SHRP 2 Renewal Project R23

Using the Existing Pavement In-Place and Achieving Long Life

PREPUBLICATION DRAFT • NOT EDITED

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

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

ACKNOWLEDGMENT
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SHRP2

STRATEGIC HIGHWAY RESEARCH PROGRAM

SHRP2 R23

Using Existing Pavement in Place and Achieving Long Life

FINAL REPORT

Prepared for
The Strategic Highway Research Program 2
Transportation Research Board
of
The National Academies

Authors:

Mr. Newton Jackson, NCE, Olympia, Washington
Dr. Joe Mahoney, UW, Seattle, Washington
Mr. Jason Puccinelli, NCE, Reno, Nevada

February 2012

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AUTHOR ACKNOWLEDGEMENTS

This work was sponsored by the Federal Highway Administration in cooperation with the American Association of State Highway and Transportation Officials. It was conducted in the second Strategic Highway Research Program (SHRP 2), which is administered by the Transportation Research Board of the National Academies. The project was managed by James W. Bryant, Jr., Senior Program Officer for SHRP 2 Renewal.

The research team consisted of:

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

Of particular importance to the outcome of this project was the contributions from a number of individuals who helped coordinate meetings, and provided comments and recommendations throughout the project.

Gregory Stukel	Illinois Tollway Authority
Mike Eacker	Michigan Department of Transportation
Dr. Shongtao Dai	Minnesota Department of Transportation
John Donahue	Missouri Department of Transportation
Bill Stone	Missouri Department of Transportation
Dr. Magdy Mikhall	Texas Department of Transportation
Trenton Clark	Virginia Department of Transportation
Alex Teklu	Virginia Department of Transportation
Jeff Uhlmeyer	Washington State Department of Transportation
Larry Scofield	American Concrete Paving Association
Dr. Dave Newcomb	National Asphalt Paving Association

ABSTRACT

This report documents the findings from the SHRP 2 R-23 project entitled, "Using Existing Pavement in Place and Achieving Long Life." The goal of this project was to develop reliable procedures that identify 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. SHRP 2 has defined long life pavements as those lasting in service for 50 years or longer without needing major rehabilitation. Through literature reviews, industry interviews, international surveys, and extensive interactions with numerous State Highway Agencies (SHAs), this project developed decision matrices, design tables, and resource documents that provide valuable information regarding all aspects of a renewal project from project assessment, renewal selection, design, specifications, and construction. Interactive computer software was also developed that intuitively steps users through the decision matrices and provides easy and organized access to all of the resource documentation.

EXECUTIVE SUMMARY

This report documents the findings from the SHRP 2 R23 project entitled, "Using Existing Pavement in Place and Achieving Long Life." This project falls within the SHRP 2 Renewal area, which focuses on improving the ability to design and construct long-lasting highway projects with minimal disruption to the traveling public. Specific to the R23 project, construction costs and time can be greatly reduced if the existing pavement can be used in place as part of the rehabilitation solution.

The goal of this project was to develop reliable procedures that identify 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. SHRP 2 has defined long life pavements as those lasting in service for 50 years or longer without needing major rehabilitation. This effort concentrated on understanding the state of the art of rapid renewal approaches currently used both nationally and internationally to construct long-lived pavement for high-volume roadways.

Through literature reviews, industry interviews, international surveys, and extensive interactions with numerous State Highway Agencies (SHAs), this project developed a list of renewal alternatives that use the existing pavement in-place. The list of alternatives included both flexible and rigid pavements (as well as composite pavement sections). Project and performance records from the SHAs as well as numerous site visits were used to gather valuable information about each renewal alternative. Pavement performance captured in the Long Term Pavement Performance (LTPP) database along with detailed analyses using the MEPDG, PerRoad, and other analytical tools, were used to evaluate the advantages and disadvantages of each approach under different site conditions as well as the features critical in achieving long life. From these analyses, criteria on when an existing pavement could be used in-place were established. Consideration was also given to situations where modification of the existing pavement structure would be needed prior to renewal activities to ensure long life. Figure ES.1 is an example of an unbounded PCC overlay in Washington that is providing excellent performance after 35 years of service.

Figure ES.1. Photo of 35 year old unbonded PCC overlay on I-90.

PROJECT DEVELOPMENT GUIDELINES

A set of decision matrices, organized as tables, were developed to aid the identification of renewal strategies. Separate matrices, with associated decision paths, were developed for selecting renewal options for the various, existing pavement types. The decision matrices account for deterioration/surface distress types present in existing pavement as well as structural response (i.e., deflections), subgrade conditions, and other site-specific constraints. The intent of the decision matrices is to provide a set of feasible long life alternatives and include both flexible and rigid renewal options as outputs.

Additionally, a series of flexible and rigid renewal thickness design tables were established to supplement the decision matrices. These thicknesses provide approximate ranges (or scoping) for long life pavement designs and are intended as a starting point for project level design. The design thicknesses were developed based on newer design approaches including the MEPDG, PerRoad, and other analytical tools.

Selecting, designing, and constructing an optimal renewal alternative that will achieve long life performance requires attention to detail. While fragments of these details have been addressed in documentation available prior to the study, a comprehensive set of resources specifically devoted to addressing long life renewal did not exist. As such, this project developed a set of resource documentation that addresses details critical to achieving long life. The documentation addresses long life concepts at every stage of a project, starting at the assessment stage and continuing through feasible approach selection, design, traffic staging considerations, life cycle cost analysis, and construction specifications. The following six documents developed as part of the study address each stage of a project.

Project Assessment Manual

The Project Assessment Manual was prepared to help obtain a systematic collection of relevant pavement-related data. The use of the manual is to compliment the design tools developed by the study. The types of data critical for making pavement-related decisions are described along with methods (analysis tools) for organizing the information for decision-making.

Best Practices for Flexible Pavements and Best Practice for Rigid Pavements

The Best Practices documents (for both flexible and rigid pavements) provide a collection of best practices for the design and construction of long life flexible pavement alternatives using existing pavements. Standard practices for added lanes and transitions to adjacent structures are also discussed.

Guide Specifications

The Guide Specifications were developed in a format that would allow SHAs to easily make additions/modifications to their existing specifications. The specifications recommendations for long life are organized into three sections, which are: (1) guide specifications for pavement components that are not contained within the American Association of State Highway and Transportation Officials (AASHTO) Guide Specifications, (2) elements that can be added to or otherwise modify existing AASHTO Guide Specifications, and (3) summaries for relevant SHAs and AASHTO specifications that were used to produce the "elements" in item 2.

Life Cycle Cost Analysis

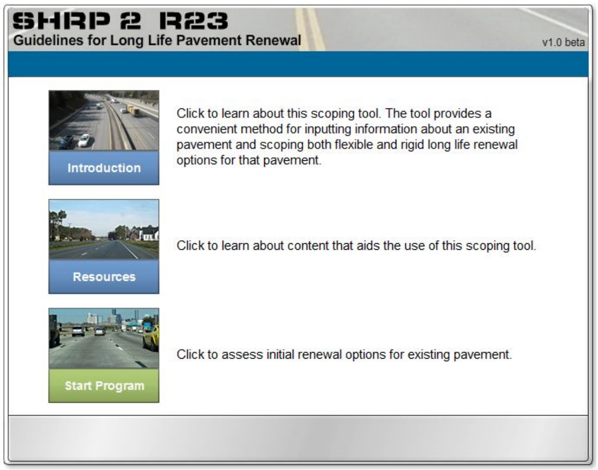
Most public agencies have specific procedures in place for life cycle cost analyses, and it is expected that those agencies will follow those procedures. Where an agency does not have a specific procedure in place, a general discussion of life cycle cost analysis is included in the Life Cycle Cost Analysis resource.

Emerging Technology

The Emerging Technology document discusses rigid and flexible pavement technologies that are not yet considered to be long life renewal options but may become so in the future as field performance is accrued.

Interactive Software

In order to provide a user-friendly means of navigating the large amount of information, and to automate the use of the decision matrices/thickness design tables, a computer-based application to guide the users through the process was established. A screenshot of the opening screen is shown in Figure ES.2.



SHRP 2 R23
Guidelines for Long Life Pavement Renewal
v1.0 beta

Click to learn about this scoping tool. The tool provides a convenient method for inputting information about an existing pavement and scoping both flexible and rigid long life renewal options for that pavement.

Click to learn about content that aids the use of this scoping tool.

Click to assess initial renewal options for existing pavement.

Figure ES.2. Opening Screen from the Interactive Software.

Product Validations

All of the products and tools described above were developed in close consultation with several SHAs. Specifically, extensive interaction took place with seven agencies (Illinois Tollway Authority, Michigan DOT, Minnesota DOT, Missouri DOT, Texas DOT, Virginia DOT, and Washington DOT). A series of visits were made to each agency over the course of the project to obtain information and solicit feedback on the products. These visits were typically structured as follows:

- ***Kick-off Meetings***: The objectives and preliminary findings of the project were discussed. Additionally, field visits were made to multiple renewal projects throughout each agency. Relevant project information was obtained from agency records.
- ***Test Case Meetings***: These meetings focused on soliciting feedback regarding the decision matrices and thickness design tables as well as the resource documentation. Access to the beta version of the interactive software was provided along with presentations explaining the development and use of the software. Coordination with each agency took place to identify and obtain information for one project to be used as a test case. This test case was used to compare the agency's standard design approach for pavement renewal with the recommendations provided by the new guidelines. In many cases, a field visit to the project was made to conduct a visual assessment of the site and capture photographs of the pavement and drainage features. A design report using the guidelines and interactive software was developed for the test case, which included feasible flexible and rigid renewal strategies. The results were compared to the agencies standard design approach for the project. The Virginia and Washington test cases both included analyses of construction productivity, lane closure alternatives, and traffic impacts using the CA4PRS software. The test case comparisons generated valuable feedback from the agencies.
- ***Workshops***: The team organized and facilitated one pilot workshop in Washington and two regional workshops in Virginia and Missouri. The workshops were attended by representative departments within the agency as well as local contractors and industry representatives. Adjacent state agency personnel were invited to attend the regional workshops. Near the end of each workshop, each participant was asked to complete a questionnaire. Overall, the participants viewed the guidelines as valuable and useful. In particular, the resource documentation was viewed by attendees as providing excellent content for pavement designers. All of the comments received were reviewed and addressed in the final guidelines.

Through these visits, meetings, workshops, and interactions, the products developed under this study have been vetted by the community and are a practical set of tools for pavement engineers and designers.

IMPLEMENTATION/RECOMMENDED RESEARCH

The guidelines developed under this project provide a single source of current information on a comprehensive list of approaches that an agency can reasonably use to design and build pavements that use existing pavements in place and achieve long life. The products were placed in an interactive program to facilitate use and implementation. The guidelines are unique in that they not only address the design approaches, but also provide guide specifications that are congruent with those approaches. To enhance implementation, this product should be housed on the web to ensure accessibility to the pavement community. Recommended future enhancements include modifying the guidelines and resource documents to include design lives of less than 50 years and enhancing the interactive software. Such enhancement would include the addition of a self-directed tutorial and conversion of static documents like the project assessment manual, guide specifications, and best practices into a content management system with cross-linked pages to aid in accessibility and to improve search capabilities of the documentation.

