levels are lower for SMA and fine-graded dense mixes according to Brown et al. (2004) ("fine graded" for the NCAT study was defined as 12.5-mm NMAS mixes with >40% passing a 2.36-mm sieve).
- Recent research studies investigated the use of lower gyration levels for designing SMA mixtures and indicate that 50 to 75 gyrations work well and should be used for SMA mix design (Timm et al., 2006). Furthermore, when fine-graded dense mixes were compared to coarse-graded dense mixes, they exhibited an equal resistance to rutting, were less likely to be permeable, were quieter, had similar friction values, were somewhat easier to compact, and had higher optimum asphalt contents (higher asphalt contents are a plus to combat aging, but the mix will cost more).
- Use of RAP in HMA reduces mix cost (Mamlouk and Zaniewski, 2011).
- On the basis of results obtained by two NCAT studies (Mallick et al., 2003; Brown et al., 2004), the following conclusions were drawn:
  - The air void level of dense-graded HMA has a significant effect on in-place permeability of pavements. This is not a new finding, but it is important to emphasize.
  - The NMAS can have a significant effect on the permeability of coarse-graded Superpave designed mixes. Furthermore, as the NMAS increased, the permeability increased by one order of magnitude. This finding is significant when choosing a wearing course gradation.
  - Fine-graded mixes are less permeable than coarse-graded mixes for the same field air void level.
  - Increasing the layer thickness decreases the mix permeability.

Binder (Intermediate) Course

The intermediate or binder layer should be designed for stability and durability. Stability can be obtained by achieving stone-on-stone contact in the coarse aggregate and using the appropriate high-temperature grading for the binder. This is especially crucial in the top 4 in. of the pavement, where high stresses induced by wheel loads can cause rutting through shear failure.

Two options to reduce cost (by lowering the asphalt content) are to use large-stone mixtures (Kandhal, 1990; Mahboub and Williams, 1990) and to consider the use of RAP. The Superpave mix design approach (Asphalt Institute, 1996b) may be used for mixtures with a nominal maximum aggregate size up to 37.5 mm. However, the use of large nominal aggregate size may lead to segregation and higher-than-desirable air voids, which can lead to the intrusion of water. Requiring a lower void content in mix design and ensuring a high level of compaction in the field are measures to mitigate against these undesirable outcomes. Smaller aggregate sizes can also be used, as long as stone-on-stone contact is maintained. The mix design should be a standard Superpave approach (Asphalt Institute, 1996b) with a design air voids level appropriate for ensuring low permeability. One test for evaluating whether stone-on-stone interlock exists is the Bailey method (Vavrik et al., 2001).

The high-temperature PG grade of the asphalt should be the same as that for the surface to resist rutting. However, the low-temperature requirement could probably be relaxed one grade, because the temperature gradient in the pavement is relatively steep and the low temperature in this layer would not be as severe as for the surface layer (Newcomb et al., 2010). The LTPPBind software can be used to determine the proper asphalt binder grade for each layer (Long-Term Pavement Performance, 2010).

It is recommended that a performance test of dense-graded mixtures be performed during mixture design. At a minimum, this should consist of rut testing (Brown et al., 2001).

Base Course

The asphalt base layer must resist against fatigue cracking. The notion of the fatigue endurance limit discussed above suggests that, at low levels of strain, there is an appreciable change to the fatigue relationship resulting in less damage per cycle. This is in part due to healing, a lack of crack propagation, and nonlinearity in fatigue relationships. Proper consideration should be given to the effects of temperature, aging, healing, and mixture composition.

The predominant mix design approach to resist fatigue cracking in the United States is to use a higher asphalt content, which (1) allows the material to be compacted to a higher density, and in turn improve its durability and fatigue resistance, and (2) provides the flexibility needed to inhibit the formation and growth of fatigue cracks. When combined with an appropriate total asphalt thickness, this helps ensure against fatigue cracking from the bottom layer. An alternative method to achieve high resistance against fatigue cracking is to design for an asphalt content that produces low air voids in place. This ensures a higher volume of binder in the voids in mineral aggregate (VMA), which is critical to durability and flexibility.

Fine-graded asphalt mixtures have also been shown to have improved fatigue life (Epps and Monismith, 1972). However, care should be taken to ensure proper rut resistance during construction if this layer is to be opened to traffic during construction (Newcomb et al., 2010).

In Europe, the concept of high-modulus pavements has been used, particularly in England and France. This solution allows for using less material and reducing the cost of long-life HMA pavements. In this design approach, a very stiff asphalt mixture is used as the base and intermediate layers. In these pavements, the base course mix is made with a stiff binder combined with a relatively high binder content and low void content. This allows for a reduction in thickness between 25% and 30% in the pavement structure (European Asphalt Pavement Association, 2009).

Because the base layer is most likely to be in prolonged contact with water, moisture susceptibility needs to be considered. A higher asphalt content, which would increase the mix density, should enhance the mixture's resistance to moisture problems, but it is advisable to conduct a moisture susceptibility test during the mix design (Newcomb et al., 2010).

HMA stripping resistance is critical for long-lasting HMA renewal solutions. As such, content about its causes, assessment, and currently applied solutions follows.

HMA STRIPPING: CAUSES, ASSESSMENT, AND SOLUTIONS

Introduction and Background

The presence of moisture combined with repetitive traffic can adversely affect the performance of asphalt pavements. Moisture damage is caused by a loss of adhesion or "stripping" of the asphalt film from the aggregate surface as shown in Figure 2.22. Moisture damage may also be caused by a loss of cohesion within the asphalt binder itself, resulting in a reduction in asphalt-mix stiffness. Furthermore, heavy traffic on a moisture-weakened asphalt pavement can result in premature rutting or fatigue cracking as shown in Figure 2.23. The presence of moisture can also accelerate the formation of potholes or promote delamination between pavement layers (Figure 2.24) (Santucci, 2002, 2010). Moisture may enter the pavement in both liquid and vapor form: through the surface by precipitation, hydraulic pressure from tire action, and irrigation, and via capillary rise of subsurface water. Moisture can also be present in the asphalt mix as a result of inadequately dried aggregate.

Factors that contribute to moisture-related distress in asphalt pavements are summarized by Hicks, Santucci, and Aschenbrener (2003). The physical and chemical characteristics of aggregates play a major role in the resistance of asphalt pavements to moisture damage.

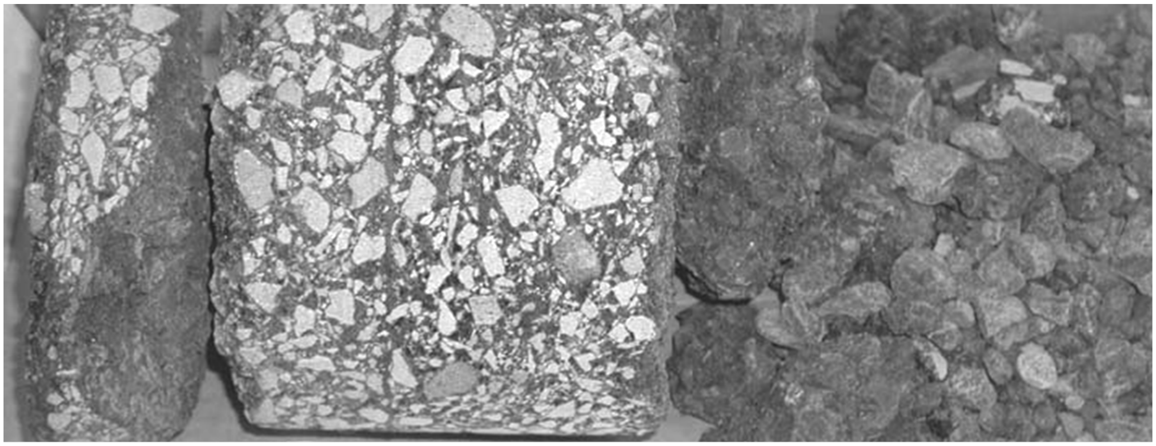

Figure 2.22. *Moisture-induced stripping.*
Photo: Courtesy of Rita Leahy.

(a) (b)

Figure 2.23. *Moisture-weakened asphalt pavement induces premature failure. (a) Rutting. (b) Fatigue cracking.*
Photo: Courtesy of Rita Leahy.

(a) (b)

Figure 2.24. *Moisture exacerbates local pavement distress. (a) Pothole. (b) Delamination.*

Physical properties such as shape, surface texture, and gradation influence the asphalt content of the mix and hence the asphalt film thickness. Thick films of asphalt resist moisture damage better than thin films. Rough-textured aggregate surfaces provide better mechanical adhesion with the asphalt than smooth-textured surfaces.

Surface chemistry of the aggregate is also important. Aggregates range from basic (limestone) to acidic (quartzite), whereas asphalt has a neutral to acidic tendency depending on the asphalt source. This suggests that asphalt adheres more readily to alkaline aggregates such as limestone than to acidic aggregates. Clay in the aggregate or present as a thin coating on the aggregate can contribute to moisture sensitivity problems. Clay expands in the presence of water and weakens the mix. As an aggregate coating, clay adversely affects the adhesive bond between the asphalt and aggregate surface.

The surface chemistry of asphalt can be altered with additives such as antistrip agents to enhance adhesion between the asphalt and aggregate. Physical properties of asphalt, such as viscosity and film thickness, are also important in preventing moisture damage. Complete coating of the aggregate surface during mixing is critical to prevent moisture infiltration at the asphalt–aggregate interface. Lowering the asphalt viscosity by raising mixing temperatures at the hot-mix plant—or, in the case of warm-mix asphalt, by using additives or foam technology—helps to ensure good coating of the aggregate. The lower asphalt viscosity allows deeper penetration into the interstices of the aggregate and thus results in a stronger physical bond between the asphalt and aggregate. The use of additives, such as polymers or rubber in asphalt, generally results in thicker films that help reduce the moisture sensitivity of the mix.

Moisture is a concern during plant production as well. Moisture from inadequately dried aggregates can escape as steam as the asphalt mix is heated or stored, potentially leading to stripping of the asphalt film from the aggregate. In some instances, water has been observed in mixes at the base of hot-mix storage silos and at the edge of windrows of hot mix placed on the roadway before paving (Santucci, Allen, and Coats, 1985).

Good construction practices can produce moisture-resistant asphalt pavements. The most important factor is good compaction. Compacting dense-graded asphalt mixes to a high density (93% to 96% of maximum theoretical density) lowers the air void content and permeability of the mix. Well-compacted mixes are less susceptible to premature rutting, fatigue cracking, and binder oxidation and thus provide a longer service life (Harvey et al., 1996; Blankenship, 2009).

Construction practices that trap moisture in pavement layers should be avoided. For example, placing an open-graded mix over a dense-graded pavement with depressions or ruts can result in water collected on the surface of the underlying pavement unless adequate drainage is provided before the overlay. Placing a high-air-void-content layer between two layers of low air void content should be avoided. Moisture can also accumulate at the interface of impermeable interlayers placed between dense-graded asphalt pavement lifts or under chip seals placed over moisture-sensitive mixes.

California Study

Recent work done in California (Qing, Harvey, and Monismith, 2007) is of special interest. Caltrans initiated and funded a study by the University of California Pavement Research Center (UCPRC) to conduct a statewide field investigation and laboratory testing to determine the severity and major factors associated with moisture damage. The study was conducted from September 2002 to September 2005. The laboratory testing determined the effect of variables such as air void and binder contents on moisture damage and developed dynamic loading test procedures to evaluate moisture sensitivity. The effectiveness of the Hamburg Wheel Track Test (HWTT) and the long-term effectiveness of hydrated lime and liquid antistrip additives were also evaluated. The HWTT will be covered in more detail shortly.

The field investigation surveyed the condition of 194 pavement sections located throughout California. The survey represented pavements encompassing a range of traffic and environmental conditions. The majority of the sections examined were dense-graded HMA, and gap-graded rubber modified asphalt concrete (R-HMA).

Based on the condition survey results, 63 sections were selected for a more intensive analysis that included field permeability measurements and the recovery of cores for testing in the laboratory. About 10% of the pavement sections showed moderate to severe moisture damage.

Air void content was found to be a major factor affecting moisture sensitivity. Dense-graded HMA sections with air void contents of 7% or less showed little or no moisture damage. Sections with air void contents greater than 7% showed medium or severe moisture damage. Based on limited data, R-HMA sections did not show an advantage in moisture resistance over dense-graded HMA using conventional binders. Severe stripping was observed on a few R-HMA sections with high air void contents. Another observation from the field survey was the importance of adequate pavement drainage systems. Drainage systems need to be well designed and maintained to ensure removal of water from the surface and within the pavement during rain events, because the amount of rainfall has a major effect on moisture damage.

The HWTT was found to be an effective predictor, correlating reasonably well with field performance, although in some cases the procedure may fail mixes that perform well in the field or give false-positive results. Suggestions made to improve the prediction accuracy of the HWTT were to (1) use a test temperature consistent with the pavement location and (2) when the standard wet test yields poor results, run the test in a dry condition.

Based on both field and laboratory data, the researchers found hydrated lime and liquid antistrip agents improved the moisture resistance of asphalt mixes. Hydrated lime and liquid antistrip agents were also effective in improving moisture resistance during a conditioning period of up to 1 year. The effectiveness of the liquid antistrip agents remained constant over the 1-year period, whereas, in some instances, the hydrated lime showed increasing effectiveness over the same time period.

Tests to Predict Moisture Sensitivity

The numerous tests developed to predict the moisture sensitivity of asphalt mixes can be grouped into three general categories:

- Tests on mix components and component compatibility,
- Tests on loose mix, and
- Tests on compacted mix.

Table 2.9 provides a summary of the tests used for moisture sensitivity.

Component and Compatibility Tests

Some of the more common tests used on asphalt-mix components to determine the potential for moisture damage include the sand equivalent test, the plasticity index, and the methylene blue test.

Tests on Loose Mix

These tests are conducted on asphalt-coated aggregates in the presence of water. Examples include film stripping, immersion (static, dynamic, or chemical), surface reaction, Texas boiling water, and pneumatic pull-off tests. Advantages of tests on

TABLE 2.9. MOISTURE SENSITIVITY TESTS

Category	Test	Output
Component, compatibility, and loose mixes	Sand equivalent (AASHTO T176)	Relative amount of clay material in the fine aggregate
	Plasticity index (ASTM D 1073)	Plastic nature of fine aggregate or soil
	Methylene blue (AASHTO TP57)	Amount of harmful clay in fine aggregate
	Net adsorption test (NAT) (SHRP Report A-341)	Amount of asphalt remaining on the aggregate surface after desorption
	Boiling water (ASTM D3652)	Visual assessment of stripping
	Ultrasonic accelerated moisture conditioning (UAMC)	Mass loss
	Surface free energy (SFE)	Conditioned-to-unconditioned adhesive bond strength ratio
	Bitumen bond strength (BBS)	Maximum pullout tensile force
Tests on compacted specimens	Original Lottman (NCHRP Report 246)	Indirect tensile strength ratio (TSR) (conditioned to unconditioned)
	Modified Lottman (AASHTO T283)	
	Tunnicliff-Root (NCHRP Report 274)	
	Immersion-compression (AASHTO T265)	Compressive strength ratio (conditioned to unconditioned)
	Energy ratio (ER)	Dissipated creep strain energy (DSCE)
	*E**/ECS (AASHTO TP62; AASHTO TP34)	Ratio of conditioned to unconditioned *E** stiffness ratio (ESR)
	Resilient modulus (ASTM D4123)	Ratio of conditioned M_R to unconditioned M_R
	Dynamic mechanical analyzer (DMA)	Ratio of conditioned-to-unconditioned crack growth index at 10,000 cycles
Repetitive loading in the presence of water	Hamburg Wheel Track Test (HWTT) (AASHTO T324)	Rut depth at 20,000 load cycles and stripping inflection point (SIP)
	Asphalt pavement analyzer (APA) (AASHTO TP63)	Ratio of conditioned-to-unconditioned rut depth
	Model Mobile Load Simulator 3 (MMLS3)	Visual stripping evaluation, conditioned-to-unconditioned rut depth ratio, and conditioned-to-unconditioned TSR
	Moisture induced stress tester (MiST)	Visual stripping evaluation, change in bulk specific gravity, and ratio of conditioned to unconditioned indirect tensile strength

loose asphalt mix are that they are quick to run, cost little, and require simple equipment and procedures. Disadvantages are that the tests do not take into account traffic action, mix properties, and the environment. Results are mostly qualitative and require the subjective judgment and experience of the person performing the test. There is little evidence that results from these tests correlate well with field performance of asphalt mixes.

Tests on Compacted Mix

A multitude of tests on compacted asphalt mixes have been developed and modified. The tests are run on laboratory compacted specimens, field cores, or slabs. Examples include moisture vapor susceptibility, immersion-compression, Marshall immersion, freeze-thaw pedestal, Lottman indirect tension (original and modified), Tunnicliff-Root, ECS and resilient modulus, and wheel tracking (Hamburg and Asphalt Pavement Analyzer) tests. Many of these tests compare the strength of the compacted mix after being exposed to defined conditions, such as temperature and freeze-thaw cycling, to the dry strength of the specimen. Advantages of these tests are that they consider traffic, mix properties, and the environment and that they produce quantitative results rather than subjective evaluations. Disadvantages include longer testing times, elaborate and expensive testing equipment, and test procedures that are laborious.

A survey conducted by the Colorado DOT in 2002 [referred to by Hicks et al. (2003) and Solaimanian et al. (2003)] revealed that most agencies used some version of retained strength tests on compacted mixes (Lottman, modified Lottman, Tunnicliff-Root, or immersion-compression) to determine moisture sensitivity of hot-mix asphalt (Table 2.10). Despite the widespread use of AASHTO T283, the success rate of predicting moisture damage in the field has been limited, as shown in Table 2.11 (Kiggundu and Roberts, 1988). In some instances, the procedure fails mixes that have a long history of good field performance. Some critics of the Lottman-type procedures question the severity of the accelerated vacuum saturation step and its effect on the asphalt–aggregate bond.

TABLE 2.10. POST-SHRP AGENCY USE OF MOISTURE SENSITIVITY TESTS

Test	Number of Agencies Using
Boiling water (ASTM D3625)	0
Lottman (NCHRP 246)	3
Tunnicliff-Root (ASTM D4867)	6
Modified Lottman (AASHTO T283)	30
Immersion-compression (AASHTO T165)	5
Wheel tracking	2

Source: Hicks et al., 2003, and Solaimanian et al., 2003.

TABLE 2.11. SUCCESS RATES OF MOISTURE SENSITIVITY TEST METHODS

Test Method	Minimum Test Criterion	Success (%)
Modified Lottman (AASHTO T283)	TSR ≥70% TSR ≥80%	67 76
Tunnicliff-Root (ASTM D4867)	TSR ≥70% TSR ≥80% TSR: 70%–80%	60 67 67
10-Minute boil test	Retained coating: 85% to 90%	58
Immersion-compression (AASHTO T165)	Retained strength: 75%	47

Note: TSR = tensile strength ratio.
Source: Kiggundu and Roberts, 1988.

More recently, agencies have found greater success with the HWTT, which measures the combined effects of rutting and moisture damage by rolling a steel wheel across the surface of asphalt-compacted specimens immersed in hot water.

The results from the HWTT define four phases of mix behavior: postcompaction consolidation, creep slope, stripping slope, and stripping inflection point (Figure 2.25). The postcompaction consolidation is the deformation measured at 1,000 passes, while the creep slope is the number of wheel passes needed to create a 1-mm rut depth due to viscous flow. The stripping slope is the number of passes needed to create a 1-mm impression from stripping. The stripping inflection point is the number of passes at the intersection of the creep slope and the stripping slope. The Colorado DOT found an excellent correlation between the stripping inflection point and pavements of known stripping performance. The stripping inflection point was more than 10,000 passes for good pavements and fewer than 3,000 passes for pavements that lasted only 1 year (Aschenbrener, 1995; Aschenbrener, McGennis, and Terrel, 1995).

Texas DOT's (TxDOT's) evaluation of the HWTT yielded similarly positive results; that is, the results were repeatable and correlated well with field performance. Also, the TxDOT researchers concluded that the device was capable of detecting the use of antistripping additives in HMA (Izzo and Tahmoressi, 1999).

Solutions: Treatment Methods and Compaction

The primary methods of treating moisture-sensitive mixes involve the use of liquid antistrip additives or lime. The use of organosilane compounds has also shown promise in reducing moisture damage in asphalt pavements (Santucci, 2002, 2010).

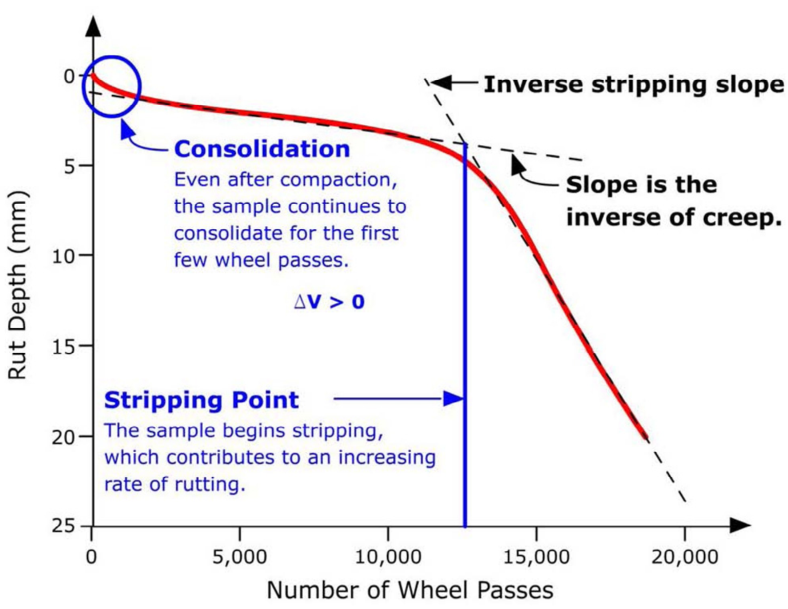

Figure 2.25. *Typical Hamburg wheel-tracking data.*
Source: Pavement Interactive, 2011.

Most liquid antistrips are amine-based compounds that are usually added to the asphalt binder at a refinery or terminal, or through in-line blending at hot-mix plants. The antistrip is typically added at a rate of 0.25% to 1.00% by weight of asphalt. Liquid antistrip additives are designed to act as coupling agents that promote better adhesion at the asphalt–aggregate interface. It is important to pretest any liquid antistrip agent with the job aggregate and asphalt to determine its effectiveness. Any change in asphalt source, aggregate source, or additive should generate additional tests to see how the changes may affect the moisture sensitivity of the mix (Santucci, 2002, 2010; Epps-Martin et al., 2011; TRB, 2003).

Lime treatment is widely used throughout the United States to improve the moisture resistance of asphalt pavements. Lime treatment helps mitigate adhesive and cohesive failure, tends to stiffen the mix, and appears to retard binder aging from oxidation, thus extending pavement life. The most common methods of lime treatment are dry lime on dry aggregate, dry lime on damp aggregate, dry lime on damp aggregate with marination, and lime slurry marination. Lime is generally added at about a rate of 1.0% to 2.0% by weight of dry aggregate or 20% to 40% by weight of asphalt. Most of these treatment methods seem to produce similar results, although some agencies feel lime slurry marination is slightly more effective. However, lime marination can be costly because of processing requirements and space limitations at the hot-mix plant site. The literature contains several reports on the effectiveness of lime treatments, the most recent being a comprehensive study by Sebaaly et al. (2010) at the University of Nevada, Reno.

The pessium voids concept, proposed by Terrel and Shute (1991), suggests that moisture damage will be less for impermeable and for free-draining asphalt mixes. The worst condition for dense-graded asphalt pavements is in the range of 8% to 12% air void contents, where moisture can readily enter the pavement but not easily escape. Improving compaction procedures to reduce the air void contents of dense-graded asphalt mixes to the 6% to 8% range go a long way toward improving moisture resistance. A recent field investigation study of moisture sensitivity in California revealed that the air void contents of dense-graded mixes ranged from 2% to 14% with a mean value of about 7%. Reducing the mean and especially the variance of these air void contents would help reduce the risk of moisture damage. Other research funded by Caltrans quantified the effect of air void content on fatigue resistance and stiffness (rut resistance) of dense-graded mixes—first with laboratory tests and later verified with full-scale heavy vehicle simulator (HVS) tests on pavement sections. More recently, laboratory testing of Kentucky dense-graded mixes revealed that a 1.5% reduction in air void content can increase mix fatigue life by 4% to 10% and increase rut resistance by 34%.

HMA Stripping–Recap

Moisture damage in asphalt pavements is caused by adhesive failure between the asphalt film and aggregate or cohesive failure within the asphalt binder itself. Factors contributing to moisture-related distress include material properties such as type, shape, and porosity of the aggregate and viscosity; film thickness; and source of the asphalt binder. Hot-mix plant production issues, including inadequately dried aggregate, can lead to moisture problems in the finished pavement. Construction practices that trap moisture in pavement layers, such as placing a high air void content mix between low air void content lifts or placing a chip seal over a moisture-sensitive pavement, need to be avoided to minimize moisture damage.

Treatment methods to minimize moisture damage involve the use of liquid antistrip additives or lime. Liquid antistrips are usually added to the asphalt at the refinery or through in-line blending at hot-mix plants. Lime treatment methods include dry lime on dry aggregate, dry lime on damp aggregate, dry lime on damp aggregate with marination, or lime slurry marination.

Good compaction procedures to reduce the air void content of dense-graded asphalt pavements have been shown repeatedly to improve moisture resistance (≥93% of TMD). Slightly tightening existing requirements for maximum theoretical density will also improve the fatigue and rut resistance of asphalt pavements. Lower air void contents will tend to lower mix permeability and limit oxidative hardening of the asphalt binder, thus improving the long-term durability of pavements.

PROJECT EVALUATION

The Basics

In any HMA pavement construction project, the foundation must be able to support paving and compaction operations during construction. When using existing pavements, the “foundation” layer materials may include existing HMA intermediate/base course, existing concrete pavement (intact or fractured), or rubblized concrete. In the former cases, the construction platform is stiff enough to support construction traffic and provide resistance to compactors. When dealing with rubblized concrete, this layer must be well compacted, smooth, and stiff enough to support construction. In situ testing for pavement foundation materials should be conducted. In the United States, the use of DCP, with correlations to CBR values, FWD tests, and GPR surveys have been prevalent.

For existing HMA pavements, the subgrade CBR value should dictate the thickness of the granular base layer, as suggested by the Illinois DOT chart (Figure 2.26). A similar foundation design practice is used in the United Kingdom, as shown in Table 2.12. The CBR of the subgrade dictates the thickness of the overlying granular layers. For a subgrade CBR of less than 15, a minimum 6-in. thickness of subbase (equivalent to high-quality base in the United States) is required. When using FWD testing, TRL set end-result requirements for the pavement foundation (both during and after its construction), stipulating a minimum required stiffness of 5,800 psi on top of the subgrade and 9,500 psi at the top of the subbase under an FWD load of 9,000 lb (Newcomb et al., 2010). Insufficient existing granular base or subbase thickness should be addressed by increasing the HMA overlay thickness to ensure that the limiting-compressive-strain criterion at the top of the subgrade is met.

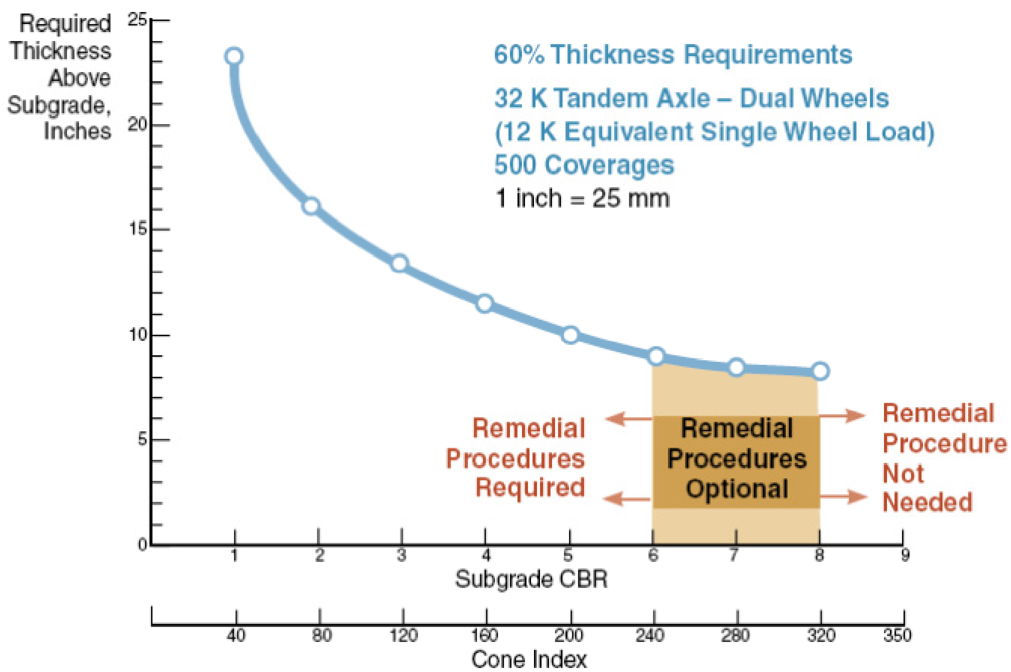

Figure 2.26. *Illinois granular thickness requirement for foundation.*
Source: Illinois DOT, 1982.

TABLE 2.12. TRANSPORT RESEARCH LABORATORY FOUNDATION REQUIREMENTS

Subgrade CBR	<12	12–15	>15
Base thickness (in.)	6	6	9
Subbase thickness (in.)	24	14	—

Note: Base course is called a subbase in the United Kingdom, and a subbase is called capping.
Source: Nunn et al., 1997.

When the existing pavement is concrete, FWD data should be collected at 0.2-mi intervals, or at intervals sufficient to obtain at least 30 drops on the project, whichever is less. FWD drops should be done in the center of the concrete slabs. If the project is jointed concrete, joint transfer tests should be randomly collected to aid in evaluating the joint transfer efficiency. FWD data should be processed with a suitable back-calculation program (Sebesta and Scullion, 2007).

For rubblized concrete pavements, test pits through the rubblized concrete, down to the subgrade foundation, should be conducted systematically throughout the rubblization process to verify the adequacy of the rubblizing equipment and to ensure that the rubblization criteria are met. The procedure recommended by Sebesta and Scullion (2007) for evaluating projects should be followed:

- *Visual condition survey.* Review the project for the overall levels of and types of distresses present. Examine and note the location of any maintenance treatments where the structure may be different. Look for low-lying areas or areas with poor drainage where subgrade conditions may be poor.
- *GPR*. Perform a GPR survey over the entire project, collecting data at 1-ft intervals. Use Colormap to analyze the GPR data to estimate pavement layer thicknesses, locate limits of potential section breaks in the pavement structure, and identify locations where the subgrade may be excessively wet. For increased reliability, survey the section again before rubblization, but after the contractor mills off all HMA.
- *FWD*. Collect FWD data on the project at 0.2-mi intervals, or at intervals sufficient to obtain at least 30 drops on the project, whichever is less. Collect the drops in the center of the concrete slabs. If the project is jointed concrete, randomly collect joint transfer tests to aid in evaluating the joint transfer efficiency. Process the FWD data with a suitable back-calculation program.
- *DCP*. From the FWD data, identify the locations with the highest and lowest deflections at the outermost deflection sensor. Perform DCP tests at these locations. Test a minimum of two locations of high outer sensor deflection with the DCP. Test at least one location with low outer sensor deflection with the DCP. Estimate the thickness of the base layer from the DCP data, and use the Army Corps of Engineers equation to convert the DCP penetration rate to CBR. Determine the CBR and thickness of the base layer. If the DCP data do not clearly detect a base layer, then use the CBR of the first 6 in. beneath the concrete as a "dummy" base layer (many older concrete pavements may not have a base beneath them). Determine the CBR of the first 6 in. of subgrade.

Top-Down Cracking

It is critical that coring of the existing flexible pavement be used to identify top-down cracking if it occurs in the existing pavement. There are at least three reasons for this: (1) there is a need to understand the origins of HMA cracking since that influences basic renewal decisions; (2) HMA quality control factors, such as density, can be impacted by this type of information; and (3) maintenance decisions for renewed pavements, such as crack sealing, will be influenced by such information.

There are numerous studies worldwide that show this is a common cracking mode for HMA surfaces. The following may be broadly concluded:

Surface-initiated cracking of HMA is widespread, particularly for asphalt pavement layers with a combined thickness exceeding about 6 in. (although there have been reports of top-down cracking in thinner HMA). Furthermore, this type of cracking has been reported for a variety of climate and traffic conditions, which are illustrated by Figures 2.27 to 2.30. Figure 2.27 shows top-down cracking in cores taken in Panama with significantly different core thicknesses. Figure 2.28 shows views of top-down cracking which occurred on both an Interstate highway and local streets in Washington State. Figure 2.29 shows longitudinal top-down cracking on a U.S.

Figure 2.27. *Top-down cracking in cores from Panama. Core thicknesses ranged from 6 to 12 in.*

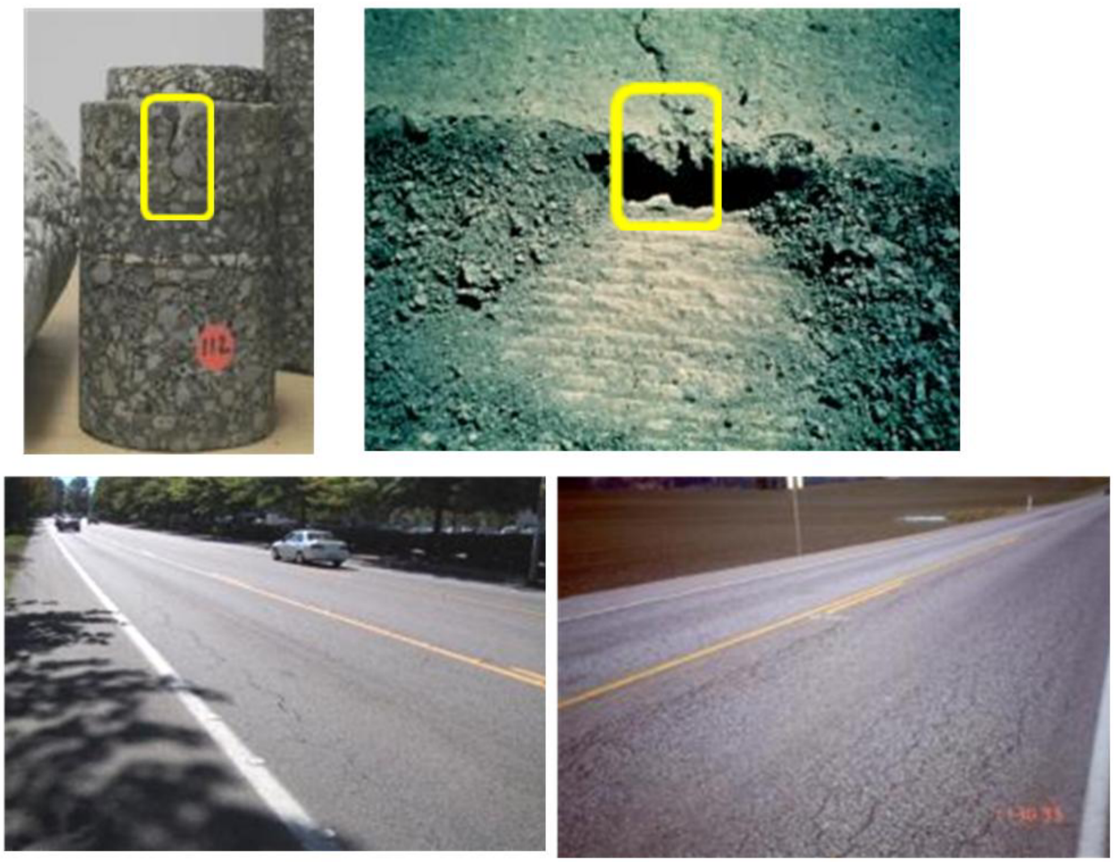

Figure 2.28. *Top-down cracking in Washington State. The top photos are from Interstate 90; the bottom photos are local streets in western Washington.*

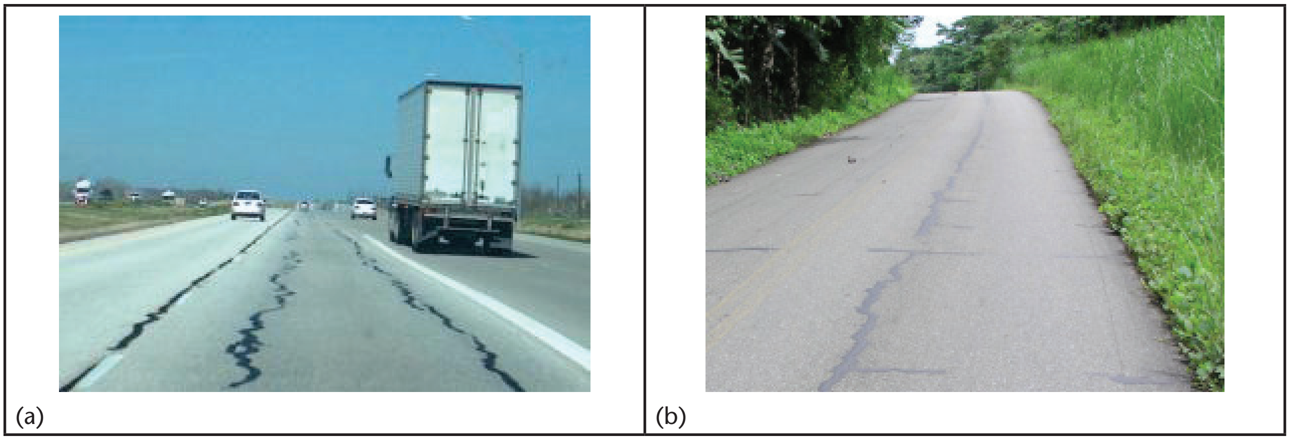

Figure 2.29. *(a) Longitudinal top-down cracking following crack sealing for a U.S. Interstate highway. (b) Longitudinal and transverse top-down cracking in Panama.*

Figure 2.30. *Longitudinal top-down cracking in Michigan. The photo on the right is HMA placed over rubblized PCC pavement.*

Interstate highway and transverse and longitudinal top-down cracking in Panama (near Colon). Figure 2.30 shows two views of top-down cracking in Michigan, including cracking over rubblized PCC pavement.

The age at which top-down surface cracking initiates ranges from 1 to 5 years following surface-course construction (Japan; Matsuno and Nishizawa, 1992), 3 to 5 years (France; Dauzats and Rampal, 1987), 5 to 10 years (Florida; Myers, Roque, and Ruth, 1998), within 10 years (United Kingdom; Nunn, 1998), and 3 to 8 years with an average of 5 years (Washington State; Uhlmeyer et al., 2000). Generally, the HMA thicknesses associated with initiation of top-down cracking ranged from 6 to 7 in.

Surface cracks are caused by a combination of truck tires, thermal stresses, and age hardening of the binder. There is limited agreement on where the critical tensile stresses occur with the surface course. Most researchers note that the critical location is at or near the tire edge. Furthermore, wide-base tires cause higher tensile stresses. Studies based on measured tire–pavement contact pressures and instrumented pavements support the view that truck tires are at least one cause of top-down cracking in HMA wearing courses.

HMA mix aging has a strong role in top-down cracking. Rolt (2001b) reported that top-down cracking is widely observed in tropical environments and appears to be related to the age hardening of the asphalt binder in the upper 2 to 3 mm of surface courses. It was found that the binder is typically 100 to 500 times more viscous in that 2- to 3-mm zone, and hence more brittle, than the binder at a depth of about 10 to 25 mm following initial aging. (Some of the results reported by Rolt noted a field aging period of 24 months.) Importantly, Rolt noted that the increase in binder viscosity was strongly related to age, but HMA mix variables such as air voids, binder content, and filler content were positive second-order factors. An additional finding was that application of a surface dressing (such as a chip seal) to the HMA pavement surface soon after construction was observed to reduce binder aging by a factor of about 50.

Observations made by Rolt (2001b) and Uhlmeyer et al. (2000) note that top-down cracking, once initiated, remains at a constant depth for some time before eventually propagating to the full depth of the HMA layer(s).

HMA CONSTRUCTION QUALITY CONTROL

Construction of a long-life pavement should not be much different than that of conventional pavements, other than requiring a heightened attention to detail and a commitment to build it with quality from the bottom up. Testing should be employed to give continuous feedback on the quality of materials and construction. Achieving uniformity is crucial for ensuring long life.

Along with a proper structural design and mix type, good construction practices are needed to ensure good performance. HMA construction issues that can be detrimental to performance include lack of density, permeability to water, lack of interface bonding, and segregation. These issues are discussed below.

HMA Density

The density of the asphalt base layer can be affected by its interlayer friction with the pavement foundation. Insufficient friction between these two layers will lead to problems in compacting the base layer because it will tend to shove out from under the rollers. This condition can occur if there is excessive dust on the foundation surface or if it has recently rained. Remedial action for such a condition may include waiting for the material to become drier, excavating the top few inches of the foundation to remove the dust, adding granular material to the top of the foundation, or using a thicker lift for the bottom of the base course. An extreme measure would be to place a chip seal on the foundation to provide the necessary friction to hold the asphalt mix in place during compaction.

Another primary issue affecting HMA density in the field is lift thickness. One needs to make sure that the lift thickness corresponds appropriately to the nominal maximum aggregate size in the mixture as provided by Newcomb and Hansen (2006) in Table 2.8.

In general, the lift thickness should be three to four times the NMAS for fine-graded mixtures and four to five times for coarse-graded mixtures (Brown et al., 2004).

The lack of density in the asphalt layers may also be caused by stiff mixes (e.g., mixes with overly oxidized binders due to overheating in the mixing process, and mixes with polymer-modified asphalt binders) that are difficult to work and compact. Industry guidelines provided by the Asphalt Pavement Environmental Council (2001) may be used to ensure the proper temperature is used in the handling and application of liquid asphalt binders. The workability of asphalt mixtures may be improved with warm-mix asphalt technologies that allow the material to be placed and compacted at temperatures anywhere from 35°F to 100°F lower than conventional asphalt mixtures (Prowell and Hurley, 2007).

Prowell and Brown (2007), in NCHRP Report 573, noted that in-place field densities between 92% and 97% of maximum theoretical density (i.e., 3% to 8% air voids) for surface courses will generally provide good performance (based on mixes with gradations passing through or above the Superpave-defined restricted zone). Furthermore, when HMA is placed has an effect on density. Prowell and Brown showed that the majority of the densification of HMA occurs in the first 3 months following construction. This is somewhat counter to prior views that most of the postconstruction densification occurs within 2 years. Furthermore, for HMA placed during cooler fall months, the rapid, additional densification may not occur in time for winter weather.

State DOTs have a range of HMA density specifications. Many of these types of specifications are statistically based with some form of lower specification limit. Based on a survey done in 2001 of several western states and federal lands (Mahoney and Economy, 2001), the reported average in-place HMA density ranged between 92% and 93% of TMD. The lower specification density requirement ranged between 91% and 92%.

Given the evidence available, it is suggested that an average density value for dense-graded mixes is ≥93% of TMD.

HMA Segregation

Segregation can be caused by a separation of fine and coarse aggregates during production, transport, and placement (AASHTO, 1997), or by temperature differentials that occur during transport and paving operations (Willoughby et al., 2002). Coarse aggregate mixtures are usually the most problematic. The danger with segregation in large-aggregate, coarsely graded mixtures is that the mix may become permeable in coarse pockets, which could lead to the infiltration of water and subsequent moisture damage (Scullion, 2006a). Segregation may be measured with infrared temperature techniques and laser texture methods such as the Rosan procedure (Stroup-Gardiner and Brown, 2000). Figure 2.31 illustrates both the open texture resulting from temperature differentials on a two-lane state highway and an infrared image that shows the cooler mix (green and yellow), which leads to lower as-compacted mix densities.

Segregation can be addressed by proper handling of the material during manufacture, transport, and laydown. The use of material-transfer devices that remix the HMA before placement can help in avoiding thermal segregation. Also, the selection

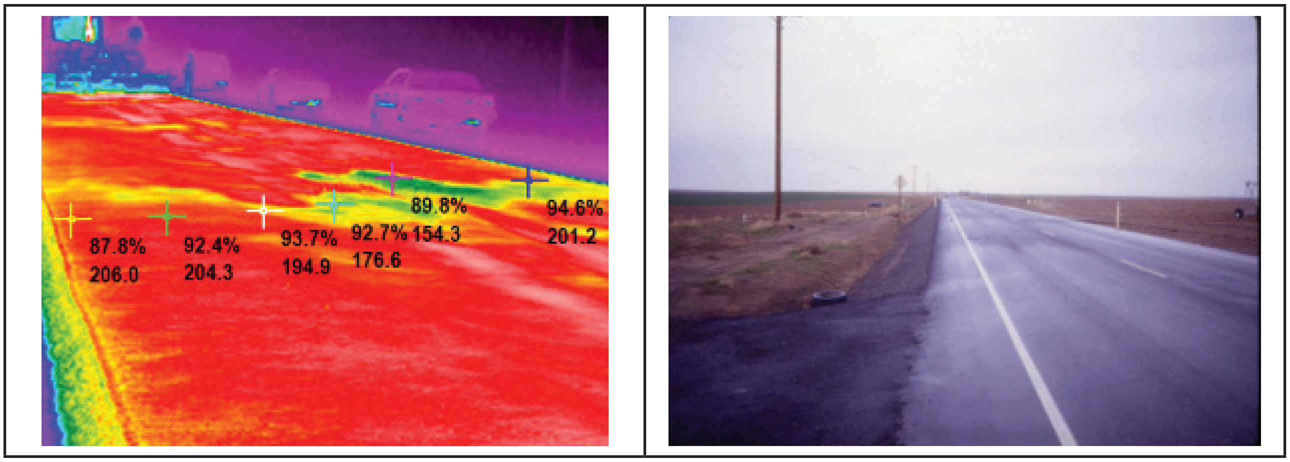

Figure 2.31. *HMA segregation caused by temperature differentials.*

of the appropriate mix design can help in avoiding many of the problems associated with segregation. For example, one should design large-stone asphalt base mixtures to a lower void content so that it is less susceptible to being permeable. Alternatively, one can choose a mix with finer total gradation, which will lessen the possibility of segregation. To ensure impermeability, one can use a fine surface mix, which will seal the surface of the pavement, preventing moisture infiltration from the top.

If temperature differentials occur during construction, but the finished pavement has a uniform density of 93% of TMD or greater for traditional dense-graded mixes, then the pavement should serve its intended length of time. Given the types of pavement distress that result from temperature differentials, it is common to see pavement surfaces that would otherwise last about 12 years require repaving in 7 to 8 years (or less). This translates to a 30% to 40% reduction in pavement surface life. Extreme cases have occurred where the reduction in pavement life is far higher. The lower densities are rarely uniform, but they group in systematic or cyclic areas as shown in Figure 2.26. Temperature variations of 50°F to 100°F or more have been observed following laydown. A rule of thumb is that for every 25°F difference (or decrease) in mat temperature, the air voids in the compacted mix are reduced by 1% (Willoughby et al., 2001).

A number of HMA specification modifications have been crafted largely by state DOTs to address nonuniform laydown temperatures and mix densities. One technique requires that density profiles be taken. That process provides a method of determining the effect of the temperature differentials in the finished product. It can locate potential areas of low density, test those areas, and provide results (via nuclear asphalt content gauge) to determine the extent of the problem. The technique gets the job done; however, the testing is time consuming and results in a large number of tests. What is clear is that typical random sampling associated with HMA density testing does not and should not be expected to identify nonuniform conditions.

A relatively new solution is to measure whether temperature variation is a major factor in a paving project by 100% sampling of the freshly laid HMA mat. The Pave-IR system (MOBA Corp.) provides this type of sampling along with providing a permanent, continuous record of paver operations. Sites for testing can be quickly selected at critical locations to measure the severity of the problem. The device attaches to the paver screen as shown in Figure 2.32.

Longitudinal Joints

Longitudinal joints are potential weakness areas in HMA pavement construction because density tends to be lower at the edges of the asphalt mat, and the mix may be more permeable at this point and more susceptible to moisture infiltration and damage. Guidance exists on the best way to construct longitudinal joints (National Asphalt Pavement Association, 2002). The use of echelon paving or full-width paving has the effect of essentially eliminating the longitudinal joint, because the two paving lanes are placed at the same time. This should be considered the best solution, although it may not always be possible to implement because of space limitations. Other ways to improve longitudinal joint performance include using techniques such as wedge joints, joint heaters, and joint sealants (Brown, 2006). Also, joints should be staggered between lifts to break any continuity in potentially weak joints. Finally, one of the most practical ways of protecting longitudinal joints in lower pavement layers is to use a fine-graded, impermeable mixture on the pavement surface, which will effectively seal the joint in addition to providing a quiet, smooth surface.

Interlayer Bonding

Bonding between asphalt layers is critical to long-term performance, because the total HMA layer would only act as one layer if full bonding between interlayers exists. Otherwise, these thinner layers will behave independently (they will slip relative to each other), thus leading to significantly higher tensile strains, which will cause

Figure 2.32. *Pave-IR thermal imaging system.*

premature cracking. This was demonstrated at the NCAT test track (Willis and Timm, 2007). Before applying any tack or bond coat, the previous layer should be clean and dust-free to ensure good adhesion. Once the tack coat is applied, precautions should be taken to ensure that the coat remains clean until the next layer is placed. This means limiting the time between the application of the tack coat and laying the next layer, and preventing any construction traffic other than that for laying the HMA. It has also been shown that milling enhances the bond in the case of asphalt overlays (West, Zhang, and Moore, 2005). Therefore, milling should be encouraged not only to remove surface defects but also to ensure the bonding of the overlay to the existing pavement surface.

QC Testing

Quality volumetric control of the mixtures is essential to ensure consistency and quality in the final product. The contractor should have access to a fully equipped and staffed quality control laboratory and should conduct periodic testing and data analysis with good quality control and inspection techniques. In-place density can be checked using either nuclear or dielectric methods of testing; ground-penetrating radar can be used as a continuous monitoring tool to check thickness; and smoothness can be evaluated with new lightweight profilometers.

HMA Quality Control and Specifications

Examples of guide specification elements are shown in Table 2.13 that are relevant for HMA quality control. The table includes a brief explanation of why the issue is of special interest along with examples from the recommendations in the Guide Specifications (Chapter 4). These specification elements are sorted by (1) HMA density, (2) HMA segregation, (3) longitudinal joints, and (4) interlayer bonding.

SUMMARY

A summary of the flexible pavement best practices is provided in Table 2.14. They are grouped by the following:

- Structural design,
- HMA mix design,
- HMA construction, and
- Process of existing PCCP layers.

TABLE 2.13. EXAMPLES OF BEST PRACTICES AND SPECIFICATIONS FOR HMA QUALITY CONTROL

Best Practice	Why This Practice?	Typical Specification Requirements
HMA density	HMA density is a function of numerous variables (e.g., mix, layer thickness, weather) and is crucial in constructing long-lasting HMA layers. Air void levels greater than 7%–8% result in accelerated fatigue and increased permeability.	• The average target percent of TMD should range between 93% and 94% for dense-graded mixes. • Use of a lift thickness governed by t/NMAS ≥ 4 will aid the compaction process.[a]
HMA segregation	HMA segregation can take at least two forms: (1) aggregate segregation, which results in an open-textured mix, and (2) temperature differentials, which result in localized low densities. Both types of segregation result in accelerated deterioration of the surface course.	• Consider use and associated measurement options of the density profile approach used by TxDOT. • Alternatively, specify the use of an approved material transfer vehicle (MTV). • Use MTV according to manufacturer recommendations.[a]
Longitudinal joints	There are two major issues: (1) achieving proper joint density and (2) staggering the joints. If the joint density is low, then high air voids are the result—a typical restriction is no more than 2% higher voids in the joint than in the middle of the HMA mat. Staggering the joints reduces the potential for water entry into the pavement structure.	• Stagger joints according to AASHTO 401. • The minimum density of all traveled-way pavement within 6 in. of a longitudinal joint, including the pavement on the traveled-way side of the shoulder joint, shall not be less than 2.0% below the specified density when unconfined.[a]
Interlayer bonding (tack coat)	If interlayer bonding is not achieved then excessive tensile strains occur, resulting in fatigue cracking. This is critical for the wearing course.	• Apply the bond coat to each layer of HMA, and to the vertical edge of the adjacent pavement, before placing subsequent layers. • Apply a thin, uniform tack coat to all contact surfaces of curbs, structures, and all joints. • Apply undiluted tack at a rate ranging from 0.05 to 0.10 gal/yd^2. • Consider the use of a hot tack (paving-grade asphalt cement).[b]

[a] For more details, refer to Elements for AASHTO Specification 401 in Chapter 4.
[b] For more details, refer to Elements for AASHTO Specification 404 in Chapter 4.