CHAPTER 1
BACKGROUND

This report documents the findings from the SHRP 2 R23 project entitled, "Using Existing Pavement in Place and Achieving Long Life." This project falls within the SHRP 2 Renewal area, which focuses on improving the ability to design and construct long-lasting highway projects quickly with minimal disruption to the traveling public. Key components to achieving these objectives include the application of innovative methods and materials for preserving, rehabilitating, and reconstructing the Nation's transportation infrastructure. Specific to the R23 project, construction costs and time can be greatly reduced if the existing pavement can be used in place as part of the rehabilitation solution.

During the last 20 years, there have been numerous projects where the existing pavement was either modified in place or used as is and a new structural pavement was placed on top. Both asphalt and concrete pavement solutions have shown promise, but there is limited in-service performance on heavy duty pavements. Techniques include rubblizing and crack and seat for asphalt over concrete pavements and concrete over concrete or over asphalt pavements.

There is a need for reliable procedures that allow agencies to identify when an existing pavement can successfully be used in place and how to incorporate it into the new structural pavement to achieve long life. The guidelines, resource documentation, specifications, manuals, and software developed as part of the SHRP 2 R23 effort focused on addressing these needs.

This effort concentrated on understanding the state of the art of rapid renewal approaches currently used both nationally and internationally to construct long-lived pavement for high-volume roadways. The project also identified promising alternatives to renewal approaches currently in use (but without substantive performance history) or imminently on the horizon. SHRP 2 has defined long life pavement as those lasting in service for 50 years or longer (details on the long life definition can be found in subsequent sections of this report).

State Highway Agency (SHA) participation and contribution to this project was critical in developing a practical and useable set of guidelines and tools. The project team recognizes the critical and substantial information and feedback provided by the following agencies:

- Illinois Tollway Authority (ITA)—Steven Gullien
- Michigan Department of Transportation (MDOT)—Michael Eacker
- Minnesota Department of Transportation (MnDOT)— Shongtao Dai
- Missouri Department of Transportation (MoDOT)—John Donahue, William Stone
- Texas Department of Transportation (TxDOT)— Magdy Mikhail

- Virginia Department of Transportation (VDOT)—Trenton Clark, Alex Teklu
- Washington Department of Transportation (WsDOT)—Jeff Uhlmeyer

PROJECT OBJECTIVES

The goal of this project was to develop reliable procedures that identify 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. To that end, the project had the following objectives:

- Identification of alternatives for using existing pavements in-place for rapid renewal.
- Analysis of advantages and disadvantages for each approach under different conditions.
- Development of detailed criteria on when an existing pavement can be used in-place, with or without significant modification.
- Identification of practices and techniques available to construct pavements with the above characteristics.
- Determination of the optimal methods to integrate the renewed pavement with adjacent pavements and structures.

SCOPE OF WORK

Project R23 was structured in two phases, with Phase 1 consisting of five tasks. These five tasks were:

1. Document current renewal approaches in use by SHAs.
2. Analyze renewal approaches to determine which factors are critical for success.
3. Develop criteria for when existing pavement can be used, with or without modification.
4. Present advantages and disadvantages of each approach under different project conditions.
5. Develop an Interim Report and Phase 2 Work Plan.

Phase 1 focused on documenting the existing practices, analyzing each approach to determine which factors are critical for success, establishing criteria on when the existing structure requires modification (i.e., pulverization, rubblization, crack/seat) as part of the renewal, and evaluating the advantages/disadvantages of renewal approaches. These findings served as the basis for developing practical design guides in consultation with seven SHAs during Phase 2. This was accomplished by working with the states to develop draft guidelines, using the guidelines on a test project in each state, and then facilitating two regional workshops with the agencies where agency personnel, industry, and contractors were able to provide input on the process. The workshops were also used to compare designs between the existing agency practice and the new procedure developed from this study.

Phase 2 consisted of the following tasks:

1. Work with seven SHAs to develop practical design guides.
2. Verify usability of design guides by designing actual projects with each SHA.
3. Compare the results from new design guides and existing SHA procedures and solicit feedback through regional workshops.
4. Revise guidelines based on comments from SHAs.
5. Develop final report and final design guidelines.

REPORT ORGANIZATION

The research approach used for the study is provided in Chapter 2. The methodology associated with the two major phases is described along with the agency interactions.

A summary of the Phase 1 and Phase 2 activities performed for this study is provided in Chapter 3. Details on the national/international literature review and survey can be found in Appendix A. The analysis conducted using test sections from the Long Term Pavement Performance (LTPP) Program can be found in Appendix B. The development of the decision matrices for both flexible and rigid renewal can be found in Appendix C. Appendix D shows how the flexible and rigid renewal thickness design tables were developed.

Chapter 3 provides an overview of the products developed from the project including the interactive software that directs the user through the guidelines and contains the primary resource documentation. The following resource documentation can also be found in Appendix E:

- Pavement Assessment Manual (Appendix E-1)
- Rigid Best Practices (Appendix E-2)
- Flexible Best Practices (Appendix E-3)
- Specification Modifications for Long Life (Appendix E-4)
- Life Cycle Cost Analysis (Appendix E-5)
- Emerging Pavement Technology (Appendix E-6)

A summary including conclusions, implementation, and suggested additional research are provided in Chapter 4.

CHAPTER 2
RESEARCH APPROACH

INTRODUCTION

This chapter is used to present the study research approach. The primary research was to confirm that long life pavements could be designed and constructed using the existing pavements as part of the structure. To do this, two major phases were conducted. Phase 1 began the study with a thorough literature review documenting the potential long life approaches using existing pavements. This included a comprehensive evaluation of SHA project records and international documentation. Detailed analyses of pavement performance, including information from the LTPP database, were used to confirm the approaches that could provide 50 years of service life. During Phase 2, additional information became available and it was used to refine the findings from Phase 1 as well as to develop the guidelines and tools delivered as part of the project.

The SHRP 2 program defined long-lived pavements as those that last 50 years or longer without requiring major structural rehabilitation or reconstruction. This definition was the primary criterion that resulted in the findings and products associated with this study.

PHASED RESEARCH APPROACH

The research conducted for this project consisted of an extensive discovery process in Phase 1, consisting of agency surveys, literature reviews, and specific queries of individuals internationally. The information gained in Phase 1 was used to develop the long life guidance in Phase 2. Additionally more detailed information was collected during site visits with a number of agencies in Phase 2. Information gathered in both Phases 1 and 2 was used to produce the research findings and products described in Chapter 3.

Phase 1 Structure

The Phase 1 discovery process consisted of the following individual tasks:

- Literature review
- National and international survey of practice
- Review of practices in 15 states
- Analysis of LTPP data to confirm long life performance of different approaches
- MEPDG and PerRoad runs to predict long life performance

A literature search for information on highway renewal using existing pavements in-place was conducted. The findings from the literature review are discussed in Chapter 3 with details in Appendix A.

Literature Review

The literature review served three purposes. First, it allowed the team to refine the definition of long life pavements as well as typical criteria that are currently used by the industry to differentiate between conventional and long life pavements. Second, it provided a means to develop a complete list of viable approaches that have been utilized by SHAs and show promise for meeting the established long life criteria. Third, the data from the literature provided insight into the design, features, and configurations of each alternative.

The following list shows the strategies that were obtained from the literature review. These approaches were analyzed as part of Phase 1 and the findings are described in Chapter 3.

- Asphalt concrete (AC) over AC renewal methods
    - AC over existing AC pavement
    - AC over rubblized AC pavement
    - AC over reclaimed AC (recycling)
    - Lane additions
- AC over PCC renewal methods
    - AC over existing CRCP
    - AC over crack and seated JPCP
    - AC over rubblized JPCP
    - Lane replacement (inlay)/lane additions
- PCC over PCC renewal methods
    - Unbonded PCC overlay of PCC pavement
    - Bonded PCC overlay of PCC pavement
    - Lane replacement (inlay)/lane addition
- PCC over AC renewal methods
    - Unbonded PCC overlay of AC pavements
    - Bonded PCC overlay of AC pavements
    - Lane replacement (inlay)/lane addition

Survey of Practice

A survey of national and international practices was part of the Task 1 effort. Questionnaires were sent to each of the SHAs, the FHWA, and industry. These were followed by a series of e-mails and phone calls to learn more about individual projects. In some cases, the team visited SHAs to obtain project-level information. The agencies surveyed are listed below with those that responded in bold.

- **National Asphalt Pavement Association (NAPA),**
- **American Concrete Pavement Association (ACPA),**
- SHAs and FHWA division offices of Alabama, **Alaska**, **Arizona**, Arkansas, California, **Colorado**, **Connecticut**, Delaware, District of Columbia, **Florida**, **Georgia**, Hawaii, Idaho, **Illinois**, **Indiana**, **Iowa**, **Kansas**, Kentucky, **Louisiana**,

Maine, **Maryland**, Massachusetts, **Michigan**, Minnesota, Mississippi, Missouri, Montana, **Nebraska**, Nevada, **New Hampshire**, **New Jersey**, **New Mexico**, **New York**, **North Carolina**, North Dakota, **Ohio**, Oklahoma, **Oregon**, Pennsylvania, Puerto Rico, Rhode Island, **South Carolina**, South Dakota, Tennessee, **Texas**, **Utah**, **Vermont**, **Virginia**, **Washington**, West Virginia, Wisconsin, and Wyoming.

- **Antigo Construction, Carlo Construction, Duit Construction,**
- Government and industry contacts in **Argentina**, **Australia**, **Belgium**, **Canada (British Columbia, Ontario, and Québec)**, Chile, Colombia, **Germany**, **Italy**, Japan, **the Netherlands**, **South Africa**, **Sweden**, **Taiwan**, and the **United Kingdom**.

To obtain more detailed information, a number of agencies and individuals were contacted. The following individuals provided information on the approaches that had been used by their agency and how they were performing:

- Colorado (Steve Olson)
- Florida (Bruce Dietrich)
- Georgia (Georgene Geary)
- Indiana (Dave Kumar)
- Iowa (Chris Brake)
- Michigan (Michael Eacker)
- New York (Wes Yang)
- North Carolina (Judith Corley-Lay)
- Ohio (Roger Green)
- Oklahoma(Jeff Dean)
- Ontario (Tom Kazmierowski)
- Oregon (John Coplantz)
- South Carolina (Andrew Johnson)
- Texas (Magdy Mikhail)
- Washington (Jeff Uhlmeyer)

These contacts provided access to the available documentation. In a number of cases, they provided project specific information including design documents, copies of plan sheets, date of construction, and current condition. A sample of this information is illustrated in the plan section (Figure 2.1) provided by the Oregon DOT.

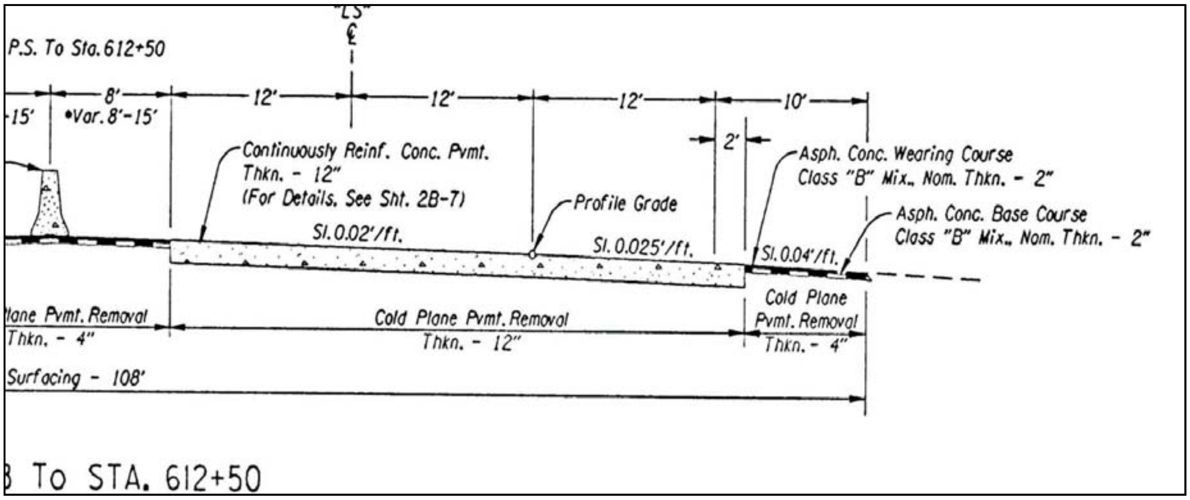


Figure 2.1. Example of Plan Section Provided by ODOT. (Courtesy of John Coplantz).

LTPP Data and MEPDG Analysis

Neither the literature review nor the survey of current practices confirmed the long life performance of the approaches being considered. There was information on the long life aspects of new pavements but not for the approaches for using existing pavements in place. To provide more information on service life, both the special pavement study (SPS) and the general pavement study (GPS) sections in the LTPP database were investigated. There were a number of SPS and GPS experiments that specifically included some of the approaches being considered.

For the flexible pavements sections the following LTPP experiments were considered:

- GPS-6A and GPS-6B (AC Overlay over AC Pavement)
- GPS-7A and GPS-7B (AC Overlay over portland cement concrete (PCC)
- SPS-5 (AC Overlay of AC Pavement)
- SPS-6 (Rehabilitation of Jointed PCC Pavement)

For rigid pavement sections they were:

- GPS-9 (Unbonded PCC overlays of PCC Pavement)
- SPS-7 (Bonded PCC overlays on PCC Pavements)

In most cases, the LTPP data indicated general trends but the performance periods were not long enough to show which approaches would provide the long life service required for the study. To provide additional insight, specific test sites for both flexible and rigid pavements were analyzed using the MEPDG. At the time of the analysis, the MEPDG did not include limiting strain criteria so PerRoad was also used (which does provide for selection of limiting strain criteria). Features that would produce long life performance and those that might limit long life performance were considered. The findings are described in Chapter 3 and the detailed analysis of both the LTPP data is included in Appendix B.

Phase 2 Structure

Phase 2 had a number of activities that continued from Phase 1. The critical activities were:

- Develop guidelines based on findings from Phase 1.
- Work with seven agencies to develop and test the guidelines.
  - Agency visits
  - Test cases
- Develop an interactive program to facilitate the use of the guidelines and provide a platform for the information needed to produce long life pavements.
- Develop the information needed to produce long life pavements which is contained in the following documents:
  - Pavement Assessment Manual
  - Best Practices for Flexible Pavements
  - Best Practices for Rigid Pavements
  - Guide Specifications
  - Life Cycle Cost Analysis
  - Emerging Pavement Technologies
- Conduct two regional workshops to get additional feedback on the guidelines.

Guidelines were developed initially in outline form and then converted to decision tables, which made them easier to use. The development of the guidelines and their refinements based on agency reviews and comments is described in Chapter 3.

Agency Visits

Seven agencies were visited during the development of guidelines. Those agencies and the primary contacts were:

- Illinois Tollway Authority (Steven Gullien)
- Michigan DOT (Michael Eacker)
- Minnesota DOT (Shongtao Dai)
- Missouri DOT (John Donahue, William Stone)
- Texas DOT (Magdy Mikhail)
- Virginia DOT (Trenton Clark, Alex Teklu)
- Washington State DOT (Jeff Uhlmeyer)

In the initial set of meetings, the team met with the agency to introduce the project and obtain appropriate details. The team asked the agencies to identify specific projects where they had used the previously identified approaches and to provide the following information, where available.

- Design procedure
- Typical thicknesses

- Construction considerations
- Specifications
- Performance
- Construction risks/issues
- Reason for any changes or modifications over time
- Reasons for abandoning approaches, if applicable

The team also made field visits to projects constructed by the agency using renewal alternatives considered for inclusion in the guidelines. Details on these field visits can be found in Chapter 3.

Test Cases

As part of the agency visits, the team and agency personnel identified potential projects that could be used as test cases to compare the use of the guidelines to what the agency had done. Those projects were:

- Michigan: I-75 in Cheboygan County
- Minnesota: I-35 in Chisago County
- Missouri: I-55 in Perry County
- Texas: US-75 Loy Lake Rd to Exit 64
- Virginia: I-95 in Caroline County
- Washington: I-5 Skagit County (at Bow Hill)

The data collected from each agency was used to develop a design report using the guidelines and interactive software. For each test case, feasible flexible and rigid renewal strategies were developed and documented. The results were compared to the agencies standard design approach for the project. Only two of the test cases dealt with long life designs by the individual agency. For the other four cases, the guidelines were compared to current practice, which were 20, 30, or 40 year designs.

Resource Development

Considerable effort was taken to develop the resource documents that go with the guidelines. There are approximately 400 pages of documents that were prepared to be used with the decision tables. Designing and building long life pavements typically require more attention to detail than simple treatment selection and thickness design. Those details that should be considered in designing and building long life pavements include:

- Project Assessment Manual
- Best Practices for Flexible Pavements
- Best Practices for Rigid Pavements
- Guide Specifications
- Life Cycle Cost Analysis

- Emerging Technology
- Traffic considerations
- Life cycle assessment.

A set of six resource documents were developed to address these details and are described in more detail in Chapter 3.

Workshops

Two regional workshops and a local pilot workshop were conducted to assess the guidelines. The team organized and facilitated one pilot workshop in Washington and two regional workshops in Virginia and Missouri. The pilot workshop was held with WSDOT employees from design, materials, construction, and traffic divisions. Additionally, local contractors and industry representatives participated. Similarly, the two regional workshops were attended by representative departments within the agency as well as local contractors and industry representatives. Adjacent state agency personnel were invited to attend and public advertisement of the workshop was conducted in accordance with the agency's protocols. Attendance by adjacent state representatives generally was not possible due to travel restrictions. However, these representatives received access to all of the material and were asked to provide comments.

The agendas distributed prior to each workshop contained information on the purpose and objective of the workshop. A link was also provided to the interactive guideline software and resource documentation so that attendees could complete some advanced reading and review of the material.

During each workshop, presentations were provided on the resource documentation developed as part of the study, as well as results from the test cases. A number of scenarios were demonstrated using the software. The group was asked for their comments and feedback based on the material presented at the workshop. All dialogue was documented and utilized to modify the deliverables of the project.

Near the end of the workshop, each participant was asked to complete a questionnaire. Overall, the participants viewed the guidelines as valuable and useful. In particular, the resource documentation was viewed by attendees as a solid tool for pavement designers. All of the comments received were reviewed and addressed in the final guidelines.

CHAPTER 3
FINDINGS AND APPLICATIONS

INTRODUCTION

The major findings from the project were assembled into one application with a number of resource documents, which collectively serve as guidelines for roadway renewal using existing pavements. Implementation and use of the guidelines will be largely dependent on the ease of use and practicality of the products. To this end, an interactive software program was developed to package the major components of the guidelines. The software and associated resource documents are described following an overview of the study development findings.

Phase 2 activities built upon and refined the findings from Phase 1 to develop a comprehensive set of decision matrices, design tables, and resource documentation that, collectively, comprise the renewal guidelines. In developing these guidelines, significant coordination took place with the agency partners as identified in Chapter 2.

LONG LIFE DEFINITIONS

Rigid Pavements

Long life concrete pavements exist in the United States as evidenced by the number of high age pavements that remain in service. Fortunately, at this time, advances in design, construction, and materials provide the knowledge and technology needed to consistently achieve a long life.

Some distress development over a concrete pavement’s service life is expected. However, the rate of distress development is managed by incorporating sound designs, durable paving materials, and quality construction practices. Generally recognized threshold values in the United States for distresses at the end of the pavement's service life are listed in Table 3.1 for jointed plain concrete pavements (JPCPs) and continuously reinforced concrete pavements (CRCPs).

Table 3.1. Threshold Values for Long Life Concrete Pavement Distresses.
(Tayabji and Lim 2007)

Distress	Threshold Value
Cracked slabs, % of total slabs (JPCP)	10 to 15
Faulting, mm (in.) (JPCP)	6 to 7 (0.25)
Smoothness (IRI), m/km (in/mi) (JPCP and CRCP)	2.5 to 3.0 (150-180)
Spalling (length and severity) (JPCP and CRCP)	Minimal
Materials-related distress (JPCP and CRCP)	None
Punchouts, #/km (mi) (CRCP)	10 to 12 (12 to 16)

Flexible Pavements

The intent of long life flexible pavements 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 eliminated for long life. However, surface initiated (top-down) cracking will still be possible in hot mix asphalt (HMA). This type of cracking is caused by a combination of pavement structure, load, environmental and material characteristics. While 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 be eliminated or significantly reduced. Another deterioration mechanism 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 is that they are difficult to model using current mechanistic-empirical methods. Some of the distresses require advanced response and/or performance models. 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.

For asphalt concrete pavements, achieving long life requires the combination of a rut/wear resistant surface layer with a rut resistant intermediate layer and a fatigue resistant base layer. As illustrated in Figure 3.1 (Newcomb et al, 2001), this requires a high quality HMA wearing surface or an open graded friction course, a thick, stiff dense graded intermediate layer and possibly a flexible (asphalt rich) bottom layer. In addition, the pavement foundation must be strong enough to satisfy the limiting strain criteria.