TABLE 2.14. SUMMARY OF FLEXIBLE PAVEMENT BEST PRACTICES FOR LONG-LASTING PAVEMENTS

Best Practice Category	Typical Requirements
Structural design	1. Long-lasting flexible pavement renewal options will be thick. Generally additional HMA thicknesses ≥6.0 in. are required. a. Minimum thickness of HMA over crack-and-seat PCCP is 6.0 in. b. Minimum thickness of HMA over rubblized PCCP is 8.0 in. c. HMA thicknesses over existing CRCP are typically ≥4.0 in. 2. Design tools such as PerRoad or the MEPDG are needed for detailed design analyses. Use the endurance limit concept for HMA thickness design. 3. Before selecting the option of PCCP rubblization, check the suitability for rubblization by use of the TxDOT criteria (PCCP thickness versus CBR). If the upper 12 in. of the subgrade has a CBR ≥7, risk associated with this process is significantly reduced.
Mix selection and design	1. Modified PG binders have been shown to significantly reduce rutting; however, the stiffer the binder, the more difficult the placement and compaction. Refer to LTPPBind for advice as to specific PG grades to use. 2. Consider use of fine-graded HMA mix. Dense HMA mixes with a fine gradation have been shown to perform as well as or better than dense coarse-graded mixes. 3. Consider use of SMA for wearing courses. They exhibit superior performance for both cracking and rutting. 4. Smaller NMAS mixes (≤12.5 mm) are better choices. This is broadly true for both SMA and dense-graded HMA mixes.
HMA construction	1. HMA average field density should be ≥93% of TMD for dense-graded HMA. Higher densities reduce the rate of surface aging in the wearing course. 2. Should use lift thicknesses (defined by t/NMAS) ≥4 and must use t/NMAS ≥ 3. 3. HMA segregation must be prevented. This is best done with a MTV. Alternatively, an aggressive testing program with infrared imaging will readily reveal potential problems during paving operations. 4. The density of longitudinal joints must be specified and be similar to that required of the overall mat (but not necessarily the same). 5. Stagger longitudinal joints in multiple HMA lifts. Exceptions can be made for crown lines. 6. Place a uniform tack coat between all HMA layers. No exceptions.
Processing of existing PCC layers	1. Cracked-and-seated PCCP is preferred over rubblization, if possible. 2. A wide range of crack spacings have been suggested for cracked-and-seated PCCP. Dimensions up to 5 ft by 6 ft have worked well. 3. Jointed reinforced concrete pavement must receive a saw, crack, and seat treatment. The crack spacing is about the same as for crack and seat. The saw-cut must sever the existing reinforcing steel. 4. The depth of cracks must be checked by coring. 5. The particle sizes for rubblized PCCP must be specified and checked.

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3 RIGID PAVEMENT BEST PRACTICES

INTRODUCTION

Long-life pavement is defined in this document as pavement sections designed and built to last 50 years or longer without requiring major structural rehabilitation or reconstruction. Periodic surface renewal activities are expected over the 50-year duration. Long-lasting concrete pavements are readily achievable, as evidenced by the number of pavements that remain in service in excess of 50 years; however, recent advances in design, construction, and materials provide the knowledge and technology needed to consistently achieve this level of performance. A more detailed working definition as suggested by Tayabji and Lim (2007) of long-life concrete pavement includes the following:

- Original concrete service life is 40+ years.
- Pavement will not exhibit premature construction and materials-related distress.
- Pavement will have reduced potential for cracking, faulting, and spalling.
- Pavement will maintain desirable ride and surface-texture characteristics with minimal intervention activities, if warranted, for ride and texture, joint resealing, and minor repairs.
- Life-cycle costs and user costs will be reduced.

The pursuit of long-life concrete pavements requires an understanding of analysis, design, and construction factors that affect short- and long-term pavement performance. This requires an understanding of how concrete pavements deteriorate and fail.

Photos of completed and under-construction jointed plain concrete pavements (JPCPs) and continuously reinforced concrete pavements (CRCPs) are shown in Figure 3.1.

Figure 3.1. *Completed and under-construction JPCP and CRCP. (a) and (b) JPCP constructed on HMA base. (c) and (d) CRCP constructed on HMA base.*
Photos: Joe Mahoney.

Pavement Distress Thresholds

Generally recognized threshold values in the United States for distresses at the end of the pavement's service life are presented in Table 3.1 for JPCP and CRCP.

These failure mechanisms can be addressed through application of best practices for structural design (layer thicknesses, panel dimensions, joint design, base selection, and drainage considerations), material selection (concrete ingredients, steel, and foundation), and construction activities (compaction, curing, saw-cut timing, surface texture, and dowel alignment). The trends in structural design of rigid pavements have generally resulted in thicker slabs and shorter joint spacings (for JPCP) along with widespread use of corrosion-resistant dowel bars and stabilized base layers (especially asphalt-stabilized base layers).

TABLE 3.1. THRESHOLD VALUES FOR CONCRETE PAVEMENT DISTRESSES

Distress	Threshold Value
Cracked slabs, % of total slabs (JPCP)	10%–15%
Faulting (JPCP)	0.25 in.
Smoothness (IRI), m/km (in./mi) (JPCP and CRCP)	2.5–3.0 (150–180)
Spalling (JPCP and CRCP)	Minimal
Material-related distress (JPCP and CRCP)	None
Punchouts, number/mi (CRCP)	12–16

Note: IRI = international roughness index.
Source: Tayabji and Lim, 2007.

Types of Concrete Overlays

To design and construct long-lasting rigid pavement overlays as applied to existing pavements, it is important to define the three types of concrete overlays. Typical concrete overlay types were described by Rasmussen and Rozycki (2004). Even though the industry has defined improved terminology and definitions for concrete overlays, these original terms are still widely used and are described below:

- Unbonded concrete overlays consist of a portland concrete cement (PCC) layer constructed on top of an existing PCC pavement, separated by a bond breaker.
- Bonded concrete overlays consist of a PCC layer constructed on top of an existing PCC pavement, bonded to the existing pavement.
- Whitetopping involves a PCC layer constructed on top of an existing hot-mix asphalt (HMA) pavement. Subcategories of whitetopping include thin whitetopping (TWT) and ultrathin whitetopping (UTW):
  - —Conventional whitetopping overlays are ≥8 in. thick,
  - —TWT overlays are >4 in. but <8 in. thick, and
  - —UTW overlays are ≤4 in. thick.

  An illustration of the different types of concrete overlays is shown in Figure 3.2.

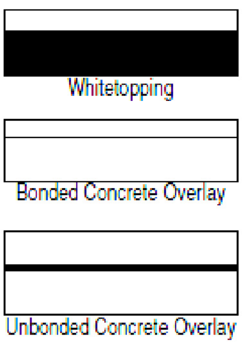


Figure 3.2. *Types of concrete overlays—earlier descriptions.*
Source: Rasmussen and Rozycki, 2004.

The Texas Department of Transportation (TxDOT) recommends a design life of only 5 to 10 years for bonded concrete overlays of asphalt pavements for a range of PCC overlay thicknesses from 4 to 7 in. (greater thickness is associated with higher truck traffic) (Texas Department of Transportation, 2011). Anecdotally, other states have reported using design lives of 20 years or more for similar bonded concrete overlay designs. TxDOT uses the term "thin whitetopping" in its *Pavement Design Guide* (PDM) (Texas Department of Transportation, 2011) to describe this type of overlay, which is normally used at intersections where rutting and shoving of HMA causes performance problems. The TxDOT PDM notes that the contraction joints are to be spaced 6 ft apart with all panels being square.

More recent concrete overlay terminology was described by Harrington (2008). The new definitions provide a simplified description of concrete overlays as shown in Figure 3.3. Two categories are shown: (1) unbonded concrete overlays and (2) bonded concrete overlays. Subcategories are defined based on the underlying pavement, which can be (1) concrete, (2) asphalt, or (3) composite pavements.

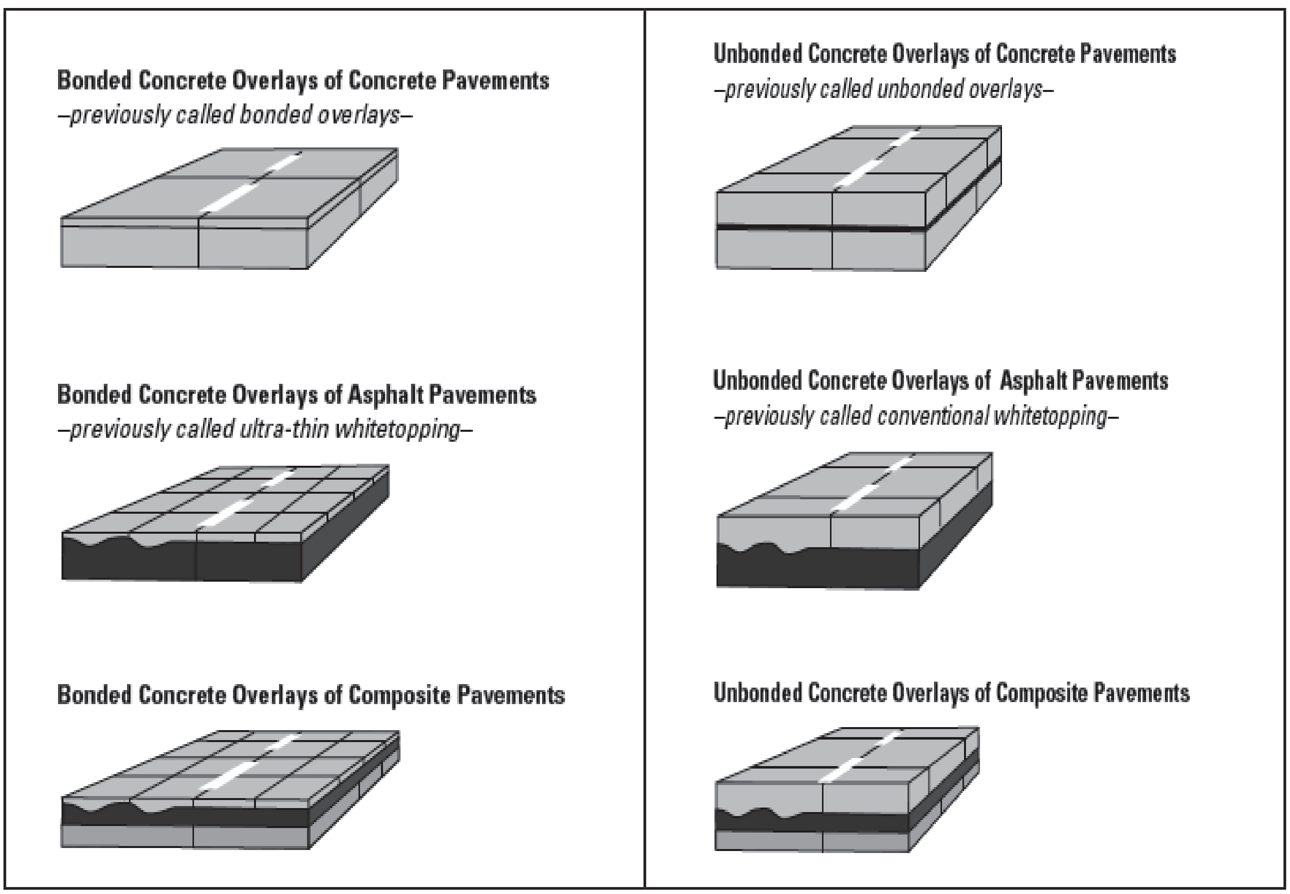


Figure 3.3. *Types of concrete overlays—more recent descriptions.*
Source: Harrington, 2008.

RIGID PAVEMENT RENEWAL STRATEGIES

The renewal strategies for long life using existing pavements as described in this best practices chapter are

- Unbonded concrete overlays of concrete pavements and
- Unbonded concrete overlays of HMA pavements.

The logic for selecting these two long-life strategies follows.

SUPPORTING DATA AND PRACTICES

Long-life renewal strategies should be designed as a system that covers a combination of materials, mixture and structural design, and construction activities. Smith, Yu, and Peshkin (2002) state that the success of long-life renewal alternatives using existing pavements hinges on two critical parameters: (1) the *timing* of the renewal and (2) the *selection* of the appropriate renewal strategy. The timing and selection of the appropriate renewal strategy are dependent on factors such as the condition of the existing pavement; the rate of deterioration of the distress; the desired performance life from the repair strategy; lane closures and traffic control considerations; and user costs. Given the definition of long-life renewal strategies and the constraints of life expectancy associated with timing and selection of pavement renewal strategies, only unbonded concrete overlays (using HMA separator layers) of existing concrete and asphalt pavements are likely to perform adequately for 50 or more years. This conclusion is based on several sets of information which includes, but is not limited to, (1) prior pavement design criteria, (2) state DOT criteria and field projects, (3) LTPP findings, (4) state field visits, and (5) information from the National Concrete Pavement Technology Center (Harrington, 2008).

It is and has been apparent that slab thickness is a major factor in long-life renewal options. Well-known design procedures for PCC systems have been available for several decades. For example, Packard (1973) used fatigue concepts for airport pavement design for the Portland Cement Association (PCA). Packard (1973) and Neville (1975) both noted that for flexural stress ratios less than 0.55 (applied flexural stress divided by modulus of rupture), the fatigue life of PCC is unlimited. Packard actually used a stress ratio of 0.50 to add a bit of conservatism to the PCA airfield design process. Additionally, Packard (1984) produced a fatigue-based highway design method for PCA. This method is also based on fatigue principles [specifically, the flexural stress is divided by the modulus of rupture (28-day cure)]. These fatigue-based approaches use Miner's hypothesis (Miner, 1945) for accumulating fatigue damage.

In addition to existing design procedures and state DOT practices, an extensive amount of pavement performance data has been collected over the past 20 years via the Long-Term Pavement Performance (LTPP) program. These results, as relevant to long-life rigid pavement renewal best practices, are summarized as follows.

Long-Term Pavement Performance (LTPP) and State DOT Information

LTPP

LTPP results were examined to see what could be learned about long-life designs. This included data from General Pavement Study 9 (GPS-9) and Special Pavement Study 7 (SPS-7) projects.

Unbonded Concrete Overlays

From the GPS-9 experiment ("Unbonded Concrete Overlays," which included unbonded JPCP or CRCP overlays placed on JPCP or CRCP), performance data reviewed for Phase 1 of this study were used. The overlay thicknesses ranged from 5.8 to 10.5 in. Separator layers included dense-graded asphalt concrete, open-graded asphalt concrete, and chip seals. The average joint spacing was about 16 ft and load-transfer mechanisms were either aggregate interlock or steel dowels. A summary of the sections and major findings from that assessment include the following:

- Of the unbonded overlays reviewed, the thicknesses were
  - ~6 in. thick, 22%;
  - ~8 in. thick, 22%;
  - ~9 in. thick, 11%; and
  - ~10 in. thick, 45%.
- The thicker JPCP overlays (≥8 in.) exhibited essentially no transverse cracks. The CRCP overlays had transverse cracks with ~4-ft spacing for overlays <10 in. thick and ~5-ft spacing for overlays >10 in. thick.
- On average, thicker GPS-9 overlays had lower IRI values.
- The overall magnitude of the faulting was well below 0.25 in. for all unbonded overlays (the threshold considered for long-life pavements). Faulting levels were significantly less for (1) thicker slabs (~10 in. thick), (2) interlayer thicknesses >2 in., and (3) use of HMA as the interlayer material.
- Thicker HMA interlayers appear to inhibit transverse cracking. This condition also contributed toward the integrity of the joint by controlling the amount of joint faulting.
- Use of dowel bars in transverse joints had a positive impact on all pavement performance measures.

Bonded Concrete Overlays

From the SPS-7 experiment ("Bonded Concrete Overlays on PCC Pavement"), these sections were examined for Phase 1 of this study and included three types of bonded overlays: JPCP, CRCP, and plain concrete pavement (PCP). The third type of overlay included PCP, which was placed on existing CRCP but without reinforcement in the overlay. The ages of overlays ranged from 7 to 11 years (the time between construction and the last condition survey). The overlay thicknesses of the various test sections ranged from a minimum of 3.1 in. to a maximum of 6.5 in. The bonding agent type used in 21 of the SPS-7 sections was water with cement grout, and in 13 sections no bonding agents were employed. The surface-preparation methods used to create bond in the various sections included shot blasting, water blasting, and milling. The major findings from that assessment follow:

- Of these overlays located in four states, the total number of sections (35) expressed as percentages associated by overlay type are

  —CRCP, 51%;

  —JPCP, 26%; and

  —PCP, 23%.

- For bonded JPCP overlays, eight sections all were located on Route 67 in Missouri—which, at the time of construction (1990), experienced about 250,000 equivalent single axle loads (ESALs)/year. The JPCP overlays ranged in thickness from 3.0 to 5.4 in., with an average of 4.3 in. These overlays were placed on existing JPCP, which had a 20-ft spacing between transverse joints. Before the bonded overlays were placed, two surface-preparation treatments were used: either shot blasting or milling. All of these SPS sections had a length of 500 ft. The actual overlay thicknesses and performance with respect to transverse cracks over 5 years following construction are shown in Table 3.2.

TABLE 3.2. OVERLAY THICKNESS AND PERFORMANCE OVER 5 YEARS

Target Overlay Thickness (in.)	Overlay Thickness Based on Cores (in.)	Number of Transverse Cracks Before Overlay (JCPC constructed in 1955, 10-in. slabs)	Number of Transverse Cracks 5 Years After Construction
3.0	4.4	1	21
3.0	3.0	0	11
3.0	3.6	9	43
3.0	3.0	0	15
5.0	4.8	6	102
5.0	4.9	3	101
5.0	5.2	2	94
5.0	5.4	4	130

Source: Smith and Tayabji, 1998; Missouri DOT, 1998.

- Given the cracking levels observed for these nominal 3- and 5-in.-thick bonded overlays, it is unlikely these sections will serve adequately for 50 years. The Missouri DOT notes the following in its *Missouri Guide for Pavement Rehabilitation* (2002): "(1) A bonded PCC overlay is a viable rehabilitation treatment that has historically been technically difficult to construct properly, and (2) unbonded PCC overlays should provide at least 20 years of good performance if properly designed and constructed. PCC thickness should be ≥8 inches with an AC interlayer

≥1 inch." Thus, use of bonded overlays is allowed but unbonded overlays are preferred with 8-in. or thicker slabs.

- The CRCP overlays ranged in thickness from 3.2 to 6.5 in. with an average of 4.6 in. All of these overlays were placed on existing CRCP.
- The CRCP overlays show more promise in that only 4 of 19 sections in the SPS-7 experiment exhibited punchouts following 5 to 7 years of service; however, the length of service precludes a clear view about longevity.
- The data suggest that, on average, thicker SPS-7 overlays (>6 in.) resulted in lower IRI values.

Given the performance of the LTPP JPCP bonded concrete overlays in Missouri and the amount of cracking observed, this study and the rigid best practices will focus only on unbonded concrete overlays over existing concrete and flexible pavements. The amount of transverse cracking suggests that a 50-year life is only likely for unbonded overlays. This is further supported by additional state experience, which follows. The exception might be bonded CRCP overlays, but additional performance data are desirable.

Texas Department of Transportation CRCP Overlays

During the conduct of the SHRP 2 R23 study, a field trip to review concrete overlays was made with TxDOT. Most of TxDOT's bonded concrete overlays are located in the Houston area and are CRCP overlays over existing CRCP. Based on observed performance of 4- to 8-in.-thick bonded overlays and views expressed by TxDOT personnel, it appears that bonded CRCP overlays within that thickness range can be expected to perform for about 25 years. One unbonded 12-in.-thick CRCP overlay approximately 10 years old at the time of visit was performing well.

Information by Kim et al. (2007) documented the performance of 4-in. bonded concrete overlays on existing CRCP in Houston on I-610. The 4-in. overlays were reinforced with either wire mesh or steel fibers. The existing CRCP was assessed to be structurally deficient with 8 in. of CRCP over 1 in. of HMA over 6 in. of CTB. After 20 years of service, the wire-mesh overlay sections provided the best performance in the experiment along with the use of limestone aggregate [a material with low coefficient of thermal expansion (CTE)].

Washington State DOT Bonded Concrete Overlays

Bonded JPCP concrete overlays constructed in 2003 over existing HMA were reviewed (Figure 3.4). Three thicknesses of concrete overlays were used: 3, 4, and 5 in., each placed on I-90 east of Spokane, Washington, which experiences about 1,000,000 ESALs/year. These sections were removed during 2011 due to pavement reconstruction; thus, they were in service for 8 years.

Each of the bonded concrete overlays was 500 ft long and used the same PCC mix. Transverse contraction joints were sawed at 5-ft spacings and the longitudinal joint split the 12-ft-wide lane (thus a joint spacing of 5 ft by 6 ft), as illustrated in Figure 3.5. The mix had a specified minimum flexural strength of 800 psi with a minimum cement content of 800 lb/yd^3. Polypropylene fibers were added at a rate of 3 lb/yd^3. A carpet

Figure 3.4. *Construction of bonded PCC overlays that were placed directly on rotomilled HMA (Washington State, July 2003).*
Photos: WSDOT.

Figure 3.5. *Condition of 3-in. bonded PCC overlay of HMA in 2011, following 8 years of service.*
Photos: WSDOT.

drag finish was applied to the surface (Anderson et al., 2006). The underlying HMA thicknesses were 9 in. for the 3-in. slab, 8 in. for the 4-in. slab, and 7 in. for the 5-in. slab. Following 1 year of service, cracking in the three bonded JPCP sections were as follows:

- 87% of the 3-in.-thick panels were cracked, and
- Each of 4- and 5-in. sections had 4% cracked panels.

At the time of removal in 2011 (Figure 3.6), the 3-in. section was severely distressed, as shown in Figure 3.5. The 4- and 5-in.-thick sections were in substantially better condition. The total accumulated ESALs at the time of removal were a bit less than 10 million.

Figure 3.6. *Removal of 3-in. PCC overlay before reconstruction of this portion of I-90. Bond between the PCC overlays was assessed visually during removal in 2011.*
Photo: WSDOT.

Minnesota DOT and MnRoad Bonded Concrete Overlays

The Minnesota DOT constructed its first set of bonded JPCP concrete overlays on existing HMA at MnRoad in 1997, and it included 3-, 4-, and 6-in.-thick sections. Following 7 years of service, the 3- and 4-in.-thick sections were removed (Burnham, 2008). The 6-in. sections remained in service through 2010. Figure 3.7 shows the 3-in.-thick sections with two different joint layouts. The conclusion was the 5 ft by 6 ft joint layout was superior to the 4 ft by 4 ft layout, but the amount of cracking for both configurations was extensive.

(a) (b)

Figure 3.7. *Condition of 3-in. bonded concrete overlays following 5 million ESALs and 6 years of service.*
(a) MnRoad Cell 95. Bonded concrete overlay 3 in. thick with a 5 ft by 6 ft joint spacing in November 2003.
(b) MnRoad Cell 94. Bonded concrete overlay 3 in. thick with a 4 ft by 4 ft joint spacing in November 2003.
Photos: MnDOT.

Table 3.3 contains a summary of the 3-, 4-, and 6-in. sections. The applied ESALs are about 1,000,000/year on this portion of I-94. The 6-in. sections have survived through 2010 achieving an age of ≥13 years. Figure 3.8 illustrates the performance of the 6-in. sections at MnRoad following 11 years of service.

TABLE 3.3. INITIALLY CONSTRUCTED MNROAD BONDED CONCRETE OVERLAY SECTIONS

Cell	Type	PCC Thickness (in.)	HMA Thickness (in.)	Panel Size (ft)	Year Start to Year End
92	TWT	6	7	10 × 12 (doweled)	1997–2010
93	UTW	4	9	4 × 4	1997–2004
94	UTW	3	10	4 × 4	1997–2004
95	UTW	3	10	5 × 6	1997–2004
96	TWT	6	7	5 × 6	1997 to present
97	TWT	6	7	10 × 12	1997–2010

Source: After Burnham, 2008.

Recap on Concrete Overlays

There are two types of bonded concrete overlays for which state and LTPP performance data are available:

- Bonded JPCP concrete overlays over HMA and
- Bonded concrete overlays over existing PCC.

Given the information summarized, the performance of bonded JPCP concrete overlays over existing HMA is a function of slab thickness and design details such as joints and remaining HMA thickness. Given Interstate types of traffic (~1 million ESALs per year), Table 3.4 shows typical pavement lives that can be expected for various slab thicknesses along with joint details. The expected lives shown are tentative and reflect an extrapolation of the field data reviewed.

TABLE 3.4. BONDED CONCRETE OVERLAYS OVER EXISTING HMA WITH 1 MILLION ESALS PER YEAR WITH SUFFICIENT EXISTING HMA THICKNESS

Slab Thickness (in.)	Joints	Dowels?	Expected Life (years)
3	5 ft by 6 ft	No	5
4	5 ft by 6 ft	No	5–10
5	5 ft by 6 ft	No	10–15
6	5 ft by 6 ft	No	15–20

Note: It is assumed for all HMA thicknesses that the existing HMA materials are in good condition and exhibit no stripping.

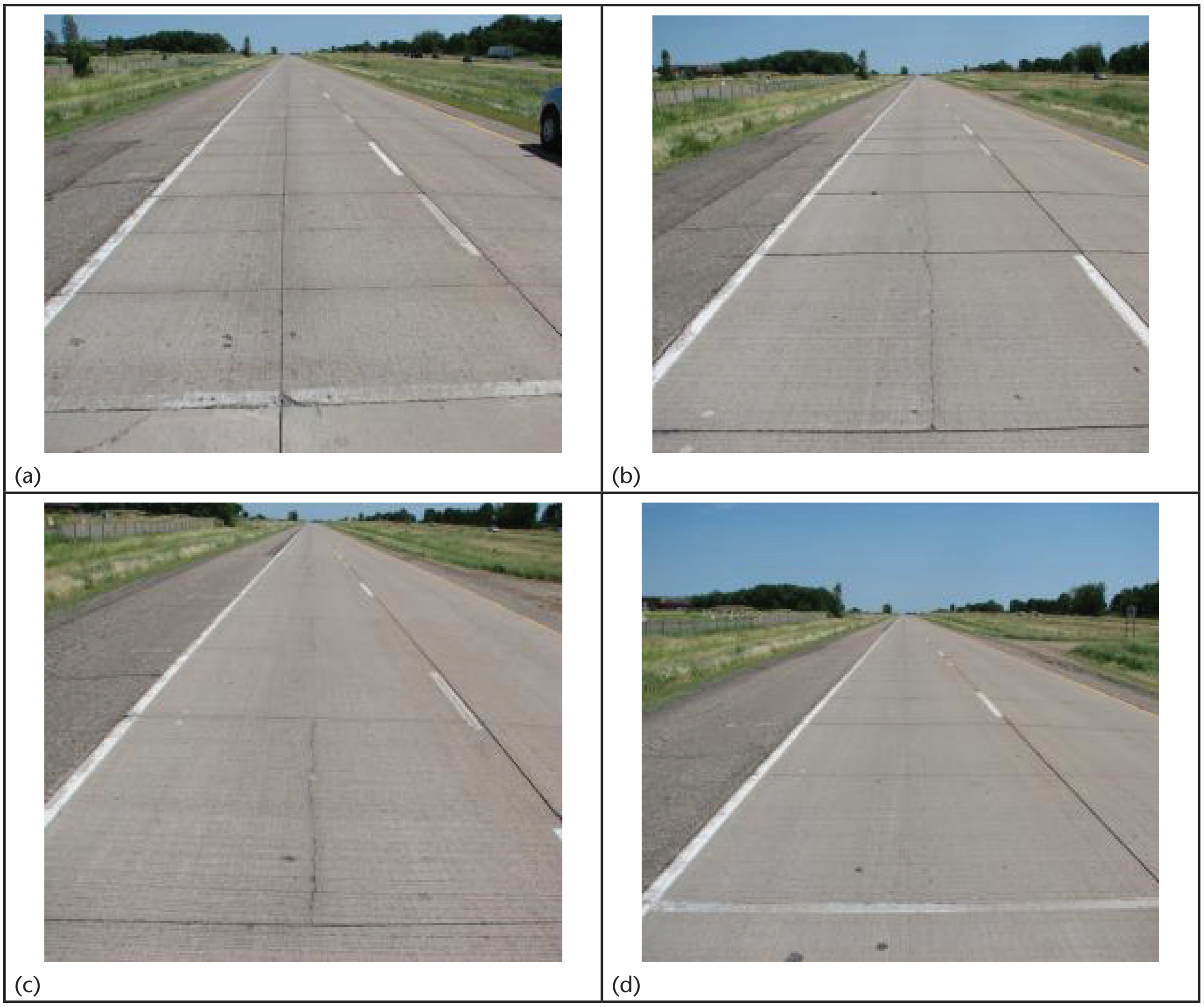


Figure 3.8. *Condition of 6-in. bonded concrete overlays following 10 million ESALs and 11 years of service at the time of the photos (Constructed in 1997). (a) MnRoad Cell 96. Bonded concrete overlay 6 in. thick with a 5 ft by 6 ft joint spacing without dowels. Performance: No cracked panels but noticeable faulting has occurred. Will be diamond ground in 2011 to improve ride. (b) MnRoad Cell 97. Bonded concrete overlay 6 in. thick with a 10 ft by 12 ft joint spacing without dowels. Performance: Excessive faulting and some longitudinal panel cracks resulted in replacement of this section in 2010. (c) and (d) MnRoad Cell 92. Bonded concrete overlay 6 in. thick with a 10 ft by 12 ft spacing with dowels. Performance: Longitudinal cracking in some panels but no faulting. Replaced in 2010.*
Photos: Tom Burnham, MnDOT, July 2008.

A recent summary report from MnRoad (2009) provides design recommendations for bonded concrete on HMA: "Under interstate traffic loads, the best performing and most economical test section at MnROAD has been the 6-inch–thick concrete over 7 inches of existing HMA, installed with 5 x 6-foot panels. This recommendation follows the national trend toward 6-inch thick concrete overlays, placed with 6 x 6-foot panels on higher volume roadways."

Limited information on bonded CRCP overlays suggests they perform better than bonded concrete overlays over HMA for equal thicknesses, given performance data from Texas (Kim et al., 2007). Sections 4 in. thick located on I-610 containing wire mesh and materials with low CTE performed adequately for 20 years. The LTPP results for bonded concrete overlays over PCC provide mixed results.

The preceding findings are supported by Harrington (2008), who states the following:

- Use bonded overlays to "add structural capacity and/or eliminate surface distress when the existing pavement is in good structure condition. Bonding is essential, so thorough surface preparation is necessary before resurfacing."
- Use unbonded overlays "to rehabilitate pavements with some structural deterioration. They are basically new pavements constructed on an existing, stable platform (the existing pavement)."

Additional State Design and Construction Practices

A best practices document by Tayabji and Lim (2007) overviewed a selection of design, materials, and construction features for new concrete pavements for four state DOTs (Illinois, Minnesota, Texas, and Washington). These practices were updated based on recent information and are summarized in Tables 3.5 and 3.6. Minnesota and Washington were grouped together in Table 3.5 because their practices are for JPCP. Illinois and Texas are summarized in Table 3.6 to reflect their CRCP practices. Although these practices were developed with new pavement construction in mind, they are also applicable to long-life concrete overlay systems.

A recurring theme emerges when examining these practices: (1) thick unbonded PCC slabs >11 in. are used, (2) design lives are all >30 years ranging up to 60 years, and (3) PCC mix and materials requirements are important. Thus, as expected, long-life PCC renewal options are not just about slab thickness, but also about materials and construction.

TABLE 3.5. EXAMPLES OF LONG-LIFE JPCP STANDARDS FOR THE MINNESOTA AND WASHINGTON DOTS

Item	Minnesota DOT	WSDOT
Design life	• 60 years	• 50 years
Typical structure	• Slab thicknesses = 11.5–13.5 in. • 3–8-in. dense-graded granular base • Subbase 12–48 in. select granular (frost-resistant)	• Slab thickness = 12–13 in. (typical) • 4-in. HMA base • 4-in. crushed stone subbase
Joint design	• Spacing = 15 ft with dowels • All transverse joints are doweled	• Spacing = 15 ft with dowels • Joints saw-cut with single pass • Hot-poured sealant
Dowel bars	• Diameter = 1.5 in. (typical) • Length = 15 in. (typical) • Spacing = 12 in. • Bars must be corrosion resistant	• Diameter = 1.5 in. • Length = 18 in. • Spacing = 12 in. • Bars must be corrosion resistant; epoxy coatings not acceptable
Outside lane and shoulder		• 14-ft lane with tied PCC or HMA • 12-ft lane with tied and dowel PCC
Surface texture	• Astroturf or broom drag • Longitudinal direction • Requires 1 mm average depth in sand patch test (ASTM E965)	• Longitudinal texturing
Alkali-silica reactivity (ASR)	• Fine aggregate must meet ASTM C1260 (ASR Mortar-Bar Method) • Expansion ≤0.15% OK. If ≥0.30%, reject. • Mitigation required by use of GGBFS or fly ash when expansion is between 0.15 and 0.30%	• Allow various combinations of Class F fly ash and ground granulated blast furnace slag (GGBFS)
Aggregate gradation	• Use a combined gradation	• Use a combined gradation
Concrete permeability	• Use GGBFS or fly ash to lower permeability of concrete • Apply ASTM C1202 for rapid chloride ion permeability test	
Air content	• 7.0% ± 1.5%	• 5.5%
Water/ cementitious ratio	• ≤0.40	• ≤0.44 • Minimum cementitious content = 564 lb/yd^3 of PCC mix
Curing	• No construction or other traffic for 7 days or flexural strength ≥350 psi	• Traffic opening compressive strength ≥2,500 psi by cylinder tests or maturity method
Construction quality	• Monitor vibration during paving	

Source: Tayabji and Lim, 2007; Minnesota DOT, 2005a, 2005b; WSDOT, 2010.

TABLE 3.6. EXAMPLES OF LONG-LIFE CRCP STANDARDS FOR THE ILLINOIS AND TEXAS DOTS

Item	Illinois DOT	Texas DOT
Design life	• 30–40 years	• 30 years
Typical structure	• Up to 14-in. CRCP slab • 4–6-in. HMA base • 12-in. aggregate subbase	• Up to 13-in. CRCP slab with one layer of reinforcing steel • 14–15-in. CRCP slab with two layers of reinforcing steel • Uses stabilized base either 6-in. CTB with 1-in. HMA bond breaker on top or 4-in. HMA • Recommends tied PCC shoulders
Tiebars	• Use at centerline and lane-to-shoulder joints • Use 1 in. × 30 in. bars spaced at 24 in.	
CRCP reinforcement	• Reinforcement ratio = 0.8% • Steel depth 4.5 in. for 14-in. slabs • All reinforcement in CRCP epoxy coated	• Increased amount of longitudinal steel • Design details for staggering splices
Aggregate requirements	• Illinois DOT applies tests to assess aggregate freeze-thaw and ASR susceptibilities	
PCC mix		• Limits the coefficient of thermal expansion of concrete to ≤6 microstrains per °F
Construction requirements	• Limits on concrete mix temperature = 50°F–90°F • Slipform pavers must be equipped with internal vibration and vibration monitoring • Curing compound must be applied within 10 min of concrete finishing and tining • Curing ≥7 days before opening to traffic	• Revised construction joint details

Source: Tayabji and Lim, 2007; TxDOT, 2009a, 2009b, 2011.

CONCEPTS FOR DEVELOPING LONG-LIFE RENEWAL STRATEGIES

Commonly accepted criteria for defining long-life concrete pavement performance (Tayabji and Lim, 2007) were described previously. For the purposes of this document, those criteria are generally applicable, although the performance life requirement has been extended to 50 years.

Long performance life, in combination with good ride quality and minimal distress, cannot be achieved with increased pavement thickness or improved structural design alone. It requires the selection of durable component materials, proper mixture proportioning, comprehensive structural design, and best practices for construction to ensure acceptable long-term performance. Furthermore, it must be recognized that changes in one design or construction parameter (thickness or curing practices, for example) may have implications for the selection of other design parameters (joint spacing, for example). In other words, the pavement structure, materials, and construction practices must be recognized as a system where the failure of any one component (whether structural, functional, or related to durability) results in a system that will not achieve the goal of long life.

One general concept or approach for developing a long-life pavement design or renewal strategy is to identify potential failure mechanisms and address each of them in the design, construction, and/or materials specifications. There are many potential failure mechanisms that may limit the performance life of a given pavement structure, and each of these mechanisms can be addressed in the materials, design, and construction specifications and procedures. Key considerations often include the following:

- Foundation support (uniformity, volumetric stability, including stabilizing treatments);
- Drainage design (moisture collection and removal and design for minimal maintenance);
- Concrete mixture proportioning and components (e.g., selected to minimize shrinkage and potential for chemical attack, low CTE, provide adequate strength);
- Dowels and reinforcing (corrosion resistance, sized and located for good load transfer);
- Accuracy of design inputs;
- Construction parameters (including paving operations, surface texture, initial smoothness); and
- Quality assurance/quality control (QA/QC; e.g., certification, prequalification, inspection).

All the potential failure mechanisms (including those associated with structural or functional deterioration) must be addressed to ensure the pavement system achieves the desired level of performance over 50 or more years. Addressing only one or two distresses or design parameters (e.g., only pavement slab thickness and joint spacing to reduce uncontrolled cracking) while ignoring others (such as durability of materials and concrete curing practices) may postpone the development of some distresses for 50 or more years without preventing the pavement from failing due to other distresses in less than 50 years. The overall pavement performance life will be only as long as the "weakest link" (or shortest life) in the chain of factors that controls the system.

The need for a "systems approach" to long-life pavement renewal or design is illustrated in Figure 3.9. The chart presents an illustration of the expected performance life of an example standard pavement (with a 35-year nominal design life) due to the impacts of various design, materials, and construction parameters. It can be seen that, for this example, all of the components being considered result in a life of about 35 years; if we consider the pavement to be "failed" when any of the component performances "fails," then the expected life of this pavement is equal to the shortest component performance life (about 28 years in this case, limited by the dowel bar corrosion).

The chart in Figure 3.10 illustrates an effort to increase the pavement performance life to 50 years by improving several design and construction parameters (e.g., slab thickness, improved drainage and foundation support). Although the development of

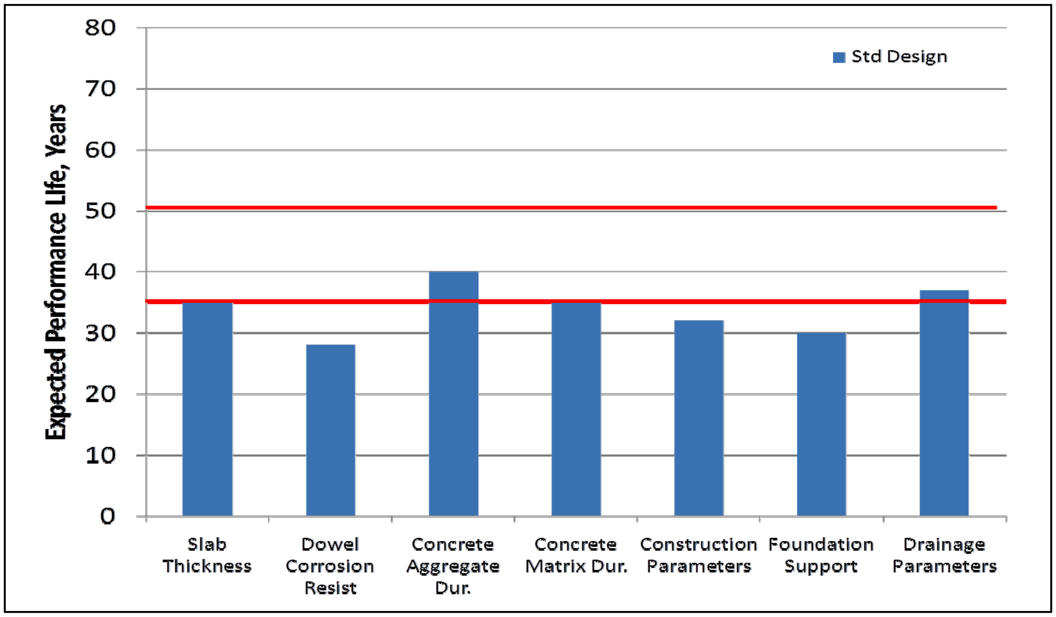


Figure 3.9. *Pavement designed and built for 35-year service life.*

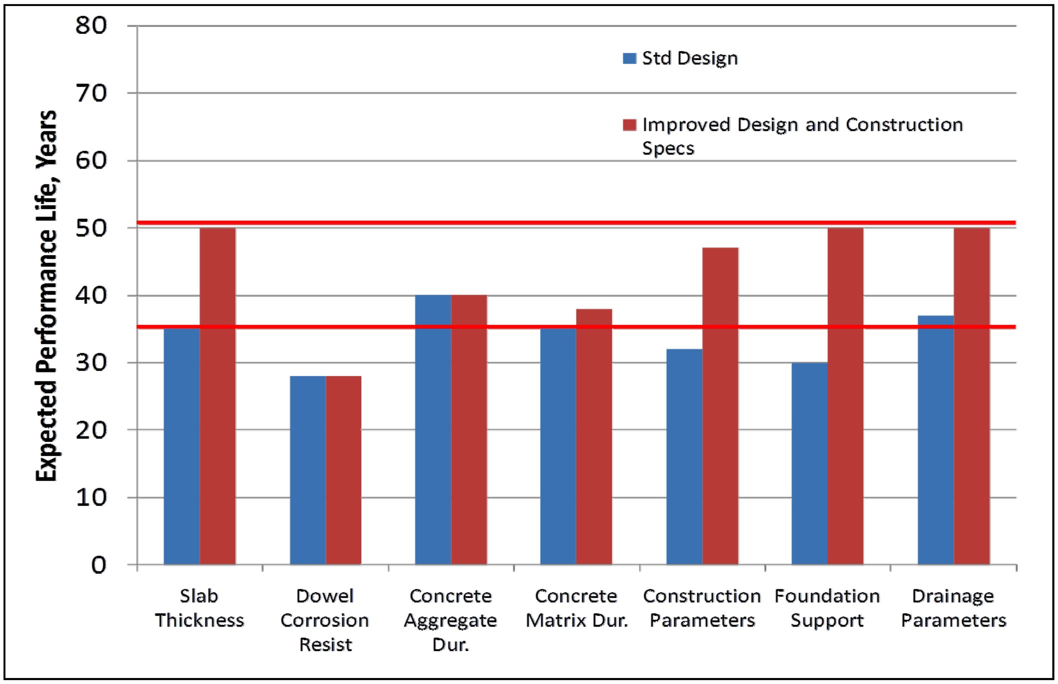


Figure 3.10. *Improved design and construction specifications.*

distresses due to these parameters is not expected to produce "failures" for at least 50 years, the overall pavement life remains controlled by the durability of the dowel bars. The goal of a 50-year performance life was not achieved. The chart in Figure 3.11 shows that the consideration of all of the potential improvement areas is necessary to ensure a performance life of at least 50 years.

MATERIAL CONSIDERATIONS

Although standard concrete pavement mixtures are suitable for the construction of unbonded concrete overlays, concrete is a complex material and involves judicious selection and optimization of various materials to produce a durable concrete (Van Dam et al., 2002). The concrete materials requirements reviewed largely focused on cementitious materials and aggregates.

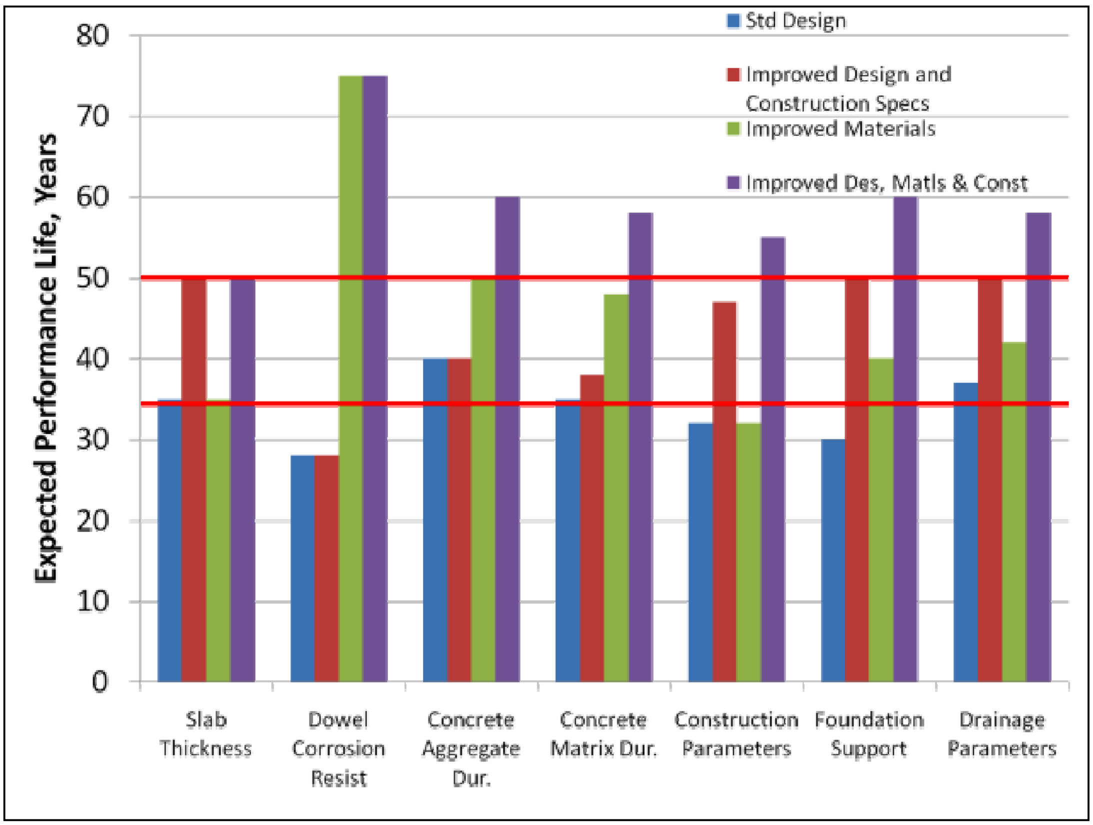


Figure 3.11. *Illustration that all areas of improvement need to be considered for long life.*

Cementitious Materials

Cementitious materials include hydraulic cements, such as portland cement, and pozzolanic materials, such as fly ash. Fly ash is also referred to as supplementary cementitious material (SCM). The current practice for paving concrete is to incorporate portland cement and an SCM. Although not a common practice, some agencies allow use of ternary concrete mixtures that incorporate portland cement and two SCMs.

Supplementary Cementitious Materials

For highway paving applications, the choice of SCM is typically limited to fly ash and ground granulated blast furnace slag (GGBFS). The replacement dosage for SCMs (fly ash and GGBFS) should be compatible with the needs for strength and durability, with upper limits generally defined by state DOT standard specifications. For paving applications, the desired SCM content should be established considering durability concerns (ASR), if applicable, along with economic and sustainability considerations.

Fly ash and slag are covered under the Environmental Protection Agency's Comprehensive Procurement Guidelines (CPG) (Environmental Protection Agency, 2011). The CPGs are federal laws that require federally funded construction projects to include certain recycled materials in construction specifications. Concrete specifications, therefore, must include provisions that allow use of fly ash and slag. The CPGs state that no preference should be given to one of these materials over another; rather, they should all be included in the specification. The enabling federal legislation is from the Resource Conservation and Recovery Act (RCRA).

Fly Ash

Fly ash must meet the requirements of ASTM C618; however, care should be taken in applying ASTM C618 because it is rather broad. Class F fly ash is the preferred choice for controlling ASR, and it also improves sulfate resistance. Selection of fly ash type and dosage for ASR mitigation should be based on local best practices. A photo of Class F fly ash is shown in Figure 3.12.

Typical dosages for Class F fly ash are generally between 15% and 25% by mass of cementitious materials. Sources must be evaluated for typical usage rates. As the amount of fly ash increases, some air-entraining and water-reducing admixtures are not as effective and require higher dosage rates due to interactions with the carbon in the fly ash. While ASTM C618 permits up to 6% loss on ignition (LOI), the state

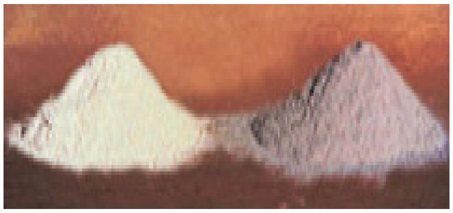


Figure 3.12. *Class F fly ash.*
Photo: FHWA.

DOTs should establish their own LOI limits. Changes in LOI can result in changes to the amount of air-entraining admixture required in the mixture. If fly ash will be used to control expansion due to ASR, the lower the CaO content the more effective it will be. Ideally, the CaO content should not exceed 8%.

Slag Cements and Ground Granulated Blast Furnace Slag (GGBFS)

In the recent past, cement typically used in concrete pavements was traditional portland cement Type I or II (or occasionally Type III for decreased cure times). Today, a wider range of cements is available, including slag cements and cements that are combinations of portland and slag cement.

Blast furnace slag is a by-product of manufacturing molten iron in a blast furnace. This granular material (Figure 3.13) results when the molten slag is quenched with water. The rapid cooling forms glassy silicates and aluminosilicates of calcium. Once ground to a suitable particle size, the end result is GGBFS. This is commonly referred to as "slag cement."

GGBFS must meet the requirements of ASTM C989. The following three grades are based on their activity index:

1. Grade 80 is the least reactive and is typically not used for highway or airport projects.
2. Grade 100 is moderately reactive.
3. Grade 120 is the most reactive, with increased activity achieved through finer grinding. Grade 120 can be difficult to obtain in some regions of the United States.

It is common that blends of slag and portland cements are made (typically designated Type IS(*X*), where *X* is the percentage of GGBFS). Typical dosages of slag should be between 25% and 50% of cementitious materials. Concrete strength at early ages (up to 28 days) may be lower using slag–cement combinations, particularly at low

Figure 3.13. *Preprocessed blast furnace slag.*
Photos: Joe Mahoney.

temperatures or at high slag percentages. The desired slag content must be established by considering the importance of early strengths for the panel-fabrication process. However, if the slag will be used to control expansions caused by ASR, the minimum slag content used is that needed to control ASR.

Aggregates

Aggregates are a key component of concrete and can affect the properties of both fresh and hardened concrete. This is, in part, due to 70% to 80% of the PCC volume being composed of aggregates. Aggregate selection should maximize the volume of aggregate in the concrete mixture to minimize the volume of cementitious paste (without compromising the durability and strength of the concrete mixture). Aggregate requirements for pavement concrete are typically established in accordance with the requirements of ASTM C33. Some of the key aggregate requirements are discussed below. Tables 3.7 and 3.8 summarize the relationship between aggregate properties and possible pavement distresses and standard test methods (Folliard and Smith, 2003) and illustrate the critical roles of competent aggregates. Figure 3.14 shows typical aggregate processing before batching concrete for paving.

Figure 3.14. *Aggregate processing, which includes stockpiles, conveyors, and screening.* Photos: Joe Mahoney.