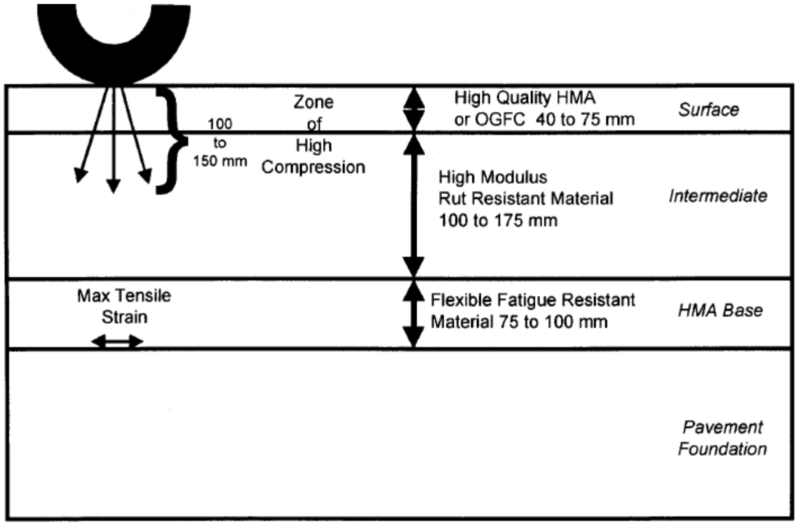


Figure 3.1. Long life Flexible Pavement Design Concept. (Newcomb et al., 2001)

When using existing pavements in the renewal process, the inhibition of reflective cracking is crucial. Reflective cracking is caused by repetitive shearing, e.g., 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 irrespective of the existing pavement type (i.e., distressed HMA or PCC) although experience shows that reflective cracking can be more predominant when the existing pavement is a rigid pavement.

BACKGROUND ON EXISTING RENEWAL APPROACHES

A search for information on highway renewal using existing pavements was conducted for both flexible and rigid renewal types. In addition, questionnaires were distributed to SHA and international representatives to solicit input on experience with the various renewal approaches. The following sections provide an overview of both relevant literature and practitioners' experience. Details on the literature review can be found in Appendix A.

Rigid Renewal

Long life rigid renewal strategies involve concrete overlays. Smith, et al (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 selection and timing is dependent on factors such as the condition of the existing pavement; the rate of deterioration of the distress; the desired performance; lane closures and traffic control considerations; and user costs.

Recent concrete overlay terminology was described by Harrington (2008). These definitions provide a straightforward description of concrete overlays as shown in Figure 3.2. 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.

Detained performance observations of bonded concrete overlays were obtained from TxDOT, WSDOT, and MnDOT. Observed performance of 4 to 8 inch bonded overlays by TxDOT personnel indicated that their thicker bonded CRCP overlays can be expected to perform up to 25 years; however, TxDOT only recommends a design life of five to 10 years for 4 to 7 inch bonded concrete overlays of asphalt pavements (TxDOT, 2011).

The literature and documented SHA experience with bonded concrete overlays is supported by the data within the Long Term Pavement Performance (LTPP) database (discussed in a subsequent section). This experience and performance data for slabs up to about 6 inches thick suggest that a 50 year life is unlikely for bonded concrete overlays.

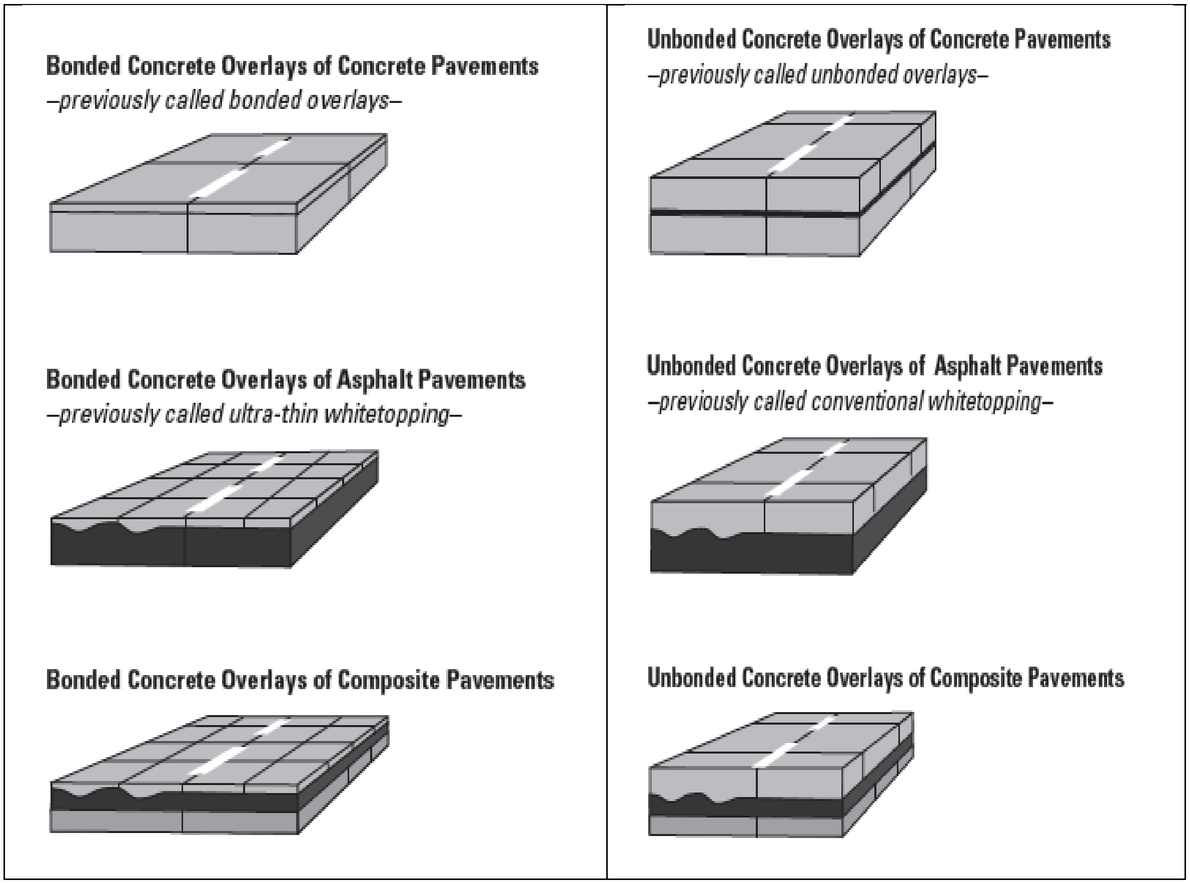


Figure 3.2. Types of Concrete Overlays.
(Harrington, 2008)

Harrington (2008) found that bonded overlays are used to “add structural capacity and/or eliminate surface distress when the existing pavement is in good structural condition. Bonding is essential, so thorough surface preparation is necessary before resurfacing.” Harrington also noted that unbonded overlays are used to “rehabilitate pavements with some structural deterioration. They are basically new pavements constructed on an existing, stable platform (the existing pavement).” This concept of unbonded concrete overlays being similar to new pavement construction is expanded below.

A best practices document by Tayabji and Lim (2007) overviewed a selection of design, materials, and construction features for new concrete pavements for four SHAs (Illinois, Minnesota, Texas, and Washington). These practices were updated based on recent information and summarized in Table 3.2 and Table 3.3. Minnesota and Washington were grouped together in Table 3.2 since their practices are for JPCP. Illinois and Texas are summarized in Table 3.3 to reflect their CRCP practices. While these practices were developed with new pavement construction in mind, they are applicable to long life concrete overlay systems. This type of information illustrates the following:

- For JPCP
  - Design lives range from 50 to 60 years.
  - Slab thicknesses range between 11.5 to 13.0 inches.
  - Joint spacings are 15 ft., doweled and corrosion resistant.
  - Maximum water/cementitious ratios range between 0.40 to 0.44
- For CRCP
  - Design lives range from 30 to 40 years.
  - Slab thicknesses range between 13.0 to 15.0 inches.

A more specific example of a long-lasting concrete overlay over pre-existing PCC comes from Washington State. WSDOT constructed an unbonded concrete overlay on I-90 over 35 years ago. Figure 3.3 is a photograph of this overlay taken in 2010. The overlay is still performing well as of 2011 with no observable distress.

Figure 3.3. Photo of 35 year old unbonded PCC overlay on I-90 MP 74.

Belgium is the only country outside the United States identified in this review as having reported appreciable experience with unbonded concrete overlays (Hall, 2007). Belgium constructed its first concrete overlay in 1960, over a concrete pavement originally constructed in 1934. The JRCP overlay is seven inches thick. Figure 3.4 shows the overlay still in service nearly 45 years later.

Table 3.2. Examples of Long life JPCP Standards for the MnDOT and WsDOT. (Tayabji and Lim, 2007; MnDOT, 2005: WsDOT, 2010)

Item	Minnesota DOT	Washington DOT
Design Life	• 60 years	• 50 years
Typical Structure	• Slab thicknesses = 11.5 to 13.5” • 3 to 8” dense-graded granular base • Subbase 12 to 48” select granular (frost-resistant)	• Slab thickness = 12 to 13” (typical) • 4” HMA base • 4” crushed stone subbase
Joint Design	• Spacing = 15’ with dowels • All transverse joints are doweled	• Spacing = 15’ with dowels • Joints saw cut with single pass • Hot poured sealant
Dowel Bars	• Diameter = 1.5” (typical) • Length = 15” (typical) • Spacing = 12” • Bars must be corrosion-resistant	• Diameter = 1.5” • Length = 18” • Spacing = 12” • Bars must be corrosion-resistant. Epoxy coatings not acceptable.
Outside Lane and Shoulder		• 14’ lane with tied PCC or HMA • 12’ 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	• 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 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%	• 5.0% ± 2.0%
Water/ Cementitious Ratio	• ≤ 0.40	• ≤ 0.44 • Minimum cementitious content = 564 lb/CY 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	

Table 3.3. Examples of Long life CRCP Standards for the Illinois and Texas DOTs. (Tayabji and Lim, 2007; TxDOT, 2011; TxDOT, 2009a; TxDOT, 2009b)

Item	**Illinois DOT**	**Texas DOT**
Design Life	• 30 to 40 years	• 30 years
Typical Structure	• Up to 14" CRCP slab • 4 to 6" HMA base • 12" aggregate subbase	• Up to 13" CRCP slab with one layer of reinforcing steel. • 14 to 15" CRCP slab with two layers of reinforcing steel. • Uses stabilized base either 6" CTB with 1" HMA bond breaker on top or 4" HMA • Recommends tied PCC shoulders
Tie Bars	• Use at centerline and lane-to-shoulder joints • Use 1" by 30" bars spaced at 24"	
CRCP Reinforcement	• Reinforcement ratio = 0.8% • Steel depth 4.5" for 14" slabs • All reinforcement in CRCP epoxy-coated	• Increased amount of longitudinal steel. • Design details for staggering splices.
Aggregate Requirements	• IDOT 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 to 90°F • Slipform pavers must be equipped with internal vibration and vibration monitoring • Curing compound must be applied within 10 minutes of concrete finishing and tining • Curing ≥ 7 days before opening to traffic	• Revised construction joint details.

Figure 3.4. Belgium's first concrete overlay after 45 years in service. (Photo: Hall 2007)

The study review found that design thicknesses of unbonded PCC overlays are typically greater than or equal to 9 inches. for Interstate applications. This is supported by data from LTPP. In a study by Smith et. al, (2002) a large number of unbonded overlay projects were identified and the Highway Agencies asked to rate their performance from good to poor. They found a strong correlation between thickness and performance as shown in Figure 3.5. This figure was generated based on expert opinion from the study perform by Smith et. al, (2002).It is evident that, for long life pavements in high traffic volume applications, the unbonded overlay thickness should be 9 inches or greater.

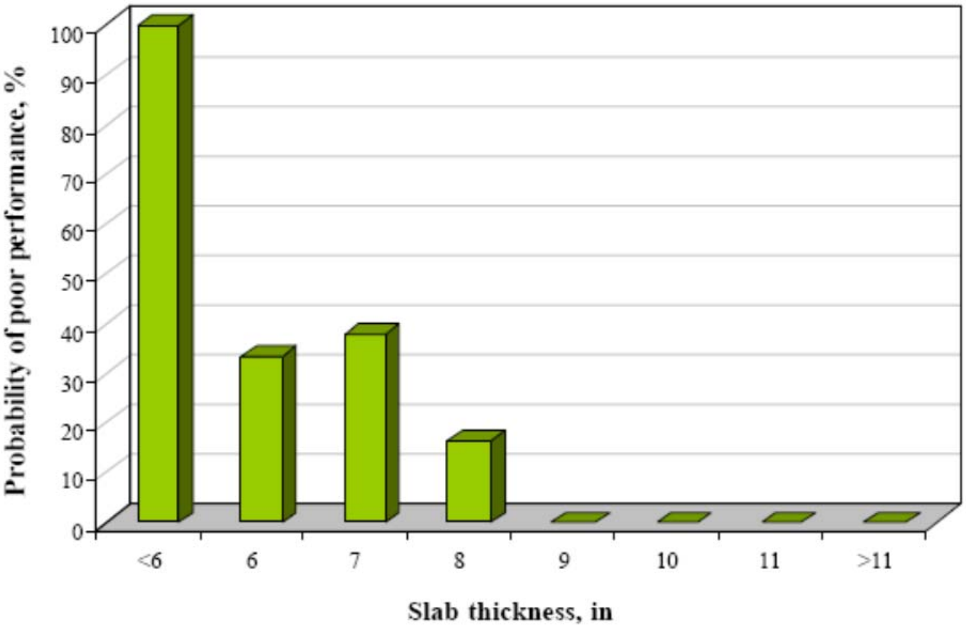


Figure 3.5. Probability of Poor Performance for Unbonded JPCP Overlays.
(Smith et al, 2002)

A recurring theme emerges in examining the literature/practices discussed above: (1) use of thick unbonded PCC slabs, (2) design lives are greater than or equal to 30 years ranging up to 60 years, and (3) PCC mix and materials requirements are important. Thus, long life PCC renewal options are not just about slab thickness but also materials and construction as expected. Key considerations include:

- Foundation support (uniformity, volumetric stability—including stabilizing treatments)
- Drainage design (moisture collection/removal and design for minimal maintenance)
- Concrete mixture proportioning and components (selected to minimize shrinkage and potential for chemical attack, low coefficient of thermal expansion (CTE), provide adequate strength, etc.)
- Dowels and reinforcing (corrosion resistance, sized and located for good load transfer)
- Construction parameters (including paving operations, surface texture, initial smoothness, etc.)
- QA/QC (certification, pre-qualification, inspection, etc.)

Flexible Renewal

The review of flexible renewal included the following seven approaches, each of which will be briefly described below.

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

HMA Overlay and Existing HMA Pavement

If there is no visible distress in the existing HMA pavement other than isolated areas, the existing pavement can be directly overlaid as long as it is determined to be structurally sound, level, clean, and capable of bonding to the overlay. However, when visible surface distress is present and it is determined (through coring) to be near the surface, milling prior to the overlay is required.

HMA Overlay and Reclaimed HMA

In cases where the surface of the existing HMA layer is in poor condition and the depth of the distress (cracking) is deeper in the pavement section, reclaiming the existing HMA pavement prior to the placement of new layers is required. 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 HMA thickness that is used to calculate the limiting tensile strain at the bottom of that layer.

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 long life. 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 include (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 de-lamination from the underlying layer; and/or (4) allowing a project to remain open for too long into the winter season. Also, excessive processing can result in higher fines content, leading to rutting due to low stability.

HMA Overlay and Crack and Seated JPCP

HMA over crack and seated JPCP is suitable for plain (unreinforced) concrete pavements. The performance of this renewal option has been variable in the U.S. This could be tied to the quality of the cracking operation. 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 reflective transverse cracks in the HMA overlay. If construction guidelines ensure closely spaced, tight, full-depth vertical cracks, then potential for long life should be achievable.

Experience in the United Kingdom has been excellent with crack and seat projects, but with a strict quality control process and a minimum HMA overlay thickness in excess of 6 inches (Jordan et al, 2008). Thinner overlays like those commonly used in the U.S. were not found to perform as well in test sections in the United Kingdom (Coley and Carswell, 2006). In addition, 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 feet by 6 feet followed by seating with five passes of a pneumatic-tired roller of at least 15 tons (Caltrans, 2008). For a number of years, the overlay thickness associated with the crack and seat process ranged from a minimum of four inches up to about six inches. Service life expectation was a minimum of 10 years with these thicknesses (or about 10 to 20 million ESALs). Starting in 2003 with the Interstate 710 rehabilitation of existing eight-inch thick PCC slabs near Long

Beach (Monismith et al, 2009), the crack and seat process is followed by HMA overlays totaling 9 in. thick. The design ESAL levels for these sections of I-710 have ranged between 200 to 300 million. This renewal strategy adopted by Caltrans implies a long life of at least 40 years.

HMA Overlay and Saw, Crack and Seat JRCP

The crack and seat technique of fracturing reinforced concrete pavements (JRCP) has generally not performed well because of the inability to shear the steel reinforcement or break the bond between the reinforcing steel and concrete. The bonded and un-sheared reinforcing steel results in thermal contraction concentrated at the existing transverse joints, thus leading to reflective cracks through the HMA layer.

An alternative solution used primarily in the United Kingdom (UK) is the saw, crack, and seat approach, which involves sawing narrow transverse cuts into the concrete deep enough to cut through the longitudinal steel reinforcement, then cracking the pavement at the locations of the sawed cuts using the same crack and seat procedure described above (Merrill, 2005). Verification coring should follow to ensure that fine, full-depth, vertical cracks are achieved. The UK Department of Transport Road Note 41 (Jordan et al, 2008) recommends a saw and crack spacing of three to six feet. Under these conditions, the critical features and limitations are the same as for the crack and seat approach. Similar to crack and seating, thicker overlays were found to perform substantially better than thinner overlays in test sections in the United Kingdom (Coley and Carswell, 2006).

HMA Overlay and Rubblized JRCP and JPCP

The rubblization approach effectively eliminates the problem of reflection cracking, since the technique is supposed to completely disintegrate the existing concrete slab and de-bond the concrete from the reinforcing steel. However, this also reduces the strength of the existing concrete pavement substantially since it renders the concrete into broken fragments resembling an unbound base course, though with “aggregate” sizes much larger than a regular crushed aggregate base layer. Thus, it is the only approach that uses the existing concrete pavement and fully addresses slab movement responsible for reflective cracking, particularly for JRCP.

Von Quintus et al. (2007) reviewed the performance of HMA overlays of PCC pavements from the 2005 LTPP database. Those findings suggest that sections without edge drains or those with rubblized pieces less than two inches in size exhibit higher levels of distress.

State SHA experience indicates construction difficulties with rubblization if the foundation underneath the existing concrete is not sufficiently strong. The rubblization process can damage the base/subbase and/or the existing subgrade and produce an unstable condition. Sebasta and Scullion (2007) refined a risk assessment methodology for rubblization first developed in Illinois (Heckel 2002) based on dynamic cone penetrometer testing (this process is fully described in the flexible best practices document provided in Appendix E-2).

HMA Overlay and Existing Composite Pavement

HMA overlays of existing HMA surfaced composite pavements is also a viable long life HMA renewal solution. Sebasta and Scullion (2007) recommend milling the old HMA overlay completely off to expose the existing PCC pavement. The PCC pavement should be modified using either the crack/seat, saw/break/seat, or rubbilization approaches described above.

HMA Overlay and Existing CRCP

HMA over existing CRCP has significant potential to provide long-life. 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 HMA overlay as long as the surface of the existing CRCP is in good condition and a good bond between the HMA overlay and the CRCP is achieved. This solution should lead to thinner HMA overlays compared to HMA over existing jointed concrete pavements. The main limitation of this renewal strategy is any untreated or improperly treated defect in the existing CRCP can develop into a major repair. Studies have shown that the placement of HMA overlays can accelerate D-cracking, resulting in poor performance of HMA overlays (Zollinger et al., 2004). Therefore, this approach would only apply to CRCP in good condition. Also, if bonding is not properly ensured, water caught between the HMA overlay and the existing CRCP can lead to stripping and HMA deterioration. Finally, the performance of HMA overlays on CRC pavements has been variable in the U.S. 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.

Regardless of the flexible renewal approaches reviewed, the following principles are required to achieve good performing long life pavements:

- 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 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 sufficient.
- Foundation uniformity is essential to reduce/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).

LTPP ANALYSES

The research team reviewed the LTPP database to provide insight into performance of various renewal approaches. A detailed analysis was made of the available, appropriate data. The analyses for both flexible and rigid pavements experiments are shown in Appendix B. In addition, selected projects from the LTPP database were examined by mechanistic-empirical design programs, (MEPDG and PerRoad), to determine whether the basic roadway sections were likely to provide long life pavements and to define critical features and limitations. The following is a summary of both the LTPP and related MEPDG analyses.

Rigid Pavements

The General Pavement Study 9 (Unbonded PCC Overlay on PCC Pavement) and Specific Pavement Study 7 (Bonded PCC Overlay on PCC Pavement) experiments were reviewed. The information for both experiments was extracted from the "LTPP DataPave Online" database (Release 21). The pavement performance criteria selected for the summary included transverse cracking, International Roughness Index (IRI), joint and crack faulting (JPCP), and punch-outs (CRCP only).

The original construction (pre-overlay) date for the unbonded PCC overlay sections ranged from the early 1950s to the mid-1970s. The actual overlays included both JPCP and CRCP. The average age of overlays until the test sections were taken out of service was about 17 years. The overlay thicknesses of the various test sections ranged from 5.8 to 10.5 inches with an average joint spacing of 16 feet. The load transfer mechanisms were either aggregate interlock or dowel bars. While a significant fraction of these unbonded PCC overlay GPS-9 test sections have potential for long life performance, all were monitored for less than 20 years. Figure 3.6 provides a summary of transverse cracking as a function of overlay thickness for JPCP overlay sections. As can be seen, there is a clear difference in performance when overlay thicknesses are greater than eight inches thick. It should be noted that this finding is very similar to that from Smith et. al (2002) and shown in Figure 3.5.