TABLE 3.7. CONCRETE PAVEMENT PERFORMANCE PARAMETERS AFFECTED BY AGGREGATE PROPERTIES

Performance Parameter	Manifestation	Mechanism(s)	PCC Properties	Aggregate Properties
Alkali-aggregate reactivity	Shallow map cracking and joint/crack spalling, accompanied by staining	Chemical reaction between alkalis in cement paste and either susceptible siliceous or carbonate aggregates		• Mineralogy • Size • Porosity
Blowups	Upward lifting of PCC slabs at joints or cracks, often accompanied by shattered PCC	Excessive expansive pressures caused by incompressibles in joints, alkali-aggregate reactivity (AAR), or extremely high temperature or moisture conditions	• Coefficient of thermal expansion	• Coefficient of thermal expansion • Mineralogy
D-cracking	Crescent-shaped hairline cracking generally occurring at joints and cracks in an hourglass shape	Water in aggregate pores freezes and expands, cracking the aggregate and/or surrounding mortar	• Air void quality	• Mineralogy • Pore size distribution • Size
Longitudinal cracking	Cracking occurring parallel to the centerline of the pavement	Late or inadequate joint sawing, presence of alkali-silica reactivity (ASR), expansive pressures, reflection cracking from underlying layer, traffic loading, loss of support	• Coefficient of thermal expansion • Coarse aggregate–mortar bond • Shrinkage	• Coefficient of thermal expansion • Gradation • Size • Mineralogy • Shape, angularity, and texture • Hardness • Abrasion resistance • Strength
Roughness	Any surface deviations that detract from the rideability of the pavement	Development of pavement distresses, foundation instabilities, or "built in" during construction	• Any that affect distresses • Elastic modulus Workability	• Any that affect distresses • Gradation • Elastic modulus
Spalling	Cracking, chipping, breaking, or fraying of PCC within a few feet of joints or cracks	Incompressibles in joints, D-cracking or AAR, curling/warping, localized weak areas in PCC, embedded steel, poor freeze-thaw durability	• Coefficient of thermal expansion • Coarse aggregate–mortar bond • Workability • Durability • Strength • Air void quality • Shrinkage	• Gradation • Mineralogy • Texture • Strength • Elastic modulus • Size

continued

TABLE 3.7. CONCRETE PAVEMENT PERFORMANCE PARAMETERS AFFECTED BY AGGREGATE PROPERTIES (*continued*)

Performance Parameter	Manifestation	Mechanism(s)	PCC Properties	Aggregate Properties
Surface friction	Force developed at tire–pavement interface that resists sliding when braking forces applied	Final pavement finish and texture of aggregate particles (mainly fine aggregates)		• Hardness • Shape, angularity, and texture • Mineralogy • Abrasion resistance
Transverse cracking	Cracking occurring perpendicular to the centerline of the pavement	PCC shrinkage, thermal shrinkage, traffic loading, curling/warping, late or inadequate sawing, reflection cracking from underlying layer, loss of support	• Shrinkage • Coarse aggregate–mortar bond • Coefficient of thermal expansion • Strength	• Coefficient of thermal expansion • Gradation • Size • Shape, angularity, and texture • Mineralogy • Hardness • Abrasion resistance • Strength
Corner breaks (jointed PCC)	Diagonal cracks occurring near the juncture of the transverse joint and the longitudinal joint or free edge	Loss of support beneath the slab corner, upward slab curling	• Strength • Coarse aggregate–mortar bond • Coefficient of thermal expansion • Elastic modulus	• Coefficient of thermal expansion • Gradation • Size • Mineralogy • Shape, angularity, and texture • Hardness • Abrasion resistance • Strength
Transverse joint faulting (jointed PCC)	Difference in elevation across transverse joints	Pumping of fines beneath approach side of joint, settlements or other foundation instabilities	• Elastic modulus	• Size • Gradation • Shape, angularity, and texture • Abrasion resistance • Elastic modulus • Coefficient of thermal expansion
Punchouts (CRCP)	Localized areas of distress characterized by two closely spaced transverse cracks intersected by a longitudinal crack	Loss of support beneath slab edges and high deflections	• Elastic modulus • Strength • Shrinkage • Coefficient of thermal expansion	• Elastic modulus • Strength • Coefficient of thermal expansion • Size • Shape, angularity, and texture • Abrasion resistance

Source: After Folliard and Smith, 2003.

TABLE 3.8. STANDARD AGGREGATE, AGGREGATE-RELATED, AND PCC TEST METHODS

Property		Test Method
Basic aggregate property	Grading	AASHTO T27
	Specific gravity	AASHTO T84
	Absorption	AASHTO T84
	Unit weight	AASHTO T19
	Petrographic analysis	ASTM C295
Durability	Soundness	AASHTO T104
	F-T resistance	AASHTO T161
	Internal pore structure	AASHTO T85
	Degradation resistance	AASHTO T96, ASTM C535
Chemical reactivity	ASR	ASTM C227, C295, C289
	Alkali-carbonate reactivity (ACR)	ASTM C295
Dimensional change	Drying shrinkage	ASTM C157
Deleterious substances		AASHTO T21
Frictional resistance		AASHTO T242
Particle shape and texture		ASTM D4791

Source: Folliard and Smith, 2003.

Maximum Aggregate Size

The concern with aggregate size involves selecting an aggregate that will maximize aggregate volume and minimize cementitious material volume. In general, the larger the maximum size of the coarse aggregate, the less cementitious material is required, potentially leading to lower costs. Use of smaller maximum-size aggregate (e.g., 0.75-in. maximum size) is required for D-cracking regions. However, the use of 0.75-in. maximum aggregate size alone does not prevent D-cracking, and many state agencies have criteria for D-cracking other than maximum aggregate size.

Aggregate Gradation

In the past, paving concrete was produced using coarse and fine aggregates. Today, agencies are moving toward the use of a combined gradation that may require use of more than two aggregate sizes. A combined gradation is based on an 8-to-18 specification. The percentage retained on all specified standard sieves should be between 8% and 18%, except for the coarsest sieve and sieves finer than the No. 30 sieve. The coarseness factor differentiates between gap-graded and well-graded aggregate gradations, whereas the workability factor determines the mix coarseness. Concrete made with combined aggregate gradation has improved workability for slipform paving applications, requires use of less cementitious materials, exhibits less drying shrinkage, and may be more economical (Richardson, 2005).

Deleterious Substances

Deleterious substances are contaminants that are detrimental to the aggregate's use in concrete. ASTM C33 lists the following as deleterious substances:

- Clay lumps and friable particles,
- Chert (with saturated surface dry specific gravity <2.40),
- Material finer than a No. 200 sieve, and
- Coal and lignite.

Inclusion of larger-than-allowable amounts of the deleterious substances can seriously impact both the strength and durability of concrete.

Soundness

The soundness test measures the aggregate's resistance to weathering, particularly frost resistance. The ASTM C88 test for soundness has a poor precision record. Aggregates that fail this test may be reevaluated using ASTM C666 or judged on the basis of local service history.

Flat and Elongated Particles

Flat and elongated particles affect the workability of fresh concrete and may negatively affect the strength of hardened concrete. The amount of such particles needs to be limited. The breakdown of aggregates, especially the breakdown of fine aggregates, during handling and later when mixed in the concrete may lead to the production of excess microfines. This aggregate breakdown tends to negatively affect concrete workability, its ability to entrain air, and constructability (i.e., placing, compacting, and finishing). Increasing water content to offset the reduction in workability would increase the w/c ratio and lead to lower strength and an increased potential of plastic and drying shrinkage (Folliard and Smith, 2003).

Los Angeles Abrasion Test

The Los Angeles abrasion test provides a relative assessment of the hardness of the aggregate. Harder aggregates maintain skid resistance longer and provide an indicator of aggregate quality.

Durability (D-Cracking)

Durability cracking (D-cracking) is a concern for coarse aggregate particles that typically are (1) sedimentary in origin, (2) have a high porosity, (3) have small pore size (about ~0.1 μm), and (4) become critically (>91%) saturated and subjected to freezing and thawing. Cracking of the concrete is caused by the dilation or expansion of susceptible aggregate particles and will develop wherever the conditions of critical saturation and freezing conditions exist. Because moisture is usually more readily available near pavement joints and cracks, patterns of surface cracking often surround and follow the joints and cracks, as shown in Figure 3.15. Also, because there is usually more moisture present at the bottom of the slab than at the surface, the extent of cracking deterioration is often much greater than what is visible at the surface.

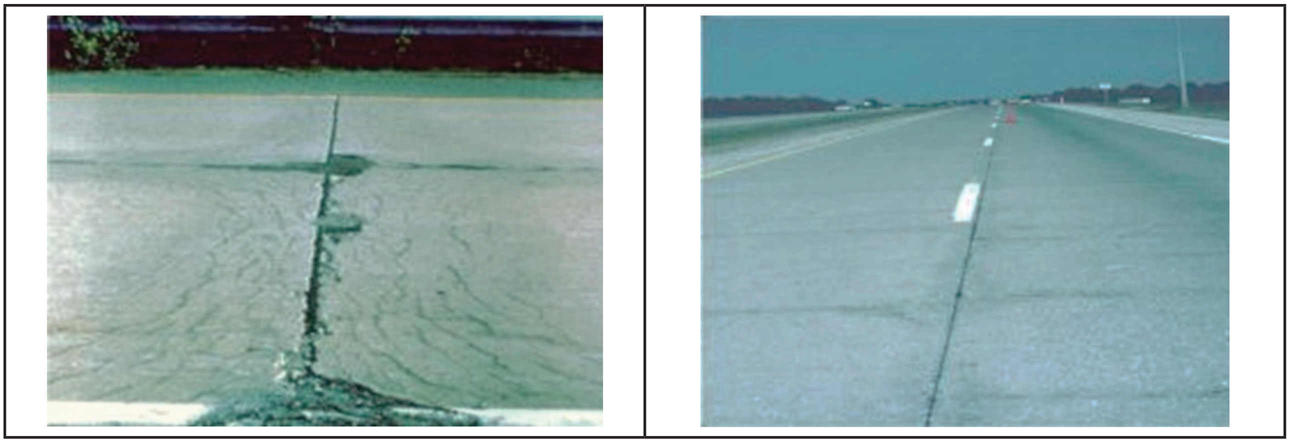


Figure 3.15. *D-cracking.*
Photos: FHWA, NHI.

Van Dam et al. (2002) hypothesized that D-cracking is caused by aggregates with a certain range of pore sizes, and the damage may be exacerbated in the presence of deicing salts for some carbonate aggregates. Coarse aggregates are the primary concern, and for each specific aggregate type, there generally exists a critical aggregate size below which D-cracking is not a problem. Coarse aggregate particles exhibiting relatively high absorption and having pore sizes ranging from 0.1 to 5 μm generally experience the most freezing and thawing problems because of higher potential for saturation. Aggregates of sedimentary origin, such as limestones, dolomites, and cherts, are most susceptible to D-cracking (Van Dam et al., 2002).

Alkali-Aggregate Reactivity (AAR)

Two types of AAR reaction are recognized, and each is a function of the reactive mineral; silicon dioxide or silica (SiO_2) minerals are associated with ASR and calcium magnesium carbonate [$CaMg(CO_3)_2$ or dolomite] minerals with alkali-carbonate reactivity (ACR) (Thomas, Fournier, and Folliard, 2008). Both types of reaction can result in expansion and cracking of concrete elements, leading to a reduction in the service life of concrete structures. A process for identifying whether there is (or could be) a problem with AAR is illustrated in Figure 3.16.

ASR is of more concern because the aggregates associated with it are common in pavement construction. ASR is a deleterious chemical reaction between reactive silica constituents in aggregates and alkali hydroxides in the hardened cement paste. This constituent of concrete has a pore structure, and the associated pore water is an alkaline solution. This alkaline condition, plus reactive silica provided by the aggregate, produces a gel. The gel, unfortunately, has an affinity for water, which in turn grows and produces expansive stresses. These stresses generate polygonal cracking within the aggregate, within the mortar, or both that over time can compromise the structural integrity of concrete. Concrete undergoing ASR often exhibits telltale signs of surface map cracking as illustrated by Figures 3.17 and 3.18. It is widely accepted

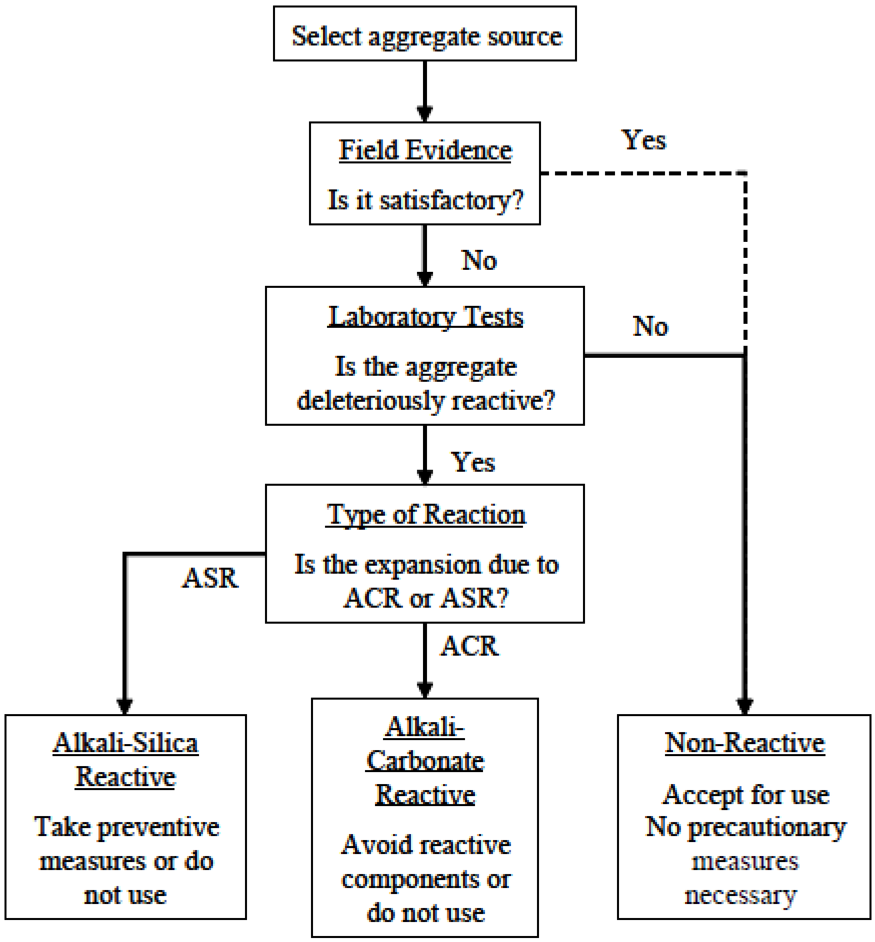


Figure 3.16. *Evaluation stages for alkali-aggregate reaction determination.*
Source: Thomas et al., 2008.

that high-pH (>13.2) pore water in combination with an optimum amount of reactive siliceous aggregate are key ingredients to initiate ASR expansion; it is also believed that a relative humidity (RH) ≥85% is essential for ASR to occur.

Although the problem is widely known, and successful mitigation methods are available, ASR continues to be a concern for concrete pavement. Aggregates susceptible to ASR are either those composed of poorly crystalline or metastable silica materials, which usually react relatively quickly and result in cracking within 5 to 10 years, or those involving certain varieties of quartz, which are slower to react in field applications. ASR research is ongoing and the provisions associated with ASR-related testing are based on best current practices. Guidelines related to ASR will continue to be updated or replaced as more research becomes available.

AASHTO has issued a Provisional Practice—AASHTO Designation PP 65-10—to address ASR. The full title of PP 65-10 is "Provisional Practice for Determining the Reactivity of Concrete Aggregates and Selecting Measures for Preventing Deleterious

Figure 3.17. *Illustration of ASR on a traffic barrier.*
Photo: FHWA.

Figure 3.18. *Illustration of ASR in concrete pavements.*
Source: D. Huft, South Dakota DOT.

Expansion in New Concrete Construction." Additionally, reports from PCA (Farney and Kosmatka, 1997) and FHWA (Thomas et al., 2008: Fournier et al., 2010) provide solid explanations of why ASR occurs, how it can be assessed, and mitigation measures that can be taken.

Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) plays an important role in PCC joint design (including joint width and slab length) and in accurately computing pavement stresses (especially curling stresses) and joint load-transfer efficiency (LTE) over the design life; thus, the lower the CTE the better for concrete pavements.

The CTE of concrete is highly dependent on the CTEs of the concrete components and their relative proportions (as well as the degree of saturation of the concrete). Cement-paste CTE increases with water-to-cement ratio, and cement pastes generally have higher CTEs than concrete aggregates (as shown in Table 3.9). Therefore, the concrete aggregate, which typically comprises 70% or more of the volume of concrete, tends to control the CTE of the hardened concrete: more aggregate and lower CTE aggregate results in concrete with lower CTE values. It should be noted that critical internal stresses may develop in the PCC if the thermal expansion characteristics of the matrix and the aggregates are substantially different, and large temperature changes take place.

TABLE 3.9. TYPICAL CTE RANGES FOR COMMON PCC COMPONENTS

Material Type	Typical CTE ($\times 10^{-6}$/°F)
Aggregate	
Limestone	3.4–5.1
Granites and gneisses	3.8–5.3
Basalt	4.4–5.3
Dolomites	5.1–6.4
Sandstones	5.6–6.5
Quartz sands and gravels	6.0–8.7
Quartzite, cherts	6.6–7.1
Cement paste with w/c ratio 0.4–0.6	10.0–11.0
Concrete cores from LTPP sections	4.0 (lowest), 5.5 (mean), 7.2 (highest)

Source: ARA, 2004.

Chemical Admixtures

A number of chemical admixtures can be added to concrete during proportioning or mixing to enhance the properties of fresh and/or hardened concrete. Admixtures commonly used in mixtures include air entrainers and water reducers. The standard specification for chemical admixtures in concrete used in the United States is AASHTO M194 (ASTM C494). The use of chemical admixtures for concrete is a well-established practice and requires no additional provisions for application. High-range water reducers are typically not used with paving concrete.

Other Materials

The characteristics of other materials used in the construction of unbonded concrete overlays are as follows:

- Dowel bars should conform to the appropriate ASTM and AASHTO standards. The standard practice in the United States is to specify use of epoxy-coated dowel bars. However, the effectiveness of the current standard epoxy coating materials and processes beyond 15 to 25 years in service is considered suspect. Figure 3.19 shows epoxy-coated dowels with less than 15 years of service in Washington State.

Figure 3.19. *Corroded epoxy-coated dowel bars in a retrofitted dowel bar project (original bars 1.5 in. by 18 in.).* Photos: WSDOT.

It is noted that these photos are from retrofit dowel projects, which present challenges in consolidating the patching mix—a situation unlikely to occur in PCC overlays; however, voids in the vicinity of dowels are a concern. Corrosion has been noted for epoxy-coated dowels by WSDOT on fully reconstructed JPCP construction following about 15 years of service. Several recent projects (in Minnesota, Illinois, Iowa, Ohio, and Washington State) have been constructed using stainless steel–clad dowel bars (Figure 3.20) and zinc-clad dowel bars with satisfactory performance (Federal Highway Administration, 2006). WSDOT requires corrosion-resistant dowel bars for concrete pavements that have a design life of greater than 15 years. The long-life dowel options used by WSDOT include (1) stainless steel–clad bars, (2) stainless steel tube bars whereby the tube is press-fitted onto a plain steel inner bar, (3) stainless steel solid bars, (4) corrosion-resistant steel bars that conform to ASTM A1035, and (5) zinc-clad bars (Washington State Department of Transportation, 2010). The Minnesota and Wisconsin DOTs have similar specifications for long-life dowel bars, with Minnesota allowing the use of hollow stainless steel tubes as an additional option, and neither state allowing the A1035 dowels (Minnesota Department of Transportation, 2005b; Wisconsin Department of Transportation, 2009). Additional guidance on dowel bar design can be found in a recent publication by the Concrete Pavement Technology Center (2011).

- Tiebars should conform to the appropriate ASTM and AASHTO standards.
- All joint cuts and sealant materials used should conform to the appropriate ASTM and AASHTO standards, or a governing state specification.

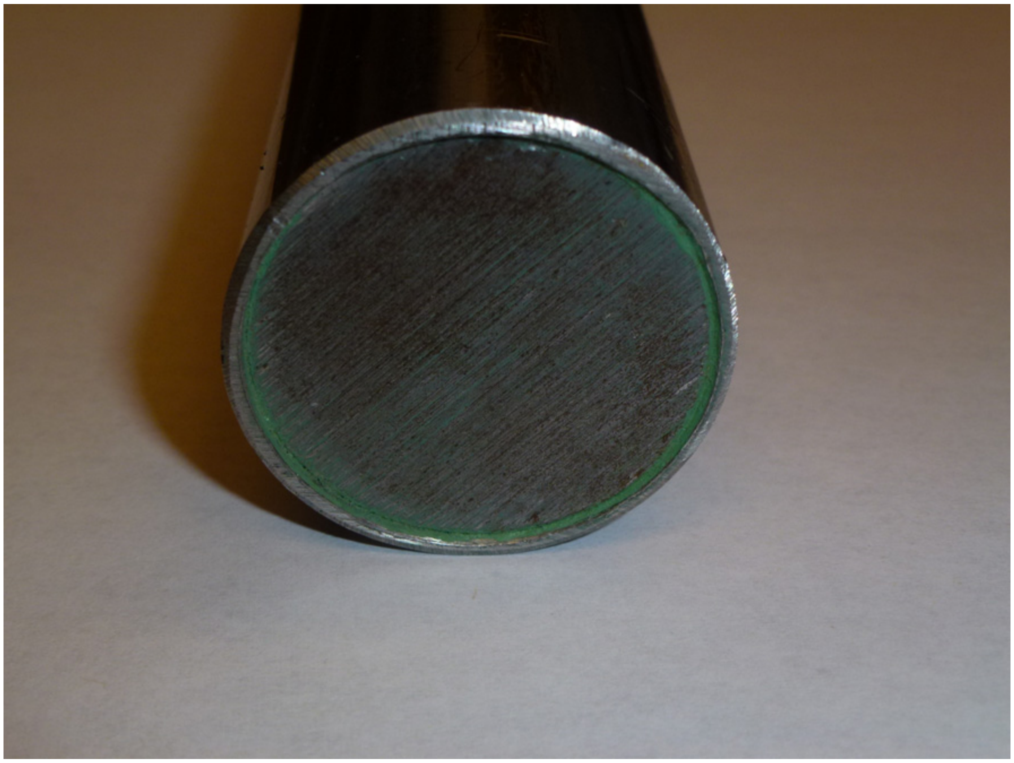


Figure 3.20. *Stainless steel dowel bar.*
Photo: Joe Mahoney.

UNBONDED CONCRETE OVERLAYS OF CONCRETE PAVEMENTS

Criteria for Long-life Potential

This renewal strategy is applicable when the existing pavement exhibits extensive structural deterioration and possible material-related distresses such as D-cracking or reactive aggregate (Smith, Yu, and Peshkin, 2002: Harrington, 2008). The success of the strategy depends on the stability (structural integrity) and the uniformity of the underlying structure. Since the concrete overlay is “separated” from the underlying pavement, the preoverlay repairs are usually held to a minimum. Figure 3.21 is a sketch of an unbonded overlay over concrete.

Figure 3.22 illustrates an in-service unbonded undoweled concrete overlay. The photo shows a 35-year-old JPCP overlay over an existing JPCP located on I-90 in Washington State.

The following sections summarize some of the design and construction issues to consider for long-life unbonded concrete overlays.

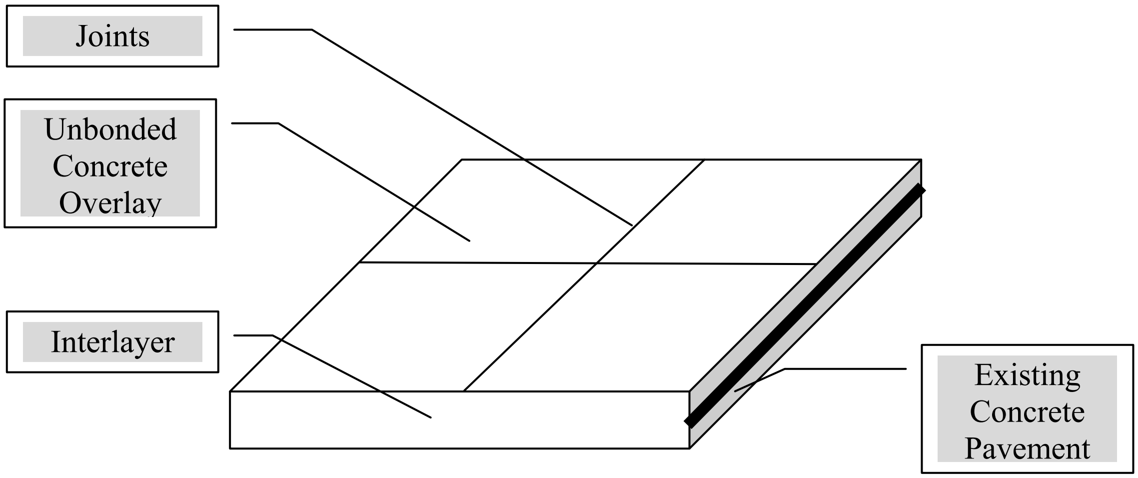


Figure 3.21. *Unbonded concrete overlay of concrete pavement.*
Illustration: Joe Mahoney.

Figure 3.22. *Unbonded 9-in. JPCP concrete overlay placed over concrete (I-90 in Washington State; overlay is 35 years old).*
Photo: WSDOT.

General Design Considerations

Smith et al. (2002) and Harrington (2008) have suggested that, when designing unbonded concrete overlays, the following factors need to be considered:

- *The type and condition of the existing pavement*. In general, unbonded concrete overlays are feasible when the existing pavement is in poor condition, including material-related distress such as sulfate attack, D-cracking, and ASR. The structural condition of the existing pavement can be established by (1) conducting visual distress surveys, (2) conducting deflection testing using a falling weight deflectometer (FWD) (the deflection magnitudes can be used to determine the load-transfer efficiency across joints, determine possible support characteristics under the slab corners and edges, back-calculate the modulus of subgrade reaction and modulus of the existing portland cement concrete pavement, and determine the variability of the foundation layers along the length of the project); and (3) extracting cores from the existing pavement. Laboratory testing of the cores is necessary if the existing pavement exhibits D-cracking or reactive aggregates.
- *Preoverlay repairs*. One of the attractive features of this renewal strategy is that extensive preoverlay repairs are not warranted. It is recommended that only those distresses need to be addressed that can lead to a major loss in structural integrity and uniformity of support. The guidelines (Harrington, 2008) for conducting preoverlay repairs are summarized in Table 3.10.

TABLE 3.10. GUIDELINES FOR PREOVERLAY REPAIRS

Existing Pavement Condition	Possible Repairs
Faulting ≤10 mm	No repairs needed
Faulting >10 mm	Use a thicker interlayer
Significant tenting, shattered slabs, pumping	Full-depth repairs
Severe joint spalling	Clean the joints
CRCP with punchouts	Full-depth repairs

Source: Harrington, 2008.

- *Separator-layer design*. The separator layer is a critical factor for the performance of the unbonded concrete overlay. The separator layer acts as a lower-modulus buffer layer that assists in mitigating cracks from reflecting up from the existing pavement to the new overlay. The separator layer does not contribute significantly to the structural enhancement.

Structural Design and Joint Design Considerations

The design thickness of unbonded PCC overlays is typically ≤≤9 in. for Interstate applications. Figure 3.23 illustrates the probability of poor performance of unbonded concrete overlays in these applications as a function of slab thickness. It is evident that, for long-life pavements in high-traffic-volume applications, the overlay thickness should

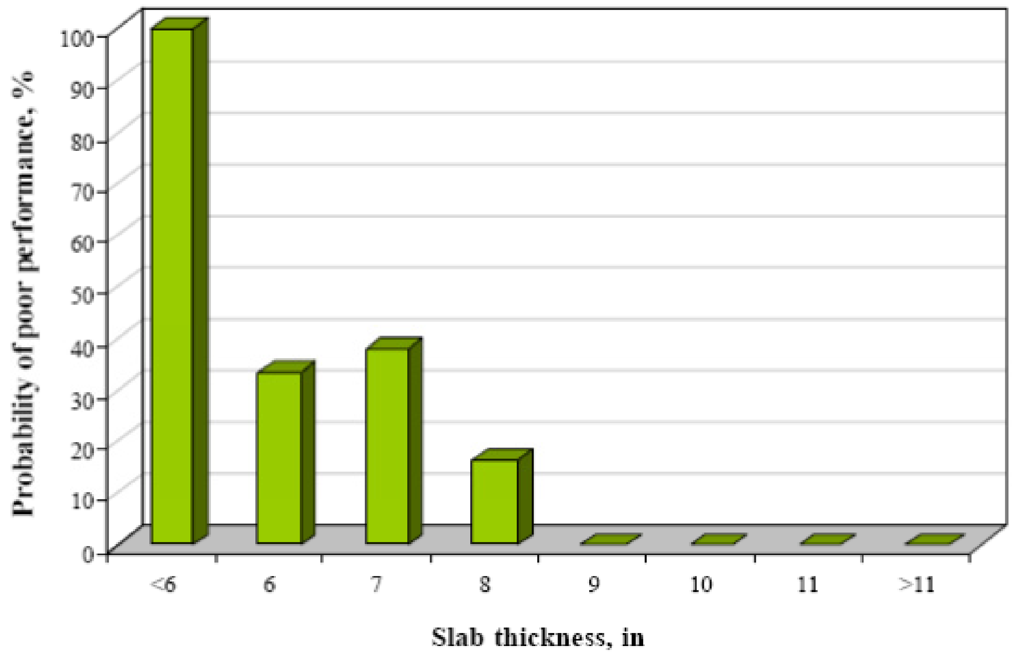


Figure 3.23. *Slab thickness versus probability of poor performance for unbonded JPCP overlays.*
Source: Smith et al., 2002.

be 9 in. or greater. It is clear that slab thickness is one of the critical design features for ensuring long service life; however, the slab thickness required for long pavement life may vary somewhat with other design details (e.g., joint design and layout), and long life cannot be achieved at any slab thickness unless sufficiently durable materials are used.

Thickness design can be performed using either the AASHTO 1993 or the Mechanistic-Empirical Pavement Design Guide (MEPDG) design methods. The key factors associated with these two methods are described below:

In the AASHTO design method (1993/1998), the overlay design is based on the concept of structural deficiency, in which the structural capacity of the unbonded concrete overlay is computed as a difference between the structural capacity of the new pavement designed to carry the projected traffic and the effective structural capacity of the existing pavement. The effective structural capacity of the existing pavement can be established using (1) the condition survey method or (2) the remaining life method. The thickness of the new pavement required to carry the projected traffic can be determined by using the AASHTO design procedure for new PCC pavements. This method of design does not take into account the interaction (friction and bonding) between the separator layer and the overlay and separator layer and the existing pavement. The 1993/1998 AASHTO overlay design method does not directly account for the effects of thermal (curling) and moisture (warping) gradients. The results tend to be conservative for high-ESAL conditions and often calculate greater concrete overlay design thicknesses than mechanistic-based procedures.

The MEPDG (or Darwin-ME) design method is based on the damage concept and uses an extensive array of inputs to estimate pavement distress for a specific set of inputs. The predicted distress types for JPCP are slab cracking, faulting, and IRI. For CRCP, the predicted distress types are punchouts and IRI. The production version of the MEPDG (Darwin-ME) from AASHTO was released during 2011.

Joint design is one of the factors affecting jointed pavement performance. It also affects the thickness design for overlays. The joint design process includes joint spacing, joint width, and load-transfer design (dowel bars and tiebars). Size, layout, and coating of the dowel bars depend on the project location and traffic levels.

Load transfer in unbonded concrete resurfacing is typically very good—comparable to that of new JPCP on HMA base, and better than that of JPCP on untreated base. Doweled joints should be used for unbonded resurfacing on pavements that will experience significant truck traffic (i.e., typically for concrete overlay thicknesses of 9 in. or more). Several studies have shown that adequately sized dowels must be provided to obtain good faulting performance (Snyder et al., 1989; Smith et al., 1997). Dowel diameter is often selected based on slab thickness, but traffic may be a more important factor for consideration. For long-life pavements, 1.5-in.-diameter bars are usually recommended. Additionally, corrosion-resistant dowels (e.g., stainless steel–surfaced, nonstainless corrosion-resistant steel (ASTM A1035), and zinc-clad steel alternatives are required by those state DOTs considering long-life designs. Details concerning the design of dowel load-transfer systems can be found in a recent publication prepared by the National Concrete Consortium (Concrete Pavement Technology Center, 2011). Examples of three state DOT specifications and special provisions for the use of corrosion-resistant dowels were cited earlier.

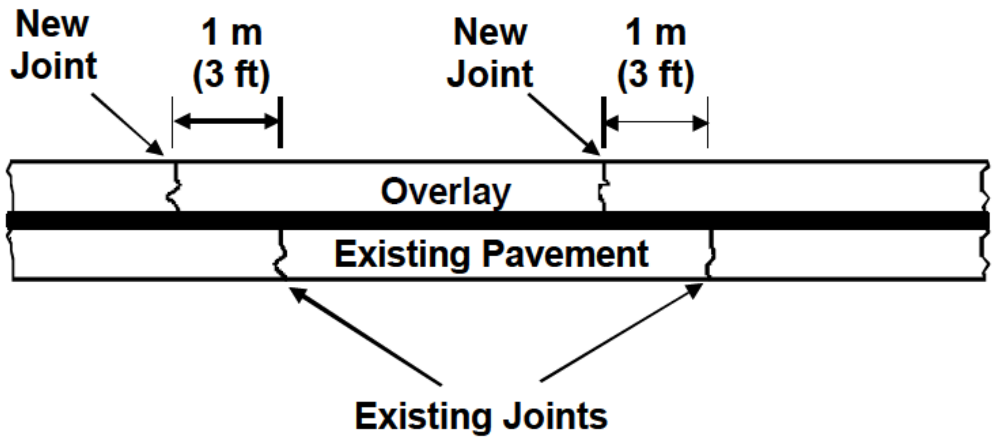


Figure 3.24. *Joint mismatching details.*
Source: Smith et al., 2002.

It is recommended that shorter joint spacings be used to reduce the risk of early cracking due to curling stresses. A maximum joint spacing of 15 ft is typically used for thick (>9 in.) long-lived concrete pavements. Figure 3.24 illustrates a typical joint mismatching detail, which should be considered for jointed concrete overlays. Prior recommendations suggest that the transverse joints should be sawed to a depth of T/4 (minimum) to T/3 (maximum) (Smith et al., 2002; Harrington, 2008).

Drainage Design

Drainage system quality significantly affects pavement performance. Overlay drainage design depends on the performance and capacity of the existing drainage system. Consequently, evaluation of the existing pavement is the first step in overlay drainage design. Depending on the outcome of this evaluation, no upgrade may be necessary. However, in the presence of distresses caused by moisture, appropriate design measures must be employed to address these issues. Distresses such as faulting, pumping, and corner breaks could be indicators of a poor drainage system. Standing water might be an indication of insufficient cross slope. Proper design, along with good construction and maintenance, will reduce these types of distresses. If asphalt interlayer drainage is inadequate in an unbonded PCC overlay, pore pressure induced by heavy traffic may cause HMA layer stripping, so careful consideration and design for interlayer drainage should be followed (Smith et al., 2002; Harrington, 2008).

Separator Layers

The separator layer is a critical factor in determining the performance of an unbonded concrete overlay. The separator layer acts as a lower-modulus buffer layer that assists in preventing cracks from reflecting up from the existing pavement to and through the new overlay. The separator layer does not contribute significantly to the structural enhancement and, therefore, the use of excessively thick (e.g., >2 in.) separator layers should be avoided (Smith et al., 2002; Harrington, 2008).

Interlayers should be between 1 and 2 in. thick (Smith et al., 2002; Harrington, 2008). Thin interlayers (e.g., 1 in.) have been used successfully when the existing pavement has little faulting or other surface distress. Thicker separator layers have been used when faulting and distress levels are high. The use of dense-graded and permeable HMA interlayers is common. Other materials used in unbonded overlay interlayers (either alone or in conjunction with HMA material) include polyethylene sheeting, liquid asphalts, geotextile fabrics, chip seals, slurry seals, and wax-based curing compounds. Not all of these materials and material combinations may be suitable for long-life pavements.

In Germany, a nonwoven fabric material is placed between the stabilized subbase and concrete slab to prevent bonding between layers and to provide a medium for subsurface drainage. This technology has been adapted for use in the United States for unbonded concrete overlay interlayers and was showcased on a 2008 unbonded concrete overlay project in Missouri (Tayabji et al., 2009). Figure 3.25 illustrates the placement of the fabric on the existing pavement surface. It is noted that no long-term performance data are currently available for the application of this technology in concrete overlays.

Figure 3.25. *Placement of nonwoven fabric as an interlayer.*
Source: Tayabji et al., 2009.

Table 3.11 summarizes the types of interlayers currently used in the construction of unbonded concrete overlays for concrete pavements. This information is based on extended meetings with pavement engineering and management professionals from the Illinois Tollway Authority and the Michigan, Minnesota, and Missouri DOTs.

As reported by Smith et al. (2002), the most commonly used separator layer is HMA (69%). Although other types of separator layers are also used, bituminous materials make up 91% of all separator layer types.

TABLE 3.11. EXAMPLE STATE OF PRACTICE REGARDING THE USE OF INTERLAYERS

State DOT	Interlayer Material
Illinois Tollway Authority	Used rich sand asphalt layer for one project.
Michigan	Experienced problems with thick sandy layers. Moved to using open-graded interlayer with a uniform thickness. The HMA separation layer is constructed in either a uniform 1-in. or 1- to 3-in. moderately wedged section. Geometric issues are corrected with the thickness of the PCC overlay.
Minnesota	Typically use an open-graded interlayer, but have also milled existing HMA to a 2-in. thickness and utilized it as an interlayer.
Missouri	Typically use a 1-in. HMA or geotextile interlayer.

Performance Considerations

The performance of unbonded concrete overlays of GPS-9 sections is presented in this section. The pavement performance criteria selected for the summary include transverse cracking, IRI (and PSI), and joint and crack faulting. The performance trends presented in this section are based on measurements documented in the latest year of monitoring available.

Transverse Cracking

Figure 3.26 shows typical transverse cracks for both airfield and highway pavements. Figure 3.27 shows the magnitude of the average number of transverse cracks per 500-ft-long section for the LTPP GPS-9 sections as a function of overlay thickness for jointed concrete pavements. As expected, the thicker overlays (>8 to 9 in.) exhibit fewer transverse cracks. It is noted that 11 of the 14 jointed concrete pavement overlays exhibited little or no cracking in 18 years of service. These test sections do exhibit the promise of long-life performance.

International Roughness Index (IRI)

Figure 3.28 illustrates the progression of IRI and PSI for the various GPS-9 sections and the impact of overlay thickness on ride quality.

Joint and Crack Faulting

Figure 3.29 illustrates transverse contraction joint faulting (faulting above 0.25 in. is significant); however, the data from GPS-9 projects do not show the degree of severity that is illustrated in Figure 3.30. The overall magnitude of the faulting is below 0.25 in. and therefore does not appear to be an issue; however, slab thicknesses greater than 9.6 in. show significantly less faulting, perhaps due to the use of dowel bars in these thicker pavements. The thinner overlays in the GPS-9 experiment were not doweled, so the trends are probably more due to the use of dowels rather than pavement

Figure 3.26. *Transverse cracking on an airport apron and an Interstate highway.*
Photos: Joe Mahoney.

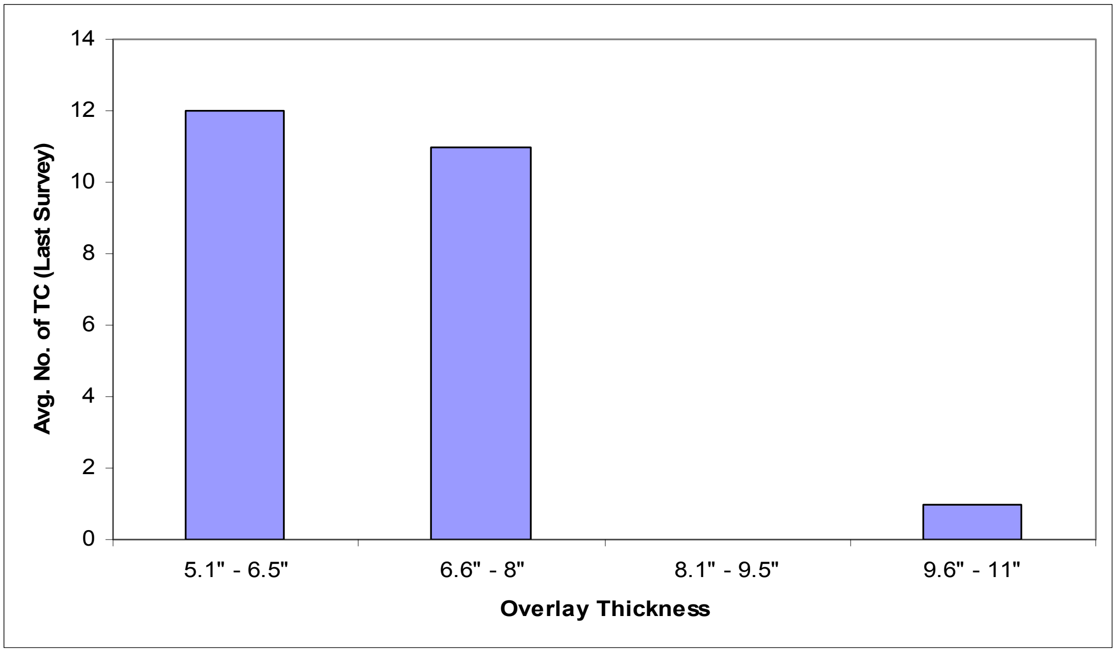


Figure 3.27. *JPCP overlay thickness versus average number of transverse cracks.*

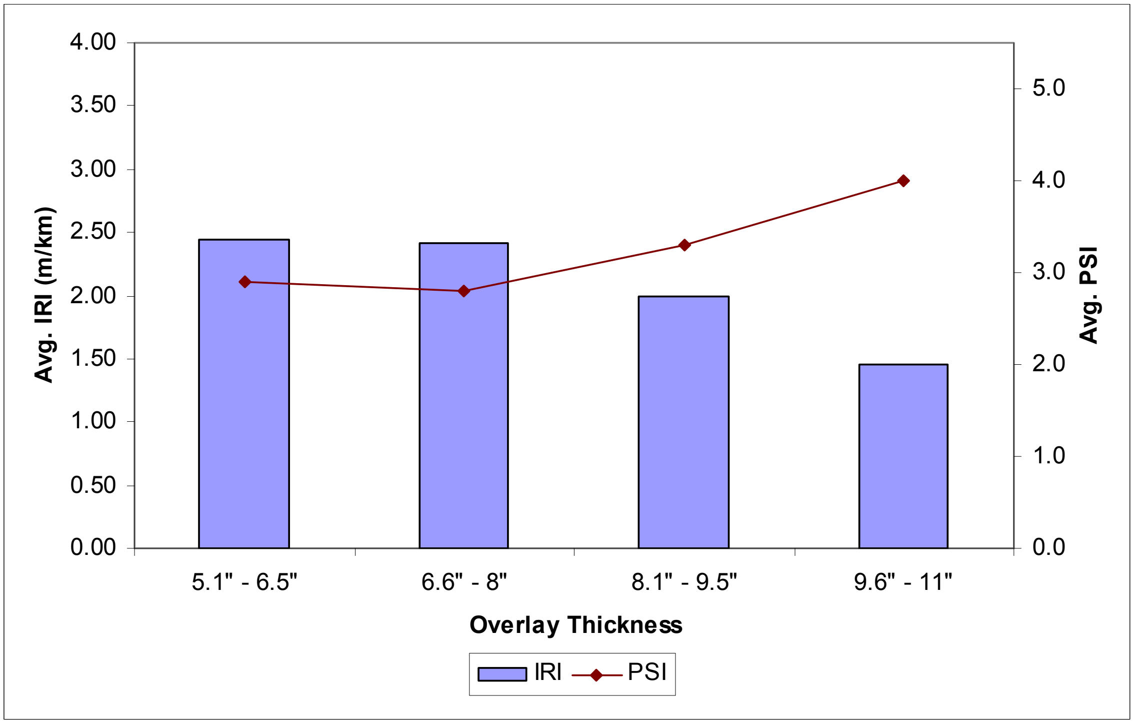


Figure 3.28. *Overlay thickness versus average IRI and average PSI (pavement age ranges from 6 to 20 years).*

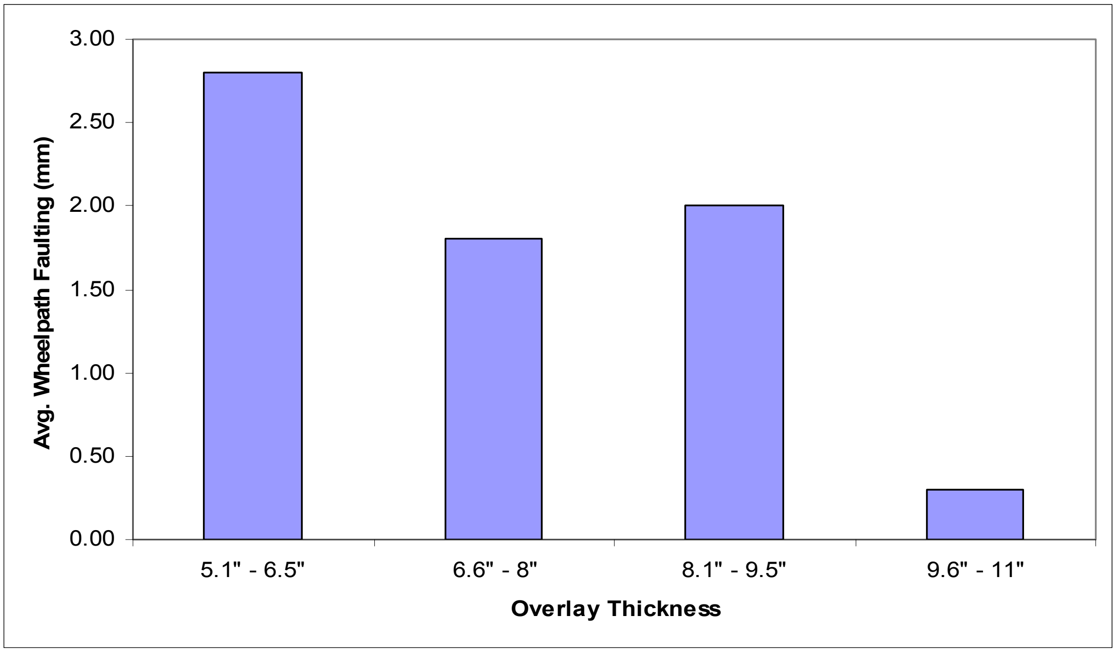


Figure 3.29. *Overlay thickness versus average wheelpath faulting.*

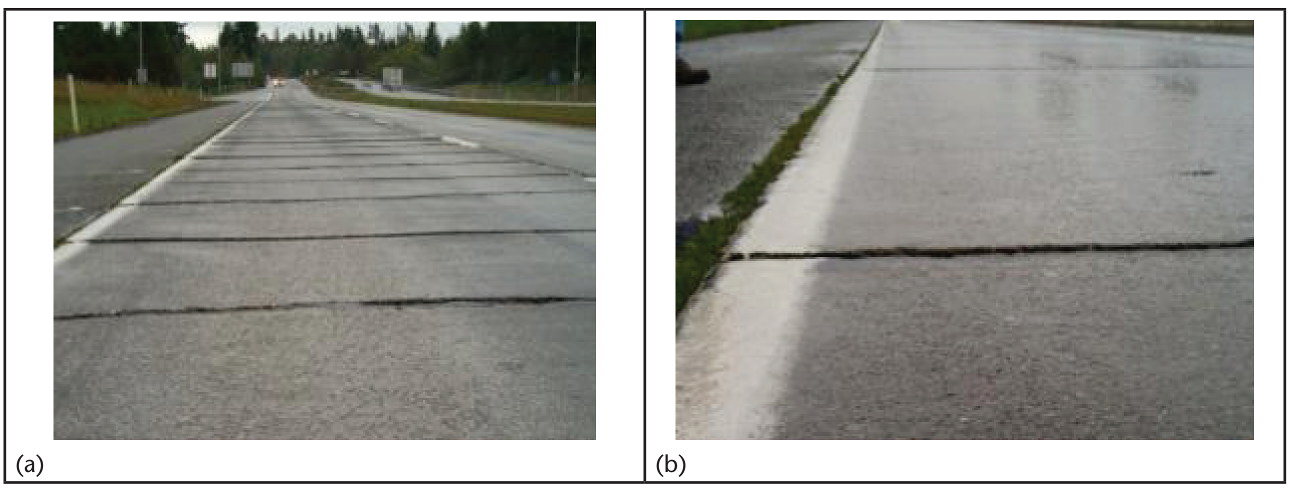


Figure 3.30. *Contraction joint faulting of JPCP. (a) Average fault ~0.25–0.5 in. (b) Average fault ~0.5 in.*
Photos: WSDOT.

thickness, but that may simply imply that the pavement needs to be thick enough to install dowels. The use of properly designed dowels in the transverse joints should essentially eliminate transverse joint faulting.

Impact of Interlayer Design on Performance

Figures 3.31 and 3.32 illustrate the impact of the interlayer type and thickness on transverse cracking of the overlay. In general, thicker interlayers tend to inhibit transverse cracking.

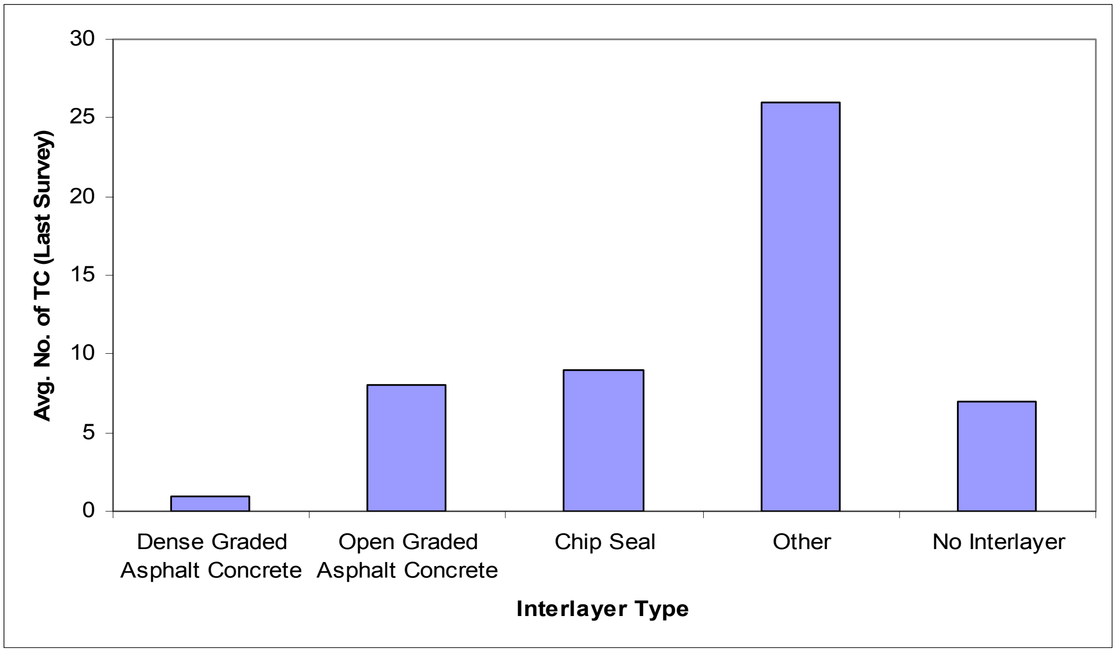


Figure 3.31. *JPCP interlayer type versus average number of transverse cracks.*

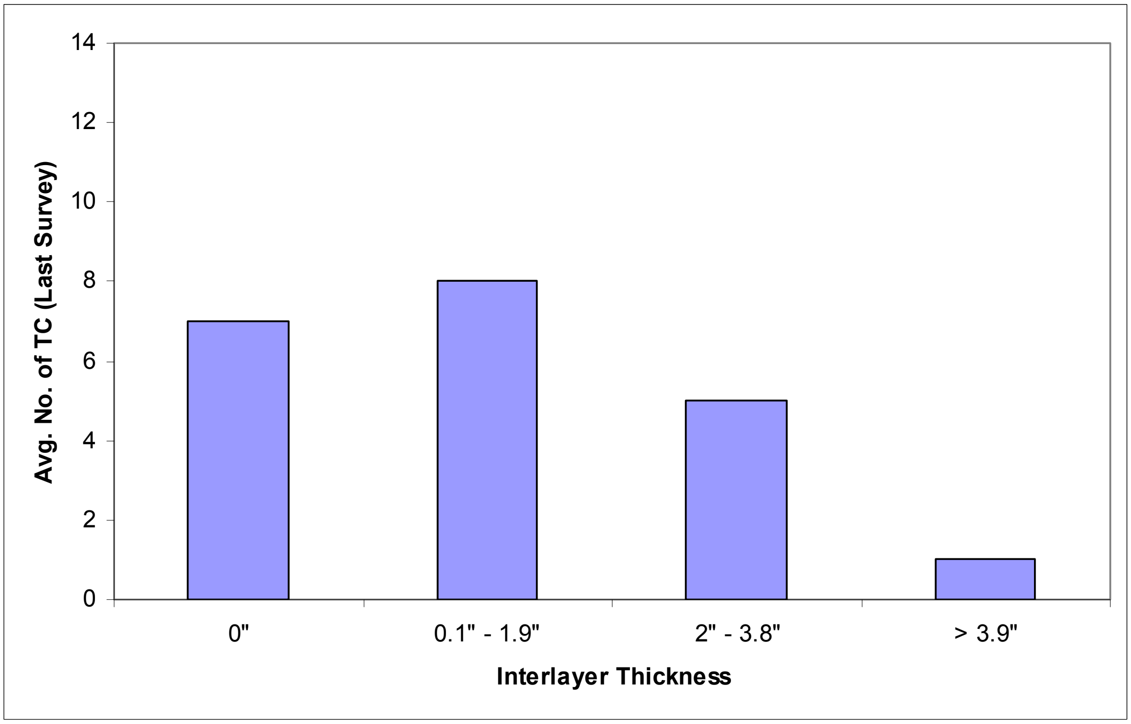


Figure 3.32. *JPCP interlayer thickness versus average number of transverse cracks.*

Figure 3.33 shows that thicker interlayers contribute to the integrity of the joint by controlling the amount of joint faulting (all other parameters being equal).

Construction Considerations

Construction of the Separator Layer

The placement of a separator layer is straightforward. The procedure depends on the interlayer material, but standard application procedures apply. The existing pavement surface needs to be swept clean of any loose materials. Either a mechanical sweeper or an air blower may be used (American Concrete Pavement Association, 1990; McGhee, 1994). With HMA separator layers, precautionary steps may be needed to prevent the development of excessively high surface temperatures prior to PCC placement. Surface watering should be used when the temperature of the asphalt separator layer is at or above 120°F to minimize the potential of early age shrinkage cracking (Harrington, 2008). There should be no standing water or moisture on the separator-layer surface at the time of overlay placement. An alternative to this is to construct the PCC overlay at night. Whitewashing of the bituminous surface using lime slurry may also be performed to cool the surface (American Concrete Pavement Association, 1990). However, this practice may lead to more complete debonding between the overlay PCC and the separator layer. Some degree of friction between the overlay PCC and the separator layer is believed to be beneficial to the performance of unbonded overlays, even if the

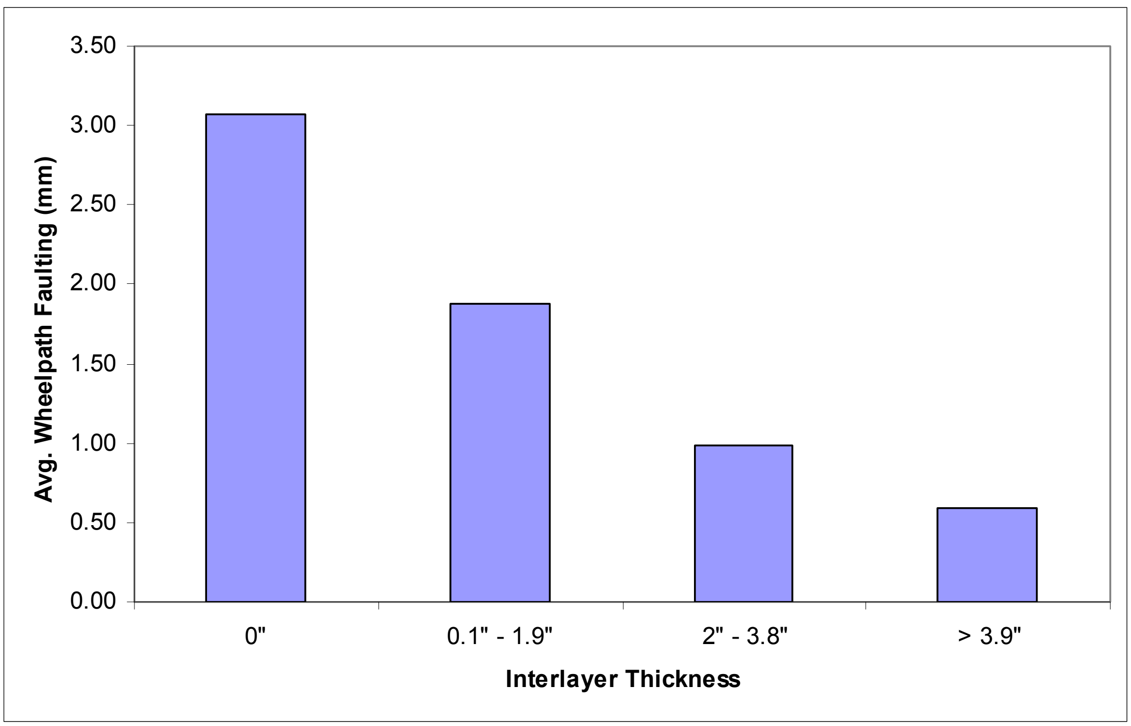


Figure 3.33. *JPCP interlayer thickness versus average wheelpath faulting.*

structural design is based on the assumption of no bond (ERES, 1999). The size of the project and geometric constraints will determine the type of paving (fixed form, slip form, or a combination) used (Smith et al., 2002).

Concrete Temperature During Construction

During construction, excessively high temperature and moisture gradients through the PCC must be avoided through the use of good curing practices (i.e., control of concrete temperature and moisture loss). Several studies have shown that excessive temperature and/or moisture gradients through the PCC slab at early ages (particularly during the first 72 hours after placement) can induce a significant amount of curling into PCC slabs, which can then result in higher slab stresses and premature slab cracking. This built-in construction curling is of particular concern for unbonded overlays because of the very stiff support conditions typically present.

Early age (less than 72 hours) characterization of the pavement should be performed to study the impact of PCC mixture characteristics and climatic conditions at the time of construction on the predicted overlay behavior and performance. An excellent tool for completing concrete pavement early age assessments is the HIPERPAV III software (High Performance Concrete Paving) (HIPERPAV, 2010). A screen shot from HIPERPAV is shown in Figure 3.34, which illustrates the predicted tensile stress and strength in the concrete over the first 72 hours following placement.

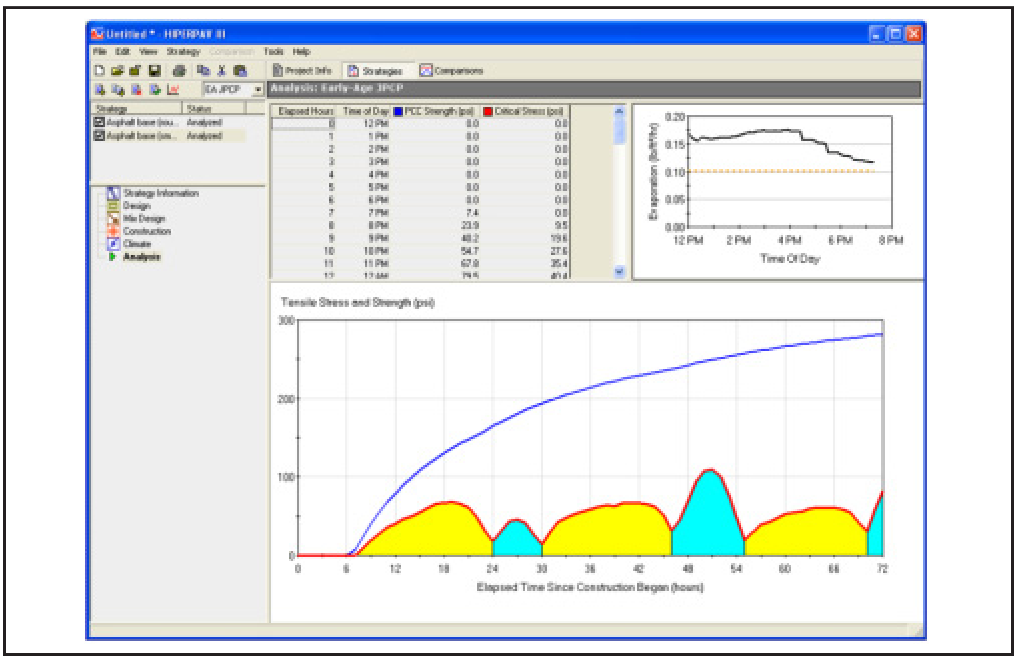


Figure 3.34. *Screen shot from HIPERPAV III software illustrating tensile stress and strength over first 72 hours.*
Source: HIPERPAV, 2010.

Surface Texture

For quieter pavements, the surface texture should be negative (i.e., grooves pointing downward, not fins) and oriented longitudinally. If the texture is placed in the transverse direction, then it should be closely spaced and randomized. Texture depth is also important for both friction and noise generation. A minimum depth is required for friction, but excessive depth of texture (particularly for transversely oriented textures) is associated with significantly greater noise generation, both inside and outside of the vehicle (American Concrete Pavement Association, 2006). It is believed that the use of siliceous sands tends to improve texture durability and friction. For diamond grinding, polish-resistant, hard and durable coarse aggregates are recommended. Narrow single-cut joints are recommended to minimize noise. Avoid faulted joints, protruding joint sealants, and spalled joints for quieter pavements (Rasmussen et al., 2008).

Dowel Placement

The use of dowel bars is critical for long-lasting JPCP. Numerous studies, including the AASHO Road Test, showed the need for doweled transverse contraction joints to survive heavy traffic conditions. A number of state DOTs during the initial construction of the Interstate system used undoweled JPCP and have now changed to doweled JPCP—largely due to faulting of the contraction joints. During construction, dowel misalignment can occur, particularly so with dowel bar inserters—although it can happen with dowel baskets as well. It is critical to avoid such misalignments, and technology developed over the past 10 years can help do so.

There are five possibilities for misalignment, as illustrated in Figure 3.35. These misalignments can cause various types of performance issues ranging from slab spalling to cracking, as shown in Table 3.12. Notably, the long-term load transfer at the contraction joints can also be affected. As shown in the table, horizontal skew and vertical tilts are likely the most critical misalignments.

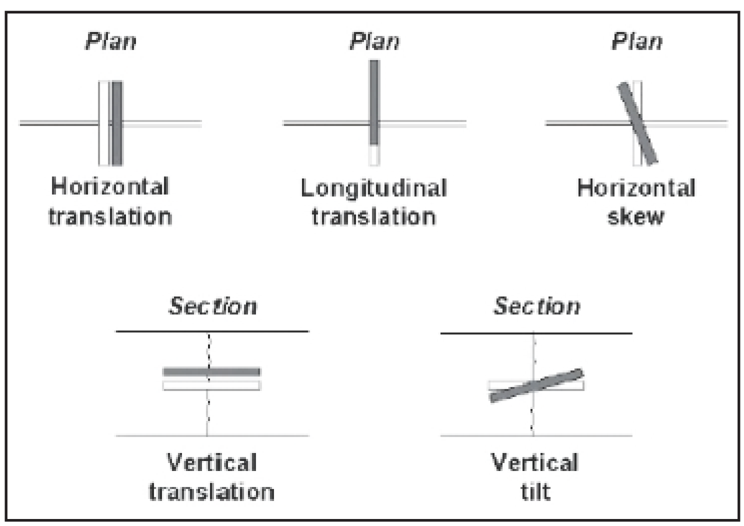


Figure 3.35. *Types of dowel bar misalignments.*
Source: Yu and Tajabji, 2007.

TABLE 3.12. DOWEL MISALIGNMENT AND EFFECTS ON PAVEMENT PERFORMANCE

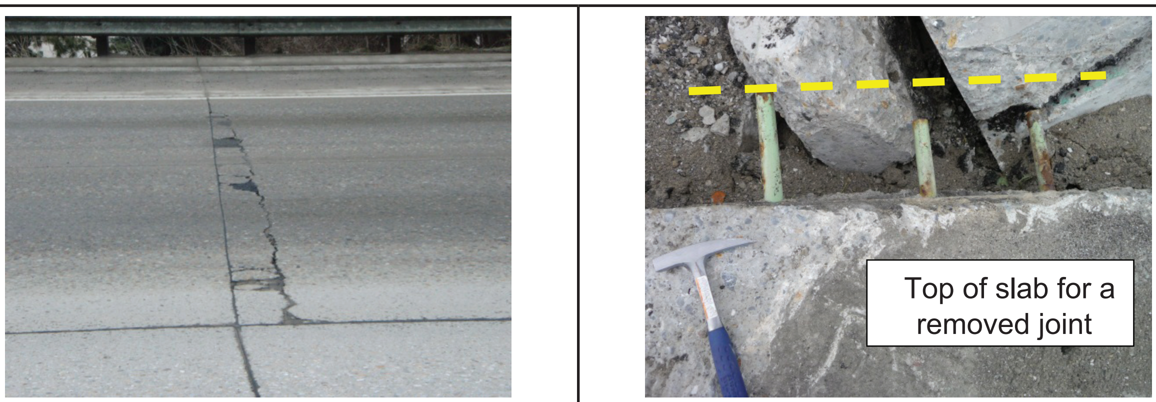
Type of Misalignment	Effect on Spalling	Slab Cracking	Load Transfer
Horizontal translation	No	No	Yes
Longitudinal translation	No	No	Yes
Vertical translation	Yes	No	Yes
Horizontal skew	Yes	Yes	Yes
Vertical tilt	Yes	Yes	Yes

Source: Federal Highway Administration, 2006.

An illustration of a failed contraction joint due to dowel misalignment is shown in Figure 3.36. Additionally, an example of dowel "longitudinal translation" is also shown.

A critical step for minimizing misalignment is to measure the postconstruction location of the dowel bars. There are multiple ways this can be done, but an instrument available from Magnetic Imaging Tools (MIT) is explored here. The device, MIT Scan-2, has been assessed and described by FHWA studies (Yu and Khazanovich, 2005; Yu, 2005) and applied on numerous paving projects. The nondestructive instrument uses magnetic tomography to locate metal objects (steel dowels for this application). This process is, in essence, an imaging technique that induces currents in steel dowels, and these currents provide the needed location information. A MIT Scan-2 device is shown in operation in Figure 3.37.

The MIT Scan-2 has daily productivity rates of about 250 doweled joints for a single lane and can be used with freshly placed or hardened concrete. The FHWA, through its Concrete Pavement Technology Program (CPTP), has three of these units available to the states for loan or on-site demonstration (as of April 2011).



(a) (b)

Figure 3.36. *Dowel misalignment from an Interstate pavement. (a) Failed contraction joint due to dowel misalignment. (b) Example of dowel longitudinal translation (joint is not the same as the one in the accompanying photo).*
Photos: Kevin Littleton and Joe Mahoney.

Figure 3.37. *MIT Scan-2.*
Source: Yu and Khazanovich, 2005.

Various studies have been performed to examine the issue of what are allowable dowel misalignments. A best practices document is available from FHWA (Yu and Tayabji, 2007).

Example Designs

Table 3.13 summarizes a selection of unbonded concrete overlays of concrete pavements constructed in the United States since 1993. The information presented in the table was compiled from National Concrete Overlay Explorer [a database provided by the American Concrete Pavement Association (2010)]. The website currently contains only a representative sampling of projects across the United States, and so the number of concrete overlay projects viewable online is expected to increase over time.

The common features for these unbonded concrete overlays in Table 3.13 include the following:

- Slab thickness ranges from 9 to 12 in.;
- Doweled joints are spaced mostly at 15 ft;
- HMA interlayers range in thickness from 1 to 3 in. with most dense-graded but some open-graded mixes; and
- Existing pavements were either jointed or CRCP.

TABLE 3.13. A SELECTION OF UNBONDED CONCRETE OVERLAYS CONSTRUCTED IN THE UNITED STATES SINCE 1993

Project Location and Details	Year of Overlay Construction	Design Details of Overlay
I-77, Yadkin, South of Elkin, North Carolina. The existing pavement is CRCP and 30 years old.	2008	• Slab thickness is 11 in. • Doweled joints spaced at 15 ft • Asphalt 1.5-in. interlayer
I-86, Olean, New York. The existing pavement is JRCP and 30 years old.	2006	• Slab thickness is 9 in. • Doweled joints spaced at 15 ft • Asphalt 3-in. interlayer • 30% truck traffic
I-35, Noble/Kay County, Oklahoma. The existing pavement is JRCP and 42 years old.	2005	• Slab thickness is 11.5 in. • Doweled joints spaced at 15 ft • Asphalt 2-in. interlayer • 25% truck traffic
I-40, El Reno, Oklahoma. The existing pavement is JPCP and 35 years old.	2004	• Slab thickness is 11.5 in. • Doweled joints spaced at 15 ft • Asphalt 2-in. interlayer
I-264, Louisville, Kentucky. The existing pavement is JRCP and 36 years old.	2004	• Slab thickness is 9 in. • Doweled joints spaced at 15 ft • Drainable asphalt 1-in. interlayer
I-40, El Reno, Oklahoma (MP 119 and east). Existing pavement is JPCP and 34 years old.	2003	• Slab thickness is 10 in. • Doweled joints • Asphalt 2-in. interlayer
I-85 (southbound), near Anderson, South Carolina. Existing pavement is JPCP and 38 years old.	2002	• Slab thickness is 12 in. • Doweled joints • Asphalt 2-in. interlayer • 35% truck traffic • The northbound lanes have been rubblized and overlaid. Performance comparison is recommended.
I-275, Circle Freeway, Kentucky. Existing pavement is JPCP and 28 years old.	2002	• Slab thickness is 9 in. • Doweled joints spaced at 15 ft • Drainable asphalt 1-in. interlayer
I-65, Jasper County, Indiana. Existing pavement is JRCP and 25 years old.	1993	• Slab thickness is 10.5 in. • Doweled joints spaced at 20 ft • Asphalt 1.5-in. interlayer • 23% truck traffic
I-40, Jackson, Tennessee. Existing pavement is JPCP.	1997	• Slab thickness is 9 in. • Doweled joints spaced at 15 ft • Asphalt 1-in. interlayer
I-85, Granville, North Carolina. Existing pavement is CRCP and 25 years old.	1998	• Slab thickness is 10 in. • Doweled joints spaced at 18 ft • Permeable asphalt 2-in. interlayer • 25% truck traffic

continued

TABLE 3.13. A SELECTION OF UNBONDED CONCRETE OVERLAYS CONSTRUCTED IN THE UNITED STATES SINCE 1993 (*continued*)

Project Location and Details	Year of Overlay Construction	Design Details of Overlay
I-265 at I-71, Jefferson County, Kentucky. Existing pavement is JRCP and was constructed in 1970.	1999	• Slab thickness is 9 in. • Doweled joints spaced at 15 ft • Drainable asphalt 1.3-in. interlayer
I-85, Newman, Georgia. Existing pavement is JPCP and 38 years old.	2009	• Slab thickness is 11 in. • CRCP overlay • Asphalt 3-in. interlayer

Source: American Concrete Pavement Association, 2010.

Summary for Unbonded Concrete Overlays of Concrete Pavements

Based on the review of the best practices and performance of pavement sections in the LTPP database and related data in these best practices, the design recommendations for long-lived unbonded concrete overlays are summarized in Table 3.14.

A selection of significant practices and specifications associated with paving unbonded concrete overlays over existing concrete were selected and included in Table 3.15. The table includes a brief explanation of why the issue is of special interest, along with examples from the recommendations in the Guide Specifications (Chapter 4). Three major practices are featured: (1) existing pavement and preoverlay repairs, (2) overlay thickness and joint details, and (3) interlayer requirements.

TABLE 3.14. RECOMMENDED DESIGN ATTRIBUTES FOR LONG-LIFE CONCRETE PAVEMENT (LLCP)

Design Attribute	Recommended Range
Slab thickness	Minimum thickness of 9 in.
Interlayer thickness	≥1 in.; 2 in. is likely optimal
Joint spacing	Maximum spacing of 15 ft
Load-transfer device	Mechanical load-transfer device, corrosion-resistant dowels to promote long-life Dowel lengths of 18 in.
Dowel diameter	1.5 in. (function of slab thickness)

TABLE 3.15. SUMMARY OF BEST PRACTICES AND SPECIFICATIONS FOR UNBONDED CONCRETE OVERLAYS OVER EXISTING CONCRETE

<table>
<tr><th>Best Practice</th><th>Why This Practice?</th><th>Typical Specification Requirements</th></tr>
<tr><td>Existing pavement and preoverlay repairs[a]</td><td>The preparation of the existing pavement is important for achieving long life from the unbonded concrete overlay.</td><td><table>
<tr><th>Existing Pavement Condition</th><th>Possible Repairs</th></tr>
<tr><td>Faulting ≤10 mm</td><td>No repairs needed</td></tr>
<tr><td>Faulting >10 mm</td><td>Use a thicker interlayer</td></tr>
<tr><td>Significant tenting, shattered slabs, pumping</td><td>Full-depth repairs</td></tr>
<tr><td>Severe joint spalling</td><td>Clean the joints</td></tr>
<tr><td>CRCP with punchouts</td><td>Full-depth repairs</td></tr>
</table></td></tr>
<tr><td>Overlay thickness and joint details[b]</td><td>Thickness and joint details are critical for long-life performance.</td><td>• Overlay thickness ≥9 in.
• Transverse joint spacing not to exceed 15 ft when slab thicknesses are in excess of 9 in.
• Joints should be doweled; dowel diameter should be a function of slab thickness. The recommended dowel bar sizes are:
— For ≥9 in.: 1.50-in.-diameter minimum
• Dowels should be corrosion resistant</td></tr>
<tr><td>Interlayer between overlay and existing pavement[b]</td><td>Interlayer thickness and conditions prior to placing the concrete overlay influence long-life performance and early temperature stress in the new slabs.</td><td>• The interlayer material shall be a minimum of 1-in.-thick new bituminous material.
• The surface temperature of HMA interlayer shall be <90°F before overlay placement.</td></tr>
<tr><td>Concrete overlay materials[b]</td><td></td><td>• Supplementary cementitious materials may be used to replace a maximum of 40%–50% of the portland cement.</td></tr>
</table>

[a] For additional details, see Elements for AASHTO Specifications 552, 557, and 558 in Chapter 4.
[b] For additional details, see Elements for AASHTO Specification 563 in Chapter 4.

UNBONDED CONCRETE OVERLAYS OF HOT-MIX ASPHALT CONCRETE PAVEMENTS

Criteria for Long-Life Potential

Unbonded concrete overlays of HMA concrete pavements are a viable long-lived renewal strategy. In general, this strategy is applied when the existing HMA pavements exhibit significant deterioration in the form of rutting, fatigue cracking, potholes, foundation issues, and pumping; however, the stability and the uniformity of the existing pavement are important for both renewal construction and long-life performance of the unbonded concrete overlay. Figure 3.38 is a sketch of an unbonded overlay over preexisting flexible pavement.

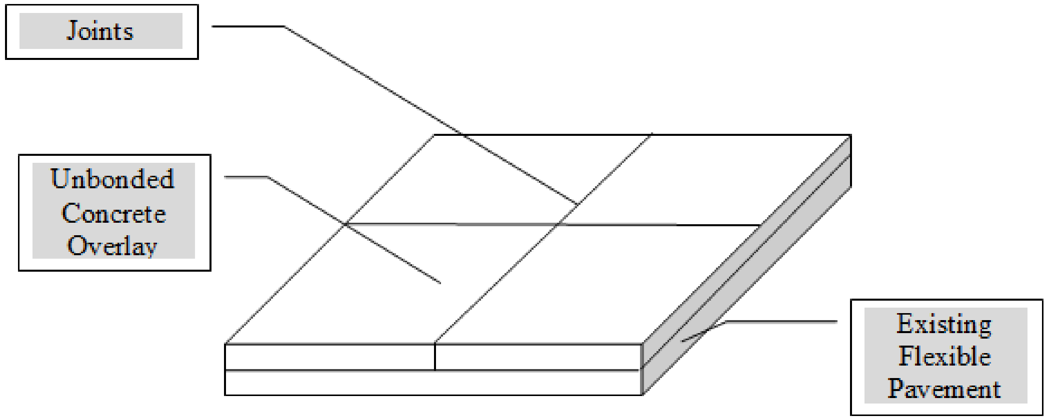


Figure 3.38. *Unbonded concrete overlay of flexible pavement.*
Illustration: Joe Mahoney.

The placement of the overlay can potentially do the following (Smith et al., 2002; Harrington, 2008):

- Restore and/or enhance structural capacity of the pavement structure;
- Increase life equivalent to that of a full-depth pavement; and
- Restore and/or improve friction, noise, and rideability.

General Design Considerations

The structural condition of the existing pavement can be established by conducting visual distress surveys and deflection testing using an FWD. The deflection information can be used to back-calculate the resilient moduli of various pavement layers (although HMA layers less than 3 in. thick are difficult to back-calculate).

Preoverlay Repairs

The preoverlay requirements are minimal at best. Table 3.16 summarizes the possible preoverlay repairs needed in preparation for the PCC unbonded concrete overlay of asphalt pavements (Harrington, 2008).

TABLE 3.16. SUGGESTED PREOVERLAY REPAIRS

Existing Pavement Condition	Possible Repairs
Potholes	Fill with asphalt concrete
Shoving	Mill
Rutting ≥2 in.	Mill
Rutting <2 in.	None or mill
Crack width ≥4 in.	Fill with asphalt

Source: Harrington, 2008.

Structural Design

The design of an unbonded concrete overlay of HMA pavement considers the existing pavement as a stable and uniform base, and the overlay thickness is designed similarly to a new concrete pavement. Furthermore, the design assumes an unbonded condition between the existing asphalt layer and the new concrete overlay. The existing asphalt thickness should be at least 4 in. of competent material to ensure an adequate load-carrying base for the concrete overlay (Smith et al., 2002; Harrington, 2008). The 1993 AASHTO design method does not consider the effects of bonding between the new overlay and the existing HMA pavement. The design method considers the composite *k* at the top of the HMA layer. Field studies have shown that there is some degree of bonding between the two layers. However, the longevity and the uniformity of this bond over the design life of the structure is not well documented. In the MEPDG design procedure, the bonding between the two layers is modeled by selecting appropriate friction factors.

In general (as documented in the literature), the unbonded overlay thickness usually ranges from 4 to 11 in.; however, to ensure long-life performance the slab thicknesses of the overlay should range from 9 to 13 in. The joint design, slab length, and joint width details are similar to unbonded concrete overlays of concrete pavements.

Performance Considerations

In general, the field performance of unbonded concrete overlays of HMA pavements has been satisfactory. The success of the renewal strategy hinges on the uniform underlying support. The underlying HMA base eliminates most of the pumping of fines so there is little to no faulting and very uniform support. The general performance of PCC over HMA has been very good.

Example Designs

Table 3.17 summarizes unbonded concrete overlays of concrete pavements constructed in the United States since 1995. The information presented in the table was compiled from the National Concrete Overlay Explorer (American Concrete Pavement Association, 2010). The website currently contains only a representative sampling of projects across the United States, and so the number of concrete overlay projects viewable online is expected to increase over time.

The common features for these unbonded concrete overlays in Table 3.17 include the following:

- Slab thicknesses range from 9 to 12 in., and
- Doweled joints are spaced mostly at 15 ft.

TABLE 3.17. OVERVIEW OF SELECTED UNBONDED CONCRETE OVERLAYS OF FLEXIBLE PAVEMENTS CONSTRUCTED IN THE UNITED STATES SINCE 1995

Project Location and Details	Year of Overlay Construction	Design Details of Overlay
Cherry Street, North to H-17, Iowa.	2004	• Slab thickness is 9 in. • Doweled joints spaced at 15 ft
Tiger Mountain, Oklahoma. Existing pavement was 9 years old.	2004	• Slab thickness is 10.5 in. • Doweled joints spaced at 15 ft • 30% truck traffic
US-412, Bakerville, Missouri. Existing pavement is 30 years old.	2004	• Slab thickness is 12 in. • Doweled joints spaced at 15 ft • 24% truck traffic
US-412, Bakerville, Missouri.	2003	• Slab thickness is 12 in. • Doweled joints spaced at 15 ft • 24% truck traffic
I-55, Vaiden, Mississippi.	2001	• Slab thickness is 10 in. • Doweled joints spaced at 16 ft
E-33, Iowa.	1998	• Slab thickness is 9 in. • Doweled joints spaced at 15 ft
P-33, Iowa.	1998	• Slab thickness is 10 in. • Doweled joints spaced at 15 ft
I-10/1-12, Louisiana.	1995	• Slab thickness is 12 in.

Source: Data from American Concrete Pavement Association, 2010.

ADDED LANES AND TRANSITIONS FOR ADJACENT STRUCTURES FOR UNBONDED PCC OVERLAYS OVER EXISTING CONCRETE AND HMA PAVEMENTS

There is little guidance found in the literature on integrating new or rehabilitated pavements into adjacent pavements and features. This document addresses adding lanes to an existing pavement structure, as well as accommodating existing features such as bridge abutments and vertical clearance restrictions within the limits of a pavement renewal project. These issues are paramount when using the existing pavement in place as part of long-life renewal, because there is typically a significant elevation change associated with each renewal alternative. The following recommendations are based on discussions with the state highway agencies surveyed in Phase 1 and those agencies that participated in Phase 2.

Bridge and Overcrossing Structure Approaches

In the transition where the unbonded PCC overlay connects to a bridge approach, or when the roadway section with an unbonded overlay passes under an existing structure, the new grade line and reduced vertical clearances usually require the construction of a new pavement section. The length of the new section depends on the elevation difference, but it is usually in the range of 300 to 500 ft before and after the structure. A typical taper rate used by a number of agencies visited is 400 to 1 to transition from the new grade line to the elevation required by the adjacent feature. Attention should be paid to the longitudinal drainage as well as to the transverse drainage when designing the new pavement section. Where possible, the existing subgrade elevation and grade should be maintained in the longitudinal direction as well as the transverse direction.

Because the new roadway section will not be as thick as the renewal approach using the existing pavement, the difference in elevation is usually made up with HMA or a combination of HMA and untreated granular base material. Because the unbonded PCC overlay requires reasonably uniform support, the transition from the old PCC pavement to the new pavement should be made as stiff as possible, which may require replacement of the PCC with full-depth HMA. Subgrade stabilization should also be considered if needed in the transition area. Specifically, the SHRP 2 Renewal Project R02 guidance for “Geotechnical Solutions for Transportation Infrastructure” and its recommendations for stabilization of the pavement working platform should be considered. Diagrams of possible transition profiles are shown in Figures 3.39 and 3.40.

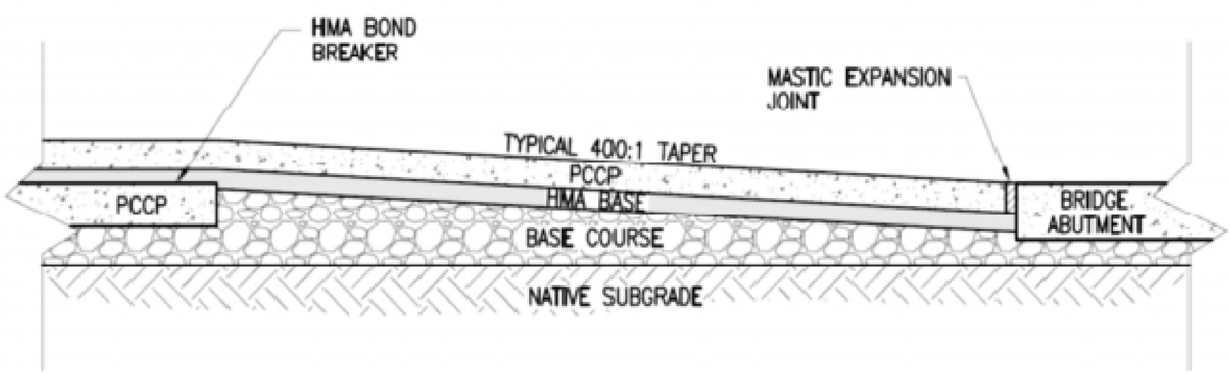


Figure 3.39. *Diagram of transition to bridge approach (unbonded PCC overlay of PCC pavement).*

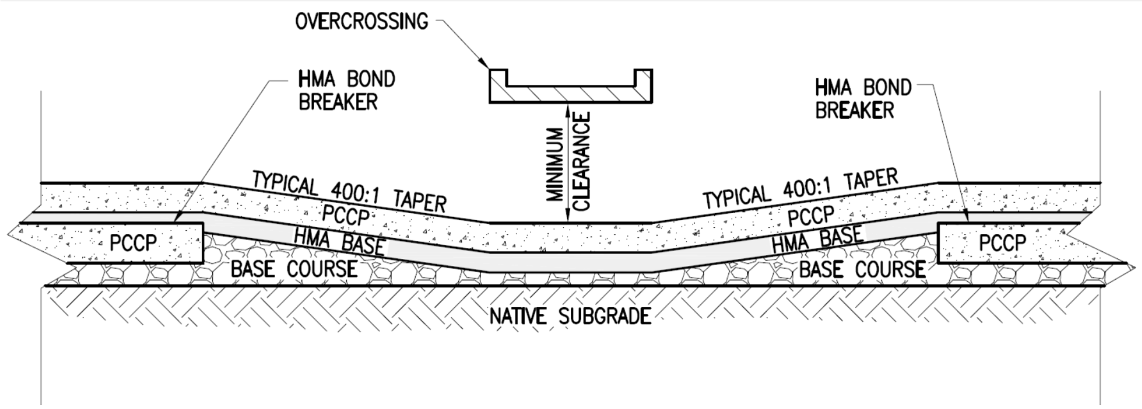


Figure 3.40. *Diagram of transition beneath structure.*

In some cases, agencies reported they were able to raise an overcrossing rather than reconstruct the roadway for less cost and reduced impact on traffic. That option may be considered where possible, particularly in more rural areas where there is little cross traffic on the overcrossing.

Added Lanes or Widening

When a project calls for additional lanes or widening, the addition of lanes often facilitates the staging of the traffic through the project, but it usually produces a mismatch in pavement sections in the transverse direction. The slope and grade line of the subgrade should be maintained so that water flowing along the contact between the base and the subgrade does not get trapped in the transverse direction. There is a risk that there may be reflection cracking between the existing pavement and the new pavement section, particularly when the existing pavement is a PCC. Also of concern is the need for stabilizing the subgrade soil, if required for widening. Subgrade stabilization will increase the stability of the roadway section, accelerate pavement construction, and help reduce some of the settlement or differential vertical deflection that causes reflection cracking along the contact with the old PCC pavement. Specifically, the SHRP 2 Renewal Project R02 guidance for “Geotechnical Solutions for Transportation Infrastructure” and its recommendations for stabilization of the pavement working platform should be considered.

Lane Widening

A number of agencies have reported they have constructed a 14-ft widened lane in the outside lane to provide improved edge support. One agency reported cracking along the edge of the old PCC pavement caused by nonuniform support at that location. They had not improved the shoulder section prior to construction of the unbonded PCC overlay. If lane widening is considered, the existing shoulder section may need to be reconstructed to provide more uniform support for the new PCC pavement.

Added Lanes

When a project calls for additional lanes or widening, the addition of lanes often facilitates the staging of the traffic through the project, but it usually produces a mismatch in pavement sections in the transverse direction. The slope and grade line of the subgrade should be maintained so that water flowing along the contact between the base and the subgrade does not get trapped in the transverse direction. Similar to widened lanes, there is a need for uniform support under the PCC overlay; thus, the shoulder will need to be reconstructed and the subgrade should be stabilized where needed.

No specific guidance could be found to provide uniform support in the widening next to the existing PCC pavement. A number of agencies have widened with HMA as part of the traffic staging and then placed the unbonded PCC pavement across both the existing PCC pavement with a HMA bond breaker, and the widened HMA pavement. Some agencies have widened the existing PCC pavement with PCC pavement, then placed the HMA bond breaker across both the old and the new PCC pavement before placing the PCC overlay. This approach provides uniform support for the PCC overlay; however, there was no indication that there was any difference in performance when the widening was constructed with PCC pavement or HMA pavement as a base for the PCC overlay. Use of HMA to widen the existing pavement does provide some advantage in traffic staging. Typical pavement sections are shown in Figures 3.41 and 3.42. The minimum thickness of the HMA in the widening is usually controlled by the traffic loading during staging, but it is usually a minimum of 6 in. thick to minimize failure risk during staging and provide more uniform support for the PCC overlay.

For unbonded PCC overlays of flexible pavement, the existing pavement is simply widened with HMA to provide the base for the PCC overlay. The pavement section should extend the subgrade line and slope out to either the contact with the in-slope of the ditch or fill slope, or to a collection point for longitudinal drains as shown in Figures 3.42 and 3.43.

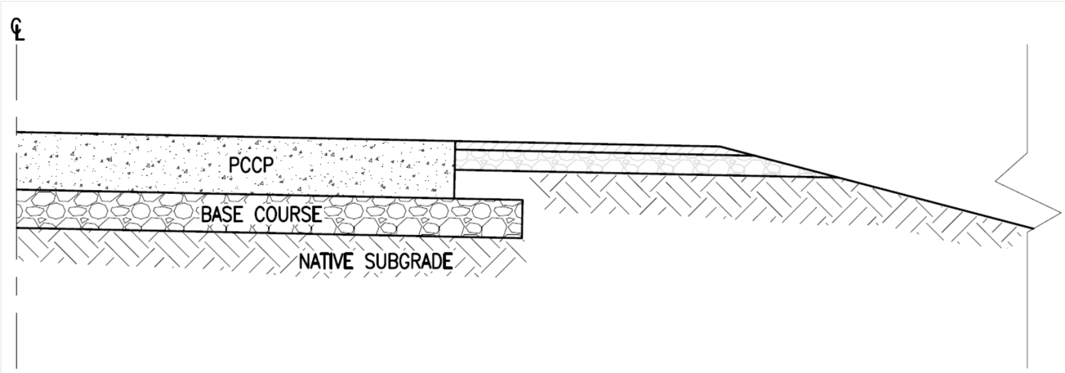


Figure 3.41. *Cross section showing existing PCC pavement without daylighted shoulders.*

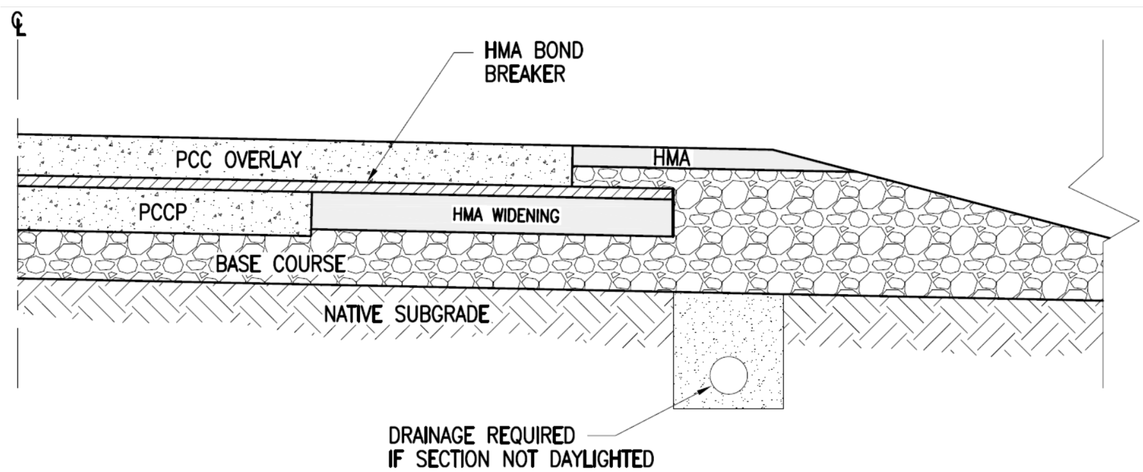


Figure 3.42. *Cross section showing widening of the shoulder with daylighting or drainage.*

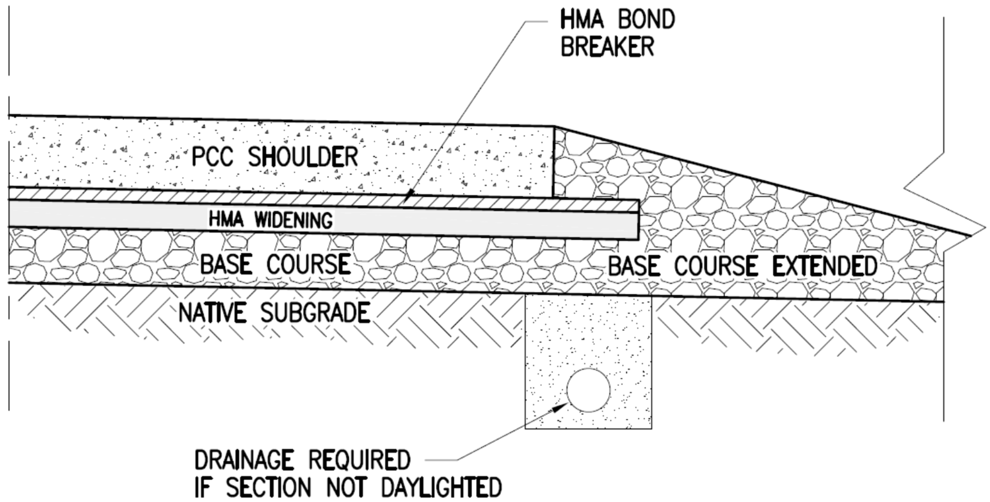


Figure 3.43. *Cross-section detail with PCC shoulder.*

BEST PRACTICES SUMMARY

The definition of long-life renewal strategies is a design life of 50 years or more. To achieve this, unbonded concrete overlays of existing pavements are recommended. This recommendation is based on several sets of information, including but not limited to (1) state DOT criteria, (2) LTPP findings, and (3) information from the National Concrete Pavement Technology Center.

To achieve a 50-year life, several practices are critical, and these include the selection of materials, knowledge of local pavement distress and its causes, structural design, and relevant construction practices. Two broad types of unbonded concrete were discussed: (1) unbonded concrete over existing concrete pavement and (2) unbonded concrete over existing HMA pavement. Concrete overlays can be either JPCP or CRCP—both perform well.

Table 3.15 is a summary of relevant best practices and related specification requirements for unbonded concrete overlays. Three major practices are featured: (1) existing pavement and preoverlay repairs, (2) overlay thickness and joint details, and (3) interlayer requirements.

The major findings are recapped in Table 3.18.

TABLE 3.18. SUMMARY OF RECOMMENDED PRACTICES FOR UNBONDED PCC OVERLAYS

Factor or Consideration	Practice
Concrete overlay thickness	≥9 in.
Type of concrete overlay	Unbonded JPCP or CRCP
Structural design	Do a complete structural design using an agency-approved method.
JPCP joint spacing	≤15 ft
JPCP load transfer	Use 1.5-in.-diameter dowel bars.
Type of dowel bar	Use corrosion-resistant dowels.
Aggregates	Use local state DOT specifications with special attention paid to eliminating the potential for ASR and D-cracking.
Cements	SCM is acceptable and may be superior to traditional portland cements; use state guidelines for max limits.
Existing pavement	Use criteria provided for preoverlay repairs.
Concrete overlay interlayer	Use an HMA interlayer 1 (minimum) to 2 in. thick.
Concrete overlay construction	Control mix and substrate temperatures during construction; tools such as HIPERPAV will help planning and execution.

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Texas Department of Transportation. Chapter 8, Section 5: Determining Concrete Pavement Thickness, “Pavement Design Guide,” Texas Department of Transportation, Austin, 2011.

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Wisconsin Department of Transportation. “HPC Pavement Requirements for Concrete Pavement 12-inch,” Special Provisions, Wisconsin Department of Transportation, Madison, 2009.

Yu, H. T. “Use of Magnetic Tomography Technology to Evaluate Dowel Bar Placement,” Report FHWA-IF-06-002, Federal Highway Administration, 2005.

Yu, H. T., and L. Khazanovich. “Use of Magnetic Tomography Technology to Evaluate Dowel Placement,” Report FHWA-IF-06-006, Federal Highway Administration, 2005.

Yu, H. T., and S. Tayabji. “Best Practices for Dowel Placement Tolerances,” Report FHWA-HIF-07-021, Federal Highway Administration, Washington, D.C., 2007.

4 GUIDE SPECIFICATIONS

INTRODUCTION

The guide specifications developed by the SHRP 2 R23 team are presented in this chapter. They are organized into three sections: (1) guide specifications for pavement components that are not contained within the AASHTO Guide Specifications, (2) elements that can be added to or otherwise modify existing AASHTO Guide Specifications, and (3) summaries for relevant state department of transportation (DOT) and AASHTO specifications that were used to produce the elements in Item 2.

The study team used AASHTO Guide Specifications as a starting point, in part, because there are a wide variety of pavement-oriented specifications developed and maintained by AASHTO committees. Furthermore, AASHTO Guide Specifications reflect national practice, which is a necessary part of this study. The approach was to review existing state DOT and AASHTO Guide Specifications, select sensible components (or elements), and place those in lists (see “Elements for AASHTO Guide Specifications”).

There were four guide specifications not contained in the AASHTO Guide Specifications that were deemed necessary for this study. These are Stone Matrix Asphalt (SMA); Open-Graded Friction Course; Rubblization of Existing Concrete Pavement; and Saw, Crack, and Seat Concrete Pavement. Guide specifications were prepared and are contained in this document (see “Specifications Not in the AASHTO Guide Specifications”).

SPECIFICATIONS NOT IN THE AASHTO GUIDE SPECIFICATIONS

SHRP 2 R23 Guide Specification: Stone Matrix Asphalt (SMA)

<table>
<tr><th>Paragraph</th><th>Content</th></tr>
<tr><td>Description</td><td>The work covered by this specification shall consist of constructing a hot-mix asphalt layer of fiber-stabilized stone matrix asphalt pavement on a prepared surface in accordance with these specifications and in conformity with the lines, grades, and typical cross section.</td></tr>
<tr><td>Materials</td><td>
1. Coarse aggregates
a. Coarse aggregate: Coarse aggregate shall be aggregate retained on the No. 4 sieve. Virgin aggregate shall be 100% crushed material.
b. Coarse aggregate flat and elongated particles: The maximum amount of flat and elongated particles in coarse aggregate for SMA is shown in the following table:
<table>
<tr><th>Test Method and Description</th><th>Flat and Elongated Particles in Coarse Aggregate (%)</th></tr>
<tr><td>Flat and Elongated % by Count 3:1 (max to min) (ASTM D4791, Section 8.4)</td><td>20%</td></tr>
<tr><td>Flat and Elongated % by Count 5:1 (max to min) (ASTM D4791, Section 8.4)</td><td>5%</td></tr>
</table>
c. Coarse aggregate soundness for SMA: The percent degradation of the source aggregate by the sodium sulfate soundness test (AASHTO T104) after five cycles of testing shall not exceed 10%.
d. Deleterious materials and absorption in coarse aggregate: The amount of deleterious substances and absorption in the coarse aggregate shall not exceed the limits in the following table:
<table>
<tr><th>Test Method and Description</th><th>Percent</th></tr>
<tr><td>Clay Lump and Friable Particles (AASHTO T112)</td><td>0.25%</td></tr>
<tr><td>Absorption (applied to the material passing the 0.75-in. sieve and retained on the No. 4 sieve) (AASHTO T85)</td><td>2.0%</td></tr>
</table>
e. Los Angeles (LA) abrasion criteria for coarse aggregate: The percent loss of the coarse aggregate by the LA abrasion test (AASHTO T96) shall not exceed 40%.
2. Fine aggregates
a. Fine aggregate shall be 100% crushed materials and conform to the following table:
<table>
<tr><th>Test Method and Description</th><th>Minimum</th><th>Maximum</th></tr>
<tr><td>Uncompacted Voids % (AASHTO T304)</td><td>45%</td><td>100%</td></tr>
<tr><td>Sand Equivalent % (AASHTO T176)</td><td>50%</td><td>100%</td></tr>
<tr><td>Liquid Limit % (AASHTO T89)</td><td>0%</td><td>25%</td></tr>
<tr><td>Plasticity Index (AASHTO T90)</td><td colspan="2">Nonplastic</td></tr>
</table>
b. Fine aggregate shall have a maximum of 1.0% clay lumps and friable particles as determined by AASHTO T112. It shall consist of hard, tough grains free of deleterious substances.
</td></tr>
</table>

continued

SHRP 2 R23 Guide Specification: SMA (*continued*)

Paragraph	Content
Materials (*continued*)	3. Mineral filler for SMA: Mineral filler shall meet the requirements of AASHTO M17. These minerals shall consist of finely divided mineral matter such as crusher fines, road dust, slag dust, hydrated lime, hydraulic cement, or fly ash (Class F) meeting the requirements of AASHTO M17. Any lime-based product shall meet the requirements of AASHTO M303. 4. Recycled asphalt pavement (RAP) and reclaimed asphalt shingles (RAS): RAP and RAS are not allowed in SMA mixes unless local practice has shown that performance is not impacted negatively. 5. Blend of aggregates: The combined aggregates shall conform to the percent passing by volume requirements given in the following table:

Sieve Size	0.5 in. Lower Limit	0.5 in. Upper Limit	0.375 in. Lower Limit	0.375 in. Upper Limit
0.75 in.	100	100		
0.5 in.	90	100	100	100
0.375 in.	26	78	90	100
No. 4	20	28	26	60
No. 8	16	24	20	28
No. 16	13	21	13	21
No. 30	12	18	12	18
No. 50	12	15	12	15
No. 200	8	10	8	10
Typical asphalt content ranges between 6.0% and 7.5% by weight of total mix.				

Paragraph	Content
	6. Asphalt binder a. Asphalt binder for SMA: The liquid asphalt binder shall be polymer modified and meet local PG binder temperature requirements. b. Binder draindown: When fiber is used, the dosage rate shall be a minimum of 0.3% for both cellulose and mineral fibers by weight of total mix and shall produce a maximum liquid asphalt binder draindown of 0.3% or less when tested in accordance with AASHTO T305. 7. Mix design: ASMA mixes shall be designed by an approved mix design process. If the Superpave gyratory compactor is used, a compactive effort of 50 gyrations shall be used. SMA mixes can also be designed using a 50-blow Marshall design. The SMA shall have minimum voids in the mineral aggregate (VMA) of 17 and air voids (V_a) of 4.0%. Voids in the coarse aggregate (VCA) should be used to ensure stone-on-stone skeleton is achieved. The SMA mix shall be designed with a minimum tensile strength ratio (TSR) of 70% according to AASHTO T283 with the test conducted at an air void level of 6.0%. The mix should be checked for rutting potential by the asphalt pavement analyzer or the Hamburg wheel-tracking device and locally determined rut criteria.

continued

SHRP 2 R23 Guide Specification: SMA (*continued*)

Paragraph	Content
Construction	1. Hot-mix plant requirements: SMA shall not be stored at elevated temperatures for more than 3 hours. SMA shall not be heated above 350°F without approval of the Engineer. 2. Weather and temperature limitations: The mixture shall be laid only upon an approved underlying course, which is dry, and only when weather conditions are suitable. SMA shall not be placed when the surface or air temperature is below 40°F. Spreading operations shall be stopped when the air temperature is below 45°F and falling. 3. Surface preparation: A tack coat shall be applied to ensure uniform and complete adherence of the overlay. 4. Compaction: The mixture, when delivered to the paver, shall have a temperature of not less than 290°F. Due to the nature of the stone matrix asphalt mixture, the surface shall be rolled immediately. Rolling shall be accomplished with steel wheel rollers. Pneumatic tire rollers shall not be used on stone matrix asphalt. Rollers shall move at a uniform speed, not to exceed 3 mph, with the drive roller nearest the paver. Rolling shall be continued until all roller marks are eliminated and the required density has been obtained, but not after the mat has cooled to 240°F. The Contractor shall monitor density during the compaction process by use of nuclear density gauges to ensure that the required density is being obtained. If vibratory compaction causes aggregate breakdown or forces liquid asphalt binder to the surface, the vibratory mode shall be turned off and the roller shall operate in static mode only. To prevent adhesion of the mixture to the rollers, it shall be necessary to keep the wheels properly moistened.
Method of Measurement and Basis of Payment	The accepted quantities of SMA wearing layer in tons will be measured. The SMA mix shall be evaluated for asphalt binder content, laboratory compacted air voids, and in-place density; pay factors will be applied. In-place density will be assessed as a percentage of theoretical maximum density (TMD) (AASHTO T209). The target density for SMA mix is 94% of TMD.

References

Alabama Department of Transportation. “Stone Matrix Asphalt (SMA) (Fiber Stabilized Asphalt Concrete),” Section 423, Standard Specifications, Alabama Department of Transportation, Montgomery, 2008.

Brown, R., and L. Cooley. “NCHRP Report 425: Designing Stone Matrix Asphalt Mixtures for Rut-Resistant Pavements,” Project 9-8, TRB, National Research Council, Washington, D.C., 1999.

Prowell, B., D. Watson, G. Hurley, and R. Brown. “Evaluation of Stone Matrix Asphalt (SMA) for Airfield Pavements,” Paper, 2010 FAA Worldwide Airport Technology Transfer Conference, Atlantic City, N.J., April 2010.

SHRP 2 R23 Guide Specification: Open-Graded Friction Course

<table>
<tr><th>Paragraph</th><th>Content</th></tr>
<tr><td>Description</td><td>The work covered by this specification shall consist of constructing a hot-mixed, hot-laid polymer-modified open-graded friction course wearing layer placed on an existing pavement.</td></tr>
<tr><td>Materials</td><td>1. Aggregates: The aggregate shall be limited to 100% crushed, virgin aggregates.
a. The aggregate shall be combined into a total blend that will produce an acceptable job mix within the gradation limits shown in the following table. The blend shall be made from at least two stockpiles of different gradations. At least 10% of the blend shall be taken from each stockpile.
<table>
<tr><th>Sieve Size</th><th>Percent Passing by Weight</th></tr>
<tr><td>0.75 in.</td><td>100</td></tr>
<tr><td>0.5 in.</td><td>85–100</td></tr>
<tr><td>0.375 in.</td><td>55–65</td></tr>
<tr><td>No. 4</td><td>10–25</td></tr>
<tr><td>No. 8</td><td>5–10</td></tr>
<tr><td>No. 200</td><td>2–4</td></tr>
</table>
b. No RAP or RAS will be allowed.
2. Asphalt binder: The liquid binder shall be polymer-modified PG graded and meet local PG grading requirements. The proportion of liquid asphalt binder to total sample by weight shall be 4.7%–9.0%. The exact proportion shall be fixed by the job mix formula. A fiber stabilizer shall be incorporated into the mix to reduce draindown.
3. Mix design: The open-graded friction course shall be designed with a minimum air void content of 12%.</td></tr>
<tr><td>Construction</td><td>1. Compaction equipment: Steel wheel tandem (7 ton [6 metric ton] minimum size) rollers shall be furnished in sufficient numbers based on the quantity of material being placed to provide effective compaction coverage within the workable time period of the mix as designated by the Engineer. Rubber-tire rollers shall not be used.
2. Weather and temperature limitations: The mixture shall be laid only upon an approved underlying course, which is dry, and only when weather conditions are suitable. SMA shall not be placed when the surface or air temperature is below 40°F. Spreading operations shall be stopped when the air temperature is below 45°F and falling.
3. Rolling: Rolling shall be as approved by the Engineer. No density tests are required.</td></tr>
<tr><td>Method of Measurement and Basis of Payment</td><td>Open-graded friction course described by this specification shall be paid for by the ton.</td></tr>
</table>

Reference

Alabama Department of Transportation. "Polymer Modified Open Graded Friction Course," Section 420, Standard Specifications, Alabama Department of Transportation, Montgomery, 2008.