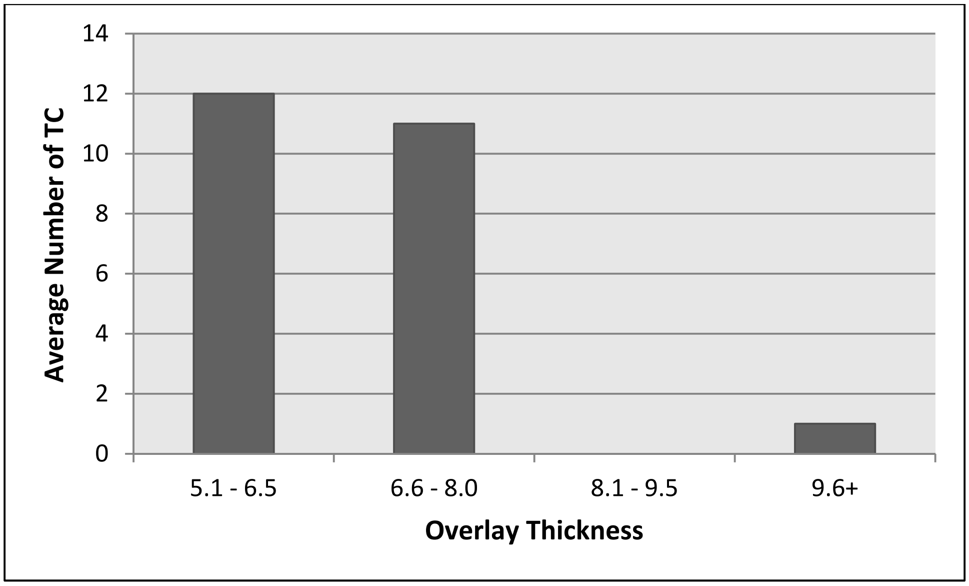


Figure 3.6. JPCP Overlay thickness versus average number of transverse cracks.

The LTPP SPS-7 experiment included bonded PCC overlays on PCC pavement. Data from 18 CRCP, nine JPCP, and eight PCC (unreinforced PCC overlays of CRCP) test sections were analyzed. However, these 35 test sections only represent pavements in four locations since multiple SPS-7 test sections were constructed at each project. This is a limited dataset given the grouping of test sections.

The average age of overlays at the time the test sections were taken out of the LTPP study (and no longer monitored) was about 15 years. The overlay thicknesses of the various test sections ranged from 3.1 to 6.5 inches., The bonded overlays exhibited significant transverse cracking after 15 years of service and are unlikely candidates for long life renewal.

Because of the limited nature of this experiment, it is difficult to assess the likelihood that bonded concrete overlays will provide a long life service. This was confirmed with numerous discussions and project evaluations with TxDOT and other SHAs during Phase 2. Observed performance along with M-E analyses implies performance lives of up to 35 years, but 50 year service lives are unlikely.

Flexible Pavements

The following LTPP experiments were reviewed to determine the pavement life achieved for HMA surfaced pavements where the existing pavement remained in place:

- GPS-6A, Existing AC Overlay of AC Pavement
- GPS-6B, AC Overlay with Conventional Binder of AC Pavement
- GPS-7A, Existing AC Overlay of PCC Pavement
- GPS-7B, AC Overlay with Conventional Binder of PCC Pavement

- SPS-5, Rehabilitation Strategies for AC Pavement
- SPS-6, Rehabilitation Strategies for PCC Pavement

Performance data including longitudinal cracking, fatigue cracking, transverse cracking, rut depth, and IRI were plotted against pavement age. Sections with the longest overlay ages were selected and traffic data (ESALs) corresponding to pavement age was extracted. The next step was to examine the better performing sections to determine potential long life pavement candidates. The majority of the test sections evaluated had overlays with ages of 16 years or less. Data from the GPS test sections with overlay ages of up to 34 years were available from the database.

While none of the test sections had overlays with 50 years of service, a selection of test sections exhibited performance that had the potential to meet the long life criteria. These test sections were analyzed using the Mechanistic-Empirical Pavement Design Guide (MEPDG) and PerRoad software to model each of the test sections for performance.

Limitations in the reflective cracking models, questionable predicted performance curves, and inability of the MEPDG to produce results using HMA over CRCP limited findings that could be applied to the long life renewal objectives of the project. Due to this, the sections were analyzed using the PerRoad software, which is based on the mechanistic-empirical approach of calculating pavement response mechanistically and estimating damage using empirical transfer functions and Miner’s rule. It uses the concept of limiting strain criteria. PerRoad estimates the time (i.e., pavement life) at which damage accumulation reaches 0.1 according to Miner’s rule.

PerRoad results indicated that actual traffic loadings produced horizontal and vertical strains well below reasonable thresholds. While the field performance observations only captured about 16 years of actual performance, there is virtually no fatigue damage observed for the sites. These findings support the notion that structural designs of flexible renewal alternatives satisfying the limiting strain criteria for fatigue and subgrade rutting have the potential to achieve long life service, assuming all other critical features are satisfied. PerRoad does not account for reflective cracking in its analysis, which must be considered when selecting the appropriate level of modification to the existing pavement structure.

ASSESSMENT OF RENEWAL APPROACHES

The results of the prior work were used to develop features and limitations for each renewal alternative, which follow.

Rigid Pavements

Table 3.4 provides an overview of critical features and limitations associated with the rigid pavement renewal approaches.

Table 3.4. Summary of Rigid Renewal Features and Limitations.

Approach	Critical Features	Limitations
Unbonded PCC overlay over existing PCC	• Overlay thickness is critical to performance • Repair locally failed areas • Stable subbase • 1.5" diameter rust resistant dowels • 15' max joint spacing • Interlayer should not trap water • Thicker HMA interlayer performed better • Adequate drainage	• Significant surface elevation increase • Consistent foundation support when widening • Consistent drainage when widening
Unbonded PCC overlay over existing HMA	• Overlay thickness is critical to performance • Locally failed areas must be repaired • Stable subbase • 1.5" diameter rust resistant dowels • 15' max joint spacing • Adequate drainage	• Significant surface elevation increase • Consistent foundation support when widening • Consistent drainage when widening
Bonded PCC overlay over existing PCC	• Adequate surface preparation • Bonding is a critical feature • Locally failed areas must be repaired • Match joint location, width, type of existing PCC • Adequate drainage	• Existing pavements with materials related distress are not good candidates • Existing pavement with voids are not good candidates • Working cracks can cause debonding of overlay • Service life up to 35 years

Flexible Pavements

Table 3. 5 provides a summary of critical features and limitations to each of the flexible pavement renewal approaches.

Table 3. 5 Summary of Flexible Renewal Features and Limitations.

Approach	Critical Features	Limitations
HMA over Existing HMA	• Absence/removal of full depth cracking • Good foundation support • No stripping • Adequate drainage	• Reconstruction required if base/subgrade is poor • Milling required to remove surface cracking
HMA over Reclaimed HMA	• Good foundation support • Adequate drainage • Proper surface prep/tack coat	• Cement/emulsion have not yet been proven in field for long life • Reclaimed layer considered as base material
HMA over Existing CRCP	• Good foundation support • Adequate drainage • No evidence of pumping • Existing pavement is structurally adequate • Absence/repair of major defects • Good bond of CRCP and HMA	• CRCP has to be in good condition with few major defects (which must be repaired) • Inadequate bonding can lead to poor performance • Unproven for 50 year life
HMA over Crack/Seat JPCP	• Good foundation support • Adequate drainage • No evidence of pumping	• Inability to break and seat JRCPhas been documented • Crack and seat is not viable for reinforced PCC
HMA over Saw, Crack /Seat JRCP	• Good foundation support • Adequate drainage • No evidence of pumping	• Saw cuts must extend below reinforcement
HMA over Rubblized	• Good foundation support • Adequate drainage • No evidence of pumping	• Pavement needs to be adequately drained prior to rubblization • Performance tied to quality of rubblization process • When subgrade conditions are inadequate, significant damage to base/subgrade has created construction problems

Advantages and Disadvantages of Renewal Approaches

Table 3.6 provides an overview of the rigid renewal approaches and Table 3.7 provides flexible renewal approaches in the context of advantages and disadvantages. Rigid pavement renewal options require less modification to the existing pavement. As such, there are fewer approaches listed for the rigid renewal as compared to the flexible alternatives.

Table 3.6. Advantages and Disadvantages of Rigid Renewal Approaches.

Rigid Renewal Approach	Advantages	Disadvantages
Unbonded Concrete Overlay over PCC	• Very good long term performance with minimal maintenance/rehabilitation • Insensitive to existing pavement condition • Best documented record of projects in place that have achieved long life	• Significant surface elevation gain • Placement/cure time may make work zone management difficult.
Bonded Concrete Overlay	• Smallest vertical elevation gain	• Unlikely to be viable for service lives above 35 years
Unbonded Concrete Overlay over HMA	• Requires little preparation to existing pavement • Easily accommodates lane addition	• Significant surface elevation gain • Placement/cure time may make work zone management difficult.

Table 3.7. Advantages and Disadvantages of Flexible Renewal Strategies.

Flexible Renewal Approach	Advantages	Disadvantages
HMA over Reclaimed/Milled HMA Pavement	• Elimination of all existing deterioration that could lead to reflective cracking	• Existing pavement is considered base material in renewal structural design • Significant thickness of new HMA required over modified existing structure
HMA over CRCP	• Utilizes CRCP in-place as part of renewal	• HMA wearing surface will need to be removed/replaced at 10-20 year cycles
HMA over Crack/Seat	• Does not diminish the structural competency as much as rubblization	• Reflection cracking risk • Performance dependent on quality of crack/seat • HMA wearing surface will need to be removed/replaced at 10-20 year cycles
HMA over Saw/Crack/Seat	• Does not diminish the structural competency as much as rubblization	• Reflection cracking risk • Performance dependent on quality of crack/seat • HMA wearing surface will need to be removed/replaced at 10-20 year cycles • Costs associated with sawing
HMA over Rubblized	• Elimination of features in existing pavement that cause reflective cracking • Stiffness of rubblized material greater than granular base • Total HMA thickness is less than over granular base	• Construction risk with weak/wet subgrade • Performance dependent on quality of rubblization process • Poorly rubblized material cannot be improved through additional rubblization • Raises surface elevations • HMA wearing surface will need to be removed/replaced at 10-20 year cycles

Based on the preceding findings, decision matrices were developed with the intent of establishing a list of feasible alternatives for a project based on existing conditions. These matrices were submitted in draft form for review and comment by the study review panel. Details are discussed in the following section.

DESIGN GUIDES

The design guide development started with development of a draft decision process based on the initial study results. The following strategies were included:

- HMA over existing HMA
- HMA over reclaimed/milled existing HMA pavement
- HMA over existing CRCP
- HMA over crack/seat existing JPCP
- HMA over saw/crack/seat existing JRCP
- HMA over rubblized existing JPCP/JRCP
- Unbonded concrete overlay over existing JPCP/JRCP/CRCP
- Unbonded concrete overlay over existing HMA

Development of Decision Matrices

A set of decision matrices, organized as tables, were developed to aid the scoping of renewal strategies. Separate matrices, with associated decision paths, were developed for selecting renewal options for the various, existing pavement types. The existing pavement types include flexible, rigid (JPCP, JRCP, and CRCP), and composite. Specific decision tables, regardless of existing pavement type, use four levels of decision-making as illustrated in Table 3.8. The surface condition of the existing pavement is the primary information required for starting the renewal decision-making process.

Table 3.8. Organization and Decision Levels for Renewal Selection.