SHRP 2 R23 Guide Specification: Rubblization of Existing Concrete Pavement

Paragraph	Content
Description	Rubblize and compact existing concrete pavement.
Equipment	Provide either a Type I or Type II rubblizer, unless otherwise shown on the plans, and necessary rollers for compacting the rubblized pavement. 1. Type I rubblizer: A self-contained, self-propelled, resonant-frequency breaker, capable of producing low-amplitude, 2,000-lb blows, at a rate not less than 44 Hz. 2. Type II rubblizer: A self-contained, self-propelled, multiple-head breaker, with each hammer independently adjustable, and capable of rubblizing a width of up to 13 ft in one pass. 3. Roller-vibratory: Drum (Type C), with a static weight ≥10 tons. 4. Roller-medium pneumatic. 5. Roller-Z grid vibratory: When rubblizing with Type II equipment, provide a steel wheel, self-propelled vibratory roller, with a minimum weight of 10 tons, and a Z-pattern cladding bolted transversely to the surface of the drum.
Construction	1. Preparatory work: Prior to initiating rubblization, the following work must be complete: a. If required, construct pavement drainage systems at least 2 weeks prior to rubblization. b. Any existing material overlaying the concrete pavement must be removed. c. Adjustments or additions to the pavement adjacent to the existing concrete must be complete to the elevation of the concrete pavement to be rubblized. d. Before rubblizing a section, cut full-depth saw-cut joints at any locations shown on the plans to protect facilities that will remain in place. 2. Rubblization and compaction: Operate equipment in a manner that will not damage the base, underground utilities, drainage structures, and other facilities on the project; in the event that damage to such features occurs, the Contractor shall be fully responsible for their repair. a. Use a Type I or Type II rubblizer to completely debond any reinforcing steel and rubblize the existing concrete pavement. Other types of rubblizing equipment will only be used if shown on the plans or approved in writing. Above the reinforcing steel or upper one-half of the pavement (if unreinforced), the equipment shall produce at least 75% of broken pieces less than 3 in. in size. At the surface of the rubblized layer, all pieces shall be less than 6 in. Below the reinforcing steel or in the lower half of the pavement, the maximum particle size shall be 9 in. Any large concrete pieces that do not meet the size requirements previously specified shall be treated as follows: i. If the affected area is less than 10 ft^2 the area may be patched with aggregate. ii. Areas greater than 10 ft^2 that do not meet the specified particle size shall be repaired with hot-mix asphalt, unless otherwise approved by the Engineer. b. Reinforcing steel exposed and projecting from the surface after rubblization or compaction shall be cut off below the surface and removed. 3. Type I rubblization: Begin at a free edge or previously broken edge and work transversely toward the other edge. In the event the rubblizer causes excessive deformation of the pavement, the Engineer may require high-flotation tires with tire pressures less than 60 psi. Any displaced areas shall be considered nonconforming and treated as described above. Compact by seating rubblized pavement with the following rolling pattern: one pass from a vibratory roller, followed by at least one pass with the pneumatic roller, followed by at least two more passes with the vibratory roller. The rolling pattern may be changed as directed.

continued

SHRP 2 R23 Guide Specification: Rubblization of Existing Concrete Pavement (*continued*)

Paragraph	Content
Construction (*continued*)	4. Type II rubblization: Unless otherwise directed, rubblize the entire lane width in one pass. Provide a screen to protect vehicles from flying particles as directed. Compact by seating the pavement with the following rolling pattern: A minimum of four passes with the Z-grid vibratory roller, followed by four passes with a vibratory roller, then at least two passes from a pneumatic roller. The rolling pattern may be changed as directed. 5. Verification of rubblization process: Before full production begins, the Engineer will select approximately 200 linear feet of one lane width to verify the rubblization operation. The contractor shall rubblize the test section, using the section to adjust equipment. From within this test section, the Engineer and Contractor shall agree upon a test pit location. At the test pit, excavate a 4 ft square test pit. The Engineer shall test the material to verify that the specified particle size distribution has been achieved through the entire depth of pavement. Additional test pits may be required during the project to confirm ongoing compliance with the particle size specification. Test pit areas shall be patched as directed either with aggregate or hot-mix asphalt. If the rubblized material from the test pit does not meet specifications, another test strip shall be conducted and tested. Should this pit also fail, rubblization operations shall be suspended until the Contractor demonstrates to the satisfaction of the Engineer that specifications can be met, at which time the Engineer shall allow the Contractor to conduct another test strip. 6. Trafficking: Public traffic shall not be allowed on the rubblized pavement, except at Engineer-approved access points, and the Contractor shall avoid unnecessary trafficking of the rubblized pavement with construction equipment. 7. Placement of surfacing: The Contractor shall coordinate construction activities so that the first overlay course is placed within 48 hours after completion of rubblization. If rain occurs after rubblization but before paving, paving shall not take place until the rubblized layer is dry and stable to the satisfaction of the Engineer.
Method of Measurement	Rubblization shall be measured by the square yard of original concrete pavement. The limits of measurement will be as shown on plans.
Payment	The work performed and materials furnished in accordance with this specification and measured as provided under "Measurement" will be paid for at the unit bid price for "Rubblization of Existing Concrete Pavement." This price is full compensation for rubblizing and compacting existing concrete pavement, saw-cutting required locations, cutting and removing exposed reinforcing steel, repairing unstable or nonconforming locations, conducting required test pits, and equipment, labor, tools, and incidentals.

Reference

Sebesta, S., T. Scullion, and C. Von Holdt. "Rubblization for Rehabilitation of Concrete Pavement in Texas: Preliminary Guidelines and Case Studies," Report FHWA/TX-06/0-4687-1, Texas Transportation Institute, College Station, February 2006.

SHRP 2 R23 Guide Specification: Saw, Crack, and Seat Concrete Pavement

Paragraph	Content
Description	Saw, crack, and seat existing jointed reinforced concrete pavement. Note: This specification is used in conjunction with elements for AASHTO Specification 567 (Cracking and Seating) later in this document on existing jointed reinforced concrete pavements.
Equipment	Provide a concrete saw capable of sawing at least 5 in. deep.
Construction	1. Preparatory work: Prior to sawing, the following work must be complete: a. If required, construct pavement drainage systems at least 2 weeks prior to saw-cutting and cracking and seating. b. Any existing material overlaying the concrete pavement must be removed. 2. Sawing: Transverse saw cuts will be made at a 4 ft to 5 ft spacing along the centerline of the pavement to the depth required to cut the reinforcing steel contained in the jointed reinforced concrete pavement. 3. Cracking and seating: Cracking and seating shall proceed in accordance with the Guide Specifications for Cracking and Seating with the additional requirement that the equipment used to crack the pavement will include a protective plate that eliminates any spalling of the saw-cut during the cracking operation.
Method of Measurement	Sawing, cracking, and seating shall be measured by the square yard of original concrete pavement. The limits of measurement will be as shown on plans.
Payment	The work performed and materials furnished in accordance with this specification and measured as provided under “Measurement” will be paid for at the unit bid price for “Saw, Crack, and Seat Existing Concrete Pavement.” This price is full compensation for sawing, cracking, and seating existing concrete pavement, repairing unstable or nonconforming locations, required coring, and equipment, labor, tools, and incidentals.

Reference

UK Department for Transport. “Manual of Contract Documents for Highway Works, Vol. 1, Series 0700, Road Pavement General,” 2009.

ELEMENTS FOR USE WITH AASHTO GUIDE SPECIFICATIONS

Recommended R23 Specification Elements: AASHTO Section 313 Open-Graded Bituminous Base (OGBB)

AASHTO Paragraph	R23 Recommendations		Source
313.02 Materials	Asphalt	1. Use only PG graded binders in accordance with AASHTO M320.	All states reviewed
		2. Do not use PG binders higher than PG 82-xx.	AASHTO M323
		3. Consider use of LTPPBind for selection of PG binder grade or verified local practice.	Study team
	Aggregate	1. General: Use AASHTO specification sections and subsections unless local conditions require otherwise.	AASHTO 313
		2. RAP is not allowed.	Virginia 313
313.03 Construction	Proportioning	Use AASHTO 313 unless other local criteria are more appropriate	AASHTO 313
	Draindown	≤0.3%	Virginia 313
	Equipment	Vibratory rollers will not be used.	Virginia 313
	Maximum Compacted Layer Thickness	≤4 in.	Missouri 302
	Compaction	Compact with 3 passes of 10-ton steel drum roller.	Michigan 303
	HMA Placement Temperatures	1. Weather limitations: Use AASHTO guidance unless other local criteria are more appropriate	AASHTO 313
		2. Plant discharge temperature range: 250°F–300°F.	Missouri 302
		3. Use an approved material transfer vehicle (MTV) for placing all HMA surface courses	Study team
	Traffic Restrictions	The Contractor shall not use the open-graded course as a haul road or storage area.	Virginia 313
	Hydraulic Efficiency	Use AASHTO 313 or Virginia 313 criteria.	AASHTO 313 or Virginia 313

Recommended R23 Specification Elements: AASHTO Section 315 Separator Fabric for Bases

AASHTO Paragraph	R23 Recommendations		Source
315.02 Materials	Fabric	1. Meet AASHTO M288 Class 1 or 2, or 2. Meet Washington Section 2-12 requirements.	AASHTO 315 Missouri 1011 Washington 2-12
315.03 Construction	Construction	1. Apply construction requirements from AASHTO 315 unless local conditions are more appropriate, or 2. Use Washington Section 2-12 requirements.	AASHTO 315 Washington 2-12

Recommended R23 Specification Elements: AASHTO Section 401 Hot-Mix Asphalt (HMA) Pavements

AASHTO Paragraph	R23 Recommendations		Source
401.02 Materials	Asphalt	Use only PG graded binders in accordance with AASHTO M320.	All states reviewed
		Do not use PG binders higher than PG 82-xx.	AASHTO M323
		Consider use of LTPPBind for selection of PG binder grade or verified local practice.	Study team
		Consider a change in the high-temperature binder grade if the mix RAP content >20%.	AASHTO M323
	Aggregate	General: Use AASHTO specification sections and subsections unless local conditions require otherwise.	AASHTO 401
		Crush or break RAP so that 100% passes a 2-in. sieve.	TxDOT 340, Virginia 211
	Warm-Mix Asphalt	The Contractor may use warm-mix asphalt (WMA) processes in the production of HMA. The Contractor shall submit for approval the process that is proposed and how it will be used in the manufacture of HMA.	Washington 5-04
401.03 Construction	Mix Design	Consider use of fine mix gradation which can be defined as ½ in. nominal maximum aggregate size (NMAS): >40%–47% passing No. 8 sieve. AASHTO M323 has a difference definition for coarse- and fine-graded mixtures.	MnDOT 2360, study team, and NCHRP 531
		Avoid use of 19-mm NMAS mixes unless local performance is acceptable	Study team
		TSR should be >80% of AASHTO T283	Missouri 403 and others
		If RAP content >30%, mix design must incorporate RAP material in the mix design gradation.	Study team
		Use AASHTO mix guidelines in AASHTO M323 with V_a = 4.0%.	AASHTO and Virginia 211
		Consider use of the Hamburg wheel tester to assess mix rutting potential. Use TxDOT criteria unless other, local criteria are available.	TxDOT 340

continued

Recommended R23 Specification Elements: AASHTO Section 401 HMA Pavements (*continued*)

AASHTO Paragraph	R23 Recommendations		Source
401.03 Construction (*continued*)	HMA Placement Temperatures	Use AASHTO guidance unless other local criteria are more appropriate.	AASHTO 401
		Do not place crusted HMA into the paver.	Michigan 502
		Use an approved MTV for placing all HMA surface courses.	Study team
		Establish minimum HMA placing temperatures (before entering the paver) or use TxDOT 340.	TxDOT 340
		When the temperature of the mat immediately behind the screed falls below 200°F, stop paving and place a transverse construction joint. If the temperature of the mat falls below 190°F before any rolling, remove and replace the mat. (An exception would be a warm mix.)	Michigan 502
		Segregation: Consider use and associated measurement options of density profile approach used by TxDOT.	TxDOT 341
	Tack	An asphalt tack coat shall be applied to existing asphalt and concrete surfaces, and to the surface of each course or lift constructed.	Minnesota 2360
	Joints	Stagger joints according to AASHTO.	AASHTO 401
		The minimum density of all traveled-way pavement within 6 in. of a longitudinal joint, including the pavement on the traveled-way side of the shoulder joint, shall not be less than 2.0% below the specified density when unconfined.	Missouri 403
	Lift Thickness	t/NMAS should conform to National Center for Asphalt Technology (NCAT) recommendations. • For fine-graded HMA: t/NMAS ≥ 3.0. • For coarse-graded HMA: t/NMA ≥ 4.0. • For SMA mixes: t/NMA ≥ 4.0.	NCHRP 531
	Compaction	Achieve a minimum compaction of 92% of TMD. The average target percent of TMD should range between 93% and 94% for dense-graded mixes.	AASHTO 401 NCAT
	Rollers and Traffic	Rollers and traffic shall not stand on or operate on the uncompacted or newly rolled pavement with a surface temperature >140°F.	Minnesota 2360 Missouri 403
	Smoothness	Use a 10-ft straightedge. Allowable deviations are as follows: • Base course mixtures: ⅜ to ¾ in. • Leveling and top course mixtures: ⅛ to ¼ in.	Michigan 502

Recommended R23 Specification Elements AASHTO Section 404 Tack Coat

AASHTO Paragraph	R23 Recommendations		Source
404.02 Materials	Binder	Use either an asphalt cement (AASHTO M320) or emulsified asphalt (AASHTO M140 or M208) in accordance with local practice.	AASHTO 404 Texas 340 Virginia 310
404.03 Construction	Weather Limitations	Apply tack coat during dry weather only.	AASHTO 404 Michigan 501
	Surface Preparation	Patch, clean, and remove irregularities from all surfaces to receive tack coat. Remove loose materials.	AASHTO 404 Minnesota 2357 Missouri 407
	Application Surfaces	1. Apply the bond coat to each layer of HMA and to the vertical edge of the adjacent pavement before placing subsequent layers. 2. Apply a thin, uniform tack coat to all contact surfaces of curbs, structures, and all joints.	Michigan 501 Texas 340
	Application Rate	1. Apply undiluted tack at a rate ranging from 0.05 to 0.10 gal/yd^2. 2. Many state DOTs allow dilution with water up to 50%.	Range generally falls within most state limits
	Application Temperatures	Use manufacturer recommendations.	Study team

Recommended R23 Specification Elements: AASHTO Section 409 Cold Milling Asphalt Pavement

AASHTO Paragraph	R23 Recommendations		Source
409.02 Materials	Not Applicable		
409.03 Construction	Milling Equipment	Equipment must consistently remove the HMA surface, in one or more passes, to the required grade and cross section producing a uniformly textured surface. Machines must be equipped with all of the following: • Automatically controlled and activated cutting drums, • Grade reference and transverse slope control capabilities, and • An approved grade referencing attachment, not less than 30 feet in length. An alternate grade referencing attachment may be used if approved by the Engineer prior to use.	Michigan 502
	Milling Operations	The pavement surface shall be milled to the depth, width, grade, and cross slope as shown in the Plans or as otherwise directed by the Engineer. Machine speeds shall be varied to produce the desired surface texture grid pattern. Milling shall be performed without excessive tearing or gouging of the underlying material.	Minnesota 2232
	Milling Operations and Traffic	The pavement surface shall be milled to the depth, width, grade, and cross slope as shown in the Plans or as otherwise directed by the Engineer. Machine speeds shall be varied to produce the desired surface texture grid pattern. Milling shall be performed without excessive tearing or gouging of the underlying material.	Minnesota 2232

Recommended R23 Specification Elements: AASHTO Section 411 In-Place Cold Recycled Asphalt Pavement

AASHTO Paragraph	R23 Recommendations		Source
411.02 Materials	Not Applicable		
411.03 Construction	Use AASHTO 411		

Recommended R23 Specification Elements: AASHTO Section 501 Portland Cement Concrete Pavements

AASHTO Paragraph	R23 Recommendations		Source
501.02 Materials	Basic PCC Mix Design Requirements	• Minimum compressive strength = 3,000 psi to 3,500 psi at 7-day cure. • Flexural strength: minimum between 550 and 650 psi at 7-day cure. • Maximum water/cement ratio: range from 0.35 to 0.45 • Cement content: range from to 560 to 598 lb/yd^3 • NMAS = 1.0 in. • Slump: 0–3 in. • Air content = 5.0%–6.5%	AASHTO 501 MnDOT 2301 Missouri 501 Virginia 217
	Supplementary Cementitious Materials	Supplementary cementitious materials may be used to replace a maximum of 35%–50% of the portland cement.	AASHTO 501 Missouri 501 Washington 5-05
	Dowel Bars	Use corrosion-resistant dowel bars. Details available via WSDOT Section 5-05.	Washington 5-05
501.03 Construction	Mix and Placing Limitations	• Protect the concrete from freezing until the concrete has attained a compressive strength of at least 1,000 psi. • Stop mixing and concreting operations if shaded ambient air temperature away from artificial heat is 40°F or less. Resume operations only when the ambient air temperature is 40°F and rising. • Place mixed concrete only when its temperature is between 50°F and 90°F.	AASHTO 501 Michigan 602 Texas 360
	Curing	• Curing systems: For membrane-forming compounds, the compound shall be applied under constant pressure at the rate of 100–150 ft^2/gal (or according to manufacturer's recommendation) by mechanical sprayers mounted on movable bridges. On textured surfaces, the rate shall be as close to 100 ft^2 as possible. • Protection in cold weather: The Contractor shall protect the concrete from freezing during the first 72 hours immediately following concrete placement. • Curing in hot or windy conditions: Care shall be taken in hot, dry, or windy weather to protect the concrete from shrinkage cracking by applying the curing medium at the earliest possible time after finishing operations and after the sheen has disappeared from the surface of the pavement.	Virginia 316

continued

Recommended R23 Specification Elements: AASHTO Section 501 Portland Cement Concrete Pavements (*continued*)

AASHTO Paragraph	R23 Recommendations		Source
501.03 Construction (*continued*)	Surface Texture or Final Finish	Two options—select one: 1. Transverse tining: Texture the final surface to form an even groove pattern perpendicular to the centerline. Provide a surface with individual grooves 1⁄16 in. to 1⁄8 in. wide and 1⁄8 in. to 3⁄16 in. deep spaced on 3⁄8-in. to ¾-in. centers. Use metal tines. 2. Longitudinal tining: The pavement shall be given an initial and a final texturing. Initial texturing shall be performed with a burlap drag or broom device that will produce striations parallel with centerline. Final texturing shall be performed with a spring steel tine device that will produce grooves parallel with the centerline. The spring steel tine device shall be operated within 5 in., but not closer than 3 in., of pavement edges. Burlap drags, brooms, and tine devices shall be installed on self-propelled equipment having external alignment control. Spring steel tines of the final texturing device shall be rectangular in cross section, 3⁄32 to 1⁄8 in. wide, on ¾-in. centers, and of sufficient length, thickness, and resilience to form grooves approximately 3⁄16 in. deep in the fresh concrete surface. Final texture shall be uniform in appearance with substantially all of the grooves having a depth between 1⁄16 and 5⁄16 in. 3. Additional texturing methods: Methods that include Astroturf drag, diamond grinding, and diamond grooving can be considered in accordance with local practice.	AASHTO 501 Michigan 602 Washington 5-05 and Amendment dated 8/2/2010
	Minimum Strength Requirements for Opening to Traffic	• Min flexural strength ranges from 350 psi for thick slabs (≥9.5 in.) to 500 psi for thin slabs (6 in.). • Min compressive strength ≥2,500 psi	MnDOT 2301 Texas 360 Washington 5-05

Recommended R23 Specification Elements: AASHTO Section 552 Subsealing and Stabilization

AASHTO Paragraph	R23 Recommendations		Source
552.02 Materials	Grout	Use AASHTO Section 552	AASHTO 552
552.03 Construction	Grout Plant	Use AASHTO Section 552	AASHTO 552

Recommended R23 Specification Elements: AASHTO Section 557 Partial-Depth Patching

AASHTO Paragraph	R23 Recommendations		Source
557.02 Materials	Concrete Mix for Patches	Use requirements in AASHTO Section 557.	AASHTO 557
557.03 Construction	Patch Preparation	1. Use of jackhammers: If jackhammers are used for removing pavement, they shall not weigh more than 30 lb, and chipping hammers shall not weigh more than 15 lb. All power-driven hand tools used for the removal of pavement shall be operated at angles less than 45° as measured from the surface of the pavement to the tool. 2. Patch limits: The patch limits shall extend beyond the spalled area a minimum of 3.0 in. Repair areas shall be kept square or rectangular. Repair areas that are within 12.0 in. of another repair area shall be combined. 3. Patches and joints: WSDOT calls for specific requirements when spall repairs involve all joint types.	Washington 5-01.3(5)
	Placing Concrete	Place concrete the same day that the existing pavement is removed. Immediately before the concrete placement, wet the faces of the existing pavement and the surface of the aggregate base with water.	Michigan 603
	Opening to Traffic	The repair areas may be opened to traffic when the new concrete has attained a flexural strength of 300 psi and all joints have been sawed.	Michigan 603

Recommended R23 Specification Elements: AASHTO Section 558 Full-Depth Patching

AASHTO Paragraph	R23 Recommendations		Source
558.02 Materials	Concrete Mix for Patches	1. Use requirements in AASHTO Section 557. 2. For shorter opening times, refer to criteria in Michigan 603 or Texas 361.	AASHTO 558 Michigan 603 Texas 361
558.03 Construction	Repair Area	Make repair areas rectangular, at least 6 ft long and at least half a full lane in width unless otherwise shown on the plans.	Texas 361
	Repair Process Steps	• Saw-cut full depth through the concrete around the perimeter of the repair area before removal. • Remove or repair loose or damaged base material, and replace or repair it with approved base material to the original top of base grade. Place a polyethylene sheet at least 4 mils thick as a bond breaker at the interface of the base and new pavement. Allow concrete used as base material to attain sufficient strength to prevent displacement when placing pavement concrete. • Broom finish the concrete surface unless otherwise shown on the plans.	Texas 361
	Joints	There shall be no new joints closer than 3.0 ft to an existing transverse joint or crack.	Washington 5-01.3(4)

Recommended R23 Specification Elements: AASHTO Section 560 Diamond Grinding Concrete Pavement

AASHTO Paragraph	R23 Recommendations		Source
560.02 Materials		No materials requirements.	
560.03 Construction	Equipment	The grinding equipment shall use diamond-tipped saw blades mounted on a power-driven, self-propelled machine that is specifically designed to smooth and texture PCC pavement. The equipment shall grind the pavement to the specified texture and smoothness tolerances. The equipment shall not damage the underlying surface of the pavement, cause excessive ravels, aggregate fractures, spalls, or otherwise disturb the transverse or longitudinal joint.	AASHTO 560 Texas 360
	Faulted Pavement	Faulted areas at transverse cracks and joints in excess of 1⁄16 in. after initial grinding must be reground until faulting is less than 1⁄16 in.	Michigan 603
	Texture	Grind to a parallel corduroy-type texture consisting of grooves 1⁄16 to 1⁄8 in. wide, 1⁄16 in. deep, and 1⁄16 to 1⁄8 in. on center. Grind to a finished uniform texture. Make the transverse slope of the pavement uniform with no depressions or misalignment of slope greater than 1⁄8 in. when checked with a 10-ft straightedge.	Michigan 603

Recommended R23 Specification Elements: AASHTO Section 561 Milling Pavement

AASHTO Paragraph	R23 Recommendations		Source
561.02 Materials		No materials requirements.	
561.03 Construction	Equipment	• Pavement milling shall be accomplished with a power-operated, self-propelled cold-milling machine capable of removing concrete and bituminous surface material as necessary to produce the required profile, cross slope, and surface texture uniformly across the pavement surface. The machine shall also be equipped with means to control dust and other particulate matter created by the cutting action. • The machine shall be equipped to accurately and automatically establish profile grades along each edge of the machine, within plus or minus 1⁄8 in., by referencing from the existing pavement by means of a ski or matching shoe, or from an independent grade control. The machine shall be controlled by an automatic system for controlling grade, elevation, and cross slope at a given rate.	Minnesota 2232
	Milling Operation	• Mill the surface in a longitudinal direction. For the initial pass, use as a reference the curb, longitudinal edge of pavement, or a string attached to the pavement surface. Furnish a milling machine with a steering guide or reference that allows the operator to follow the guidance reference within 2 in. When milling next to previously milled pavement, use the edge of the milled trench as the longitudinal reference for succeeding passes. • Provide a milled surface with a uniform texture free of excessive gouges, ridges, and grooves. • Provide an end transition on a 4:1 slope to the existing pavement surface at each end of the milling work each day. End the milling passes as close to each other as practical. Do not leave longitudinal joints more than 2 in. deep exposed during nonworking hours.	AASHTO 561

Recommended R23 Specification Elements: AASHTO Section 563 Portland Cement Concrete Unbonded Overlays

AASHTO Paragraph	R23 Recommendations		Source
563.02 Materials	Basic PCC Mix Design Requirements	• Minimum compressive strength = 3,000–3,500 psi at 7-day cure. • Flexural strength: minimum between 550 and 650 psi at 7-day cure. • Maximum water/cement ratio: range from 0.35 to 0.45 • Cement content: range from 560 to 598 lb/yd^3 • NMAS = 1.0 in. • Slump: 0–4 in. • Air content = 5.0%–6.5%	AASHTO 501 and 563 MnDOT 2301 Missouri 501 Virginia 217
	Supplementary Cementitious Materials	Supplementary cementitious materials may be used to replace a maximum of 40%–50% of the portland cement.	AASHTO 501 Missouri 501
	Interlayer	The interlayer material shall be a minimum of 1-in.-thick new bituminous material.	Missouri 506.20
563.03 Construction	Mix and Placing Limitations	• Protect the concrete from freezing until the concrete has attained a compressive strength of at least 1,000 psi. • Stop mixing and concreting operations if shaded ambient air temperature away from artificial heat is 40°F or less. Resume operations only when the ambient air temperature is 40°F and rising. • Place mixed concrete only when its temperature is between 50°F and 85°F.	AASHTO 501 Michigan 602 Texas 360
	Surface Preparation	All holes greater than 2 in. wide and 1 in. deep in the surface of the traffic lanes, excluding shoulders, shall be filled with patching material and shall be compacted to a flat, tight surface.	Missouri 506.20
	Surface Texture	Same as recommendations for AASHTO 501.	
	Bituminous Interlayer	The surface temperature of a bituminous interlayer shall not exceed 90°F prior to the overlay placement. The temperature may be controlled with any means approved by the Engineer, including, but not limited to white curing compound and water misting.	Missouri 506.20

continued

Recommended R23 Specification Elements: AASHTO Section 563 Portland Cement Concrete Unbonded Overlays (*continued*)

AASHTO Paragraph	R23 Recommendations		Source
563.03 Construction (*continued*)	Curing	• Cure the concrete for at least 3 days immediately after the finishing operation. • Curing systems: For membrane-forming compounds, the compound shall be applied under constant pressure at the rate of 100 to 150 ft2/gal by (or according to manufacturer's recommendation) mechanical sprayers mounted on movable bridges. On textured surfaces, the rate shall be as close to 100 ft2 as possible. • Protection in cold weather: The Contractor shall protect the concrete from freezing during the first 72 hours immediately following concrete placement. • Curing in hot or windy conditions: Care shall be taken in hot, dry, or windy weather to protect the concrete from shrinkage cracking by applying the curing medium at the earliest possible time after finishing operations and after the sheen has disappeared from the surface of the pavement.	AASHTO 561 Virginia 316
	Minimum Strength Requirements for Opening to Traffic	• Minimum flexural strength opening ranges from 350 psi for thick slabs (≥9.5 in.) to 500 psi for thin slabs (6 in.) (mostly MnDOT 2301). • The unbounded concrete overlay may be opened for lightweight traffic when the concrete has attained a minimum compressive strength of 2,500 psi. The concrete pavement shall not be opened to all types of traffic until the concrete has attained a minimum compressive strength of 3,000 psi (Missouri 506.20).	MnDOT 2301 Missouri 506.20 Texas 360

Recommended R23 Specification Elements: AASHTO Section 567 Cracking and Seating

AASHTO Paragraph	R23 Recommendations		Source
567.02 Materials		No materials-related specifications.	AASHTO 567
567.03 Construction	General Construction	Use AASHTO Section 567.	AASHTO 567
	Cracking Operations	AASHTO 567 recommends a cracking pattern that result in PCC pieces of 1.2–1.8 ft^2 in area. Other state experience, such as that of Caltrans, suggests that a much larger cracking pattern can work well for jointed plain concrete pavement (JPCP) such as 6 ft by 5 ft (for a 12-ft-wide lane with 15 ft contraction joint spacing results in a lane cracked in half and approximately at the third points). Confirmed by United Kingdom, which calls for cracking every 0.75 to 2 m.	Study team UK Dept. of Transport Specifications (Section 716)
		Given the variability of the specifications available, the study team recommends the minimum distance from a contraction joint to initiate cracking be 3 ft. This should ensure that the cracked areas be dimensioned with a 2-to-1 ratio or less. This assumes the slab is longitudinally cracked down the middle.	Study team
	Seating Operations	• AASHTO 567 recommends seating using a 10-ton steel wheel vibratory roller, with sufficient passes to seat the slabs. • The UK Department of Transport, Section 716, calls for a minimum of six passes with a 20-tonne pneumatic tire roller. • Past reports by NCHRP and NAPA have recommended use of a 35- to 50-ton pneumatic tire roller.	UK Dept. for Transport Specifications (Section 716)

AASHTO AND STATE DOT SPECIFICATION SUMMARIES

AASHTO Specification Designation 313 "Description": Open-Graded Bituminous Base (OGBB)

Agency/Organization	Specification Section: Description
AASHTO (Section 313)	"Construct a permeable base course of aggregate and bituminous material mixed in a central plant and spread and compacted on a prepared foundation."
Michigan DOT (Section 303)	"Construct an open-graded drainage course (OGDC) on an approved surface." NOT BITUMINOUS STABILIZED.
Minnesota DOT	Not available.
Missouri DOT (Section 302)	"This work shall consist of furnishing and placing a stabilized permeable base material. The mixture shall be placed, spread and compacted as shown on the plans or as directed by the engineer." Stabilized permeable base shall be either asphalt binder stabilized or portland cement stabilized at the option of the contractor. Asphalt-stabilized base is described.
Texas DOT (Item 247)	Not available.
Virginia DOT (Section 313)	"This work shall consist of furnishing and placing a course of asphalt-stabilized open-graded material on a prepared subbase or subgrade in accordance with the required tolerances in these specifications and in conformity with the lines and grades shown on the plans or established by the Engineer."
Washington State DOT	Not available.

AASHTO Specification Designation 313 "Materials": Open-Graded Bituminous Base (OGBB)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>AASHTO (Section 313)</td><td>1. Asphalt cement/binder: Meet AASHTO M20 for pen graded, AASHTO M320 for PG graded, or AASHTO M226 for viscosity graded.
2. Aggregates: Major tests and properties
<table><tr><td>LA abrasion, % wear, maximum</td><td>40%</td></tr><tr><td>Mechanically fractured faces (of material retained on No. 4 (4.75-mm) sieve), % minimum</td><td>75% with 2 or more fractured faces</td></tr><tr><td>Flat or elongated pieces on combined and retained on No. 4 (4.75-mm) sieve, % maximum</td><td>15%</td></tr></table><table><tr><th rowspan="2">Sieve Size</th><th colspan="2">Percent Passing</th></tr><tr><th>Min</th><th>Max</th></tr><tr><td>1.5 in.</td><td>100</td><td>100</td></tr><tr><td>1.0 in.</td><td>95</td><td>100</td></tr><tr><td>½ in.</td><td>25</td><td>60</td></tr><tr><td>No. 4</td><td>0</td><td>10</td></tr><tr><td>No. 10</td><td>0</td><td>5</td></tr><tr><td>No. 200</td><td>0</td><td>3</td></tr></table></td></tr>
<tr><td>Michigan DOT (Section 303)</td><td>Materials—refer to Section 902.</td></tr>
<tr><td>Minnesota DOT</td><td>Not available.</td></tr>
<tr><td>Missouri DOT (Section 302)</td><td>1. Asphalt cement/binder: Mixtures shall be composed of the base aggregate and 2.5% asphalt binder by weight (mass) of the total mixture. PG 64-22, PG 70-22, or PG 76-22 asphalt binder shall be used.
2. Aggregates: Major tests and properties—refer to Section 1009.</td></tr>
<tr><td>Texas DOT (Item 247)</td><td>Not available.</td></tr>
<tr><td>Virginia DOT (Section 313)</td><td>1. Asphalt cement/binder: Shall be PG 70-22. Asphalt content 4.3% ± 0.3%.
2. Aggregates: Major tests and properties:
<table><tr><th rowspan="2">Sieve Size</th><th colspan="2">Percent Passing</th></tr><tr><th>Min</th><th>Max</th></tr><tr><td>1 in.</td><td>100</td><td>100</td></tr><tr><td>¾ in.</td><td>88</td><td>100</td></tr><tr><td>½ in.</td><td>70</td><td>90</td></tr><tr><td>No. 8</td><td>0</td><td>15</td></tr><tr><td>No. 200</td><td>0.5</td><td>4.5</td></tr></table>3. Hydrated lime shall be added at 0.5% by weight of total dry aggregate.
4. RAP is not allowed.
5. Coarse aggregate shall conform to Grade A Section 203.
6. Fine aggregate shall conform to Section 202.</td></tr>
<tr><td>Washington State DOT</td><td>Not available.</td></tr>
</table>

AASHTO Specification Designation 313 "Construction": Open-Graded Bituminous Base (OGBB)

Agency/Organization	Specification Section: Construction	
AASHTO (Section 313)	Major construction-related items	
	Proportioning	PG 64-22, percentage by weight (mass) of 2.5 ± 0.3 of the mix
	Equipment	Standard paving equipment as for HMA (AASHTO Section 401)
	Prime Coat	If required, apply prime coat as per AASHTO Section 405
	Surface Tolerance	Shall not exceed 0.5-in. deviation longitudinal or transverse by use of Method 1 (10-ft straightedge).
	Weather Limitations	If layer thickness less than 3 in., minimum air temp = 40°F and surface temp = 45°F. If greater than 3 in., minimum air temp = 30°F and surface temp = 35°F.
	Traffic Restrictions and Curing Period	No vehicles or construction equipment on the OGBB until cooled to ambient temperature.
	Hydraulic Efficiency	Apply 0.26 gal (1 L) of water to surface. Must be totally absorbed into base within 15 s.
Michigan DOT (Section 303)	Major construction-related items	
	Equipment	Compact with three passes of 10-ton (minimum) steel drum roller.
	Surface Tolerance	Shall not exceed 0.75-in. deviation.
	Traffic Restrictions and Curing Period	Limit vehicles and construction equipment on the layer.
Minnesota DOT	Not available.	
Missouri DOT (Section 302)	Major construction-related items	
	Equipment	Compact with three passes of 5- to 10-ton steel drum roller.
	Plant Discharge Temperature	250°F–300°F
	Maximum Compacted Layer Thickness	≤4 in.
Texas DOT (Item 247)	Not available.	

continued

AASHTO Specification Designation 313 "Construction": OGBB (*continued*)

Agency/Organization	Specification Section: Construction	
Virginia DOT (Section 313)	Major construction-related items	
	Draindown	≤0.3%
	Equipment	Vibratory rollers shall not be used. Asphalt-stabilized open-graded material shall be placed in one layer by approved equipment conforming. Compaction shall begin when the internal mat temperature is approximately 150°F–200°F. A static, steel, two-wheel roller shall compact the material in one to three passes in an established pattern. An 8- to 10-ton roller is recommended for such use. The mat shall be compacted sufficiently to support the placement of the next layer but not to the point that it is not free draining or that the aggregate is crushed.
	Mix Temperature	Mixtures shall be between 250°F and 280°F.
	Surface Tolerance	The finished surface of the stabilized open-graded material shall be uniform and shall not vary at any point more than 0.5 in. above or below the grade shown on the plans.
	Weather Limitations	Atmospheric temp >40°F and the surface temp ≥35°F.
	Traffic Restrictions	The Contractor shall not use the open-graded course as a haul road or storage area. Construction traffic will not be permitted on the open-graded course except for equipment required to place the next layer.
	Hydraulic Efficiency	Stabilized open-graded material shall be designed to have an in-place coefficient of permeability of at least 1,000 ft/day when tested in accordance with VTM-84.
Washington State DOT	Not available.	

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 315 "Description": Separator Fabric for Bases

Agency/Organization	Specification Section: Description
AASHTO (Section 315)	"Furnish and install geotextiles for subgrade separation." "Separation geotextile shall be used as a separation material to prevent mixing of dissimilar material, and to control migration of backfill material through joints in structural elements."
Michigan DOT	Not available.
Minnesota DOT	Not available.
Missouri DOT (Section 1011)	"This specification covers geotextile for use in subsurface drainage, sediment control and erosion control, or as a permeable separator."
Texas DOT	Not available.
Virginia DOT	Not available.
Washington State DOT (Section 2-12)	"The Contractor shall furnish and place construction geosynthetic in accordance with the details shown in the Plans."

AASHTO Specification Designation 315 "Materials": Separator Fabric for Bases

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>AASHTO (Section 315)</td><td>Separator fabric: Meet AASHTO M288 for separation.</td></tr>
<tr><td>Michigan DOT</td><td>Not available.</td></tr>
<tr><td>Minnesota DOT</td><td>Not available.</td></tr>
<tr><td>Missouri DOT (Section 1011)</td><td>1. The material shall be either AASHTO M288 Class 1 or Class 2. (Note: Geotextile Classes 1 and 2 relate to grab, sewn seam, tear, and puncture strengths as well as permittivity.)
2. The minimum permittivity shall be 1.0 s^{-1}.</td></tr>
<tr><td>Texas DOT</td><td>Not available.</td></tr>
<tr><td>Virginia DOT</td><td>Not available.</td></tr>
<tr><td>Washington State DOT (Section 2-12)</td><td>1. Geosynthetic roll identification, storage, and handling shall be in conformance to ASTM D4873.
2. During periods of shipment and storage, the geosynthetic shall be stored off the ground.
3. The geosynthetic shall be covered at all times during shipment and storage such that it is fully protected from ultraviolet radiation including sunlight, site construction damage, precipitation, chemicals that are strong acids or strong bases, flames including welding sparks, temperatures in excess of 160°F, and any other environmental condition that may damage the physical property values of the geosynthetic.
4. Geosynthetics for separation shall conform to the following:
<table>
<tr><th rowspan="2">Geotextile Property</th><th rowspan="2">ASTM Test</th><th colspan="2">Geotextile Property Requirements</th></tr>
<tr><th>Woven</th><th>Nonwoven</th></tr>
<tr><td>AOS</td><td>D4751</td><td colspan="2">No. 30 max</td></tr>
<tr><td>Water Permittivity</td><td>D4491</td><td colspan="2">0.02 s–1 min.</td></tr>
<tr><td>Grab Tensile Strength</td><td>D4632</td><td>250 lb min.</td><td>160 lb min.</td></tr>
<tr><td>Grab Failure Strain</td><td>D4632</td><td><50%</td><td>≥50%</td></tr>
<tr><td>Seam Breaking Strength</td><td>D4632</td><td>220 lb min.</td><td>140 lb min.</td></tr>
<tr><td>Puncture Resistance</td><td>D6241</td><td>495 lb min.</td><td>310 lb min.</td></tr>
<tr><td>Tear Strength</td><td>D4533</td><td>80 lb min.</td><td>50 lb min.</td></tr>
<tr><td>UV Radiation Stability</td><td>D4355</td><td colspan="2">50% strength retained minimum after 500 h in xenon arc device.</td></tr>
</table></td></tr>
</table>

AASHTO Specification Designation 315 "Construction": Separator Fabric for Bases

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>AASHTO (Section 315)</td><td>Major construction-related items
<table>
<tr><td>Protecting and Storing Geotextiles</td><td>Wrap geotextile in a protective covering to prevent damage during shipping and handling.</td></tr>
<tr><td>Preparing the Surface</td><td>Prepare the surface to receive the geotextile to a smooth condition, free of obstructions and debris that may damage the fabric during installation.</td></tr>
<tr><td>Placing Geotextiles</td><td>Place the fabric in the manner and at the locations shown on the plans.</td></tr>
<tr><td>Constructing Seams</td><td>To join separate geotextile sheets, either provide a minimum 18-in. overlap or provide sewn seams. If overlapped, place the fabric so that the preceding roll overlaps the following roll in the direction the base material is being spread. If sewn, ensure the seam strength is at least 70% of the required tensile strength of the unaged fabric.</td></tr>
<tr><td>Applying Cover Material</td><td>Cover the fabric with the base material within two weeks of its placement. Apply cover material by back dumping in a manner that prevents slippage of the fabric. Apply a minimum cover of 3 in. Bituminous mix material may be laid by a tracked laydown machine.</td></tr>
</table></td></tr>
<tr><td>Michigan DOT</td><td>Not available.</td></tr>
<tr><td>Minnesota DOT</td><td>Not available.</td></tr>
<tr><td>Missouri DOT (Section 1011)</td><td>No major construction-related items listed in Section 1011.</td></tr>
<tr><td>Texas DOT</td><td>Not available.</td></tr>
<tr><td>Virginia DOT</td><td>Not available.</td></tr>
<tr><td>Washington State DOT (Section 2-12)</td><td>1. The area to be covered by the geosynthetic shall be graded to a smooth, uniform condition free from ruts, potholes, and protruding objects such as rocks or sticks.
2. The geosynthetic shall be spread immediately ahead of the covering operation. The geosynthetic shall not be left exposed to sunlight during installation for a total of more than 14 calendar days. The geosynthetic shall be laid smooth without excessive wrinkles.
3. Under no circumstances shall the geosynthetic be dragged through mud or over sharp objects which could damage the geosynthetic.
4. The cover material shall be placed on the geosynthetic such that the minimum initial lift thickness required will be between the equipment tires or tracks and the geosynthetic at all times.
5. Construction vehicles shall be limited in size and weight, to reduce rutting in the initial lift above the geosynthetic, to not greater than 3 in. deep to prevent overstressing the geosynthetic. Turning of vehicles on the first lift above the geosynthetic will not be permitted.
6. The geotextile shall either be overlapped a minimum of 2 ft at all longitudinal and transverse joints, or the geotextile joints shall be sewn together. The initial lift thickness shall be 6 in. or more.</td></tr>
</table>

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

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Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 401 "Description": Hot-Mix Asphalt Pavements

Agency/Organization	Specification Section: Description
AASHTO (Section 401)	"Construct one or more courses of hot mix asphalt (HMA) mixtures on a prepared foundation."
Michigan DOT (Section 501)	"Plant mixed hot mix asphalt (HMA) consists of asphalt binder, aggregates, mineral filler, and other additives."
Minnesota DOT (Section 2360)	"This work consists of the construction of one or more pavement courses of hot plant mixed asphalt-aggregate mixture on the approved prepared foundation, base course or existing surface...."
Missouri DOT (Section 403)	"... work shall consist of providing a bituminous mixture to be placed in one or more courses on a prepared base or underlying course...."
Texas DOT (Items 340 and 341)	"Construct a pavement layer composed of a compacted, dense-graded mixture of aggregate and asphalt binder mixed hot in a mixing plant."
Virginia DOT (Sections 211 and 315)	"This work shall consist of constructing one or more courses of asphalt concrete on a prepared foundation in accordance with the requirements of these specifications and within the specified tolerances for the lines, grades, thicknesses, and cross sections shown on the plans or as established by the Engineer."
Washington State DOT (Section 5-04)	"This Work shall consist of providing and placing 1 or more layers of plant-mixed hot mix asphalt (HMA) on a prepared foundation or base in accordance with these Specifications and the lines, grades, thicknesses, and typical cross-sections shown in the Plans. The manufacture of HMA may include warm mix asphalt (WMA) processes in accordance with these Specifications. WMA processes include organic additives, chemical additives, and foaming."

AASHTO Specification Designation 401 “Materials”: Hot-Mix Asphalt Pavements

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td rowspan="6">AASHTO (Section 401)</td><td>1. Asphalt cement/binder: Meet AASHTO M20 for pen graded, AASHTO M320 for PG graded, or AASHTO M226 for viscosity graded.
2. Aggregates: Major tests and properties
a. Coarse aggregate: Meet ASTM D692 and AASHTO M323. Provide aggregate of crushed stone, crushed slag, crushed gravel, or natural gravel.
b. Fine aggregate: Meet AASHTO M29 and AASHTO M323. Provide aggregate of natural sand, manufactured sand, stone screenings, slag screenings, or a combination of these materials.
c. Mineral filler: Meet AASHTO M17.
d. Lime for asphalt mixtures: Meet AASHTO M303.</td></tr>
<tr><td>Maximum PG binders: Binders stiffer than PG 82-xx should be avoided. (AASHTO M323)</td></tr>
<tr><td>Binder Selection Guidelines for RAP Mixtures (AASHTO M323)
<table>
<tr><th>Recommended Virgin Binder Grade</th><th>RAP Percentage</th></tr>
<tr><td>No change</td><td><15%</td></tr>
<tr><td>One grade softer</td><td>15%–25%</td></tr>
<tr><td>Follow recommendations from blending charts</td><td>≥25%</td></tr>
</table></td></tr>
<tr><td>Nominal maximum aggregate size: Combined aggregate shall have a NMAS of 4.75–19.0 mm for surface courses and no larger than 37.5 mm for HMA subsurface courses (AASHTO M323).</td></tr>
<tr><td>Gradation classification: Combined aggregate gradation classified as “coarse graded” when it passes below the primary control sieve (PCS). All other gradations above the PCS are “fine graded” (AASHTO M323).
<table>
<tr><th>NMAS (mm)</th><td>37.5</td><td>25.0</td><td>19.0</td><td>12.5</td><td>9.5</td></tr>
<tr><th>PCS (mm)</th><td>9.5</td><td>4.75</td><td>4.75</td><td>2.36</td><td>2.36</td></tr>
<tr><th>PCS Control Point % Passing</th><td>47%</td><td>40%</td><td>47%</td><td>39%</td><td>47%</td></tr>
</table></td></tr>
<tr><td>Minimum Sand Equivalent (AASHTO M323)
<table>
<tr><th>Design ESALs</th><th>Minimum Sand Equivalent (%)</th></tr>
<tr><td><0.3</td><td>40%</td></tr>
<tr><td>0.3 to <3</td><td>40%</td></tr>
<tr><td>3 to <10</td><td>45%</td></tr>
<tr><td>10 to <30</td><td>45%</td></tr>
<tr><td>≥30</td><td>50%</td></tr>
</table></td></tr>
<tr><td>Michigan DOT (Section 501)</td><td>Materials—refer to Section 902.</td></tr>
</table>

continued

AASHTO Specification Designation 401 "Materials": Hot-Mix Asphalt Pavements (*continued*)

Agency/Organization	Specification Section: Materials
Minnesota DOT (Section 2360)	Major materials-related items

Design Air Void Content

Location from Surface	≤4 in.	>4 in.
Design Air Voids (Va)	4.0%	3.0%

PG Binder Selection with RAP

	Specified PG	PG to Be Used with RAP	
		≤20% RAP	>20% RAP
Overlay	64-22	64-22	64-28
	All others	No adjust.	No adjust.
New Const.	52-34	52-34	Not allow
	58-28	58-28	58-28
	58-34	58-34	Not low
	64-28	64-28	64-28
	64-34	64-34	Not allow
	All others	No adjust.	Not allow

VMA as a Function of Fine and Coarse Gradations

NMAS (in.)	Fine Mix % Pass No. 8	Min VMA	Coarse Mix % Pass No. 8	Min VMA
3/8	–	15.0	–	–
½	>47	15.0	≤47	14.5
¾	>39	14.0	≤39	13.5
1	>35	13.0	≤35	12.5

continued

AASHTO Specification Designation 401 "Materials": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Materials</th></tr>
<tr><td rowspan="4">Missouri DOT (Section 403)</td><td colspan="2">Major materials-related items</td></tr>
<tr><td>VMA</td><td><table><tr><th>NMAS</th><th>Minimum VMA (%)</th></tr><tr><td>9.5 mm</td><td>15.0</td></tr><tr><td>12.5 mm</td><td>14.0</td></tr><tr><td>19.0 mm</td><td>13.0</td></tr><tr><td>25.0 mm</td><td>12.0</td></tr></table></td></tr>
<tr><td>RAP</td><td>Recycled asphalt pavement (RAP) may be used in any mixture, except SMA mixtures. Mixtures may be used with more than 30% virgin binder replacement provided testing according to AASHTO M323 is included with the job mix formula that ensures the combined binder meets the grade specified in the contract. All RAP material, except as noted below, shall be tested in accordance with AASHTO TP58, "Method of Resistance of Coarse Aggregate Degradation by Abrasion in the Micro-Deval Apparatus."</td></tr>
<tr><td>Moisture Susceptibility</td><td>For all mixtures except SMA, the mixture shall have a tensile strength ratio (TSR) greater than 80% when compacted to 95 mm with 7% ± 0.5% air voids and tested in accordance with AASHTO T283. SMA mixtures shall have a TSR greater than 80% when compacted to 95 mm with 6% ± 0.5% air voids and tested in accordance with AASHTO T283.</td></tr>
</table>

continued

AASHTO Specification Designation 401 "Materials": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>Texas DOT
(Item 340 and 341—
Dense Graded Hot Mix
Asphalt (Method) and
(QC/QA)</td><td>
<table>
<tr><td>Sand Equivalent (SE)</td><td>For combined aggregate, the minimum SE shall be 45%.</td></tr>
<tr><td>RAP</td><td>RAP is salvaged, milled, pulverized, broken, or crushed asphalt pavement. Crush or break RAP so that 100% of the particles pass the 2-in. sieve.

When RAP is allowed by plan note, use no more than 30% RAP in Type A or B mixtures (coarse and fine base mixes) unless otherwise shown on the plans. For all other mixtures, use no more than 20% RAP unless otherwise shown on the plans.</td></tr>
<tr><td>VMA</td><td>
<table>
<tr><th>Aggregate Desc.</th><th>Approx. NMAS</th><th>Design VMA, min %</th><th>Plant Produced VMA, min %</th></tr>
<tr><td>Coarse Base (A)</td><td>37.5 mm</td><td>12.0</td><td>11.0</td></tr>
<tr><td>Fine Base (B)</td><td>25.0 mm</td><td>13.0</td><td>12.0</td></tr>
<tr><td>Coarse Surface (C)</td><td>19.0 mm</td><td>14.0</td><td>13.0</td></tr>
<tr><td>Fine Surface (D)</td><td>12.5 mm</td><td>15.0</td><td>14.0</td></tr>
<tr><td>Fine Surface (E)</td><td>9.5 mm</td><td>16.0</td><td>15.0</td></tr>
</table>
</td></tr>
<tr><td>Hamburg Wheel Test Requirements</td><td>
<table>
<tr><th>PG High Temp. Grade</th><th>Minimum Number of Passes at 0.5-in. Rut Depth, Tested at 122°F</th></tr>
<tr><td>PG 64 or lower</td><td>10,000</td></tr>
<tr><td>PG 70</td><td>15,000</td></tr>
<tr><td>PG 76 or higher</td><td>20,000</td></tr>
</table>
</td></tr>
</table>
</td></tr>
</table>

continued

AASHTO Specification Designation 401 "Materials": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Materials</th></tr>
<tr><td rowspan="6">Virginia DOT
(Sections 211 and 315)</td><td>Mix Tensile Strength Ratio (Section 211)</td><td>The mixture shall produce a tensile strength ratio (TSR) not less than 0.80 for the design and production tests. The TSR shall be determined in accordance with AASHTO T283.</td></tr>
<tr><td>Mixes and PG Binders (Section 211)</td><td><table>
<tr><th>Mix</th><th>ESALs (millions)</th><th>PG Binder</th></tr>
<tr><td rowspan="3">9.0 mm</td><td>0–3</td><td>64-22</td></tr>
<tr><td>3–10</td><td>70-22</td></tr>
<tr><td>>10</td><td>76-22</td></tr>
<tr><td rowspan="3">9.5 mm</td><td>0–3</td><td>64-22</td></tr>
<tr><td>3–10</td><td>70-22</td></tr>
<tr><td>>10</td><td>76-22</td></tr>
<tr><td rowspan="3">12.5 mm</td><td>0–3</td><td>64-22</td></tr>
<tr><td>3–10</td><td>70-22</td></tr>
<tr><td>>10</td><td>76-22</td></tr>
<tr><td rowspan="2">19.0</td><td><10</td><td>64-22</td></tr>
<tr><td>≥10</td><td>70-22</td></tr>
<tr><td>25.0</td><td>≥10</td><td>70-22</td></tr>
</table></td></tr>
<tr><td>RA (Section 211)</td><td>RAP shall be processed in such a manner as to ensure that the maximum top size introduced into the mix shall be 2 in.</td></tr>
<tr><td>PG Grades and RAP (Section 211)</td><td><table>
<tr><th rowspan="2">Mix Type by NMAS</th><th colspan="2">% RAP in Mix</th></tr>
<tr><th>0%–20%</th><th>>20%</th></tr>
<tr><td>9.0, 9.5, and 12.5 mm (9.0- and 9.5-mm mixes are considered as NMAS = 9.5 mm)</td><td>64-22
70-22
76-22</td><td>58-28
64-28
70-28</td></tr>
<tr><td>19 mm</td><td>64-22
70-22</td><td>58-28
64-28</td></tr>
<tr><td>25 mm</td><td>64-22
70-22</td><td>64-22
70-22</td></tr>
</table>
Other conditions and exceptions apply. Refer to VDOT 211 for additional details.</td></tr>
<tr><td>Design Air Voids, Va (Section 211)</td><td>Asphalt content should be selected at 4.0% air voids.</td></tr>
</table>

continued

AASHTO Specification Designation 401 "Materials": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>Washington State DOT
(Sections 5-04 and 9-03)</td><td>Major materials-related items
<table>
<tr><td>RAP</td><td>The Contractor may choose to utilize recycled asphalt pavement (RAP) in the production of HMA. If utilized, the amount of RAP shall not exceed 20% of the total weight of the HMA. The RAP may be from pavements removed under the Contract, if any, or pavement material from an existing stockpile.</td></tr>
<tr><td>Warm-Mix Asphalt</td><td>The Contractor may use warm-mix asphalt (WMA) processes in the production of HMA. The Contractor shall submit to the Engineer for approval the process that is proposed and how it will be used in the manufacture of HMA.</td></tr>
<tr><td>Gradation</td><td>
<table>
<tr><th colspan="5">Aggregate Gradation Control Points</th></tr>
<tr><th>Sieve
% Passing</th><th>3/8 in.</th><th>1/2 in.</th><th>3/4 in.</th><th>1 in.</th></tr>
<tr><td>1.5 in.</td><td></td><td></td><td></td><td>100</td></tr>
<tr><td>1.0 in.</td><td></td><td></td><td>100</td><td>90–100</td></tr>
<tr><td>0.75 in.</td><td></td><td>100</td><td>90–100</td><td>90 max</td></tr>
<tr><td>0.5 in.</td><td>100</td><td>90–100</td><td>90 max</td><td></td></tr>
<tr><td>0.375 in.</td><td>90–100</td><td>90 max</td><td></td><td></td></tr>
<tr><td>No. 4</td><td>90 max</td><td></td><td></td><td></td></tr>
<tr><td>No. 8</td><td>32–67</td><td>28–58</td><td>23–49</td><td>19–45</td></tr>
<tr><td>No. 200</td><td>2.0–7.0</td><td>2.0–7.0</td><td>2.0–7.0</td><td>1.0–7.0</td></tr>
</table>
</td></tr>
</table>
</td></tr>
</table>

AASHTO Specification Designation 401 "Construction": Hot-Mix Asphalt Pavements

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>AASHTO (Section 401)</td><td>Major construction-related items
<table>
<tr><td>Spreading and Placing</td><td>Offset longitudinal joints 6–12 in. from the joint in the layer immediately below. Create the longitudinal joint in the top layer along the centerline of two-lane highways or at the lane lines of roadways with more than two lanes.</td></tr>
<tr><td>HMA Placement Temperature Limitations</td><td>
<table>
<tr><th>Paving Course</th><th>Thickness (in.)</th><th>Min Air Temp (°F)</th><th>Surface Temp (°F)</th></tr>
<tr><td>Surface</td><td>All</td><td>50</td><td>55</td></tr>
<tr><td>Subsurface</td><td><3</td><td>40</td><td>45</td></tr>
<tr><td>Subsurface</td><td>≥3</td><td>30</td><td>35</td></tr>
</table>
</td></tr>
<tr><td>Compaction</td><td>Achieve the minimum [92] percent of theoretical maximum density. Discontinue paving if unable to achieve the specified density before the mixture cools to 175°F.</td></tr>
<tr><td>Joints</td><td>Apply a tack coat on transverse and longitudinal joint contact surfaces immediately before paving. Stagger longitudinal and transverse joints on succeeding lifts approximately 6 in. Construct all longitudinal joints within 12 in. of the lane lines.</td></tr>
</table>
</td></tr>
</table>

continued

AASHTO Specification Designation 401 "Construction": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>Michigan DOT (Section 502)</td><td>Major construction-related items
<table>
<tr><td>Transportation of Mixtures</td><td>Do not place crusted HMA in the paver.</td></tr>
<tr><td>Laydown Temperatures</td><td>Reject all loads having a temperature below 250°F or above 350°F at time of discharge from the hauling unit. A tolerance of ±20°F from the specified target placement temperature is acceptable (see table below).
<table>
<tr><th rowspan="3">Temperature of Surface Overlaid (°F)</th><th colspan="3">Application of HMA Material (lb/yd²)</th></tr>
<tr><th><120</th><th>120–200</th><th>>200</th></tr>
<tr><th colspan="3">Target Placement Temperatures (°F)</th></tr>
<tr><td>35–39</td><td></td><td></td><td>330</td></tr>
<tr><td>40–49</td><td></td><td>330</td><td>315</td></tr>
<tr><td>50–59</td><td>330</td><td>315</td><td>300</td></tr>
<tr><td>60–69</td><td>315</td><td>300</td><td>285</td></tr>
<tr><td>70–79</td><td>300</td><td>285</td><td>270</td></tr>
<tr><td>80–89</td><td>285</td><td>270</td><td>270</td></tr>
<tr><td>≥90</td><td>270</td><td>270</td><td>270</td></tr>
</table></td></tr>
<tr><td>Paving Temperatures</td><td>When the temperature of the mat immediately behind the screed falls below 200°F, stop paving and place a transverse construction joint. If the temperature of the mat falls below 190°F before any rolling, remove and replace the mat.</td></tr>
<tr><td>Longitudinal Joints</td><td>Construct either vertical or tapered longitudinal joints.</td></tr>
<tr><td>Smoothness</td><td>Use a 10-ft straightedge. Allowable deviations are
• Base course mixtures: ⅜ to ¾ in.
• Leveling and top course mixtures: ⅛ to ¼ in.</td></tr>
</table></td></tr>
</table>

continued

AASHTO Specification Designation 401 "Construction": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>Minnesota DOT (Section 2360)</td><td>Major construction-related items
<table>
<tr><td>Tack Coat</td><td>An asphalt tack coat shall be applied to existing asphalt and concrete surfaces, and to the surface of each course or lift constructed.</td></tr>
<tr><td>Compaction</td><td>Rollers shall not stand on the uncompacted or newly rolled pavement with a surface temperature >140°F.</td></tr>
<tr><td>Minimum lift thicknesses</td><td>
<table>
<tr><th>Aggregate Size</th><th>Thickness</th></tr>
<tr><td>⅜ in.</td><td>¾ in.</td></tr>
<tr><td>½ and ¾ in.</td><td>1.5 in.</td></tr>
<tr><td>1 in.</td><td>2.5 in.</td></tr>
</table>
</td></tr>
</table>
<table>
<tr><td>Compaction Pay Schedule</td><td>
<table>
<tr><th>% Gmm Depth from surface ≤4 in.</th><th>% Gmm Depth from surface >4 in.</th><th>% Payment</th></tr>
<tr><td>≥93.6</td><td>≥94.6</td><td>104</td></tr>
<tr><td>93.1–93.5</td><td>94.1–94.5</td><td>102</td></tr>
<tr><td>92.0–93.0</td><td>93.0–94.0</td><td>100</td></tr>
<tr><td>91.0–91.9</td><td>92.0–92.9</td><td>98</td></tr>
<tr><td>90.5–90.9</td><td>91.5–91.9</td><td>95</td></tr>
<tr><td>90.0–90.4</td><td>91.0–91.4</td><td>91</td></tr>
<tr><td>89.5–89.9</td><td>90.5–90.9</td><td>85</td></tr>
<tr><td>89.0–89.4</td><td>90.0–90.4</td><td>70</td></tr>
<tr><td>Less than 89.0</td><td>Less than 90.0</td><td>Other</td></tr>
</table>
Average % Gmm for a lot.</td></tr>
</table>
</td></tr>
<tr><td>Missouri DOT (Section 403)</td><td>Major construction-related items
<table>
<tr><td>Joints</td><td>Longitudinal joints shall be formed by the use of an edging plate fixed on both sides of the finishing machine. The minimum density of all traveled-way pavement within 6 in. of a longitudinal joint, including the pavement on the traveled-way side of the shoulder joint, shall not be less than 2.0% below the specified density when unconfined.</td></tr>
<tr><td>Traffic</td><td>The contractor shall keep traffic off the asphaltic concrete until the surface of the asphaltic concrete is ≤140°F.</td></tr>
<tr><td>Rollers/Rolling HMA</td><td>Rollers shall not be used in the vibratory mode when the mixture temperature is below 225°F. When warm-mix technology is used, rollers shall not be used in the vibratory mode when the mixture temperature is below 200°F.</td></tr>
<tr><td>HMA Density</td><td>The final, in-place density of the mixture shall be 94.5% ±2.5% of the theoretical maximum specific gravity for all mixtures except SMA. SMA mixtures shall have a minimum density of 94.0% of the theoretical maximum specific gravity.</td></tr>
</table>
</td></tr>
</table>

continued

AASHTO Specification Designation 401 "Construction": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td rowspan="5">Texas DOT
(Items 340 and 341)</td><td>Weather Conditions (Items 340 and 341)</td><td>Place mixture when the roadway surface temperature is ≥60°F unless otherwise approved. Measure the roadway surface temperature with a handheld infrared thermometer.</td></tr>
<tr><td>Minimum Placement Temperature (Suggested) (Item 340)</td><td><table>
<tr><th>High Temp. PG Grade</th><th>Minimum Placement Temperature (Before Entering Paver)</th></tr>
<tr><td>PG 64 or lower</td><td>260°F</td></tr>
<tr><td>PG 70</td><td>270°F</td></tr>
<tr><td>PG 76</td><td>280°F</td></tr>
<tr><td>PG 82 or higher</td><td>290°F</td></tr>
</table></td></tr>
<tr><td>Maximum Production Temperature (Item 341)</td><td>TxDOT will not pay for or allow placement of any mixture produced at more than 350°F.</td></tr>
<tr><td>Air Void Control (Item 340)</td><td>Compact dense-graded hot-mix asphalt to contain from 5% to 9% in-place air voids. Do not increase the asphalt content of the mixture to reduce pavement air voids.</td></tr>
<tr><td>Segregation (Density Profile) (Item 341)</td><td>Unless otherwise approved, perform a density profile every time the screed stops, on areas that are identified by either the Contractor or the Engineer as having thermal segregation, and on any visibly segregated areas. If the temperature differential is greater than 25°F, the area will be deemed as having thermal segregation. Take corrective action to eliminate areas that have thermal segregation. Unless otherwise directed, suspend operations if the maximum temperature differential exceeds 50°F. Criteria are as follows:
<table>
<tr><th>Mixture Type</th><th>Max Allowable Density Range (Highest to Lowest)</th><th>Max Allowable Density Range (Average to Lowest)</th></tr>
<tr><td>Types A and B</td><td>8.0 pcf</td><td>5.0 pcf</td></tr>
<tr><td>Types C, D, and E</td><td>6.0 pcf</td><td>3.0 pcf</td></tr>
</table>
Tex-244-F ("Thermal Profile of Hot Mix Asphalt") requires the use of one of three temperature measurement systems:
1. Noncontact infrared thermometer,
2. Thermal camera behind the paver, or
3. Paver-mounted infrared bar (Pave-IR system)
The temperature measurements are applied to the 150-ft longitudinally measured portion of the mat behind the paver.</td></tr>
</table>

continued

AASHTO Specification Designation 401 "Construction": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td>Texas DOT
(Items 340 and 341)
(continued)</td><td>Longitudinal Joint Density (Item 341)</td><td>While establishing the rolling pattern, perform joint density evaluations and verify that the joint density is no more than 3.0 pcf below the density taken at or near the center of the mat. Adjust the rolling pattern if needed to achieve the desired joint density. Perform additional joint density evaluations at least once per sublot unless otherwise directed.</td></tr>
<tr><td rowspan="5">Virginia DOT (Section 315)</td><td>HMA Placement and t/NMAS</td><td>Asphalt concrete Superpave pavement courses shall be placed in layers ≤4.0 times the nominal maximum size aggregate in the asphalt mixture. The minimum thickness for a pavement course shall be ≥2.5 times the nominal maximum size aggregate in the asphalt mixture.</td></tr>
<tr><td>Longitudinal Joints</td><td>The longitudinal joint in one layer shall offset that in the layer immediately below by approximately 6 in. However, the joint in the wearing surface shall be at the centerline of the pavement.</td></tr>
<tr><td>Transverse Joints</td><td>Transverse joints shall be formed by cutting back on the previous run to expose the full depth of the course. A coat of asphalt shall be applied to contact surfaces of transverse joints just before additional mixture is placed against the previously rolled material.</td></tr>
<tr><td>Surface Tolerance</td><td>The surface will be tested by using a 10-ft straightedge. The variation of the surface from the testing edge of the straightedge between any two contacts with the surface shall be not more than ¼ in.</td></tr>
<tr><td>Density Requirements and Payment</td><td>
<table>
<tr><th>Mix Type</th><th>Minimum Control Strip Density as a function of % of TMD</th></tr>
<tr><td>9.5–12.5 mm</td><td>92.2%–92.5%</td></tr>
<tr><td>19.0 mm</td><td>92.0%–92.2%</td></tr>
<tr><td>25.0 mm</td><td>91.5%</td></tr>
</table>
The control strip density is a function of design ESAL levels which are not shown.
<table>
<tr><th>% of Target Control Strip Density</th><th>% of Payment</th></tr>
<tr><td>>102</td><td>95</td></tr>
<tr><td>98–102</td><td>100</td></tr>
<tr><td>97 to <98</td><td>95</td></tr>
<tr><td>96 to <97</td><td>90</td></tr>
<tr><td><96</td><td>75</td></tr>
</table>
</td></tr>
</table>

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AASHTO Specification Designation 401 "Construction": Hot-Mix Asphalt Pavements (*continued*)

Agency/Organization	Specification Section: Construction	
Washington State DOT (Section 5-04)	MTV	1. Direct transfer of HMA from the hauling equipment to the paving machine will not be allowed in the top 0.30 ft of the pavement section of hot-mix asphalt (HMA) used in traffic lanes with a depth of 0.08 ft or greater. A material transfer device or vehicle (MTD/V) shall be used to deliver the HMA from the hauling equipment to the paving machine. 2. HMA placed in irregularly shaped and minor areas such as road approaches, tapers, and turn lanes are excluded from this requirement. 3. The MTD/V shall mix the HMA after delivery by the hauling equipment and prior to laydown by the paving machine. Mixing of the HMA shall be sufficient to obtain a uniform temperature throughout the mixture. 4. If a windrow elevator is used, the length of the windrow may be limited in urban areas or through intersections, at the discretion of the Project Engineer.
	Cyclic Density	1. The Project Engineer may also evaluate the HMA for low cyclic density of the pavement in accordance with WSDOT procedures. Low-cyclic-density areas are defined as spots or streaks in the pavement that are less than 90.0% of the reference maximum density. 2. A $500 price adjustment will be assessed for any 500-ft section with two or more density readings below 90.0% of the reference maximum density.
	Longitudinal Joint Density	1. The Project Engineer will evaluate the HMA wearing surface for low density at the longitudinal joint in accordance with WSDOT procedures. Low density is defined as less than 90.0% of the reference maximum density. 2. If one density reading, at either longitudinal joint, is below 90.0% of the reference maximum density, a $200 price adjustment will be assessed for that sublot.

continued

AASHTO Specification Designation 401 "Construction": Hot-Mix Asphalt Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td rowspan="2">NCAT (Brown et al., 2004)</td><td>Recommendations included the following:
1. For fine-graded HMA: lift thickness/NMAS (or t/NMAS) ≥ 3.0.
2. For coarse-graded HMA: t/NMA ≥ 4.0.
3. For SMA mixes: t/NMA ≥ 4.0.</td></tr>
<tr><td>Coarse- and fine-graded mixes as defined by NAPA
<table>
<tr><th>Mixture NMAS</th><th>Coarse Graded</th><th>Fine Graded</th></tr>
<tr><td>25.0 mm</td><td><40% Passing 4.75 sieve</td><td>>40% Passing 4.75 sieve</td></tr>
<tr><td>19.0 mm</td><td><35% Passing 2.36 sieve</td><td>>35% Passing 2.36 sieve</td></tr>
<tr><td>12.5 mm</td><td><40% Passing 2.36 sieve</td><td>>40% Passing 2.36 sieve</td></tr>
<tr><td>9.5 mm</td><td><45% Passing 2.36 sieve</td><td>>45% Passing 2.36 sieve</td></tr>
</table>
Source for table: National Asphalt Pavement Association, Information Series 128, "HMA Pavement Mix Type Selection Guide." Control sieves and % passing are similar to AASHTO 401 but are not identical.</td></tr>
</table>

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Brown, R., R. Hainin, A. Cooley, and G. Hurley. "NCHRP Report 531: Relationship of Air Voids, Lift Thickness, and Permeability in Hot Mix Asphalt Pavements," Transportation Research Board of the National Academies, Washington, D.C., 2004.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 404 "Description": Tack Coat

Agency/Organization	Specification Section: Description
AASHTO (Section 404)	"Apply an asphalt binder tack coat to a prepared existing surface."
Michigan DOT (Section 501)	"Apply the bond coat uniformly to the clean, dry, surface with a pressure distributor."
Minnesota DOT (Section 2357)	"This work shall consist of treating an existing bituminous or concrete surface with bituminous material preparatory to placing a bituminous course or seal coat thereon."
Missouri DOT (Section 407)	"This work shall consist of preparing and treating an existing bituminous or concrete surface with bituminous material, and blotter material if required, in accordance with these specifications, as shown on the plans or as directed by the engineer."
Texas DOT (Item 340)	The tack specification was largely contained within Item 340 "Dense-Graded Hot Mix Asphalt."
Virginia DOT (Section 310)	"This work shall consist of preparing and treating an existing asphalt or concrete surface with asphalt in accordance with the requirements of these specifications and in conformity with the lines shown on the plans or as established by the Engineer."
Washington State DOT (Section 5-04)	Tack coat requirements are contained in Section 5-04 "Hot Mix Asphalt."

AASHTO Specification Designation 404 "Materials": Tack Coat

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>AASHTO (Section 404)</td><td>1. AASHTO references to Section 702, which lists the following:
a. Asphalt cements/binders: AASHTO M20, M320, or M226.
b. Cutback asphalt: AASHTO M81 for rapid cure and AASHTO M82 for medium cure.
c. Emulsified asphalt: AASHTO M140 or M208.
2. Temperature application ranges (see table):
<table>
<tr><th>Type and Grade of Material</th><th>Spray Temperature (°F)</th></tr>
<tr><td>RC 70</td><td>80–150</td></tr>
<tr><td>RC 250</td><td>100–175</td></tr>
<tr><td>RC 800</td><td>160–225</td></tr>
<tr><td>RC 3000</td><td>200–275</td></tr>
<tr><td>MC 30</td><td>50–120</td></tr>
<tr><td>MC 70</td><td>80–150</td></tr>
<tr><td>MC 250</td><td>100–200</td></tr>
<tr><td>MC 800</td><td>185–260</td></tr>
<tr><td>MC 3000</td><td>225–275</td></tr>
<tr><td>All Emulsions</td><td>50–160</td></tr>
<tr><td>Asphalt Cements (all grades)</td><td>400 max</td></tr>
</table>
</td></tr>
<tr><td>Michigan DOT</td><td>Additional details are provided in MDOT Sections 506 and 507 but the applications are slurry seals and microsurfacing, respectively.</td></tr>
<tr><td>Minnesota DOT
(Section 2357)</td><td>Tack coats are typically limited to use of emulsified asphalts except during freezing weather:
• Anionic: SS-1, SS-1H, MS-2, RS-1, RS-2
• Cationic: CSS-1, CSS-1H, CRS-1, CRS-2</td></tr>
<tr><td>Missouri DOT (Section 407)</td><td>Emulsified asphalts are used and can include SS-1, SS-1H, CSS-1, or CSS-1H.</td></tr>
<tr><td>Texas DOT (Item 340)</td><td>1. Use CSS-1H, SS-1H, or a PG binder with a minimum high-temperature grade of PG 58 for tack coat binder in accordance with Item 300.
2. Do not dilute emulsified asphalts at the terminal, in the field, or at any other location before use.</td></tr>
<tr><td>Virginia DOT (Section 310)</td><td>1. Asphalt for tack coat shall be CRS-1, CRS-2, CRS-1h, or CSS-1h.
2. CMS-2 may be used during the winter months. CMS-2 is not allowed to be diluted.
3. Asphalt for tack coat may be diluted with 50% water provided that resulting material produces a uniform application of the tack.</td></tr>
<tr><td>Washington State DOT</td><td>1. Unless otherwise approved by the Engineer, the tack coat shall be CSS-1, CSS-1h, or STE-1 emulsified asphalt.
2. The CSS-1 and CSS-1h emulsified asphalt may be diluted with water at a rate not to exceed one part water to one part emulsified asphalt.
3. The tack coat shall not exceed the maximum temperature recommended by the emulsified asphalt manufacturer.</td></tr>
</table>

AASHTO Specification Designation 404 "Construction": Tack Coat

Agency/Organization	Specification Section: Construction	
AASHTO (Section 404)	Major construction-related items	
	Weather Limitations	Apply tack coat during dry weather only.
	Equipment	*Distributors.* Use a distributor capable of uniformly dispensing asphalt to the required section at a pressure from [0.05 to 2.0 ± 0.02 gal/yd²]. Maintain uniform asphalt temperature. Equip distributors with a tachometer, pressure gauges, volume-measuring devices or a calibrated tank, tank thermometer, power unit for the pump, and full circulation spray bars adjustable laterally and vertically.
	Prepare Existing Surface	Patch, clean, and remove irregularities from all surfaces to receive tack coat. Remove loose materials.
	Applying Asphalt	Use a calibrated pressure distributor to apply a uniform tack coat. Tack irregular or inaccessible areas using hand-hose application methods. Apply at a rate of [0.033 to 0.15 gal/yd²]. Obtain approval before diluting emulsified asphalt.
Michigan DOT (Section 501)	Major construction-related items	
	Application	Apply the bond coat to each layer of HMA and to the vertical edge of the adjacent pavement before placing subsequent layers.
	Weather and Seasonal Limitations	Do not place HMA or apply bond coat when precipitation is imminent or when moisture on the existing surface will prevent satisfactory curing.

continued

AASHTO Specification Designation 404 "Construction": Tack Coat (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td rowspan="5">Minnesota DOT
(Section 2357)</td><td colspan="2">Major construction-related items</td></tr>
<tr><td>Road Surface Preparation</td><td>At the time of applying bituminous material, the road surface shall be dry and clean, and all necessary repairs or reconditioning work shall have been completed. All objectionable foreign matter on the road surface shall be removed and disposed of by the Contractor as the Engineer approves. Preparatory to placing an abutting bituminous course, the contact surfaces of all fixed structures and the edge of the in-place mixture in all courses at transverse joints and in the wearing course at longitudinal joints shall be given a uniform coating of liquid asphalt or emulsified asphalt, applied by methods that will ensure uniform coating.</td></tr>
<tr><td>Application Rates</td><td>The bituminous material shall be applied at a uniform rate not to exceed
(1) 0.05 gal/yd^2 for cutback asphalt and undiluted asphalt emulsion (as supplied from the refinery).
(2) 0.20 gal/yd^2 for diluted asphalt emulsion (with water added in the field).</td></tr>
<tr><td>Application Temperatures</td><td>Emulsified asphalts (1) SS-1, SS-1H, MS-2, CSS-1, CSS-1H: 70°F–160°F, (2) RS-1: 70°F–140°F, and (3) SS-2, CRS-1, CRS-2: 120°F–185°F</td></tr>
<tr><td>Dilution with Water</td><td>Grades SS-1, SS-1H, CSS-1, and CSS-1H: water may be added up to 50% by volume to improve the material application and distribution characteristics. However, the added water will be excluded from the pay quantities.</td></tr>
<tr><td rowspan="4">Missouri DOT (Section 407)</td><td colspan="2">Major construction-related items</td></tr>
<tr><td>Preparation of Surface</td><td>The existing surface shall be free of all dust, loose material, grease or other foreign material at the time the tack is applied.</td></tr>
<tr><td>Application Rates</td><td>Asphalt emulsion shall be applied uniformly with a pressure distributor at the rate specified in the contract or as revised by the engineer to be within a minimum of 0.02 gal/yd^2 and a maximum of 0.10 gal/yd^2.</td></tr>
<tr><td>Dilution with Water</td><td>Water may be added to the asphalt emulsion in such a proportion that the resulting mixture will contain no more than 50% of added water. The contractor shall notify the engineer of the exact quantity of added water. The application of the resulting mixture shall be such that the original emulsion will be spread at the specified rate.</td></tr>
</table>

continued

AASHTO Specification Designation 404 "Construction": Tack Coat (*continued*)

Agency/Organization	Specification Section: Construction
Texas DOT (Item 340)	Major construction-related items **Preparation of Surface:** Clean the surface before placing the tack coat. **Application Rates:** Unless otherwise approved, apply tack coat uniformly at the rate directed by the Engineer. The Engineer will set the rate between 0.04 and 0.10 gal. of residual asphalt per square yard of surface area. **Tacked Surfaces:** Apply a thin, uniform tack coat to all contact surfaces of curbs, structures, and all joints. **Adhesion Properties:** The Engineer may use Tex-243-F to verify that the tack coat has adequate adhesive properties.
Virginia DOT (Section 310)	Major construction-related items **Preparation of Surface:** The existing surface shall be patched, cleaned, and rendered free from irregularities to the extent necessary to provide a reasonably smooth and uniform surface. **Tacked Surfaces:** The edges of existing pavements that are to be adjacent to new pavement shall be cleaned to permit adhesion of asphalt. **Application Rates:** Undiluted asphalt shall be applied at the rate of 0.05–0.10 gal/yd^2. Diluted asphalt shall be applied at the rate of 0.10–0.15 gal/yd^2.
Washington State DOT (Section 5-04)	A tack coat of asphalt shall be applied to all paved surfaces on which any course of HMA is to be placed or abutted. Tack coat shall be uniformly applied to cover the existing pavement with a thin film of residual asphalt free of streaks and bare spots. A heavy application of tack coat shall be applied to all joints. For roadways open to traffic, the application of tack coat shall be limited to surfaces that will be paved during the same working shift. The spreading equipment shall be equipped with a thermometer to indicate the temperature of the tack coat material. Equipment shall not operate on tacked surfaces until the tack has broken and cured. If the Contractor's operation damages the tack coat it shall be repaired prior to placement of the HMA.

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 409 "Description": Cold Milling Asphalt Pavement

Agency/Organization	Specification Section: Description
AASHTO (Section 409)	"Cold mill and remove existing asphalt pavement."
Michigan DOT (Section 502)	No specific description. Cold-milling specification information largely contained in MDOT Section 502, "Hot Mix Asphalt Construction Practices."
Minnesota DOT (Section 2232)	"This work shall consist of improving the profile, cross slope, and surface texture of an existing pavement surface by machine (cold) milling preparatory to placement of another course thereon."
Missouri DOT	No information found.
Texas DOT	No files available.
Virginia DOT	No files available.
Washington State DOT	No specification available.

AASHTO Specification Designation 409 "Materials": Cold-Mill Asphalt Pavement

Agency/Organization	Specification Section: Materials
AASHTO (Section 409)	AASHTO does not list any materials-related specifications for Section 409.
Michigan DOT (Section 502)	MDOT does not list any materials-related specifications for cold-milled asphalt pavement.
Minnesota DOT (Section 2232)	MnDOT does not list any materials-related specifications for cold-milled asphalt pavement.
Missouri DOT	No available information found.
Texas DOT	No files available.
Virginia DOT	No files available.
Washington State DOT	No specification available.

AASHTO Specification Designation 409 "Construction": Cold-Mill Asphalt Pavement

Agency/Organization	Specification Section: Construction	
AASHTO (Section 409)	Major construction-related items	
	Milling Equipment	Use self-propelled milling equipment capable of maintaining accurate cut depth and slope. Ensure the equipment can accurately and adequately establish profile grade and control cross slope. Equip the milling machine with integral material pickup and truck discharges, if specified. Ensure the milling machine has effective means for dust control.
	Milling Operations	Cold mill the existing pavement to the specified profile grade and cross section. Taper the transverse joint at the end of each day's run. Unless specified otherwise, dispose of the reclaimed pavement in a manner approved by the Engineer.
	Surface Tests	Meet the specified surface tolerance, as verified using a 10-ft rolling straightedge operated parallel to centerline. Ensure no variation greater than [¼ in.].
Michigan DOT (Section 502)	Major construction-related items	
	Milling Equipment	Equipment must consistently remove the HMA surface, in one or more passes, to the required grade and cross section producing a uniformly textured surface. Machines must be equipped with all of the following: • Automatically controlled and activated cutting drums, • Grade reference and transverse slope control capabilities, and • An approved grade referencing attachment, not less than 30 ft in length. An alternate grade referencing attachment may be used if approved by the Engineer prior to use.
	Milling Operations	1. Remove the HMA surface to the depth, width, grade, and cross section specified. Backfill, and compact, all depressions left by removal of material below the specified grade. 2. Immediately after cold milling, clean the surface. Dispose of the material removed from the surface. Do not incorporate the material into the HMA.

continued

AASHTO Specification Designation 409 "Construction": Cold-Mill Asphalt Pavement (*continued*)

Agency/Organization	Specification Section: Construction	
Minnesota DOT (Section 2232)	Major construction-related items	
	Milling Equipment	Pavement milling shall be accomplished with a power-operated, self-propelled cold-milling machine capable of removing concrete and bituminous surface material as necessary to produce the required profile, cross slope, and surface texture uniformly across the pavement surface. The machine shall also be equipped with means to control dust and other particulate matter created by the cutting action. The machine shall be equipped to accurately and automatically establish profile grades along each edge of the machine, within plus or minus 1⁄8 in., by referencing from the existing pavement by means of a ski or matching shoe, or from an independent grade control. The machine shall be controlled by an automatic system for controlling grade, elevation, and cross slope at a given rate.
	Milling Operations	The pavement surface shall be milled to the depth, width, grade, and cross slope as shown in the Plans or as otherwise directed by the Engineer. Machine speeds shall be varied to produce the desired surface texture grid pattern. Milling shall be performed without excessive tearing or gouging of the underlying material.
	Milling Operations and Traffic	Milling operations shall be conducted so that the entire pavement width is milled to a flush surface at the end of each work period, whenever the pavement is open to traffic.
	Milled Material	The surfacing removed in conjunction with the milling operations may be recycled for use on the project in accordance with the applicable specifications, or disposed of.
Missouri DOT	No information found.	
Texas DOT	No files available.	
Virginia DOT	No files available.	
Washington State DOT	No specification available.	

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 411 "Description": In-Place Cold Recycled Asphalt Pavement

Agency/Organization	Specification Section: Description
AASHTO (Section 411)	"Construct an in-place cold recycled asphalt pavement."
Michigan DOT	Not available.
Minnesota DOT	Not available.
Missouri DOT	No files available.
Texas DOT	No files available.
Virginia DOT	No files available.
Washington State DOT	Not available.

AASHTO Specification Designation 411 "Materials": In-Place Cold Recycled Asphalt Pavement

Agency/Organization	Specification Section: Materials
AASHTO (Section 411)	No specific information provided unique to Section 411.
Michigan DOT	Not available.
Minnesota DOT	Not available.
Missouri DOT	No files available.
Texas DOT	No files available.
Virginia DOT	No files available.
Washington State DOT	Not available.

AASHTO Specification Designation 411 "Construction": In-Place Cold Recycled Asphalt Pavement

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>AASHTO (Section 411)</td><td>Major construction-related items
<table>
<tr><td>Weather Limitations</td><td>Work when the atmospheric temperature is at least [60°F] and when there is no precipitation.</td></tr>
<tr><td>Pulverizing</td><td>Mill and pulverize existing asphalt pavement to the specified depth. Use a self-propelling pulverizing machine capable of maintaining a uniform grade and cross slope. Ensure pulverized material meets the following gradation:
<table>
<tr><th>Sieve Size</th><th>% Passing</th></tr>
<tr><td>2.0 in.</td><td>100</td></tr>
<tr><td>1.5 in.</td><td>90–100</td></tr>
</table>
Reject pulverized asphalt pavement contaminated with base or subgrade material.</td></tr>
<tr><td>Mixing</td><td>Combine an asphalt binder with the pulverized material at the specified rate, using one of the following methods to ensure a consistent mixture:
1. Incorporate with the liquid used to cool the cutter teeth. Ensure even application across the width of the cut and uniformly blend.
2. Incorporate into the pulverized asphalt windrow with a separate mechanical mixing device and uniformly blend.
3. Incorporate through a paving machine during combined mixing and placing operation.</td></tr>
<tr><td>Placing and Compacting</td><td>Place the surface course only when the final moisture content of the recycled mixture is less than [1.5]%. Apply tack, prime, and fog coats to the existing subgrade or surface when specified. Blot excess asphalt with fine sand.
1. Placing by blade. Use self-propelled, pneumatic-tired graders to spread the windrowed material to the required section and grade. Establish a test strip to verify the rolling pattern and maximum placement thickness. Meet density, cross-section, and profile grade requirements.
2. Placing by paver. Place the recycled mixture with a self-propelled asphalt paver. Spread the material in one or more lifts.

Compact as specified.</td></tr>
</table></td></tr>
<tr><td>Michigan DOT</td><td>Not available.</td></tr>
<tr><td>Minnesota DOT</td><td>Not available.</td></tr>
<tr><td>Missouri DOT</td><td>No files available.</td></tr>
<tr><td>Texas DOT</td><td>No files available.</td></tr>
<tr><td>Virginia DOT</td><td>No files available.</td></tr>
<tr><td>Washington State DOT</td><td>Not available.</td></tr>
</table>

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 501 "Description": Portland Cement Concrete Pavements

Agency/Organization	Specification Section: Description
AASHTO (Section 501)	"Construct a portland cement concrete pavement on a prepared subgrade or base course."
Michigan DOT (Sections 601 and 602)	"Construct a jointed Portland cement concrete pavement, unbonded overlay, base course, or shoulder, with or without reinforcement." Both MDOT Sections 601 ("Portland Cement Concrete Pavements") and 602 ("Concrete Pavement Construction") were reviewed.
Minnesota DOT (Section 2301)	"This work shall consist of constructing Portland cement concrete pavement on a prepared base."
Missouri DOT (Sections 501, 502)	"502. This work shall consist of constructing a Portland cement concrete base or pavement, with or without reinforcement as specified, shown on the plans or directed by the engineer." "501. Concrete shall consist of a mixture of cement, fine aggregate, coarse aggregate and water, combined in the proportions specified for the various classes. Admixtures may be added as specifically required or permitted." Brief mention is made of Section 507, "Strength of Concrete Using the Maturity Method."
Texas DOT (Item 360)	"Construct hydraulic cement concrete pavement with or without curbs on the concrete pavement."
Virginia DOT (Sections 217 and 316)	Section 316: "This work shall consist of constructing reinforced, non-reinforced, or continuously reinforced hydraulic cement concrete pavement and approach slabs composed of hydraulic cement concrete, with or without reinforcement as specified, on a prepared subgrade or base course in accordance with the requirements of these specifications and within the specified tolerances for the lines, grades, thicknesses, and cross sections shown on the plans or as established by the Engineer."
Washington State DOT (Section 5-05)	"This Work shall consist of constructing a pavement composed of Portland cement concrete on a prepared Subgrade or base in accordance with these Specifications and in conformity with the lines, grades, thicknesses, and typical cross-sections shown in the Plans or established by the Engineer."

AASHTO Specification Designation 501 "Materials": Portland Cement Concrete Pavements

Agency/Organization	Specification Section: Materials	
AASHTO (Section 501)	Major materials-related items	
	Portland Cement	Conform to AASHTO M85.
	Fine Aggregate	Conform to AASHTO M6.
	Coarse Aggregate	Conform to AASHTO M80.
	Load Transfer Devices	Conform to AASHTO M31.
	Joint Filler	Conform to AASHTO M282, "Poured Joint Sealants for Pavements."
	Reinforcing Steel	1. Conform to AASHTO M31 or M322. 2. Furnish deformed bars for concrete structures meeting the tensile properties for the grade specified.
	Curing Materials	1. Burlap Cloth: AASHTO M182 2. Sheet Materials: AASHTO M171 3. Liquid Membrane Compounds: AASHTO M148
	Air-Entraining Admixtures	Conform to AASHTO M154.
	Chemical Admixtures	Conform to AASHTO M194 as applied to (1) water reducing, (2) set retarding, and (3) set accelerating.
	Fly Ash	Conform to AASHTO M295.
	Ground Granulated Blast Furnace Slag (GGBFS)	Conform to AASHTO M302.
	Water	Conform to AASHTO M157. Potable-quality water requires no testing.

continued

AASHTO Specification Designation 501 "Materials": Portland Cement Concrete Pavements (*continued*)

Agency/Organization	Specification Section: Materials	
Michigan DOT (Section 601)	Major materials-related items	
	Cement	Section 901
	GGBFS	Section 901
	Fly Ash	Section 901
	Coarse Aggregate	Section 902
	Fine Aggregate	Section 902
	Concrete Admixtures	Section 903
	Water	Section 911
	Certified Batch Plants	Supply portland cement concrete from certified portable and stationary concrete batch plant facilities meeting the requirements of the National Ready Mixed Concrete Association (NRMCA) certification program for automatic control and automatic systems. When no fully automated NRMCA certified facility is within 25 mi of the project limits, the Engineer may waive NRMCA certification and/or automation requirements.
	Additional Water at Placement Site	Do not add more water than the approved concrete mix design will allow based on maximum water content and maximum water/cementitious material ratio.
	Concrete Placing Temperature	Concrete must be between 45°F and 90°F at the time it is placed.
	Air Content	At the time of placement, concrete must have 6.5% ± 1.5% entrained air. However, concrete furnished for slipform placement and having a slump of 1.5 in. or less, may have a minimum of 4.5% entrained air.

continued

AASHTO Specification Designation 501 "Materials": Portland Cement Concrete Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>Minnesota DOT
(Section 2301)</td><td>Major materials-related items
<table>
<tr><td>Minimum Cementitious Content</td><td>530 lb/yd³ with a minimum of portland cement = 400 lb/yd³ when using fly ash or GGBFS.</td></tr>
<tr><td>Total Alkalis in Portland Cement</td><td>0.60%</td></tr>
<tr><td>Total Alkalis in Cementitious Material</td><td>≤5 lb/yd³</td></tr>
<tr><td>Water/Cement Ratio</td><td>The target w/c ratio is 0.40 for large paving projects (>5,000 yd³). Incentives and disincentives associated with lower or higher w/c ratios are shown below.
<table>
<tr><th>Mean Value of W/C (termed QI)</th><th>Payment Incentive or Disincentive per Cubic Yard ($/yd³)</th></tr>
<tr><td>≤0.35</td><td>+4.00</td></tr>
<tr><td>0.36</td><td>+3.00</td></tr>
<tr><td>0.37</td><td>+2.00</td></tr>
<tr><td>0.38</td><td>+1.25</td></tr>
<tr><td>0.39</td><td>+0.50</td></tr>
<tr><td>0.40</td><td>0.00</td></tr>
<tr><td>0.41</td><td>–0.50</td></tr>
<tr><td>0.42</td><td>–1.25</td></tr>
<tr><td>0.43</td><td>–2.00</td></tr>
<tr><td>0.44</td><td>–3.00</td></tr>
<tr><td>≥0.45</td><td>Determined by the Concrete Engineer</td></tr>
</table></td></tr>
</table></td></tr>
</table>

continued

AASHTO Specification Designation 501 "Materials": Portland Cement Concrete Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Materials</th></tr>
<tr><td rowspan="17">Missouri DOT
(Section 501, 507)</td><td colspan="2">Major materials-related items</td></tr>
<tr><td>Cement</td><td>Section 1019</td></tr>
<tr><td>GGBFS</td><td>Section 1017</td></tr>
<tr><td>Fly Ash</td><td>Section 1018</td></tr>
<tr><td>Coarse Aggregate</td><td>Section 1005.2</td></tr>
<tr><td>Fine Aggregate</td><td>Section 1005.3</td></tr>
<tr><td>Concrete Admixtures</td><td>Section 1054</td></tr>
<tr><td>Water</td><td>Section 1070</td></tr>
<tr><td>Cement Requirement for Pavement Concrete</td><td>560 lb/yd^3</td></tr>
<tr><td>Minimum Compressive Strength for Pavement Concrete</td><td>4,000 psi (cure period not stated in Section 501).</td></tr>
<tr><td>Max Water/ Cementitious Ratio</td><td>0.50 for air-entrained concrete.
0.53 for non-air-entrained concrete.</td></tr>
<tr><td>Air Entrainment</td><td>If air-entrained concrete is used, the designated quantity of air by volume shall be a minimum of 5.0%.</td></tr>
<tr><td>Supplementary Cementitious Materials</td><td>1. Supplementary cementitious materials may be used to replace a maximum of 40% of the portland cement.
2. Fly ash: Class C or Class F fly ash may be used to replace a maximum of 25% of the portland cement on a pound-for-pound basis in all concrete.
3. GGBFS: GGBFS may be used to replace a maximum of 30% of the portland cement on a pound-for-pound basis in all concrete.</td></tr>
<tr><td>Maturity Method</td><td>Specification in Section 507 covers the maturity method as a nondestructive means of determining in-place concrete strength for pavement or structural applications. This method requires the establishment of a relationship between compressive strength and calculated maturity indices for a specific concrete mixture prior to placement of the mixture in the field. The contractor may use the maturity method in accordance with Section 507 to estimate the compressive strength of the in-place concrete.</td></tr>
</table>

continued

AASHTO Specification Designation 501 "Materials": Portland Cement Concrete Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Materials</th></tr>
<tr><td rowspan="2">Texas DOT (Item 360)</td><td colspan="2">Major materials-related items</td></tr>
<tr><td>Minimum Strength</td><td>Provide Class P concrete designed to meet a minimum average flexural strength of 570 psi or a minimum average compressive strength of 3,500 psi at 7 days or a minimum average flexural strength of 680 psi or a minimum average compressive strength of 4,400 psi at 28 days.</td></tr>
<tr><td rowspan="5">Virginia DOT (Section 217)</td><td colspan="2">Major materials-related items</td></tr>
<tr><td>Basic PCC Mix Design Requirements for Pavement</td><td>1. Minimum compressive strength = 3,000 psi
2. Maximum water/cement ratio = 0.49
3. NMAS = 1.0 in.
4. Slump: 0–3 in.
5. Air content = 6% ± 2%</td></tr>
<tr><td>Cementitious Materials</td><td>Limits for fly ash, GGBFS, and silica fume
1. Fly ash, Class F ≤30%
2. GGBFS ≤50%
3. Silica fume ≤10%</td></tr>
<tr><td>Ready-Mixed Concrete Time Limitation</td><td><table>
<tr><th rowspan="2">Equipment</th><th colspan="3">Concrete Temperature</th></tr>
<tr><th><80°F</th><th>80°F–90°F</th><th>>90°F</th></tr>
<tr><td>Agitator Type Haul Equipment
• Retarded
• Unretarded</td><td>
2.5 h
1.5 h</td><td>
2.0 h
1.25 h</td><td>
1.5 h
1.0 h</td></tr>
<tr><td>Nonagitator-Type Haul Equipment—All Concrete</td><td>1.0 h</td><td>0.75 h</td><td>0.5 h</td></tr>
</table></td></tr>
<tr><td>Placement Temperature Limitation</td><td>When paving concrete is placed by slipform and contains a water reducer, placement temperature limits of 40°F–95°F apply.</td></tr>
<tr><td rowspan="4">Washington State DOT (Section 5-05)</td><td colspan="2">Major materials-related items</td></tr>
<tr><td>Cementitious Materials</td><td>Limits for fly ash, GGBFS, and silica fume
1. Fly ash, Class F ≤35% with max CaO content of 15%.
2. GGBFS ≤25%
3. Max GGBFS + fly ash ≤35% by weight of total cementitious materials.</td></tr>
<tr><td>Minimum Cementitious Materials</td><td>≥564 lb/yd³</td></tr>
<tr><td>Water/Cementitious Ratio</td><td>≤0.44</td></tr>
</table>

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td rowspan="3">AASHTO (Section 501)</td><td colspan="2">Major construction-related items</td></tr>
<tr><td>Mix Design Options</td><td>1. Mix based on minimum strength. Must meet properties shown in table below:

<table>
<tr><th>Property</th><th>Value</th><th>AASHTO Test Method</th></tr>
<tr><td>Compressive Strength (min)</td><td>3,500 psi</td><td>T22</td></tr>
<tr><td>Flexural Strength (min)</td><td>550 psi</td><td>T97</td></tr>
<tr><td>Flexural Strength (min)</td><td>650 psi</td><td>T177</td></tr>
<tr><td>Slump</td><td>3/8–3 in.</td><td>T119</td></tr>
<tr><td>Cement Content
Without Air (min)
With Air (min)</td><td>
564 lb/yd³
598 lb/yd³</td><td></td></tr>
<tr><td>Fly Ash
Type C
Type F</td><td>
30% max[1]
25% max[1]</td><td>Note 1: % max cement replacement</td></tr>
<tr><td>GGBFS</td><td>50% max[1]</td><td>See Note 1</td></tr>
<tr><td>Water/Cementitious Ratio
Without Air (max)
With Air (max)</td><td>
0.53
0.49</td><td></td></tr>
<tr><td>Entrained Air</td><td>5%–8%</td><td>T152, T196, or T199</td></tr>
</table>

2. Contractor proposed mix.
3. Mix based on predetermined cement content—use table above.</td></tr>
<tr><td>Mixing and Placing Limitations</td><td>1. Stop mixing and concreting operations if shaded ambient air temperature away from artificial heat is 40°F or less. Resume operations only when the ambient air temperature is 40°F and rising.
2. Place mixed concrete only when its temperature is between 50°F and 85°F.</td></tr>
</table>

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

Agency/Organization	Specification Section: Construction		
AASHTO (Section 501) (*continued*)	Major construction-related items (*continued*)		
	Longitudinal Joints	Dimensions	Saw the first cut or insert the joint material to one-third of the depth.
		Tiebars	Place [30-in.]-long No. 5 tiebars of Grade 60 steel, spaced [30 in.] center-to-center to one-half of the depth of the portland concrete cement pavement (PCCP). Ensure that tiebars are placed perpendicular to the face of the joint, centered in the slab depth, and parallel to the finished surface.
		Construction	Form or saw longitudinal joints in the plastic concrete. Saw the joints within 4–24 h after placing the concrete and immediately after completing the transverse joints. Allow only the saw on the pavement during sawing operations.
		Sealing	Seal joints after the curing period and before opening the pavement to traffic. Use sandblasting followed by an oil-free air jet to clean the faces and joint openings before sealing. Seal joints only when they are completely dry. Do not dry joints with a heat lance. Use an approved backer rod to seal the lower portion of the joint groove to a uniform depth to prevent sealant from entering beneath the specified depth. Ensure that backer rod is compatible with the sealant type specified and install according to manufacturers' recommendations.
	Contraction Joints	Location and Dimensions	Form or saw joints as narrowly as possible, to at least one-third of the pavement depth.
		Load Transfer	Install load transfer dowel bars of specified grade and size, spaced at […] centers, and secured with a wire basket or implanted mechanically. Place dowel bars one-half of the depth parallel to the surface and pavement edge to an alignment tolerance of (±¼ in.). Vibrate concrete around all dowel bars without misaligning them.
		Construction	Place formed joints while the concrete is plastic. Begin relief-cut joint sawing immediately after the concrete hardens to the stage that it can be sawed without raveling. Saw all joints between 4 and 24 h after placing concrete but before uncontrolled shrinkage cracking develops.
		Sealing	Similar to longitudinal joint construction.

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

Agency/Organization	Specification Section: Construction
AASHTO (Section 501) (*continued*)	Major construction-related items (*continued*)

Transverse Construction Joints	Install transverse construction joints at the end of each day's placement. Form bulkheads when stopping the placement in an emergency or at the end of each day's pour.
Surface Tolerances	AASHTO provides for two profile measurement methods: Straightedge: This method applies to all paving. Test the surface with a 10-ft straightedge at random locations. The Engineer will identify pavement areas that deviate more than [3/16 in.] from the straightedge as defective work. Profilograph: Describes a California-type profilograph.
Curing	Cure the concrete for at least 3 days immediately after the finishing operation. Protect the concrete for at least 10 days or until the concrete achieves a compressive strength of [2,200 psi].
Tolerance and Price Adjustments for Pavement Thickness	Determine pavement thickness according to AASHTO T148. Price adjustments in accordance with the table below.

Deficiency in Thickness as Determined by Cores (in.)	Contract Price Allowed
0–0.20	100
0.21–0.30	80
0.31–0.40	72
0.41–0.50	68
0.51–0.75	57
0.76–1.00	50
>1.00	Remove and Replace

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

Agency/Organization	Specification Section: Construction	
Michigan DOT (Section 602)	Major construction-related items	
	Surface Texture	When the pavement has set sufficiently to maintain texture, drag the surface longitudinally using one or two layers of an approved damp fabric material. Maintain fabric contact with the surface across the entire width of concrete being placed. Immediately after dragging, groove all surfaces other than concrete base courses and shoulders. Orient the grooves generally perpendicular to the centerline and form the grooves in the plastic concrete cleanly without slumping of the edges or severe tearing of the surface. Provide a surface texture consisting of ⅛-in.-wide grooves spaced ½ in. on center and ⅛ in. to ¼ in. deep.
	Sealing Joints with Hot-Poured Sealants	Seal the joints immediately after the joints are cleaned. Joint surfaces must be dry when sealed. Do not place sealant when temperature is less than 50°F.
	Profile	While the concrete is still plastic, test the slab surface for compliance with the required grade and cross section using a 10-ft straightedge. If high or low spots exceed ⅛ in. over 10 ft, suspend paving operations and correct the finishing procedures. Correct high or low spots in pavements that exceed the tolerances.
	Weather and Temperature Limitations	1. Protect the concrete from freezing until the concrete has attained a compressive strength of at least 1,000 psi. 2. Do not place concrete if portions of the base, subbase, or subgrade layer are frozen, or if the grade exhibits poor stability from excessive moisture levels. 3. Do not place concrete when the temperature of the plastic concrete at the point of placement is above 90°F.