Decision Levels	**First Level**		**Third Level**	**Fourth Level**	
Basis for Decision	Identify Distress Category	Identify Specific Distress Type within a Distress Category	Select Renewal Pavement Type	Renewal Action	Design Resources
Details associated with each decision. Decision tables a function of the existing pavement type.	**Existing Flexible** • Environmental Cracking • Materials Caused Distress • Full Depth Fatigue Cracking • Top Down Cracking	• Transverse or Block Cracking • Stripping • Longitudinal or Alligator Cracking	Either a **flexible or rigid** option can be selected for each specific distress or category.	Describes what is to be done to the existing pavement and the type of renewal strategy.	Describes the design resources to be used to complete the scoping process.
	Existing JPCP or JRCP • Materials Caused Distress • Pavement Cracking • Joint Faulting and Movement • Pumping	• D-Cracking • Alkali-Silica Reactivity (ASR) • Fault Depth • Joint Deflection			
	Existing CRCP • Punch-outs	--			
	Existing Composite • Surface Course Condition	--			

The guidelines were developed to help designers in selecting either a rigid or flexible reconstruction approach that can reasonably be expected to provide long life pavement performance. For this project, long life performance was defined as providing 50 years or service without major structural deterioration. It is anticipated that any approach selected will require some form of rehabilitation or resurfacing during the service life of the pavement. The final selection of the most appropriate design should be based on a life cycle cost analysis of the various approaches including all rehabilitation or resurfacing costs over the life of the pavements.

The development of the decision matrices followed a process where team members laid out an outline of the decision process in outline form. The outline had the basic form seen on the tables with pavement type, distress present and potential renewal approaches for those conditions. The outline was circulated to the full team and modified as additional considerations were added. The outline was presented at the kickoff meetings then circulated among the participating Agencies for comment, and again adjustments were made. To make the process clearer the decision matrix was put in a set of tables. The tables were then circulated again to the full R23 Team and the participating Agencies who provided more comment (most likely because the tables were easier to follow than the outlines). The tables were then used to build an interactive Flash based program that would simplify using the decision matrix. In building the logic for the interactive program, a few more decision points were added based on the more rigorous nature of that process. After the program was developed, it was evaluated through a series of trials that included a wide range of potential applications. The decision tables were adjusted again based on errors or omissions found in that process. The interactive program and the decision tables were again presented to the participating Agencies for review and comment and final adjustments were made to the program and the tables presented in this report.

The decision matrices account for deterioration/surface distress types present in existing pavement as well as structural response (i.e., deflections) and subgrade conditions. They provide a feasible set of alternatives based on in situ conditions of the existing pavement. For example, if full-depth cracking is present and the quantity is large enough to make full depth patching cost-prohibitive, the decision matrix eliminates the alternative to mill existing HMA and overlay with new HMA. Instead, for the flexible alternative, full depth pulverization or reclamation is the recommended along with HMA overlay. The decision process would also recommend an unbonded concrete overlay as a rigid renewal alternative.

The intent of the decision matrices is to provide a set of feasible long life alternatives. Both flexible and rigid renewal options are included as outputs. The decision matrices are shown below in Table 3.9 (renewal of existing flexible pavement), Table 3.10 through Table 3.11 (renewal of existing JPCP and JRCP), Table 3.12 (renewal of existing CRCP), and Table 3.13 (renewal of composite pavements).

Table 3.9. Feasible Renewal Alternatives for Existing Flexible Pavements.

Distress Category	Specific Distress Description	Distress present?	Renewal Pavement Type Option	Action	Design Resources
Environ-mental Cracking	Transverse or Block Cracking	Yes	Flexible	Pulverize pavement structure full-depth followed by a thick AC overlay.	Pulverize and use residual material as untreated base (50 ksi). Apply AC thickness from Tables 3.14-3.16.
					Pulverize and treat residual material with emulsion or foamed asphalt resulting in a treated base (100 ksi). Apply AC thickness from Tables 3.14-3.16.
			Rigid	No mitigation required, place an unbonded PCC overlay.	Use Table 3.17 for thickness determination of an unbonded PCC overlay.
		No	--	Continue to Materials Caused Distress.	--
Materials Caused Distress	Stripping	Yes	Flexible	If stripping is found through all layers, pulverize pavement structure full-depth followed by a thick AC overlay.	Pulverize and use residual material as untreated base (50 ksi). Apply AC thickness from Tables 314-3.16.
					Pulverize and treat residual material with emulsion or foamed asphalt resulting in a treated base (100 ksi). Apply AC thickness from Tables 3.14-3.16.
				If stripping is found in specific layers, remove AC to maximum depth of stripping followed by a thick AC overlay.	Use Tables 3.14-3.16 with 30 ksi base and the subgrade M_R to determine total depth of AC thickness then subtract remaining AC thickness to determine overlay thickness.
			Rigid	Place unbonded PCC overlay. If grade limits require, mill existing pavement. AC overlay over stripped pavement may be required to stabilize HMA.	Use Table 3.17 for thickness determination of an unbonded PCC overlay.
		No	--	Continue to Full Depth Fatigue Cracking.	--
Full Depth Fatigue Cracking	Longitudinal or Alligator Cracking in Wheel paths	Yes	Flexible	<15% fatigue cracking: patch and repair, moderate thickness AC overlay.	Use Tables 3.14-3.16 with 30 ksi base for AC overlay thickness, then subtract existing AC thickness to determine overlay thickness.
				>15% fatigue cracking: pulverize pavement structure full-depth followed by a thick AC overlay.	Pulverize and use residual material as untreated base. Apply AC thickness from Tables 3.14-3.16 with 50 ksi base.
					Pulverize and treat residual material with emulsion or foamed asphalt resulting in a treated base. Apply AC thickness from Tables 3.14-3.16 with 100 ksi base.
			Rigid	Patch severely cracked areas, place an unbonded PCC overlay. Profile elevation may require milling existing AC pavement.	Use Table 3.17 for thickness determination of an unbonded PCC overlay.
		No	--	Continue to Top Down Cracking.	--
Top Down Cracking	Longitudinal or Alligator Cracking in Wheel paths	Yes	Flexible	< 15% patch and overlay	Use Tables 3.14-3.16.with 30 ksi base and the subgrade M_R to determine total depth of AC thickness, then subtract the thickness milled out to eliminate the top down cracking (unless indicated the assumed depth is 2 inches). Where patching only, subtract existing depth to calculate overlay.
				>15% Mill down to bottom of cracking followed by a moderate thickness AC overlay.	
			Rigid	Place an unbonded PCC overlay.	Use Table 3.17 for thickness determination of an unbonded PCC overlay.

Table 3.10. Feasible Renewal Alternatives for Existing JPCP and JRCP Pavements.

Distress Category	Specific Distress Description	Distress Present?	Renewal Pavement Type Option	Action	Design Resources
Materials Caused Distress	D-Cracking with Light Severity	Yes	Flexible option for JPCP	Rubblization or crack and seat JPCP followed by a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Apply Rule 2.
			Flexible option for JRCP	Rubblization or saw, crack and seat JRCP with a thick overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Saw, crack and seat existing PCC followed by application of AC overlay from Table 3.14-3.16; otherwise, Rule 2 applies.
			Rigid option	Apply 2 inch AC overlay bond breaker followed by an unbonded PCC overlay.	Apply Rule 3 shown below.
		No	--	Continue to next level of D-Cracking.	--
	D-Cracking with Moderate to High Severity	Yes	Flexible option with rubblization if subgrade meets TTI guidelines	Rubblize followed by a thick AC overlay. For rubblization, apply TTI guidelines.	Apply Rule 1.
			Flexible option if does not meet TTI guidelines for rubblization	Do not use the existing pavement, requires all new pavement.	--
			Rigid option	Full depth patch and apply 2 inch AC overlay bond breaker followed by an unbonded overlay.	Apply Rule 3.
		No	--	Continue to ASR.	--
	Alkali-Silica Reactivity (ASR)	Yes	Flexible option	Rubblize followed by thick AC overlay. For rubblization, apply TTI guidelines.	Apply Rule 1.
			Rigid option	Patch plus 2 inch AC bond breaker followed by unbonded PCC overlay.	Apply Rule 3.
		No	--	Continue to Pavement Cracking.	--
Pavement Cracking	% Multiple Cracked Panels	Yes	Flexible option for low to moderate multiple cracked panels (1 to 10% of panels)	Rubblization or crack and seat JPCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
			Rigid option for low to moderate multiple cracked panels (1 to 10% of panels)	Place a 2 inch AC bond breaker followed by an unbonded PCC overlay.	Apple Rule 3.
			Flexible option for moderate to high multiple cracked panels (>10% of panels)	If subgrade meets or exceeds TTI criteria, apply rubblization followed by a thick AC overlay.	Apply Rule 1.
				If subgrade does not meet TTI criteria, options include crack and seat or do not use existing pavement.	Apply Rule 2
			Rigid option for moderate to high multiple cracked panels (>10% of panels)	Replace rocking or shattered slabs followed by a 2 inch AC overlay bond breaker followed by an unbonded PCC overlay.	Apply Rule 3.
		No	Continue to Joint Faulting (Table 2b)		

Table 3.11. Feasible Renewal Alternatives for Existing JPCP and JRCP Pavements (continued).

Distress Category	Specific Distress Description	Distress Present?	Renewal Pavement Type Option	Action	Design Resources
Joint Faulting	--	Yes	Flexible option for low faulting (< 0.25 inches)	Rubblization or crack and seat JPCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Apply Rule 2.
				Rubblization or saw, break and seat JRCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Saw, crack and seat existing PCC followed by application of AC overlay from Table 3.14-3.16; otherwise, Rule 2 applies.
			Rigid option for low faulting (< 0.25 inches)	Place a 2 inch AC overly followed by an unbonded PCC overlay.	Apply Rule 3.
		Yes	Flexible option for high faulting (> 0.25 inches)	Rubblization or crack and seat JPCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Apply Rule 2.
				Rubblization or saw, break and seat JRCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Saw, crack and seat existing PCC followed by application of AC overlay from Table 3.14-3.16; otherwise, Rule 2 applies.
			Rigid option for high faulting (> 0.25 inches)	Place a 2 inch AC overlay followed by an unbonded PCC overlay. If joint deflections > 40 mils (0.040 inches), then consider crack and seat JPCP or saw, break and seat JRCP to stabilize slabs.	Apply Rule 3.
		No	--	Continue to Pumping.	--
Pumping	--	Yes	Flexible	Crack and seat JPCP with a thick AC overlay if the drainage can be improved.	Apply Rule 2.
				Saw, crack and seat JRCP with a thick AC overlay if the drainage can be improved.	Saw, crack and seat existing PCC followed by application of AC overlay from Table 3.14-3.16; otherwise, Rule 2 applies.
				If drainage cannot be improved, then AC based renewal should not be used.	--
			Rigid	If joint deflections > 40 mils (0.040 inches), consider crack and seat followed by a 2 inch AC bond breaker followed by an unbonded PCC overlay. Drainage must be improved.	Apply Rule 3.
		No	--	--	--

Table 3.12. Feasible Renewal Alternatives for Existing CRCP Pavements.