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>Minnesota DOT
(Section 2301)</td><td>Major construction-related items
<table>
<tr><td>High-Early Strength Concrete</td><td>High-early concrete is defined as a concrete mixture having a cementitious content greater than 600 lb/yd³. High-early mixes shall be designed to provide a maximum water/cementitious ratio of 0.40 and a minimum flexural strength of 500 psi or a minimum compressive strength of 3,000 psi in 48 h. High-early mixes may have up to 100% portland cement. High-early mixes are not eligible for incentive payments for water/cementitious ratio.</td></tr>
<tr><td>Minimum Strength Requirement for Opening Pavements to Construction and General Public Traffic</td><td>New pavement shall be closed to use by construction and general public traffic for 7 days or according to the values listed in the table below, whichever is the shorter.
<table>
<tr><th>Slab Thickness (in.)</th><th>Flexural Strength (psi)</th></tr>
<tr><td>6.0</td><td>500</td></tr>
<tr><td>6.5</td><td>500</td></tr>
<tr><td>7.0</td><td>500</td></tr>
<tr><td>7.5</td><td>480</td></tr>
<tr><td>8.0</td><td>460</td></tr>
<tr><td>8.5</td><td>440</td></tr>
<tr><td>9.0</td><td>390</td></tr>
<tr><td>9.5</td><td>350</td></tr>
<tr><td>10.0</td><td>350</td></tr>
<tr><td>≥10.5</td><td>350</td></tr>
</table></td></tr>
<tr><td>Placing Concrete</td><td>1. All main line pavement constructed by standard or vibratory machine placement methods shall be constructed in a single layer of concrete.
2. Water shall not be added to the surface of the concrete to aid in finishing without the approval of the Engineer. The Engineer will only give this approval to replace evaporated surface water directly behind the paver caused by a halt in forward progress from a short-term breakdown in equipment or supply of concrete.
3. Should placement of concrete be temporarily suspended, the placement operations shall be resumed in such manner that will not result in a cold joint or honeycombing. If the suspension period exceeds 90 min, a standard header joint shall be constructed.</td></tr>
<tr><td>Joint Construction</td><td>Initial joint sawing shall be approximately ⅛ in. wide and to the full joint depth. The initial sawing shall be accomplished as soon as the condition of the concrete will permit without raveling and before random cracking occurs. The sequence of initial sawing shall be at the Contractor's option.</td></tr>
</table></td></tr>
</table>

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td rowspan="5">Minnesota DOT
(Section 2301)
(continued)</td><td colspan="2">Major construction-related items
(continued)</td></tr>
<tr><td>Surface Finish</td><td>MnDOT uses a standard longitudinal carpet drag followed by transverse tining.</td></tr>
<tr><td>Concrete Curing</td><td>The Contractor shall
1. Cure and protect the concrete by the blanket curing method or one of the membrane curing methods.
2. Cure the entire pavement surface and edges as soon as surface conditions permit after the finishing operations.
3. Continue curing and protecting the concrete for at least 72 h.
4. Place the curing media on the pavement edges within 30 min after removal of the forms when side forms are used.
5. Extend the minimum curing period to 96 h when fly ash or portland-pozzolan cement substitutions are used.</td></tr>
<tr><td>Surface Smoothness</td><td>The Contractor shall test the pavement surface for surface smoothness and ride quality. Surface Smoothness and Ride Quality shall be measured with a 25-ft California-type profilograph, or a lightweight inertial profiler (IP), which produces a profilogram (profile trace of the surface tested). Either type of device must be certified according to the procedure on file in the MnDOT Concrete Engineering Unit.</td></tr>
<tr><td>Thickness Requirements</td><td>Where the cores show a thickness deficiency exceeding ½ in., but less than 1 in., the pavement represented by those cores will not be excluded from the pay quantities; however, a deduction will be made from the moneys due the Contractor equal to the product of the defective areas and $20.00 per square yard. Pavement represented by cores showing a thickness deficiency of 1 in. or more will be excluded from all payments plus a deduction will be made from the moneys due the Contractor equal to the product of the defective areas and $20.00 per square yard. These deductions will be assessed in lieu of removing and replacing the areas of pavement which are deficient in thickness.</td></tr>
</table>

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

Agency/Organization	Specification Section: Construction	
Missouri DOT (Section 502)	Major construction-related items	
	Weather Limitations wrt Freezing Conditions	All concrete shall be effectively protected from freezing until a minimum compressive strength of 3,500 psi has been attained.
	Added Finishing Water	Moisture in any form shall not be applied to the surface of the concrete except for emergency conditions.
	Required Texture Depth	The results of ASTM E965 shall show a texture depth of any sublot, as defined in Section 502.10.1, to have a minimum value of 1.00 mm. Any sublot showing a texture depth of less than 1.00 mm shall require diamond grinding of the pavement represented by this sublot to attain the necessary texture. All testing of the surface texture shall be completed no later than the day following pavement placement.
	Curing	Immediately after the finishing operations have been completed and as soon as marring of the concrete will not occur, the entire surface and exposed edges of the newly placed concrete shall be covered and cured in accordance with one of the following methods. The concrete shall not be left exposed for more than 30 min between stages of curing or during the curing period. 1. White Pigmented Membrane: The contractor shall provide satisfactory equipment to ensure uniform mixture and coverage of curing material, without loss, on the pavement at the rate of not less than 1 gal for each 200 ft^2. 2. Burlap
	Straightedge	As soon as practical, the engineer will straightedge all segments of the paved surface not profilographed, including shoulders. Any variations exceeding 1⁄8 in. in 10 ft will be marked. Areas more than 1⁄8 in. high shall be removed.
	Air Entrainment during Paving Operations	Tests for entrained air content shall be performed on a random basis for each 500 yd^3 of material produced. The minimum air content in front of the paver shall be 5.0% plus the air loss through the paver. The air loss through the paver is determined a minimum of once per half-day production by sampling the concrete ahead of the paver and behind the paver and subtracting the value obtained ahead of the paver from the value obtained behind the paver.

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

Agency/Organization	Specification Section: Construction	
Texas DOT (Item 360)	Concrete Placement	1. Do not allow the pavement edge to deviate from the established paving line by more than ½ in. at any point. Place the concrete as near as possible to its final location, and minimize segregation and rehandling. Where hand spreading is necessary, distribute concrete using shovels. Do not use rakes or vibrators to distribute concrete. 2. Consolidate all concrete by approved mechanical vibrators operated on the front of the paving equipment. Use immersion-type vibrators that simultaneously consolidate the full width of the placement when machine finishing. Keep vibrators from dislodging reinforcement. Use hand-operated vibrators to consolidate concrete in areas not accessible to the machine-mounted vibrators. Do not operate machine-mounted vibrators while the paving equipment is stationary.
	Temperature Restrictions	Place concrete that is between 40°F and 95°F when measured in accordance with Tex-422-A at the time of discharge, except that concrete may be used if it was already in transit when the temperature was found to exceed the allowable maximum. Take immediate corrective action or cease concrete production when the concrete temperature exceeds 95°F.
	Early Opening	Concrete pavement may be opened after curing is complete and the concrete has attained a flexural strength of 450 psi or a compressive strength of 2,800 psi. The maturity method, Tex-426-A, may be used to estimate concrete strength for early opening pavement to traffic.
	Tolerance and Price Adjustments for Pavement Thickness	See table below

Deficiency in Thickness as Determined by Cores (in.)	Contract Price Allowed
Not Deficient	100
>0 to 0.20	100
>0.20 to 0.30	80
>0.30 to 0.40	72
>0.40 to 0.50	68
>0.50 to 0.75	57
>0.75	Zero pay or removal

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td rowspan="5">Virginia DOT (Section 316)</td><td>Concrete Base Course</td><td>The construction of a hydraulic cement concrete base course shall conform to the requirements of these Specifications except for floating and final finishing of the surface. The surface shall be finished so that there will be no deviation of more than ¼ in. between any two contact points when tested with a 10-ft straightedge placed parallel with the centerline. A heavy broomed texture shall be applied.</td></tr>
<tr><td>Curing</td><td>The following apply to curing:
1. Curing systems: Membrane-forming compounds: The compound shall be applied under constant pressure at the rate of 100 to 150 ft²/gal by mechanical sprayers mounted on movable bridges. On textured surfaces, the rate shall be as close to 100 ft² as possible.
2. Protection in cold weather: The Contractor shall prevent the temperature at the surface of the concrete from falling below 40°F during the first 72 h immediately following concrete placement.
3. Curing in hot or windy conditions: Care shall be taken in hot, dry, or windy weather to protect the concrete from shrinkage cracking by applying the curing medium at the earliest possible time after finishing operations and after the sheen has disappeared from the surface of the pavement.</td></tr>
<tr><td>Joint Sealers</td><td>VDOT allows three basic types of joint sealers:
1. Performed
2. Hot-poured
3. Silicone</td></tr>
<tr><td>Thickness Price Adjustments</td><td><table><tr><th>Deficiency in Thickness (in.)</th><th>% of Contract Price Allowed</th></tr><tr><td>0–0.20</td><td>100</td></tr><tr><td>0.21–0.30</td><td>80</td></tr><tr><td>0.31–0.40</td><td>72</td></tr><tr><td>0.41–0.50</td><td>68</td></tr><tr><td>0.51–0.75</td><td>57</td></tr><tr><td>0.76–1.00</td><td>50</td></tr><tr><td>>1.00</td><td>Either zero pay or remove and replace</td></tr></table></td></tr>
<tr><td>Opening to Traffic</td><td>Pavement shall not be opened to traffic until specimen beams have attained a modulus of rupture strength of 600 psi when tested by the center point loading method in accordance with the requirements of ASTM C293. In the absence of such tests, pavement shall not be opened to traffic until 14 days after concrete is placed. Prior to opening to traffic, pavement shall be cleaned and joints sealed and trimmed.</td></tr>
</table>

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

Agency/Organization	Specification Section: Construction	
Washington State DOT (Section 5-05)	Subgrade	1. The subgrade shall be prepared and compacted a minimum of 3 ft beyond each edge of the area which is to receive concrete pavement in order to accommodate the slipform equipment. 2. Concrete shall not be placed on a frozen subgrade or during heavy rainfall. 3. The subgrade shall be moist before the concrete is placed. When placing concrete on a treated base, the surface temperature shall not exceed 90°F.
	Contraction Joints	1. All transverse and longitudinal contraction joints shall be formed with suitable power-driven concrete saws. The Contractor shall provide sufficient sawing equipment capable of completing the sawing to the required dimensions and at the required rate to control cracking. The Contractor shall provide adequate artificial lighting facilities for night sawing. 2. Joints shall not vary from the specified or indicated line by more than ¾ in. 3. Commencement of sawing transverse contraction joints will be dependent upon the setting time of the concrete and shall be done at the earliest possible time following placement of the concrete without tearing or raveling the adjacent concrete excessively. 4. Longitudinal contraction joints shall be sawed as required to control cracking and as soon as practical after the initial control transverse contraction joints are completed. 5. Any damage to the curing material during the sawing operations shall be repaired immediately after the sawing is completed. When cement concrete pavement is placed adjacent to existing cement concrete pavement, the vertical face of all existing working joints shall be covered with a bond-breaking material such as polyethylene film, roofing paper, or other material as approved by the Engineer.

continued

AASHTO Specification Designation 501 "Construction": Portland Cement Concrete Pavements (*continued*)

Agency/Organization	Specification Section: Construction	
Washington State DOT (Section 5-05) (*continued*)	Dowel Bars	1. Corrosion-resistant dowel bars shall be placed at all transverse contraction joints as shown in the Contract or in accordance with the Standard Plans. 2. All dowel bars shall have a parting compound, such as curing compound, grease, or other Engineer-approved equal applied to them prior to placement. 3. Any dowel bar delivered to the project that displays rust/oxidation, pinholes, questionable blemishes, or deviates from the round shall be rejected. 4. Corrosion-resistant dowel bars shall be 1½-in.-outside-diameter plain round steel bars 18 in. in length and meet the requirements one of the following types (details available in WSDOT Section 9-07.5(2)): • Stainless steel–clad dowel bars • Stainless steel tube dowel • Stainless steel solid dowel bars • Corrosion-resistant, low-carbon, chromium plain steel bars • Zinc-clad dowel bars
	Cold Weather Work	When the air temperature is expected to reach the freezing point during the day or night and the pavement has not reached 50% of its design strength or 2,500 psi, whichever is greater, the concrete shall be protected from freezing.
	Opening to Traffic	The pavement may be opened to traffic when the concrete has developed a compressive strength of 2,500 psi as determined from cylinders, made at the time of placement, cured under comparable conditions, and tested in accordance with AASHTO T22.

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 552 "Description": Subsealing and Stabilization

Agency/Organization	Specification Section: Description
AASHTO (Section 552)	"Find and fill existing voids in the pavement system by drilling injection holes, placing material, monitoring the pavement profile, testing for deflection after grouting, and resealing pavement joints."
Michigan DOT	Not available.
Minnesota DOT	Not available.
Missouri DOT	Not available.
Texas DOT	Not available.
Virginia DOT	Not available.
Washington State DOT	Not available.

AASHTO Specification Designation 552 "Materials": Subsealing and Stabilization

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>AASHTO (Section 552)</td><td>AASHTO references to Subsection 551.02 which lists:
<table>
<tr><th>Material</th><th>AASHTO Subsection</th></tr>
<tr><td>Portland cement</td><td>701.02</td></tr>
<tr><td>Limestone dust</td><td>703.14</td></tr>
<tr><td>Chemical admixtures</td><td>713.03(B)</td></tr>
<tr><td>Fly ash</td><td>713.03(C)(1)</td></tr>
<tr><td>Grout for pavement jacking, subsealing, and stabilization</td><td>713.04(A)</td></tr>
<tr><td>Water</td><td>714.01(A)</td></tr>
</table></td></tr>
<tr><td>Michigan DOT</td><td>Not available.</td></tr>
<tr><td>Minnesota DOT</td><td>Not available.</td></tr>
<tr><td>Missouri DOT</td><td>Not available.</td></tr>
<tr><td>Texas DOT</td><td>Not available.</td></tr>
<tr><td>Virginia DOT</td><td>Not available.</td></tr>
<tr><td>Washington State DOT</td><td>Not available.</td></tr>
</table>

AASHTO Specification Designation 552 "Construction": Subsealing and Stabilization

Agency/Organization	Specification Section: Construction
AASHTO (Section 552)	All construction-related items are as follows: **Grout Plant**: The grout plant shall conform to Subsection 551.03(A) and the following: The Contractor may substitute a paddle-type mixer for the high-speed colloidal mixer when using limestone dust grout. Furnish an injection pump with a pressure capability of 250–300 psi when pumping a grout slurry mixed to a 12-s flow cone time. Furnish an injection pump that can continuously pump at rates as low as 1.5 gal/min. **Vertical Movement Testing**: **Drilling and Subsealing**: **Radial Cracks**: **Hole Patching**: Agency should specify drill hole fill material. **Weather Conditions**: **Unanticipated Conditions**: **Resealing Pavement Joints**:
Michigan DOT	Not available.
Minnesota DOT	Not available.
Missouri DOT	Not available.
Texas DOT	Not available.
Virginia DOT	Not available.
Washington State DOT	Not available.

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 557 "Description": Partial-Depth Patching

Agency/Organization	Specification Section: Description
AASHTO (Section 557)	"Construct partial-depth patches of spalls, potholes, and corner breaks in portland cement concrete pavements."
Michigan DOT (Section 603) Concrete Pavement Restoration	"Restore pavement condition." "Concrete pavement restoration will include, but not be limited to: (1) Repairing portions of a concrete pavement with reinforced and nonreinforced Portland cement concrete and with the type of joint specified, (2) Diamond grinding Portland cement concrete pavement, (3) Resawing and sealing existing longitudinal pavement joints, and (4) Sawing, cleaning, and sealing cracks in concrete pavements."
Minnesota DOT	Does not have a specific related specification.
Missouri DOT	Does not have a specific related specification.
Texas DOT	Does not have a specific related specification.
Virginia DOT	Does not have a specific related specification.
Washington State DOT (Section 5-01.3(5))	Partial Depth Spall Repair

AASHTO Specification Designation 557 "Materials": Partial-Depth Patching

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>AASHTO (Section 557)</td><td>AASHTO references to Subsection 557.02, which lists
<table>
<tr><th>Material</th><th>AASHTO Subsection</th></tr>
<tr><td>Portland cement</td><td>701.02</td></tr>
<tr><td>Coarse aggregate for concrete</td><td>703.01(B)</td></tr>
<tr><td>Masonry mortar aggregate</td><td>703.13</td></tr>
<tr><td>Chemical admixtures</td><td>713.03(B)</td></tr>
<tr><td>Water</td><td>714.01(A)</td></tr>
<tr><td>Calcium chloride</td><td>714.02</td></tr>
<tr><td>Rapid-setting patching materials</td><td>Approved List</td></tr>
<tr><td>Fine aggregate for epoxy concrete</td><td>Gradation specified by manufacturer</td></tr>
</table></td></tr>
<tr><td>Michigan DOT (Section 603)</td><td>For concrete repairs, the type of mix to be used is based on time from casting to traffic opening as follows:
<table>
<tr><th>Time from Casting to Traffic Opening</th><th>Grade of Concrete</th></tr>
<tr><td>≤8 h</td><td>Type P-MS</td></tr>
<tr><td>12–72 h</td><td>Type P-NC</td></tr>
<tr><td>3 days</td><td>Grade HE</td></tr>
<tr><td>≥7 days</td><td>Grade P1</td></tr>
</table></td></tr>
<tr><td>Minnesota DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Missouri DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Texas DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Virginia DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Washington State DOT (Sections 5-01.3(1)A and 5-01.3(5))</td><td>The Contractor shall use either concrete patching materials or portland cement concrete for the rehabilitation of cement concrete pavement. Concrete patching materials shall be used for spall repair and dowel bar retrofitting and may be used for concrete panel replacement; portland cement concrete is only allowed for concrete panel replacement.</td></tr>
</table>

AASHTO Specification Designation 557 "Construction": Partial-Depth Patching

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>AASHTO (Section 557)</td><td>All construction-related items are as follows:
<table>
<tr><td>Concrete Mix Design for Patches</td><td>Provide one of the following concrete designs for partial-depth and full-depth patches, as specified in the contract:
1. Accelerated strength portland cement concrete patch mixtures: Use Type I or Type III portland cement to provide concrete with a minimum strength of 3,000 psi in 24 h.
2. Normal set portland cement concrete patch mixture
3. Rapid-setting patching materials: Rapid-setting patching materials must reach a minimum compressive strength of 3,000 psi in 24 h.
4. Epoxy resin patching mortars: Use only Agency-approved materials. Prepare epoxy resin patching mortars according to the manufacturer's recommendations.</td></tr>
<tr><td>Preparation of Partial-Depth Patch Area</td><td>Construct partial-depth patches at specified locations or as directed by the Engineer. Make a vertical saw-cut around the perimeter of the patch area to a minimum depth of 2 in. Use pneumatic tools to remove concrete within the patch area to a minimum depth of 2 in. until sound and clean concrete is exposed. If the depth of the repair exceeds 4 in., remove the entire area to full depth and replace as specified in AASHTO Section 558 ("Full Depth Patching"). Limit the maximum size of pneumatic hammers to 30 lb. Sandblast exposed concrete faces to remove loose particles, oil, dust, traces of asphalt concrete, and other contaminates before patching. Remove sandblasting residue before placing the bonding agent.</td></tr>
<tr><td>Placing Patch Material</td><td>Place and consolidate the patch mixture to eliminate voids at the interface of the patch and existing concrete. If a partial-depth repair area joins a working joint, use an insert, or other bond-breaking medium, to maintain working joints or cracks. Form the new joint to the same width as the existing joint or crack.

Details are contained in AASHTO Section 557 that are applicable for each of the concrete mix designs noted above.</td></tr>
</table>
</td></tr>
</table>

continued

AASHTO Specification Designation 557 “Construction”: Partial-Depth Patching (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>Michigan DOT (Section 603)</td><td>Relevant construction-related items are as follows:
<table>
<tr><td>Size of Patches</td><td>Make repairs 6 ft or longer. When the area to be repaired leaves a section of pavement less than 6 ft from an existing joint or less than 15 ft from the next area to be repaired, remove that section also. For repairs more than 15 ft long, cast the repair area in adjacent lanes, ramps, or shoulders separately.</td></tr>
<tr><td>Placing Concrete</td><td>Place concrete the same day that the existing pavement is removed. Immediately before the concrete placement, wet the faces of the existing pavement and the surface of the aggregate base with water.</td></tr>
<tr><td>Opening to Traffic</td><td>The repair areas may be opened to traffic when the new concrete has attained a flexural strength of 300 psi and all joints have been sawed.</td></tr>
</table></td></tr>
<tr><td>Minnesota DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Missouri DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Texas DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Virginia DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Washington State DOT (Section 5-01.3(5))</td><td>1. If jackhammers are used for removing pavement, they shall not weigh more than 30 lb, and chipping hammers shall not weigh more than 15 lb. All power-driven hand tools used for the removal of pavement shall be operated at angles less than 45° as measured from the surface of the pavement to the tool.
2. The patch limits shall extend beyond the spalled area a minimum of 3.0 in. Repair areas shall be kept square or rectangular. Repair areas that are within 12.0 in. of another repair area shall be combined.
3. A vertical saw-cut shall be made to a minimum depth of 2.0 in. around the area to be patched. The Contractor shall remove material within the perimeter of the saw-cut to a depth of 2.0 in., or to sound concrete. The surface patch area shall be sandblasted and all loose material removed. All sandblasting residue shall be removed using dry oil-free air.
4. Spall repair shall not be done in areas where dowel bars are encountered.
5. When a partial-depth repair is placed directly against an adjacent longitudinal joint, a bond-breaking material such as polyethylene film, roofing paper, or other material as approved by the Engineer shall be placed between the existing concrete and the area to be patched.
6. Patches that abut working transverse joints or cracks require placement of a compressible insert. The new joint or crack shall be formed to the same width as the existing joint or crack. The compressible joint material shall be placed into the existing joint 1.0 in. below the depth of repair. The compressible insert shall extend at least 3.0 in. beyond each end of the patch boundary.
7. Patches that abut the lane/shoulder joint require placement of a formed edge, along the slab edge, even with the surface. The patching material shall be mixed, placed, consolidated, finished, and cured according to manufacturer’s recommendations.</td></tr>
</table>

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 558 "Description": Full-Depth Patching

Agency/Organization	Specification Section: Description
AASHTO (Section 558)	"Construct full-depth patches of portland cement concrete pavement."
Michigan DOT (Section 603)	Refer to AASHTO 557 summary.
Minnesota DOT	Does not have a specific related specification
Missouri DOT	Does not have a specific related specification
Texas DOT (Item 361)	"Repair concrete pavement to full depth."
Virginia DOT	Does not have a specific related specification
Washington State DOT (Section 5-01.3(4))	Replace Portland Cement Concrete Panel

AASHTO Specification Designation 558 "Materials": Full-Depth Patching

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>AASHTO (Section 558)</td><td>AASHTO references to Subsection 558.02, which lists
<table>
<tr><th>Material</th><th>AASHTO Subsection</th></tr>
<tr><td>Portland cement</td><td>701.02</td></tr>
<tr><td>Aggregate for untreated base course</td><td>703.03</td></tr>
<tr><td>Reinforcing steel</td><td>711.01</td></tr>
<tr><td>Chemical admixtures</td><td>713.03(B)</td></tr>
<tr><td>Fly ash</td><td>713.03(C)(1)</td></tr>
<tr><td>Calcium chloride</td><td>714.02</td></tr>
<tr><td>Epoxy resin adhesives</td><td>AASHTO M235</td></tr>
</table></td></tr>
<tr><td>Michigan DOT (Section 603)</td><td>For concrete repairs, the type of mix to be used is based on time from casting to traffic opening as follows:
<table>
<tr><th>Time from Casting to Traffic Opening</th><th>Grade of Concrete</th></tr>
<tr><td>≤8 h</td><td>Type P-MS</td></tr>
<tr><td>12–72 h</td><td>Type P-NC</td></tr>
<tr><td>3 days</td><td>Grade HE</td></tr>
<tr><td>≥7 days</td><td>Grade P1</td></tr>
</table></td></tr>
<tr><td>Minnesota DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Missouri DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Texas DOT (Item 361)</td><td>The following materials-related items apply:
<table>
<tr><td>Hydraulic Cement Concrete for Pavement</td><td>If the time allowed for opening to traffic is less than 72 h following concrete placement, provide Class HES concrete designed to attain a minimum average flexural strength of 255 psi or a minimum average compressive strength of 1,800 psi within the designated time frame. Otherwise provide Class P concrete conforming to Item 360, "Concrete Pavement." Type III cement is permitted for Class HES concrete.</td></tr>
<tr><td>Asphalt Concrete</td><td>If required, furnish asphalt concrete material for overlay and asphalt shoulder repair in accordance with Item 340, "Dense-Graded Hot-Mix Asphalt (Method)."</td></tr>
</table></td></tr>
<tr><td>Virginia DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Washington State DOT (Section 5-01.3(4))</td><td>Portland cement concrete is only allowed for concrete panel replacements (as opposed to patching materials).</td></tr>
</table>

AASHTO Specification Designation 558 "Construction": Full-Depth Patching

Agency/Organization	Specification Section: Construction	
AASHTO (Section 558)	All construction-related items are as follows:	
	Concrete Mix Design for Patches	Provide one of the following concrete designs for partial-depth and full-depth patches, as specified in the contract: 1. Accelerated strength portland cement concrete patch mixtures: Use Type I or Type III portland cement to provide concrete with a minimum strength of 3,000 psi in 24 h. 2. Normal set portland cement concrete patch mixture 3. Rapid-setting patching materials: Rapid-setting patching materials must reach a minimum compressive strength of 3,000 psi in 24 h. 4. Epoxy resin patching mortars: Use only Agency-approved materials. Prepare epoxy resin patching mortars according to the manufacturer's recommendations.
	Preparation of Patch Area	Repair in accordance with specified full-depth patching requirements for the following pavement types: 1. Mesh-reinforced, plain-doweled, and plain-jointed pavement 2. Continuously reinforced concrete 3. Detailed patching requirements are provided in AASHTO Section 558.03(C).
Michigan DOT (Section 603)	Refer to AASHTO 557 summary.	
Minnesota DOT	Does not have a specific related specification.	
Missouri DOT	Does not have a specific related specification.	
Texas DOT (Item 361)	Construction-related items are as follows:	
	Repair Area	Make repair areas rectangular, at least 6 ft long and at least half a full lane in width unless otherwise shown on the plans.
	Repair Process Steps	1. Saw-cut full depth through the concrete around the perimeter of the repair area before removal. 2. Schedule work so that concrete placement follows full-depth saw-cutting by no more than 7 days unless otherwise shown on the plans or approved. 3. Remove or repair loose or damaged base material, and replace or repair it with approved base material to the original top of base grade. Place a polyethylene sheet at least 4 mils thick as a bond breaker at the interface of the base and new pavement. Allow concrete used as base material to attain sufficient strength to prevent displacement when placing pavement concrete. 4. Broom finish the concrete surface unless otherwise shown on the plans.
Virginia DOT	Does not have a specific related specification.	

continued

AASHTO Specification Designation 558 "Construction": Full-Depth Patching (*continued*)

Agency/Organization	Specification Section: Construction
Washington State DOT (Section 5-01.3(4))	1. Concrete slabs to be replaced as shown in the Plans shall be at least 6.0 ft long and the full width of an existing pavement panel. The portion of the panel to remain in place shall have a minimum dimension of 6 ft in length and full panel width; otherwise the entire panel shall be removed and replaced. 2. There shall be no new joints closer than 3.0 ft to an existing transverse joint or crack. 3. A vertical full-depth saw-cut is required along all longitudinal joints and at transverse locations and, unless the Engineer approves otherwise, an additional vertical full-depth relief saw-cut located 12 to 18 in. from and parallel to the initial longitudinal and transverse saw-cut locations is also required. 4. Removal of existing cement concrete pavement shall not cause damage to adjacent slabs that are to remain in place. 5. In areas that will be ground, slab replacements shall be performed prior to pavement grinding. When new concrete pavement is to be placed against existing cement concrete pavement.

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 560 "Description": Diamond Grinding Concrete Pavement

Agency/Organization	Specification Section: Description
AASHTO (Section 560)	"Grind and texture existing portland cement concrete pavement longitudinally using a diamond grinder."
Michigan DOT (Section 603) Concrete Pavement Restoration	"Restore pavement condition." "Concrete pavement restoration will include, but not be limited to: (1) Repairing portions of a concrete pavement with reinforced and nonreinforced Portland cement concrete and with the type of joint specified, (2) Diamond grinding Portland cement concrete pavement, (3) Resawing and sealing existing longitudinal pavement joints, and (4) Sawing, cleaning, and sealing cracks in concrete pavements."
Minnesota DOT	Has related specifications for new construction but not a full specific diamond-grinding specification.
Missouri DOT	Has related specifications for new construction but not a full specific diamond-grinding specification.
Texas DOT (Item 585)	"Measure and evaluate the ride quality of pavement surfaces."
Virginia DOT	No specific specification.
Washington State DOT (Section 5-01.3(9))	Portland cement concrete pavement grinding.

AASHTO Specification Designation 560 "Materials": Diamond Grinding Concrete Pavement

Agency/Organization	Specification Section: Materials
AASHTO (Section 560)	There are no materials requirements in AASHTO Section 560.
Michigan DOT (Section 603)	There are no materials requirements in Michigan DOT Section 603 for diamond grinding.
Minnesota DOT	Not applicable.
Missouri DOT	Not applicable.
Texas DOT	There are no relevant materials requirements in TxDOT Item 585.
Virginia DOT	No specific specification
Washington State DOT (Section 5-01.3(9))	There are no relevant materials requirements in WSDOT Section 5-01.3(9).

AASHTO Specification Designation 560 "Construction": Diamond Grinding Concrete Pavement

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>AASHTO (Section 560)</td><td>Construction-related items are as follows:
<table>
<tr><td>Diamond Grinding and Texture</td><td>1. Uniformly grind and texture the entire pavement surface area until the surface on both sides of the transverse joints and all cracks are in the same plane and meet the required smoothness. Exclude shoulders.
2. Begin and end grinding from locations normal to the pavement centerline.
3. Texture: Provide the surface of the ground pavement with a corduroy-type texture consisting of parallel grooves between $^{3}/_{32}$ and $^{5}/_{32}$ in. wide, with a distance between the grooves of $^{1}/_{16}$ to $^{1}/_{8}$ in. and a difference between the peaks of the ridges and the bottom of the grooves of ____ in.</td></tr>
<tr><td>Equipment</td><td>1. Furnish a self-propelled grinding machine with diamond blades mounted on a multiblade arbor and a minimum cutting head width of 3 ft.</td></tr>
<tr><td>Tolerances</td><td>1. After the Contractor completes grinding and texturing, the Engineer will test the pavement surface for smoothness to ensure it meets the surface tolerance for new pavement specified in AASHTO Subsection 401.03(K)(1). Grind the adjacent shoulders or pavement to provide the required cross slope for drainage.
2. Provide a uniform pavement cross slope without depressions or misalignment of slope greater than ____in. in ____ft when tested by stringline or straightedge placed perpendicular to the centerline.</td></tr>
</table></td></tr>
<tr><td>Michigan DOT (Section 603)</td><td>Relevant construction-related items are as follows:
<table>
<tr><td>Faulted Pavement</td><td>Faulted areas at transverse cracks and joints in excess of $^{1}/_{16}$ in. after initial grinding must be reground until faulting is less than $^{1}/_{16}$ in.</td></tr>
<tr><td>Texture</td><td>Grind to a parallel corduroy-type texture consisting of grooves $^{1}/_{16}$ to $^{1}/_{8}$ in. wide, $^{1}/_{16}$ in. deep, and $^{1}/_{16}$ to $^{1}/_{8}$ in. on center. Grind to a finished uniform texture. Make the transverse slope of the pavement uniform with no depressions or misalignment of slope greater than $^{1}/_{8}$ in. when checked with a 10-ft straightedge.</td></tr>
</table></td></tr>
<tr><td>Minnesota DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Missouri DOT</td><td>Does not have a specific related specification.</td></tr>
<tr><td>Texas DOT (Item 361)</td><td>Relevant construction-related items are as follows:
<table>
<tr><td>Equipment</td><td>When grinding is required, provide self-propelled powered grinding equipment that is specifically designed to smooth and texture pavements using circular diamond blades. Provide equipment with automatic grade control capable of grinding at least 3 ft of width longitudinally in each pass without damaging the pavement.</td></tr>
</table></td></tr>
</table>

continued

AASHTO Specification Designation 560 "Construction": Diamond Grinding Concrete Pavement (*continued*)

Agency/Organization	Specification Section: Construction
Virginia DOT	Does not have a specific related specification.
Washington State DOT (Section 5-01.3(9))	1. The pavement shall be ground in a longitudinal direction beginning and ending at lines normal to the pavement centerline. The minimum overlap between longitudinal passes shall be 2.0 in. Ninety-five percent of the surface area of the pavement to be ground shall have a minimum of $^1/_8$ in. removed by grinding. 2. The final surface texture shall be uniform in appearance with longitudinal corduroy-type texture. The grooves shall be between $^3/_{32}$ and $^5/_{32}$ in. wide, and no deeper than $^1/_{16}$ in. The land area between the grooves shall be between $^1/_{16}$ and $^1/_8$ in. wide.

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 561 "Description": Milling Pavement

Agency/Organization	Specification Section: Description
AASHTO (Section 561)	"Strip pavement by a cold milling process before resurfacing."
Michigan DOT	Limited specification information.
Minnesota DOT (Section 2232)	"This work shall consist of improving the profile, cross slope, and surface texture of an existing pavement surface by machine (cold) milling preparatory to placement of another course thereon."
Missouri DOT	Limited specification information.
Texas DOT (Item 585)	Limited specification information.
Virginia DOT	Limited specification information.
Washington State DOT	Limited specification information.

AASHTO Specification Designation 561 "Materials": Milling Pavement

Agency/Organization	Specification Section: Materials
AASHTO (Section 561)	There are no materials requirements in AASHTO Section 561.
Michigan DOT (Section)	Not applicable.
Minnesota DOT	There are no materials requirements in MnDOT Section 2232.
Missouri DOT	Not applicable.
Texas DOT	Not applicable.
Virginia DOT	Not applicable.
Washington State DOT	Not applicable.

AASHTO Specification Designation 561 "Construction": Milling Pavement

Agency/Organization	Specification Section: Construction	
AASHTO (Section 561)	Construction-related items are as follows:	
	Milling Setup	1. Mill the surface in a longitudinal direction. For the initial pass, use as a reference the curb, longitudinal edge of pavement, or a string attached to the pavement surface. Furnish a milling machine with a steering guide or reference that allows the operator to follow the guidance reference within 2 in. When milling next to previously milled pavement, use the edge of the milled trench as the longitudinal reference for succeeding passes. 2. Provide a milled surface with a uniform texture free of excessive gouges, ridges, and grooves. 3. Provide an end transition on a 4:1 (1:4) slope to the existing pavement surface at each end of the milling work each day. End the milling passes as close to each other as practical. Do not leave longitudinal joints more than 2 in. deep exposed during nonworking hours.
Michigan DOT	Not applicable.	
Minnesota DOT (Section 2232)	Construction-related items are as follows:	
	Equipment	1. Pavement milling shall be accomplished with a power-operated, self-propelled cold-milling machine capable of removing concrete and bituminous surface material as necessary to produce the required profile, cross slope, and surface texture uniformly across the pavement surface. The machine shall also be equipped with means to control dust and other particulate matter created by the cutting action. 2. The machine shall be equipped to accurately and automatically establish profile grades along each edge of the machine, within plus or minus $\frac{1}{8}$ in., by referencing from the existing pavement by means of a ski or matching shoe, or from an independent grade control. The machine shall be controlled by an automatic system for controlling grade, elevation, and cross slope at a given rate.
Missouri DOT	Does not have a specific related specification.	
Texas DOT	Does not have a specific related specification.	
Virginia DOT	Does not have a specific related specification.	
Washington State DOT	Does not have a specific related specification.	

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 563 "Description": Portland Cement Concrete Unbonded Overlays

Agency/Organization	Specification Section: Description
AASHTO (Section 563)	"Place portland cement concrete unbonded overlays, including pavement patching of existing surface, applying a bond breaker, repairing curb, and disposing of removed material."
Michigan DOT (Sections 601 and 602)	"Construct a jointed Portland cement concrete pavement, unbonded overlay, base course, or shoulder, with or without reinforcement." Both MDOT Sections 601 (Portland Cement Concrete Pavements) and 602 (Concrete Pavement Construction) were reviewed.
Minnesota DOT (Section 2301)	No specific specification for PCC unbonded overlays. Presumably Section 2301 applies and a summary of Section 2301 is included.
Missouri DOT (Sections 506.20 and 506.30)	[506.20] "This work shall consist of placing an interlayer material on an existing concrete pavement and constructing an unbonded concrete overlay in accordance with the details and locations shown on the plans. The standard unbounded concrete overlay design thickness is either 8 or 5 inches. The eight-inch overlays are constructed similarly to new concrete pavement in terms of joint spacing and use of dowel bars and tiebars. The five-inch overlays are sawed into smaller panels and require no steel. The overlay shall be placed in accordance with Section 502, except as herein stated." [506.30] "This work shall consist of constructing an unbonded concrete overlay on an existing asphalt surface in accordance with the details and locations shown on the plans. All work shall be performed in accordance with Section 506.20, except that an interlayer shall not be used."
Texas DOT	No specific specification for PCC unbonded overlays.
Virginia DOT	No specific specification for PCC unbonded overlays.
Washington State DOT	No specific specification for PCC unbonded overlays.

AASHTO Specification Designation 563 "Materials": Portland Cement Concrete Unbonded Overlays

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>AASHTO (Section 563)</td><td>Major materials-related items
<table>
<tr><td>Portland Cement</td><td>1. AASHTO Subsection 701.02. Meets AASHTO M85
2. Use only Type I or Type II cement</td></tr>
<tr><td>Asphalt Cements</td><td>AASHTO Subsection 702.01(A). Meets AASHTO M320</td></tr>
<tr><td>Asphalt Concrete</td><td>Place a uniform layer to a minimum depth of 1 in.</td></tr>
<tr><td>Curing Materials</td><td>AASHTO Subsection 713.02 includes three options:
1. Burlap cloth (AASHTO M182)
2. Sheet materials (AASHTO M171)
3. Liquid-membrane-forming compounds (AASHTO M148)</td></tr>
<tr><td>Water</td><td>AASHTO Subsection 714.01(A). Meets AASHTO M157.</td></tr>
<tr><td>Reinforcing Steel</td><td>Use deformed epoxy-coated bars.</td></tr>
</table></td></tr>
</table>

continued

AASHTO Specification Designation 563 "Materials": Portland Cement Concrete Unbonded Overlays (*continued*)

Agency/Organization	Specification Section: Materials	
Michigan DOT (Section 601)	Major materials-related items	
	Cement	Section 901
	GGBFS	Section 901
	Fly Ash	Section 901
	Coarse Aggregate	Section 902
	Fine Aggregate	Section 902
	Concrete Admixtures	Section 903
	Water	Section 911
	Certified Batch Plants	Supply portland cement concrete from certified portable and stationary concrete batch plant facilities meeting the requirements of the National Ready Mixed Concrete Association (NRMCA) certification program for automatic control and automatic systems. When no fully automated NRMCA certified facility is within 25 mi of the project limits, the Engineer may waive NRMCA certification and/or automation requirements.
	Additional Water at Placement Site	Do not add more water than the approved concrete mix design will allow based on maximum water content and maximum water/cementitious material ratio.
	Concrete Placing Temperature	Concrete must be between 45°F and 90°F at the time it is placed.
	Air Content	At the time of placement, concrete must have 6.5% ± 1.5% entrained air. However, concrete furnished for slipform placement and having a slump of 1.5 in. or less may have a minimum of 4.5% entrained air.

continued

AASHTO Specification Designation 563 "Materials": Portland Cement Concrete Unbonded Overlays (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Materials</th></tr>
<tr><td>Minnesota DOT
(Section 2301)</td><td>Major materials-related items
<table>
<tr><td>Minimum Cementitious Content</td><td>530 lb/yd³ with a minimum of portland cement = 400 lb/yd³ when using fly ash or GGBFS.</td></tr>
<tr><td>Total Alkalis in Portland Cement</td><td>0.60%</td></tr>
<tr><td>Total Alkalis in Cementitious Material</td><td>≤5 lb/yd³</td></tr>
<tr><td>Water/Cement Ratio</td><td>The target w/c ratio is 0.40 for large paving projects (>5,000 yd³). Incentives and disincentives associated with lower or higher w/c ratios are shown below:
<table>
<tr><th>Mean Value of w/c (termed QI)</th><th>Payment Incentive or Disincentive per Cubic Yard ($/yd³)</th></tr>
<tr><td>≤0.35</td><td>+4.00</td></tr>
<tr><td>0.36</td><td>+3.00</td></tr>
<tr><td>0.37</td><td>+2.00</td></tr>
<tr><td>0.38</td><td>+1.25</td></tr>
<tr><td>0.39</td><td>+0.0</td></tr>
<tr><td>0.4</td><td>0.00</td></tr>
<tr><td>0.41</td><td>–0.50</td></tr>
<tr><td>0.42</td><td>–1.25</td></tr>
<tr><td>0.43</td><td>–2.00</td></tr>
<tr><td>0.44</td><td>–3.00</td></tr>
<tr><td>≥0.45</td><td>Determined by the Concrete Engineer</td></tr>
</table>
</td></tr>
</table>
</td></tr>
<tr><td>Missouri DOT
(Section 506.20)</td><td>
<table>
<tr><td>Interlayer</td><td>The interlayer material shall be a minimum of 1-in.-thick new bituminous....</td></tr>
<tr><td>Concrete</td><td>Materials for an unbonded overlay shall be in accordance with MoDOT Section 502. That information states that all material for the concrete shall conform to Section 501.</td></tr>
</table>
</td></tr>
<tr><td>Texas DOT</td><td>No specific specification for PCC unbonded overlays.</td></tr>
<tr><td>Virginia DOT</td><td>No specific specification for PCC unbonded overlays.</td></tr>
<tr><td>Washington State DOT</td><td>No specific specification for PCC unbonded overlays.</td></tr>
</table>

AASHTO Specification Designation 563 "Construction": Portland Cement Concrete Unbonded Overlays

Agency/Organization	Specification Section: Construction
AASHTO (Section 563)	Major construction-related items

Surface Preparation and Pavement Patching	1. Patching pavement: Fill deep spalls with asphalt concrete or concrete before placing the interlayer treatment. 2. Full-depth removal and patching: Remove pavement full depth or stabilize as specified in AASHTO Section 558. Construct full-depth patches before placing the overlay.
Placing and Finishing Concrete	Concrete overlay: Concrete must meet AASHTO Subsection 501.03. Subsection 501.03 recap follows: 1. Mix design options

Property	Value	AASHTO Test Method
Compressive Strength (min)	3,500 psi	T22
Flexural Strength (min)	550 psi	T97
Flexural Strength (min)	650 psi	T177
Slump	3/8–3 in.	T119
Cement Content Without Air (min) With Air (min)	 564 lb/yd^3 598 lb/yd^3	
Fly Ash Type C Type F	 30% max[1] 25% max[1]	Note 1: % max cement replacement
GGBFS	50% max[1]	See Note 1
Water/Cementitious Ratio Without Air (max) With Air (max)	 0.53 0.49	
Entrained Air	5%–8%	T152, T196, or T199

2. Mixing and placing limitations
    a. Stop mixing and concreting operations if shaded ambient air temperature away from artificial heat is 40°F or less. Resume operations only when the ambient air temperature is 40°F and rising.
    b. Place mixed concrete only when its temperature is between 50°F and 85°F.

continued

AASHTO Specification Designation 563 "Construction": Portland Cement Concrete Unbonded Overlays (*continued*)

Agency/Organization	Specification Section: Construction		
AASHTO (Section 563) (*continued*)	Major construction-related items (*continued*)		
	Placing and Finishing Concrete (*continued*)	3. Longitudinal joints	
		Dimensions	Saw the first cut or insert the joint material to one-third of the depth.
		Tiebars	Place [30-in.]-long No. 5 tiebars of Grade 60 steel, spaced [30 in.] center-to-center to one-half of the depth of the PCCP. Ensure that tiebars are placed perpendicular to the face of the joint, centered in the slab depth, and parallel to the finished surface.
		Construction	Form or saw longitudinal joints in the plastic concrete. Saw the joints within 4–24 h after placing the concrete and immediately after completing the transverse joints. Allow only the saw on the pavement during sawing operations.
		Sealing	Seal joints after the curing period and before opening the pavement to traffic. Use sandblasting followed by an oil-free air jet to clean the faces and joint openings before sealing. Seal joints only when they are completely dry. Do not dry joints with a heat lance. Use an approved backer rod to seal the lower portion of the joint groove to a uniform depth to prevent sealant from entering beneath the specified depth. Ensure that backer rod is compatible with the sealant type specified and install according to manufacturers' recommendations.

continued

AASHTO Specification Designation 563 "Construction": Portland Cement Concrete Unbonded Overlays (*continued*)

Agency/Organization	Specification Section: Construction		
AASHTO (Section 563) (*continued*)	Major construction-related items (*continued*)		
	Placing and Finishing Concrete (*continued*)	4. Contraction joints	
		Location and Dimensions	Form or saw joints as narrowly as possible, to at least one-third of the pavement depth.
		Load Transfer	Install load transfer dowel bars of specified grade and size, spaced at […] centers, and secured with a wire basket or implanted mechanically. Place dowel bars one-half of the depth parallel to the surface and pavement edge to an alignment tolerance of [±¼ in.]. Vibrate concrete around all dowel bars without misaligning them.
		Construction	Place formed joints while the concrete is plastic. Begin relief-cut joint sawing immediately after the concrete hardens to the stage that it can be sawed without raveling. Saw all joints between 4 and 24 h after placing concrete but before uncontrolled shrinkage cracking develops.
		Sealing	Similar to longitudinal joint construction.
		5. Transverse construction joints: Install transverse construction joints at the end of each day's placement. Form bulkheads when stopping the placement in an emergency or at the end of each day's pour.	

continued

AASHTO Specification Designation 563 "Construction": Portland Cement Concrete Unbonded Overlays (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td rowspan="4">AASHTO (Section 563)
(continued)</td><td colspan="2">Major construction-related items
(continued)</td></tr>
<tr><td>Placing and Finishing Concrete (continued)</td><td>
6. Surface tolerances: AASHTO provides for two profile measurement methods:
a. Straightedge: This method applies to all paving. Test the surface with a 10-ft straightedge at random locations. The Engineer will identify pavement areas that deviate more than [3/16 in.] from the straightedge as defective work.
b. Profilograph: Describes a California-type profilograph.
7. Curing
a. Cure the concrete for at least 3 days immediately after the finishing operation.
b. Protect the concrete for at least 10 days or until the concrete achieves a compressive strength of [2,200 psi].
8. Tolerance and price adjustments for pavement thickness: Price adjustments in accordance with the table below:
<table>
<tr><th>Deficiency in Thickness as Determined by Cores (in.)</th><th>Contract Price Allowed</th></tr>
<tr><td>0–0.20</td><td>100</td></tr>
<tr><td>0.21–0.30</td><td>80</td></tr>
<tr><td>0.31–0.40</td><td>72</td></tr>
<tr><td>0.41–0.50</td><td>68</td></tr>
<tr><td>0.51–0.75</td><td>57</td></tr>
<tr><td>0.76–1.00</td><td>50</td></tr>
<tr><td>>1.00</td><td>Remove and Replace</td></tr>
</table>
</td></tr>
<tr><td>Opening to Traffic</td><td>Compressive strength—opening to traffic: Do not open the overlay to traffic or construction equipment until the concrete bonded overlay attains a minimum compressive strength of 3,500 psi and all joints have been cleaned and filled with joint material.</td></tr>
<tr><td>Test Properties</td><td>Slump: Use concrete having a maximum slump, as determined of 3 in. for concrete placed by vibration or 4 in. for hand-placed concrete.</td></tr>
</table>

continued

AASHTO Specification Designation 563 "Construction": Portland Cement Concrete Unbonded Overlays (*continued*)

Agency/Organization	Specification Section: Construction	
Michigan DOT (Section 602)	Major construction-related items	
	Surface Texture	When the pavement has set sufficiently to maintain texture, drag the surface longitudinally using one or two layers of an approved damp fabric material. Maintain fabric contact with the surface across the entire width of concrete being placed. Immediately after dragging, groove all surfaces other than concrete base courses and shoulders. Orient the grooves generally perpendicular to the centerline and form the grooves in the plastic concrete cleanly without slumping of the edges or severe tearing of the surface. Provide a surface texture consisting of ⅛-in.-wide grooves spaced ½ in. on center and ⅛ to ¼ in. deep.
	Sealing Joints with Hot-Poured Sealants	Seal the joints immediately after the joints are cleaned. Joint surfaces must be dry when sealed. Do not place sealant when temperature is less than 50°F.
	Profile	While the concrete is still plastic, test the slab surface for compliance with the required grade and cross section using a 10-ft straightedge. If high or low spots exceed ⅛ in. over 10 ft, suspend paving operations and correct the finishing procedures. Correct high or low spots in pavements that exceed the tolerances.
	Weather and Temperature Limitations	1. Protect the concrete from freezing until the concrete has attained a compressive strength of at least 1,000 psi. 2. Do not place concrete if portions of the base, subbase, or subgrade layer are frozen, or if the grade exhibits poor stability from excessive moisture levels. 3. Do not place concrete when the temperature of the plastic concrete at the point of placement is above 90°F.

continued

AASHTO Specification Designation 563 "Construction": Portland Cement Concrete Unbonded Overlays (*continued*)

Agency/Organization	Specification Section: Construction	
Minnesota DOT (Section 2301)	Major construction-related items	
	High-Early Strength Concrete	High-early concrete is defined as a concrete mixture having a cementitious content greater than 600 lb/yd^3. High-early mixes shall be designed to provide a maximum water/cementitious ratio of 0.40 and a minimum flexural strength of 500 psi or a minimum compressive strength of 3,000 psi in 48 h. High-early mixes may have up to 100% portland cement. High-early mixes are not eligible for incentive payments for water/cementitious ratio.
	Minimum Strength Requirements for Opening Pavements to Construction and General Public Traffic	New pavement shall be closed to use by construction and general public traffic for 7 days or according to the values listed in the table below, whichever is the shorter.
	Placing Concrete	1. All main line pavement constructed by standard or vibratory machine placement methods shall be constructed in a single layer of concrete. 2. Water shall not be added to the surface of the concrete to aid in finishing without the approval of the Engineer. The Engineer will only give this approval to replace evaporated surface water directly behind the paver caused by a halt in forward progress from a short-term breakdown in equipment or supply of concrete. 3. Should placement of concrete be temporarily suspended, the placement operations shall be resumed in such manner that will not result in a cold joint or honeycombing. If the suspension period exceeds 90 min, a standard header joint shall be constructed.
	Joint Construction	Initial joint sawing shall be approximately ⅛ in. wide and to the full joint depth. The initial sawing shall be accomplished as soon as the condition of the concrete will permit without raveling and before random cracking occurs. The sequence of initial sawing shall be at the Contractor's option.

Table within Minimum Strength Requirements for Opening Pavements to Construction and General Public Traffic:

Slab Thickness (in.)	Flexural Strength (psi)
6.0	500
6.5	500
7.0	500
7.5	480
8.0	460
8.5	440
9.0	390
9.5	350
10.0	350
≥10.5	350

continued

AASHTO Specification Designation 563 "Construction": Portland Cement Concrete Unbonded Overlays (*continued*)

Agency/Organization	Specification Section: Construction	
Minnesota DOT (Section 2301) (*continued*)	Major construction-related items (*continued*)	
	Surface Finish	MnDOT uses a standard longitudinal carpet drag followed by transverse tining.
	Concrete Curing	The Contractor shall (1) Cure and protect the concrete by the blanket curing method or one of the membrane curing methods. (2) Cure the entire pavement surface and edges as soon as surface conditions permit after the finishing operations. (3) Continue curing and protecting the concrete for at least 72 h. (4) Place the curing media on the pavement edges within 30 min after removal of the forms when side forms are used. (5) Extend the minimum curing period to 96 h when fly ash or portland-pozzolan cement substitutions are used.
	Surface Smoothness	The Contractor shall test the pavement surface for surface smoothness and ride quality. Surface Smoothness and Ride Quality shall be measured with a 25-ft California-type profilograph, or a lightweight inertial profiler (IP), which produces a profilogram (profile trace of the surface tested). Either type of device must be certified according to the procedure on file in the MnDOT Concrete Engineering Unit.
	Thickness Requirements	Where the cores show a thickness deficiency exceeding ½ in., but less than 1 in., the pavement represented by those cores will not be excluded from the pay quantities; however, a deduction will be made from the moneys due the Contractor equal to the product of the defective areas and $20.00/yd^2. Pavement represented by cores showing a thickness deficiency of 1 in. or more will be excluded from all payments plus a deduction will be made from the moneys due the Contractor equal to the product of the defective areas and $20.00/yd^2. These deductions will be assessed in lieu of removing and replacing the areas of pavement which are deficient in thickness.

continued

AASHTO Specification Designation 563 "Construction": Portland Cement Concrete Unbonded Overlays (*continued*)

Agency/Organization	Specification Section: Construction	
Missouri DOT (Section 506.20)	Major construction-related items	
	Surface Preparation	All holes greater than 2 in. wide and 1 in. deep in the surface of the traffic lanes, excluding shoulders, shall be filled with patching material and shall be compacted to a flat, tight surface.
	Bituminous Interlayer	The surface temperature of a bituminous interlayer shall not exceed 90°F prior to the overlay placement. The temperature may be controlled with any means approved by the Engineer, including, but not limited to white curing compound and water misting.
	Dowel Bars	Dowel bars for 8-in. unbounded overlays shall be installed the full width of the unbonded overlay and the baskets, if used, shall be firmly anchored to the interlayer surface.
	Tiebars	Tiebars shall be installed between lanes in an 8-in. unbounded concrete overlay.
	Concrete Temperature	The concrete temperature shall not exceed 95°F when delivered to the site.
	Contraction Joints	Sawing of the contraction joints shall not cause excessive raveling. Standard joint spacing for a 5-in. unbounded concrete overlay is 6 ft transversely and longitudinally. Standard joint spacing for an 8-in. unbounded overlay is 15 ft transversely and 12 ft across the full lane width. New transverse joints will not be required to match existing transverse joints. The minimum depth of the sawed joints shall be one-third the pavement thickness and the width of the joint shall be ⅛ in. maximum. The joints shall not be sealed, unless open more than ¼ in. but shall be cleaned of all deleterious material after sawing. Concrete panels with cracking outside of the sawed joints shall be considered unacceptable.
	Opening Strength	The unbounded concrete overlay may be opened for lightweight traffic when the concrete has attained a minimum compressive strength of 2,500 psi. The concrete pavement shall not be opened to all types of traffic until the concrete has attained a minimum compressive strength of 3,000 psi. Compressive strength for opening to traffic shall be determined either by compressive strength tests in accordance with AASHTO T22 or the maturity method.
Texas DOT	No specific specification for PCC unbonded overlays.	
Virginia DOT	No specific specification for PCC unbonded overlays.	
Washington State DOT	No specific specification for PCC unbonded overlays.	

References

AASHTO. "Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, Washington, D.C., 2008.

Michigan Department of Transportation. "Standard Specifications for Construction," Michigan Department of Transportation, Lansing, 2003.

Minnesota Department of Transportation. "Mn/DOT Standard Specifications for Construction," Minnesota Department of Transportation, St. Paul, 2005.

Missouri Department of Transportation. "Missouri Standard Specifications for Highway Construction," Missouri Department of Transportation, Jefferson City, 2004.

Texas Department of Transportation. "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges," Texas Department of Transportation, Austin, 2004.

Virginia Department of Transportation. "Road and Bridge Specifications," Virginia Department of Transportation, Richmond, 2007.

Washington State Department of Transportation. "Standard Specifications for Road, Bridge, and Municipal Construction," M41-10, Washington State Department of Transportation, Olympia, 2010.

AASHTO Specification Designation 567 "Description": Cracking and Seating

Agency/Organization	Specification Section: Description
AASHTO (Section 567)	"Crack existing portland cement concrete pavement and roll the broken concrete until surface material is well-seated before placing an asphalt pavement overlay."
UK Department for Transport Specifications (Section 716)	No general description.
Michigan DOT	No specific specification.
Minnesota DOT	No specific specification.
Missouri DOT	No specific specification.
Texas DOT	No specific specification.
Virginia DOT	No specific specification.
Washington State DOT	No specific specification.

AASHTO Specification Designation 567 "Materials": Cracking and Seating

Agency/Organization	Specification Section: Materials
AASHTO (Section 567)	No materials-related specifications.
United Kingdom (Section 716)	No materials-related specifications.
Michigan DOT	No specific specification.
Minnesota DOT	No specific specification.
Missouri DOT	No specific specification.
Texas DOT	No specific specification.
Virginia DOT	No specific specification.
Washington State DOT	No specific specification.

AASHTO Specification Designation 567 "Construction": Cracking and Seating

Agency/Organization	Specification Section: Construction	
AASHTO (Section 567)	Construction-related items are as follows:	
	Cracking and Seating Equipment	1. Use a device to crack the concrete pavement that exerts a minimum of 12,000 ft-lb of energy with a spade or guillotine-type cracker mounted on a vehicle with controlled forward and transverse movement. Crack the pavement full depth, while maintaining aggregate interlock between the pieces. Do not use any device that causes undue displacement of the concrete or damages drainage facilities, utilities, or other property, or destabilizes the base or subgrade. 2. Seat the cracked concrete with a vibratory roller. 3. Furnish vibratory rollers with separate controls for energy and propulsion. Furnish vibratory rollers with a variable amplitude and frequency system capable of producing a frequency of 2,000 vibrations per minute and meeting the following requirements: • Diameter of drum, 4 ft • Length of drum, 6.5 ft • Unit static force on drum, 125 lb/in. of width • Total applied force on drum, 325 lb/in. of width
	Surface Preparation	Remove existing asphalt patching or overlay before cracking the pavement.
	Test Section	The Engineer will designate test sections to be used before full production cracking operations begin. Crack the test sections using varying energy and striking patterns until a pattern is established that cracks the pavement to the extent required. Use the pattern established to crack the remaining pavement as long as the crack pattern meets the specified size requirements. If the production pattern stops producing cracks to the extent required, use another test section to identify a new successful pattern. Furnish and apply water to dampen the pavement surface after cracking so the extent of breakage can be seen.
	Cracking Operations	1. Perform cracking one lane at a time to produce pieces approximately 1.2–1.8 ft^2 in area. Orient the greatest dimension of the pieces transverse to the pavement centerline. Prohibit cracking within 2.5 ft of any transverse joint or other location. 2. Produce cracks that are continuous without extensive spalling along the crack. Extensive spalling is spalling more than 1 in. deep. Do not shatter the pavement or base during cracking operations. 3. Apply water randomly once each day to the surface to verify the specified extent of breakage. Adjust the energy or striking pattern based on these check sections.

continued

AASHTO Specification Designation 567 "Construction": Cracking and Seating (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td rowspan="3">AASHTO (Section 567)
(continued)</td><td colspan="2">Construction-related items are as follows:
(continued)</td></tr>
<tr><td>Seating Operations</td><td>1. After cracking, roll the concrete to seat firmly and lay the cracked pieces to an even surface. Continue rolling until the surface material is well seated and uniformly compacted.
2. Remove soft spots or rocking pieces detected and undercut unsuitable material as directed. Backfill these areas with crushed aggregate base to the bottom of adjacent portland cement concrete pavement and cover the crushed aggregate base with hot-mix asphalt concrete.
3. Perform rolling only under dry pavement conditions.</td></tr>
<tr><td>Maintenance</td><td>Maintain the pavement according to the traffic control plan if the pavement is opened to traffic after the cracking and seating operation and before placing the first asphalt concrete course. Maintain the pavement for traffic according to the Traffic Control Plan. Perform asphalt concrete pavement construction within two weeks of completing the cracking and seating operations.</td></tr>
<tr><td rowspan="3">UK Department for Transport Specifications (Section 716 and NG 716)
Cracking and Seating of Existing Jointed Unreinforced Concrete Pavements and Hydraulically Bound Mixture Bases</td><td colspan="2">Construction-related items are as follows:</td></tr>
<tr><td>Cracking and Seating Equipment</td><td>1. Layers shall be cracked and seated with plant and equipment to which the Overseeing Organization's consent has been given and shall comply with this clause.
2. Suitable plant with a guillotine action capable of delivering variable preset impact loads to the concrete surface. The plant used to crack the hydraulically bound pavement layer or layers shall be self-propelled and have all wheels fitted with rubber tires.</td></tr>
<tr><td>Surface Preparation</td><td>Any existing asphalt overlay and surfacing shall be removed from the area to be treated for the full width of each lane.</td></tr>
</table>

continued

AASHTO Specification Designation 567 "Construction": Cracking and Seating (*continued*)

<table>
<tr><th>Agency/Organization</th><th colspan="2">Specification Section: Construction</th></tr>
<tr><td rowspan="2">UK Department for Transport Specifications (Section 716 and NG 716)
Cracking and Seating of Existing Jointed Unreinforced Concrete Pavements and Hydraulically Bound Mixture Bases
(continued)</td><td colspan="2">Construction-related items are as follows:
(continued)</td></tr>
<tr><td>Test Section</td><td>1. The test section shall be no less than 250 m^2 nor greater than 420 m^2.
2. The work on the test section shall proceed as follows:
a. Cracking shall proceed in stages as directed by the Overseeing Organization in groups of four to six bays [slabs] in jointed concrete pavements. Each group that is cracked and seated shall be assessed in accordance with clauses contained in the UK specification.
b. In Stage 1 of the main trial the Contractor shall set up his plant and equipment and demonstrate that he can produce the required pattern and quality of transverse cracks in accordance with the UK specification.
c. In Stage 2 and each subsequent stage of the main trial, a group of four bays [slabs] in jointed concrete pavement shall be cracked starting from one end to produce transverse cracks at each of the spacings stated.
d. Seating: After cracking in both Stage 1 and Stage 2, the pavement shall be seated with the number of roller passes specified in the UK specification.
3. Compliance with the cracking and seating requirements for the main trial shall be assessed as follows:
a. The surface pattern of cracking shall be checked before seating but after applying clean water and allow to dry as specified.
b. The depth and the vertical direction of cracking shall be determined by coring through the full depth of the hydraulically bound pavement layer symmetrically at the crack position. Core diameter shall be in accordance with items in the UK specification. In Stage 2 and in subsequent stages of the main trial, the number of cores shall be in accordance with requires in the UK specification. In cases where cracks are not visible in the surface, the locations of cores will be generally within the impact points and transversely in line with the impact points. If any shattering or multiple cracking is present in the extracted core then there is deemed to have been "shattering failure."</td></tr>
</table>

continued

AASHTO Specification Designation 567 "Construction": Cracking and Seating (*continued*)

<table>
<tr><th>Agency/Organization</th><th>Specification Section: Construction</th></tr>
<tr><td>UK Department for Transport Specifications (Section 716 and NG 716)
Cracking and Seating of Existing Jointed Unreinforced Concrete Pavements and Hydraulically Bound Mixture Bases
(continued)</td><td>Construction-related items are as follows:
(continued)
<table>
<tr><td>Cracking Operations</td><td>1. Proceed with pavement cracking at spaces determined by test section based on effective stiffness modulus computed from falling weight deflectometer (FWD) tests (refer to UK specification 717). Generally a 0.75- to 2-m spacing.
2. Surface cracking checked by applying water on all areas, allowing it to surface dry and then core every 300 m^2 or less of surface treated. If the cores indicate multiple cracks, shattered base or no cracking then the operation is suspended and new test cycle required.
3. Any longitudinal cracking in wheelpaths that extends beyond two transverse cracks is considered a failure and requires a new test cycle and slab repair.</td></tr>
<tr><td>Seating Operations</td><td>Minimum of six passes with a 20-tonne pneumatic-tired roller. Effective stiffness modulus confirmed with FWD tests after seating.</td></tr>
<tr><td>Maintenance</td><td>Surface of cracked and seated pavement will be cleaned of all debris before contractor conducts FWD tests. Computed effective stiffness modulus must be accepted before paving. Does not appear that they allow traffic before paving.</td></tr>
</table></td></tr>
<tr><td>Michigan DOT</td><td>No specific specification.</td></tr>
<tr><td>Minnesota DOT</td><td>No specific specification.</td></tr>
<tr><td>Missouri DOT</td><td>No specific specification.</td></tr>
<tr><td>Texas DOT</td><td>No specific specification.</td></tr>
<tr><td>Virginia DOT</td><td>No specific specification.</td></tr>
</table>

continued

AASHTO Specification Designation 567 "Construction": Cracking and Seating (*continued*)

Agency/Organization	Specification Section: Construction	
Washington State DOT	Construction-related items are as follows:	
	Cracking and Seating Equipment	1. Equipment shall be self-propelled and self-contained guillotine-type drop weight. 2. Equipment shall impact the pavement with a variable force which can be controlled in force and point of impact.
	Surface Preparation	1. Prior to cracking, any existing HMA shall be removed from the PCCP to be cracked.
	Test Section	1. A test section will be used to assess early cracking operations (numerous details are associated with the test section).
	Cracking Operations	1. Pavement shall be cracked into segments nominally measuring 6 ft transversely and 4 ft longitudinally. (Note: Most WSDOT JPCP slabs are 12 ft wide and 15 ft between contraction joints.) 2. The pavement-cracking tool shall not impact the pavement within 1 ft of another break line, pavement joint, or edge of pavement. 3. Cracking of the slabs shall not deviate from vertical by more than 4 in. between the surface and bottom of the pavement. 4. Longitudinal cracks shall not be closer than 5 ft from the longitudinal edge of the panel.
	Seating Operations	1. Seating shall be by a pneumatic roller not less than 35 tons. Tires must be inflated to 60 psi minimum. 2. Roller speed shall not exceed 5 mph. 3. Seating must be done with not less than five passes over the cracked concrete. A pass shall be one movement of a roller in either direction.
	Maintenance	1. Public traffic shall not be allowed on the cracked pavement until a minimum of 0.35 ft of HMA has been placed.

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5 LIFE-CYCLE COST ANALYSIS

INTRODUCTION

The SHRP 2 R23 Guide provides a number of possible alternative designs using either rigid or flexible pavements. There is usually not a single design that meets the design criteria but rather a number of alternative designs that can be considered viable solutions. The method of selecting the best possible approach may consist of an economic evaluation, a decision matrix, or a combination of those approaches. There are several types of economic or criteria-based evaluations that can be carried out as part of conducting a life-cycle cost analysis (LCCA)—for example, cost-benefit analysis, cost-effectiveness analysis, multicriteria analysis, and risk-benefit analysis. At one extreme lies the purely multicriteria analysis, which employs weights from a variety of sources that contain a large degree of subjective assessment. At the other extreme lies the purely cost-benefit analysis that exclusively employs monetary valuation and has generally more explicitly defined criteria. Most highway agencies have established some form of selection process, and it is expected that those agencies will apply their methodology to select between different options. For those agencies that do not have a formal selection procedure in place, the following guidance for conducting life-cycle cost analysis is provided and recommended to aid the selection process.

LIFE-CYCLE COST ANALYSIS (LCCA) PROCEDURE

Most agree that life-cycle cost analysis can be carried out using a few standardized steps. The process of a typical LCCA can be divided as follows:

- Establish strategies for a 50-year service period.
- Establish activity timing.

- Estimate agency costs.
- Estimate user costs.
- Develop expenditure streams.
- Compute net present value (NPV).
- Conduct risk analysis.
- Reevaluate strategies.

These steps are explained more fully in the content that follows.

Establish Strategies for a 50-Year Service Period

The primary purpose of an LCCA is to quantify the implications of initial pavement design decisions regarding the future costs of maintenance and rehabilitation activities over 50 years. This assumes that a high level of service is maintained to preclude the use of full-depth patching and other major repairs. Having a clear picture of the pavement performance over that period is critical to the selection of the most cost-efficient alternative for that particular location and project. The timing of needed minor repairs, if properly managed, will efficiently preserve the pavement condition over the 50-year design period at what would be expected to be the lower total cost.

It is anticipated a 50-year analysis period will be long enough to incorporate multiple rehabilitation activities repeated through the service period. Figure 5.1 shows a typical analysis period for a given pavement design alternative. Guidelines for the preservation of long-life pavements are included based on the work performed in SHRP 2 Project R26, Preservation Approaches for High-Traffic-Volume Roadways.

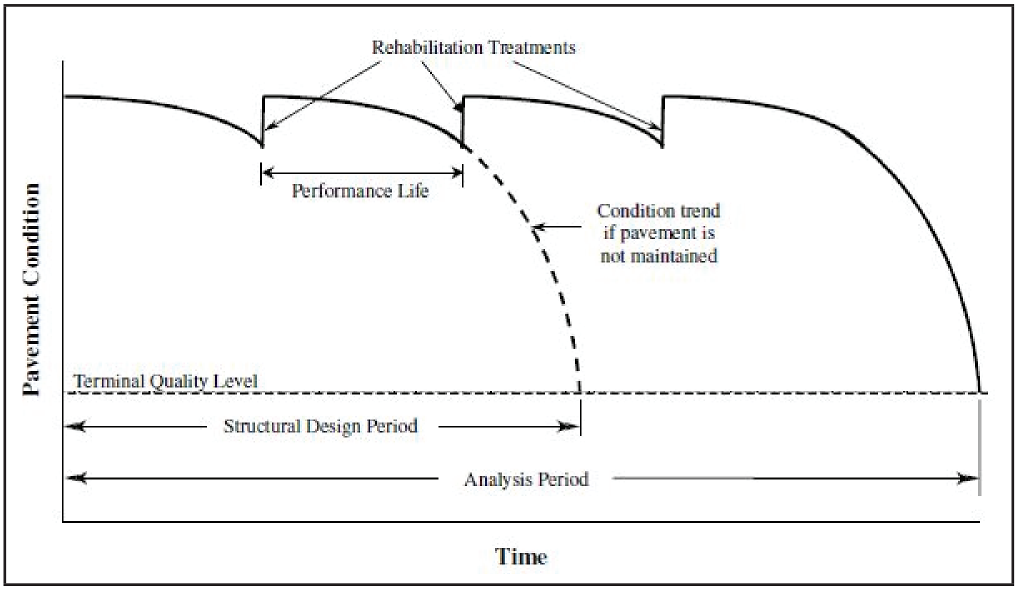

Figure 5.1. *Example of pavement performance life.*
Source: WSDOT, 2010.

Preservation treatments and approaches recommended in those guidelines should be considered in the reaccruing maintenance or preservation costs associated with each design alternative. A simplified illustration of the activity and timing is shown in Figure 5.1.

Typically, each design alternative will have an expected initial design life, periodic maintenance treatments, and rehabilitation. In terms of the LCCA, it is important to identify the developing distress condition, timing, and cost of the key activities. State highway agencies (SHAs) have historically planned to employ a variety of rehabilitation strategies to keep highway facilities in a functional condition. For example, Table 5.1 shows the Washington State Department of Transportation (WSDOT) maintenance and rehabilitation framework representing a conventional approach to maintain new and reconstructed pavements over a 50-year period in their LCCA procedure (WSDOT Pavement Guide, Vol. 1, 2009).

TABLE 5.1. REHABILITATION SCENARIOS FOR HMA AND PCC PAVEMENTS

Year	HMA Pavement	PCC Pavement
0	Construction or reconstruction	Construction or reconstruction
15	1.8-in. mill and HMA overlay	
20		Diamond grinding
30	1.8-in. HMA overlay	
40		Diamond grinding
45	1.8-in. mill and HMA overlay	
50	Salvage value (if applicable)	Salvage value (if applicable)

Establish Activity Timing

Performance life for the initial pavement design and subsequent rehabilitation activities has a major impact on LCCA results. It directly affects the frequency of agency intervention on the highway facility, which in turn affects agency cost as well as user costs during maintenance activities. SHAs can determine specific performance information for various pavement strategies through analysis of pavement management data and historical experience as a basis of calibration of performance-related models and tools. Operational pavement management systems can provide the data to evaluate pavement condition and performance to identify performance trends. Current FHWA efforts to analyze pavement performance data collected as part of the Long-Term Pavement Performance (LTPP) program should provide an additional valuable resource to SHAs.

Work-zone requirements for initial construction, maintenance, and rehabilitation directly affect highway user costs and should be estimated along with pavement strategy development. The frequency, duration, severity, and year of work-zone requirements are critical factors in developing user costs for the alternatives being considered.

Estimate Agency Costs

Construction quantities and costs are directly related to the initial design and subsequent rehabilitation strategy. The first step in estimating agency costs is to determine construction quantities and unit prices. Unit prices can be determined from SHA historical data on previously bid jobs of comparable scale. Other data sources include the Bid Analysis Management System, if used by the SHA.

LCCA comparisons are always made between mutually exclusive competing alternatives only, reflecting differential costs between alternatives. In other words, costs that are common to all alternatives will simply cancel each other out in the LCCA calculations. In the past, many agencies did not include traffic control costs because they were relatively common to different approaches for new construction. For the existing high-volume highway facilities considered in these guidelines, traffic management costs may be a large part of the total costs and significantly different between alternative designs. Therefore, traffic management costs should be considered in comparing alternative design costs.

Agency costs include all costs incurred directly by the agency over the life of the project. These typically include initial preliminary engineering, contract administration, construction supervision and construction cost, and the associated condition monitoring cost. Routine or preservative maintenance must be proactively rather than reactively applied to be effective in preserving the condition of the pavement. Even though routine preservative-type maintenance costs are generally not excessively high, their role in maintaining a relatively high performance level cannot be overstated. Unfortunately, many SHAs may not have tracked routine maintenance timing or costs, providing few data regarding the differences between most alternative pavement strategies. It may also be true that, when discounted to the present, the direct routine maintenance and associated monitoring cost differences have negligible effects on NPV and may perhaps be ignored. Nonetheless, when effectively employed, the routine maintenance may often indirectly affect the NPV due to the longer service life before more costly treatments are used.

Salvage value, which at times is included as a negative cost, represents the value of an investment alternative at the end of the analysis period and consists of two fundamental components—residual value and serviceable life. Residual value refers to the net value from recycling the pavement. The differential residual value between pavement design strategies is generally not very large, and, when discounted over the performance period, tends to have little effect on LCCA results.

Serviceable life represents the more significant salvage value component; it is the remaining life in a pavement alternative at the end of the analysis period. It is primarily used to account for differences in remaining pavement life between alternative pavement design strategies at the end of the analysis period. For example, over a 50-year analysis, Alternative A reaches terminal serviceability at year 50, while Alternative B requires rehabilitation at year 40. In this case, the serviceable life of Alternative A at year 50 would be 0, as it has reached its terminal serviceability. Alternative B may still have 5 years of serviceable life at year 50, the year the analysis terminates. The value of the serviceable life of Alternative B at year 50 could be calculated as a percentage of design life remaining at the end of the analysis period (5 of 15 years or 33%) multiplied by the cost of Alternative B's rehabilitation at year 40.

Estimate User Costs

User costs are an aggregation of three separate cost components: vehicle operating costs, user delay costs, and crash costs that are incurred by the highway user over the life of the project. In LCCA, highway user costs of concern are the differential costs incurred by the motoring public between competing alternative highway improvements and associated maintenance and rehabilitation strategies over the analysis period. In the pavement design arena, the user costs of interest are further limited to the differences in user costs resulting from differences in long-term pavement design decisions and the supporting maintenance and rehabilitation implications. There are user costs associated with both normal operations and work-zone operations. In terms of long-life designs, user costs associated with *normal operations* pertain to service periods free of maintenance and/or rehabilitation activities that typically would limit flow capacity. User costs in these circumstances would be expected to be insignificant because they are mainly a function of pavement roughness, which is anticipated to be maintained at a high level. During these operating conditions, there should be little difference between crash costs and delay costs resulting from pavement design decisions. Furthermore, it may be difficult to ascertain any difference between vehicle operating costs since roughness will be maintained at a high level.

Consequently, relative to the user costs associated with *work-zone operations* (which pertain to user costs associated with periods of construction, maintenance, and/or rehabilitation activities), the only relevant costs would be those related to delays caused by monitoring or repair activities, as these would be key to achieving the long performance life.

Pavement maintenance and rehabilitation alternatives are often selected based on LCCA evaluations. To make consistent and cost-effective decisions, LCCA should take into account all costs. Simple models to evaluate the additional road-user costs in work zones can be employed to assist in determining life-cycle costs of various repair alternatives. CA4PRS, discussed in the section "Construction Productivity and Traffic Impacts" in Chapter 1 of this Guide, can accomplish this and is gaining use among SHAs in the United States.

There is a range for the dollar value of time delay used by various agencies. Table 5.2 gives the recommended dollar value used by WSDOT in 2010 dollars (WSDOT Pavement Guide, Vol. 1, 2009).

TABLE 5.2. RECOMMENDED DOLLAR VALUES PER VEHICLE HOUR OF DELAY

	Value per Vehicle Hour	
Vehicle Class	Value ($)	Range ($)
Passenger vehicles	15.10	13–17
Single-unit trucks	24.16	22–26
Combination trucks	29.08	27–31

Note: FHWA, adjusted to 2010 dollars (http://data.bls.gov/cgi-bin/cpicalc.pl).

Compute Net Present Value

In its broadest sense, LCCA is a form of economic analysis used to evaluate the long-term economic efficiency between alternative investment options. Economic analysis focuses on the relationship between costs, timing of costs, and discount rates employed. Once all costs and their timing have been developed, future costs are often discounted to the base year and added to the initial cost to determine the NPV for the LCCA alternative. As noted earlier, NPV is the amount at various points in time back to some base year:

$$\text{NPV} = \text{Initial Cost} + \sum_{k=1}^{N} \text{Future Cost}_k \times \left[\frac{1}{(1+i)^{n_k}} \right] \tag{5.1}$$

where

i = discount rate and

n = year of expenditure.

The component within the bracket of the formula is referred to as the present value (PV) factor for a single future amount. PV factors for various combinations of discount rates and future years are available in discount factor tables (more commonly referred to as interest rate tables). PV for a particular future amount is determined by multiplying the future amount by the appropriate PV factor. For example, if the initial cost is \$26 million and the future cost is \$9 million, with a discount rate of 4%, if the year of expenditure is 20 years, the NPV will become \$30.1 million by Equation 5.1 as depicted by Figure 5.2. The NPV can be categorized in two ways: one is the agency NPV, and the other is the user-cost NPV. Because user costs may dominate total NPV, agency costs and user costs must be computed separately.

Discount rates are typically set by a SHA and are rarely changed; however, the federal Office of Management and Budget sets these rates annually via Circular A-94—and they do vary from year to year. For example, the real discount rates for 30-year-plus analyses have varied from a low of 2.7% (for 2009 and 2010) to a high of 7.9% (for 1982). On average, the real discount rate over a span of about 30 years is 4.3%.

Risk Analysis

The concept of risk comes from the uncertainty associated with future events (i.e., the inability to know what the future will bring in response to a given action today). Risk can be subjective or objective. Subjective risk is based on personal perception (i.e., intuitively deciding how risky a situation may be). For example, you may view flying as more risky than driving. This perception of risk may be related to the consequences of failure as well as the inability to control the situation. Objective risk is based on theory, experiment, or observation. Because individuals' perceptions of risk vary, decisions incorporating risk-management concepts will depend to a large extent on the decision maker's tolerance for risk.

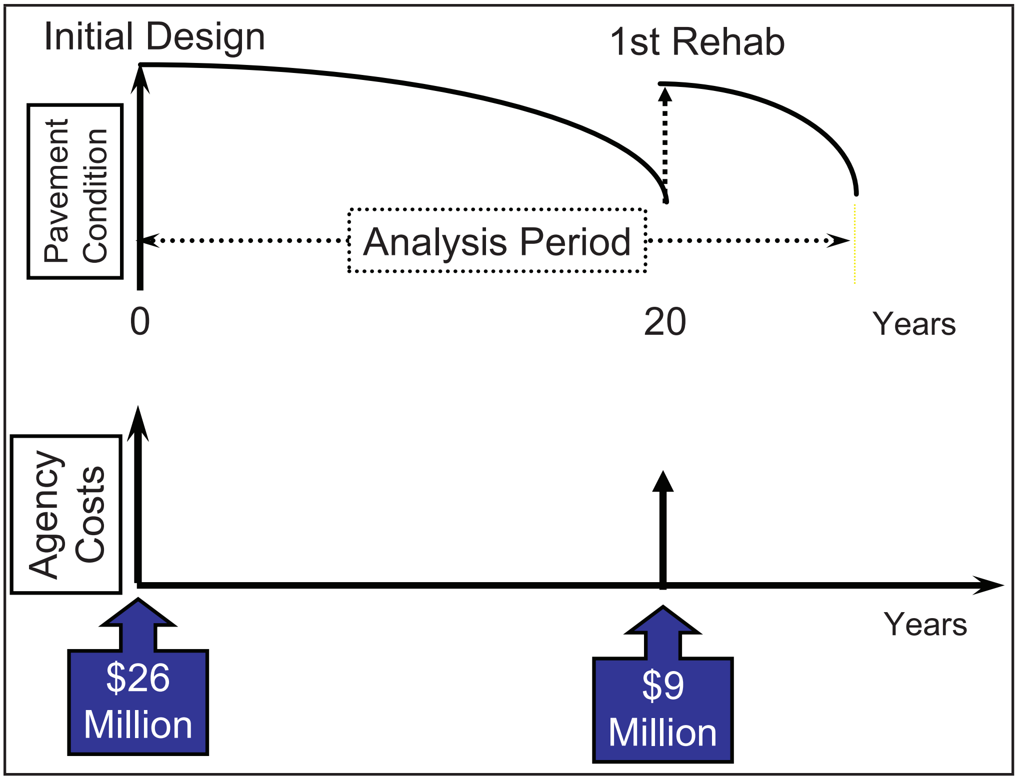

Figure 5.2. *Net present value computation example.*
Source: Walls and Smith, 1998.

Risk analysis is concerned with three basic questions: (1) What can happen? (2) How likely is it to happen? and (3) What are the consequences of its happening? Risk analysis attempts to answer these questions by combining probabilistic descriptions of uncertain input parameters with computer simulation to characterize the risk associated with future outcomes. It exposes areas of uncertainty typically hidden in the traditional deterministic approach to LCCA, and it allows the decision maker to weigh the probability of an outcome actually occurring.

Many analytical models treat input variables as discrete fixed values, as if the values were certain. In fact, the majority of input variables are uncertain. Economic models used in a typical LCCA are no exception. In conducting LCCA, it is important to be aware of the inherent uncertainty surrounding the variables used as inputs into the analysis. Uncertainty results from the assumptions, estimates, and projections made in conducting the analysis. Table 5.3 summarizes LCCA input variables and the general basis used to determine their values.

TABLE 5.3. LCCA INPUT VARIABLES

LCCA Component	Input Variable	Source
Initial and future agency costs	Preliminary engineering	Estimate
	Construction management	Estimate
	Construction	Estimate
	Maintenance	Assumption
Timing of costs	Payment performance	Projection
User costs	Current traffic	Estimate
	Future traffic	Projection
	Hourly demand	Estimate
	Vehicle distributions	Estimate
	Dollar value of delay time	Assumption
	Work-zone configuration	Assumption
	Work-zone hours of operation	Assumption
	Work-zone duration	Assumption
	Work-zone activity years	Projection
	Crash rates	Estimate
	Crash cost rates	Assumption
Net present value (NPV)	Discount rate	Assumption

This uncertainty is often ignored in an LCCA. For example, the analyst may make a series of best guesses of the values for each input variable and compute a single deterministic result. The problem with this approach is that it often excludes information that could improve the decision.

In some cases, a limited sensitivity analysis may be conducted, whereby various combinations of inputs are selected to qualify their effect on analysis results. However, even with a sensitivity analysis, this deterministic approach to LCCA often conceals areas of uncertainty that may be crucial to the decision-making process.

The need to make strategic long-term investment decisions under short-term budget constraints is encouraging SHAs to consider risk as a criterion for judging a course of action. Risk analysis exposes areas of uncertainty for the decision maker. Based on this information, the decision maker has the opportunity to take mitigating action to decrease exposure to risk. With the emergence of user-friendly computer software, like RealCoast (available from FHWA), a SHA should consider integrating quantitative risk analysis concepts into the decision-making process (Figure 5.3).

Reevaluate Strategies

Once the NPVs have been computed for each alternative and limited sensitivity analysis performed, the analyst needs to reevaluate the competing design strategies. The overall benefit of conducting LCCA is not necessarily to obtain LCCA results themselves, but rather to learn how the designer can use the information resulting from the analysis to modify the proposed alternatives and develop more cost-effective strategies.

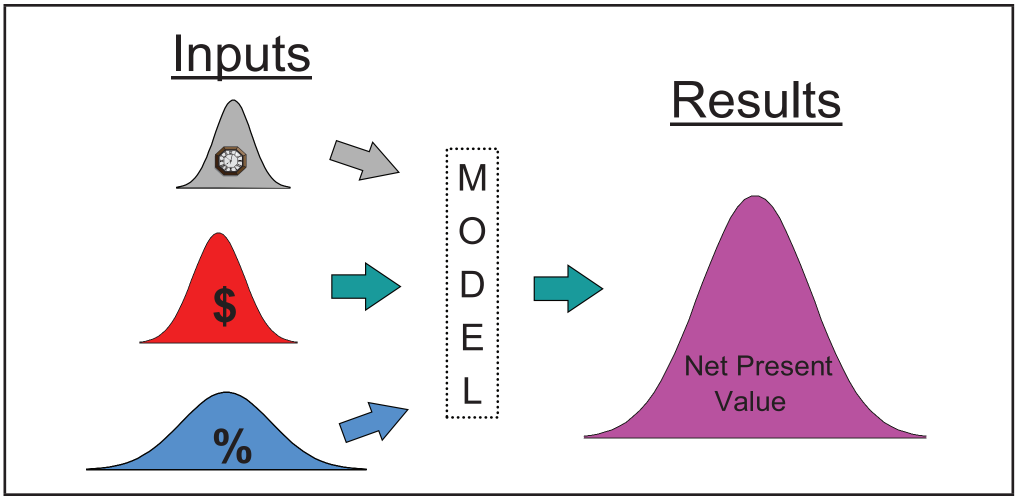

Figure 5.3. *Risk analysis approach.*
Source: Walls and Smith, 1998.

For example, if user costs dwarf agency costs for all alternatives, the analysis may indicate that none of the alternatives analyzed are viable. It could indicate that the designer needs to evaluate the current design strategies' impacts on future traffic maintenance and ensure that the design strategies reflect the need for additional capacity in the out-years to mitigate the impact on highway users. The solutions might include the following:

- The use of the shoulders in subsequent rehabilitation traffic control plans,
- Enhanced structural design of the mainline pavement to minimize the frequency of subsequent rehabilitation efforts,
- Reduction of the overall construction period,
- Restriction of contractor work hours or imposition of lane rental fees, and
- Planning for additional lanes and/or routes and shifting to alternative modes of travel.

It is important to note that restricting the contractor's hours of operation or the number of work days allowed will increase agency cost.

LCCA results are just one of many factors that influence the ultimate selection of a pavement design strategy. The final decision may include a number of additional factors outside the LCCA process, such as local politics, availability of funding, industry capability to perform the required construction, and agency experience with a particular pavement type, as well as the accuracy of the pavement design and rehabilitation models. Chapter 3 of the *AASHTO Guide for Design of Pavement Structures* (1993) discusses these other factors in greater detail. When these other factors weigh heavily in the final pavement design selection, it is imperative to document their influence on the final decision.

The accuracy of LCCA results depends directly on the analyst's ability to reasonably forecast such variables as future costs, pavement performance, and traffic years into the future. To deal effectively with the uncertainty associated with these forecasts, a probabilistic risk analysis approach is increasingly essential to quantitatively capture the uncertainty associated with input parameters in LCCA results.

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6 EMERGING PAVEMENT TECHNOLOGY

INTRODUCTION

There are portland cement concrete (PCC) and flexible pavement technologies that cannot, as yet, be considered long-life renewal options but may become so in the future. One technology reviewed, precast concrete pavement, is likely a long-lasting renewal option at this time. The limitation is that there are few projects under traffic to make that type of assessment. Thus, the term “emerging pavement technologies” does not necessarily imply that the concept is new. Several of these promising technologies were selected for a brief overview and include the following:

- Rigid pavements
  - Ultrathin continuously reinforced concrete pavement (CRCP) overlays
  - Precast concrete pavement
- Flexible or composite pavements
  - Resin-modified pavement

Without doubt, there are other technologies that could be featured; however, this is not the primary purpose of this study. This short treatment simply suggests that technologies exist that should be monitored as they continue to evolve and which may be or may become viable components for long-lasting pavement renewal.

RIGID PAVEMENTS

Ultrathin CRCP Overlays (UTCRCP)

This innovative pavement rehabilitation treatment was first reported in 2004 as an overlay system for steel bridges. This technology is not to be confused with ultrathin fiber-reinforced concrete overlays, which have been more widely evaluated in the United

States (such as the examples provided by Kuo, Armaghani, and Scherling, 1999). The UTCRCP approach has been extensively investigated in South Africa (Kannemeyer et al., 2008). Figure 6.1 illustrates some of the heavy vehicle simulator (HVS) testing that was recently completed for UTCRCP test sections near Johannesburg.

The South African experimental sections were mostly 50-mm thick and placed on various bases ranging from hot-mix asphalt (HMA) to natural gravel. Continuous steel mesh was used for reinforcement along with two types of steel fibers (straight and hooked). The continuous reinforcement as a percentage of the cross-sectional area is higher than for traditional CRCP—about 1.0% as opposed to typical values of 0.6% for CRCP. For the recent test conditions that used a granular base, it is estimated that a 50-mm UTCRCP has a minimum life of 25 million equivalent single axle loads (ESALs). Kannemeyer et al. estimated that this type of overlay would last between 14 and 55 years, depending on average daily truck traffic. (Kannemeyer et al. assumed that each truck applied 5 ESALs/truck.)

A 50-mm UTCRCP overlay was placed on the N12 highway in Johannesburg (project completion date November 2010) along with 200-mm CRCP in the slow lanes. The UTCRCP was placed on the "fast" or inside lanes for these multilane highways. The underlying base is HMA. Two types of reinforcement were used on the project: (1) a wire diameter of 5.6 mm with a 100 mm by 50 mm spacing, and (2) a wire diameter of 4 mm with a 50 mm by 50 mm spacing.

A second UTCRCP project was constructed on the N1 highway northeast of Paarl (near Cape Town), South Africa (Figure 6.2). This project currently serves 12,000 vehicles per day with 20% trucks (Ebels and Burger, 2010) and serves as a climbing lane. The design loading is 40 million E80s over a 25-year span and it has a 50-mm thickness. The mix makes use of polypropylene fibers, 5.6-mm-diameter steel mesh with a spacing of 50 mm by 100 mm, maximum nominal size aggregate of 6.7 mm, and various admixtures. This results in a mix with a compressive strength of about 15,000 psi and a minimum flexural strength of 1,500 psi.

Potential for Long-Term Performance

The South African experience with UTCRCP should be monitored because it has been carefully assessed by use of HVS experiments and is now deployed on actual highways.

Precast Panels and Precast Prestressed Concrete Pavement (PPCP)

PPCP Case Studies

Several precast concrete pavements have been built in the United States over the past 10 years; three well-documented projects include locations in Texas (completed in 2001), California (completed in 2004; Merritt, McCullough, and Burns, 2005), and Minnesota (completed in 2005; Burnham, 2007). Subsequently, projects have been completed in Missouri (2005) and Iowa (2006; Federal Highway Administration, 2009). The purpose of these projects was to assess the viability of precast concrete pavements for rapid construction and rehabilitation. These projects are relatively short. The longest is the Texas I-35 frontage road project at 2,300 ft. The Caltrans I-10 project was 248 ft, the Missouri project 1,010 ft, and the Iowa project 4,300 ft^2.

Figure 6.1. *Thin CRCP being tested via the heavy vehicle simulator in South Africa. (a) Testing of thin CRCP near Heidelberg, South Africa. (b) HVS testing of 50-mm thin CRCP. (c) Testing includes substantial instrumentation for in situ measurements. (d) HVS testing typically continues until a failure condition is reached. (e) Previously tested section illustrating reinforcing. (f) Close-up of reinforcing.*
Photos: Joe Mahoney.

Figure 6.2. *UTCRCP on the N1 Highway near Cape Town, South Africa.*
Photos: Wynand Steyn.

Earlier projects built in South Dakota, Japan, and Texas were documented by Merritt et al. (2000). The earliest Texas project was built in 1985 as a 6-in.-thick cast-in-place prestressed pavement.

Merritt et al. (2000) noted that for thickness design a reasonable lower limit for precast panel thickness would not be less than 50% to 60% of conventional concrete pavement. An analysis comparing a precast concrete pavement versus a more traditional CRCP suggested that 14-in.-thick CRCP would be equivalent to 8-in.-thick precast concrete panels. It was also noted that the 6-in. Texas-built cast-in-place prestressed pavement exhibited no distress following 15 years of in-service traffic.

The concept for the 2001 Texas project was stated as follows: "to develop a concept for a precast concrete pavement—one that meets the requirements for expedited construction and that is feasible from the standpoint of design, construction, economics, and durability. The proposed concept should have a design life of 30 or more years to make it comparable to conventional cast-in-place pavements currently being constructed" (Merritt et al., 2000). This project, as noted earlier, was 2,300 ft long with panels either 10 ft by 20 ft or 10 ft by 36 ft, all 8 in. thick (Federal Highway Administration, 2009). The posttensioned sections were 7 at 250 ft, 1 at 225 ft, and 1 at 325 ft. The panel installation rate was 25 panels per 6 hours. Figure 6.3 provides an aerial view of the project and Figure 6.4 shows photos taken during December 2010 to illustrate performance to date. The pavement was 9 years old at the time the photographs were taken and at the time exhibited no distress other than a few tightly closed longitudinal cracks. It should be noted that this road receives limited heavy traffic.

The 2004 Caltrans project used 8-ft precast panels, which resulted in a total of 31 panels to achieve the 248-ft length. The panel thicknesses were 10 in.—a thickness required to match an existing pavement. Each panel weighed 21.5 tons, which limited delivery to one panel per truck cycle to the job site. The expansion joints were designed

Figure 6.3. *PPCP section: Texas I-35 frontage road.*

for an opening ≤1 in. The panel installation rate was 15 panels over 3 hours. It was estimated that the design life would range from 30 to 57 years. The total in-place cost of this project was $224/yd^2.

The 2005 Missouri project was built on I-57 near Sikeston. The project length was 1,010 ft (two lanes plus shoulders), which used 10 ft by 38 ft panels ranging from 5.75 to 11.0 in. thick (the thinner sections are associated with the shoulders). The posttensioned sections were 4 at 250 ft. The panel installation rate was 12 panels per 6 hours. The precast panels were placed on a 4-in.-thick permeable asphalt base.

Precast Panels

Precast panels were used to replace a short section of jointed reinforced concrete pavement (JRCP) in Minnesota. The project was built on Trunk Highway (TH) 62 during June 2005 in the vicinity of the Minneapolis–St. Paul International Airport (Burnham, 2007). The original pavement was 8-in.-thick JRCP. Joint repairs were made in 1986, but the pavement was in need of additional rehabilitation about 20 years later. In 2005, TH-62 had concrete rehabilitation repairs made, along with the addition of

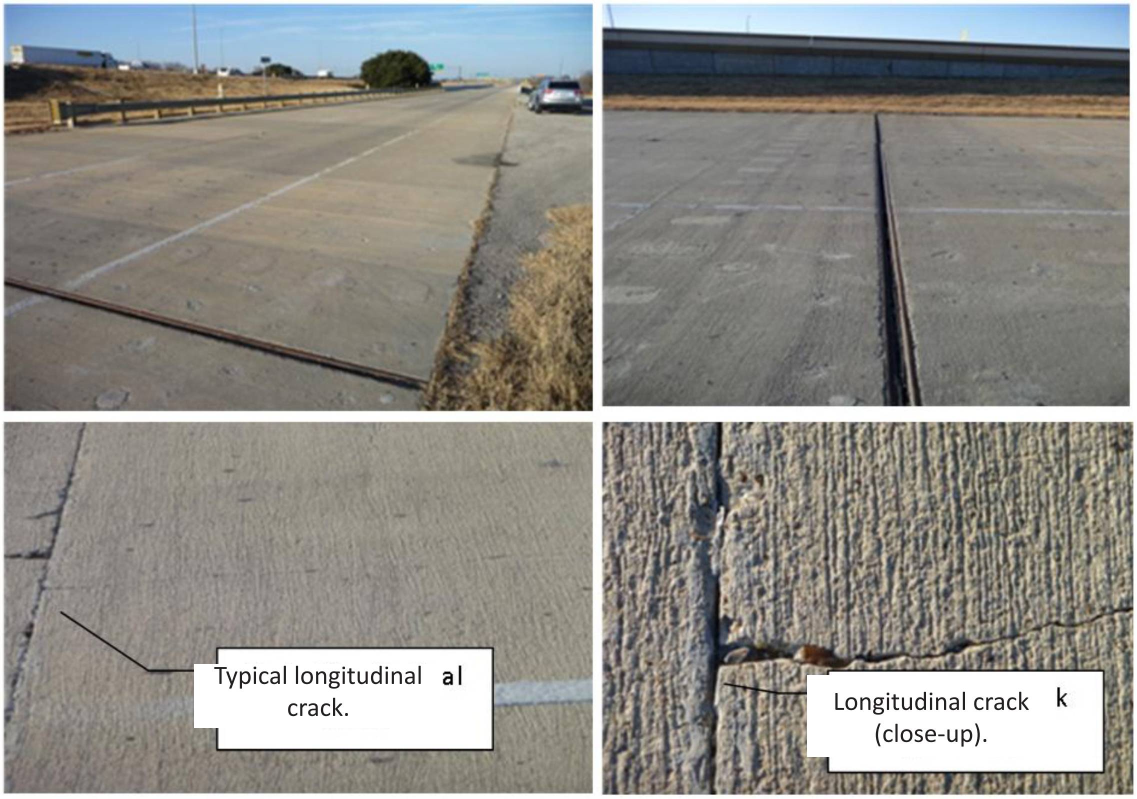

Figure 6.4. *PPCP section, Texas I-35 frontage road, December 2010.*
Photos: Joe Mahoney.

a precast test section (Figure 6.5). The precast test section was 216 ft long by 12 ft wide, which required 18 panels (the Fort Miller Co. precast system). Each panel was 12 ft long by 12 ft wide by 9.25 in. thick. The precast panels were not tied to the adjacent JRCP lane, nor were they posttensioned; rather they were doweled at the transverse joints. The test section was ground about 5 months after construction with the international roughness index (IRI) results summarized in Table 6.1. Load transfer efficiency measurements for the transverse joints were about 90% to 95% one year after construction.

TABLE 6.1. SUMMARY OF IRI RESULTS FOR PRECAST PANELS, MINNESOTA TH-62

Time and Activity	Average IRI for Both Wheelpaths (in./mi)
TH-62 before construction	150
New precast panels (Fall 2005)	140
After grinding panels (Fall 2005)	76
Six months following grinding (April 2006)	50

Source: After Burnham, 2007.

Figure 6.5. *Precast section, Minnesota TH-62.*
Source: Burnham, 2007.

Potential for Long-Term Performance

Precast concrete pavements show significant promise. Tracking performance of the existing pavements is needed. Cost and construction times will likely drop as larger projects are constructed.

FLEXIBLE OR COMPOSITE PAVEMENTS

Resin-Modified Pavement (RMP)

RMP was described by Ahlrich and Anderton (1991) as a "semi-rigid, semi-flexible" surface course. It is an open-graded HMA layer with about 25% to 30% air voids, which are filled with a resin-modified cement slurry grout. As noted by Ahlrich and Anderton, "RMP is a tough and durable surfacing material that combines the flexible characteristics of an asphalt concrete material with the fuel, abrasion, and wear resistance of a portland cement concrete." The original concept for RMP was developed in Europe during the 1960s.

The basic process for RMP is as follows (after Ahlrich and Anderton, 1991):

1. Place an open-graded HMA layer. This layer determines the thickness of the RMP.
2. Pour the grout material (portland cement, fine aggregate, water, and a resin additive) onto the HMA, squeegee over the surface, and vibrate into the voids with a small vibratory roller.
3. Cure the grout material with standard white-pigment sprayed curing compound.

Ahlrich and Anderton (1991) reported accelerated pavement testing by use of the FHWA ALF device at Turner Fairbank. The trafficking used dual tires loaded to 19,000 lb with tire pressure of 140 psi. Following 80,000 passes, the RMP surface performed well with no deterioration.

At the time of U.S. Army Corps of Engineers testing, the cost of RMP ranged between that of traditional HMA and that of PCC.

More recent studies on RMP include a 5-year performance assessment by Battey and Whittington (2007) in Mississippi (see construction of RMP, Figure 6.6). Three systems were assessed for use in signalized intersections on US-72 in Corinth, Mississippi:

1. RMP wearing course 2 in. thick,
2. Ultrathin whitetopping 3 in. thick, and
3. HMA overlay with PG 82-22 binder.

The comparison of these three options was assessed following 5 years of service. The order of comparison revealed the overall best option was the PG 82-22 HMA overlay, followed by the ultrathin whitetopping, and RMP last. However, the assessment also showed that the RMP exhibited no rutting but was the most expensive. The ultrathin whitetopping began to crack after 2 years of service and was eventually removed from service.

RMP is also being evaluated in South Africa. The photos shown in Figure 6.7 were taken in 2009 of a RMP that had been in service for 2 years at a truck weigh station (whereby all traffic involves trucks moving at a slow speed). As of 2009, no rutting or significant cracking had occurred.

Figure 6.6. *Construction of RMP. Application of the grout to the open-graded HMA.*
Source: Battey and Whittington, 2007.

(a) (b)

Figure 6.7. *Resin-modified pavement. (a) Resin-modified pavement at a truck weigh station on the N3 near Johannesburg, South Africa. (b) Close-up of the resin-modified cement that was placed on open-graded HMA.* Photos: Joe Mahoney.

Potential for Long-Term Performance

RMP appears to be a system appropriate only for wearing courses (largely due to cost and construction challenges). The performance appears quite good, particularly with regard to rutting resistance, but whether RMP will outperform traditional dense-graded HMA is as yet unclear. Hopefully, those that have built this type of pavement will continue to monitor performance and report their findings.

COST COMPARISONS

Cost comparisons for emerging technologies are a challenge on several levels because of their use in experimental projects, limited production, exchange rates, etc. Furthermore, materials and construction costs for pavements are rather volatile along with elusive, up-to-date national statistics. As such, background based on costs obtained from the Washington State Department of Transportation (WSDOT) for asphalt concrete and concrete paving materials are shown below (costs as of September 2010; WSDOT, 2010), along with available data from other projects (see Table 6.2). Table 6.3 provides performance lives and cost estimates for typical preservation treatments, developed as part of the SHRP 2 R26 study, which provide additional cost perspectives.

SUMMARY

The three emerging technologies illustrated in this document are only a sample of promising pavement developments. Whether the concepts illustrated ultimately contribute widely to long-lasting renewal options is yet unclear. On a national basis, systematic reporting on these types of technologies is needed (along with others yet to be identified).

TABLE 6.2. CONVENTIONAL AND EMERGING TECHNOLOGY COST ESTIMATES

Traditional Paving Systems	Typical Cost	Basis per Ton	Basis per yd²
Asphalt concrete (HMA) at 12 in. thick	\$64/ton	\$64/ton	\$42/yd²
Portland cement concrete at 12 in. thick	\$130/yd³	\$64/ton	\$43/yd²
HMA overlay, 2 in. thick	\$64/ton	\$64/ton	\$7/yd²
Chip seal	—	—	\$2/yd²
	Project Cost	**Miscellaneous Info**	**Basis per yd²**
UTCRCP (N1 Freeway, South Africa, completed June 2010). Section contained ~16,000 m² of UTCRCP paving.	R590/m²	~\$85/m²	~\$70/yd²
TH-62 Minnesota: precast panels 12 ft × 12 ft × 9.25 in. (completed June 2005). Cost/yd² excludes traffic control, grinding, and striping. Cost/yd² does include removal of preexisting 8-in. JRCP. Contained 288 yd² of precast panels.	—	Test section was small, ~288 yd²	\$575/yd²
Caltrans precast posttensioned test section, constructed in 2004.	—	Test section was ~1,000 yd²	\$224/yd²

Note: Per yd² basis based on equal thickness of HMA and PCC. Only the material costs were considered. Assumed densities are 145 lb/ft³ for HMA and 150 lb/ft³ for PCC.

TABLE 6.3. EXPECTED PERFORMANCE AND COSTS ASSOCIATED WITH A SELECTION OF PAVEMENT PRESERVATION TREATMENTS

Pavement Type	Expected Treatment Performance (years)	Estimated Unit Cost
Existing HMA Surfaced Pavement		
Crack filling	2–4	\$0.10–\$1.20/ft
Crack sealing	3–8	\$0.75–\$1.50/ft
Slurry seal	3–5	\$0.75–\$1.00/yd²
Chip seal, single course	3–7	\$1.50–\$4.00/yd²
Thin HMA overlay (dense graded; 0.875–1.5 in. thick)	5–12	\$3.00–\$6.00/yd²
Profile milling	2–5	\$0.35–\$0.75/yd²
Ultrathin whitetopping (2–4 in. thick)	NA	\$15.00–\$25.00/yd²
Existing PCC Surfaced Pavement		
Joint resealing	2–8	\$1.00–\$2.50/ft
Crack sealing	4–7	\$0.75–\$2.00/ft
Diamond grinding	8–15	\$1.75–\$5.50/yd²
Partial-depth concrete patching	5–15	\$75–\$150/yd² (based on patched area)
Full-depth concrete patching	5–15	\$75 to 150/yd² (based on patched area)
Dowel bar retrofit	10–15	\$25.00 to 35.00/bar
Thin HMA overlay (0.875–1.5 in. thick)	6–10	\$3.00 to 6.00/yd²

Source: After Peshkin et al., 2010.

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RELATED SHRP 2 RESEARCH

Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform (R02)
Precast Concrete Pavement Technology (R05)
Using Both Infrared and High-Speed Ground Penetrating Radar for Uniformity Measurements on New HMA Layers (R06C)
Nondestructive Testing to Identify Delaminations Between HMA Layers (R06D)
Assessment of Continuous Pavement Deflection Measuring Technologies (R06F)
Composite Pavement Systems (R21)
Preservation Approaches for High-Traffic-Volume Roadways (R26)