Distress Category	Specific Distress Description	Distress Present?	Renewal Pavement Type Option	Action	Design Resources
Punchouts	--	Yes	Flexible option with ≤5 punchouts per mile	Repair all punchouts; place thick AC overlay to achieve a longer service life.	Apply AC overlay from Tables 3.14-3.16. The selection of the AC thickness is based on a drop-down menu of subgrade moduli = 5,000 psi, 10,000 psi, or 20,000 psi. The existing pavement shall be characterized by one of four possible moduli to select from: 30,000 psi, 50,000 psi, 75,000 psi, or 100,000 psi.
			Rigid option with ≤5 punchouts per mile	Repair major punchouts if slab load support in question. Follow repairs with a 2 inch AC bond breaker followed by an unbonded PCC overlay.	Apply Rule 3.
			Flexible option with >5 punchouts per mile	Rubblization of CRCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
			Rigid option with >5 punchouts per mile	Repair major punchouts if slab load support in question. Follow repairs with a 2 inch AC bond breaker followed by an unbonded PCC overlay.	Apply Rule 3.
		No	--	--	--

Table 3.13. Feasible Renewal Alternatives for Existing Composite Pavements.

Distress Category	Specific Distress Description	Distress Present?	Renewal Pavement Type Option	Action	Design Resources
Surface course in fair to poor condition	Can be a range of distress types. For the underlying PCC, these are mostly cracking related.	Yes	Flexible Option	Remove existing AC surface(s). Apply rubblization if meets TTI criteria.	Apply **Rule** 1.
				Remove existing AC surface(s). Use crack and seat or saw, crack and seat.	Following crack and seat or saw, crack and seat of existing PCC pavement, apply **Rule** 2.
			Rigid option	Place unbonded PCC overlay. If grade limits require, mill existing AC pavement.	Apply **Rule** 3.
Surface course in very poor condition	Can be a range of distress types. For the underlying PCC, these can include severe D-cracking, ASR, etc.	Yes	Flexible option	Remove and replace existing pavement structure.	--
			Rigid option	Place unbonded PCC overlay. If grade limits require, mill existing AC pavement.	Apply **Rule** 3.

Three rules are commonly referenced in Table 3.9 through Table 3.13 under the "Design Resources" column. Each rule is described below:

> **Rule 1**: Rubblization of existing PCC followed by application of AC overlay are detailed in Appendix E. Rubblization guidelines include the following:
>
> - If the subgrade MR < 6,000 psi or CBR < 4%, do not rubblize, thus defaulting to crack and seat only.
> - If the subgrade MR ≥ 6,000 psi but < 10,000 psi, consult the TTI rubblization guidelines as to whether rubblization is viable (Sebesta and Scullion, 2006).
> - If the subgrade MR ≥ 10,000 psi, then rubblization is a viable option.
>
> The selection of the AC thickness is based on a drop-down menu of subgrade moduli = 5,000 psi, 10,000 psi, or 20,000 psi. The existing pavement shall be characterized by one of four possible moduli: 30,000 psi, 50,000 psi, 75,000 psi, or 100,000 psi. It is recommended that an existing pavement modulus = 50,000 psi be used to reflect rubblized PCC.
>
> **Rule 2**: Crack and seat existing PCC followed by application of AC overlay from Table 3.14 through Table 3.16 (following). The selection of the AC thickness is based on a drop-down menu of subgrade moduli = 5,000 psi, 10,000 psi, or 20,000 psi. The existing pavement shall be characterized by one of four possible moduli: 30,000 psi, 50,000 psi, 75,000 psi, or 100,000 psi. It is recommended that an existing pavement modulus = 75,000 psi be used for crack and seated PCC to produce thickness in line with those recommended in TRL Road Note 41.
>
> **Rule 3**: Use Table 3.17 (following) for thickness determination of an unbonded PCC overlay and place on a 2 inch thick AC bond breaker. The unbonded PCC overlay thickness is independent of subgrade support conditions.

Development of Renewal Thickness Designs

The thickness design tables referenced in the decision matrices were developed as part of this study. These thicknesses provide approximate ranges for scoping purposes. They can also be used as a starting point for project level design, but the agency specific design methodologies should be used to develop the final thickness design.

The flexible renewal thickness design tables were developed using the limiting strain approach via the PerRoad, Version 3.5 software. Numerous scenarios were analyzed using PerRoad, including combinations to account for the following factors:

- Traffic levels
- Subgrade stiffness
- Base stiffness
- Base thickness

- Seasonal temperatures (from climatic data) for 5 locations
- Standard PG binder specifications for 5 locations
- Tensile strains at bottom of HMA layer
- Damage ratio scenarios (0.1 at 10 years and 0.1 at 50 years)

For each combination of factors, PerRoad was run iteratively to find the HMA thickness that would provide a damage ratio less than or equal to 0.1 at 10 years and 50 years of service life. Details about the analyses can be found in Appendix E. The final design thicknesses used in the guidelines are shown in Table 3.14 through Table 3.16. These thicknesses are representative of analyses from five U.S. locations. It is expected that any agency using the guidelines will refine the design thickness using their standard design procedure.

Table 3.14. Flexible Renewal Designs (Subgrade M_R = 5,000 psi)

ESALs (millions)	**Existing Pavement or Base Modulus**			
	30,000 psi	**50,000 psi**	**75,000 psi**	**100,000 psi**
≤10	10.0	9.0	8.0	6.0
10-25	11.0	10.0	8.5	6.5
25-50	12.0	11.0	9.0	7.0
50-100	13.0	11.5	9.5	7.5
100-200	14.0	12.0	10.0	7.5

Table 3.15. Flexible Renewal Designs (Subgrade M_R = 10,000 psi).

ESALs (millions)	**Existing Pavement or Base Modulus**			
	30,000 psi	**50,000 psi**	**75,000 psi**	**100,000 psi**
≤10	10.0	8.0	7.0	6.0
10-25	11.0	9.0	8.0	6.5
25-50	12.0	9.5	8.5	7.0
50-100	12.0	10.0	8.5	7.0
100-200	13.0	11.0	9.0	7.0

Table 3.16. Flexible Renewal Designs (Subgrade M_R = 20,000 psi).

ESALs (millions)	**Existing Pavement or Base Modulus**			
	30,000 psi	**50,000 psi**	**75,000 psi**	**100,000 psi**
≤10	9.5	7.5	6.5	5.5
10-25	10.0	8.5	7.0	6.0
25-50	11.0	9.0	7.5	6.5
50-100	11.5	9.5	8.0	6.5
100-200	12.0	10.0	8.5	7.0

The rigid renewal thickness design tables were developed in a similar fashion as the flexible renewal alternatives. AASHTO 1993 as well as the MEDPG version 1.1 were used in the development of the rigid thickness design tables. AASHTO 1993 was used during a preliminary investigation on thickness requirements. Numerous iterations were conducted using the MEPDG to account for the following factors:

- Traffic levels
- Performance criteria thresholds
- Mixture properties
- Shoulder support
- Geographic location

Initial evaluations indicated that for purposes of thickness tables for the guidelines, Baltimore, Maryland would provide results that were representative for the range of climates found in the U.S. The default calibration coefficients in the MEPDG were used in the analysis and yielded results that were similar to those of other geographic locations. The results were also compared to thickness catalog recently developed by WSDOT for long life concrete pavement projects based on MEPDG runs calibrated to actual pavement performance in Washington State.

The final design thicknesses selected for use in the guidelines are provided in Table 3.17. Results from the assessment of the LTPP test sections along with findings from prior studies suggest that unbonded concrete overlay thicknesses greater than 8.5 inches exhibit long life potential. Complete details on the analysis conducted in developing the rigid design thickness table can be found in Appendix D.

Table 3.17. Rigid Renewal Designs (AASHTO 93, MEPDG, and WsDOT Results).

ESALs (millions)	**AASHTO 93 for k = 500 pci**	**Design Thicknesses from WSDOT Pavement Policy**	**Thickness Range for MEPDG for M_R = 5 to 10 ksi**	**PCC Slab Thickness For R23 Study (inches)**
≤ 10	10.0	9.0	7.75-8.25	**8.5**
10-25	11.5	10.0	8.75-9.0	**9.5**
25-50	12.5	11.0	9.25	**10.5**
50-100	14.0	12.0	11.5-12.25	**11.5**
100-200	15.5	13.0	11.25-15.5	**13.0**

Validation

There were a number of visits made to the participating agencies throughout this project to solicit feedback on the guidelines.

During the first set of visits, the findings from Phase 1 along with objectives of the project were discussed. Additionally, field visits were made to multiple renewal projects throughout each agency.

A second round of visits with the agencies focused on soliciting feedback on the decision matrices and thickness design tables, as well as the resource documentation. The interactive software was in beta version for many of these meetings. As such, the team provided access to the beta version along with presentations explaining the development and use of the software. Through this process, valuable comments were received from the agencies.

The team also worked with each agency to identify one project to be used as a test case—as noted in Chapter 2. This test case would be used to compare the agency's standard design approach for pavement renewal with the recommendations provided by the new guidelines. During the visit, the team acquired detailed design information on the project from each agency to be used as a test case. This included design traffic levels, existing pavement structure, subgrade conditions, Falling Weight Deflectometer (FWD) data (if available), materials test results, and any project constraints (e.g., maintenance of traffic, vertical clearances). In many cases, the team made a field visit to the project to conduct a visual assessment of the site and capture photographs of the pavement and drainage features. The following projects were used as test cases for this study:

- Michigan: I-75 in Cheboygan County
- Minnesota: I-35 in Chisago County
- Missouri: I-55 in Perry County
- Texas: US-75 Loy Lake Road to Exit 64
- Virginia: I-95 in Caroline County
- Washington: I-5 in Skagit County at Bow Hill

The data collected from each agency was used to develop a design report using the guidelines and interactive software. For each test case, feasible flexible and rigid renewal strategies were developed and documented. The results were compared to the agencies standard design approach for each project.

As an example, the test case for the Virginia DOT was on I-95, a major traffic corridor for that state. Maintenance of traffic was a major concern and a primary limiting constraint of the renewal strategy selected. For this particular test case, the analysis was expanded to include construction productivity, lane closure alternatives, and traffic impacts. Each of the scenarios was analyzed using CA4PRS and the results were summarized in a report.

In most cases, the recommendations differed between the guidelines and agency standard practice. This was mostly due to differences in thickness design methodologies and design life. The guidelines provide recommendations for 50 year service life while many of the agencies were designing for 20 to 30 years. In other cases, the agency adopted the recommendations that came out of the guidelines.

Table 3.18 provides a summary of the renewal strategies for each test case.

For the Washington test case, WSDOT estimated that using the existing pavement in the renewal process reduced the costs by over 25 percent compared to removing and replacing the existing pavement. There was also a comparable reduction in the time required for construction.

Table 3.18. Comparison of Study and Agency Renewal Approaches.

Agency	R23 Recommendation (Flexible)	R23 Recommendation (Rigid)	Agency Renewal Approach
MDOT	9" HMA over rubblized or 8" HMA over saw/crack/seat	9.5" UBCOL with 2" HMA bond breaker	8.5" HMA over rubblized PCCP
MnDOT	9" HMA over Pulverized ACP	10.5" UBCOL	6" bonded PCC OL (20 year design)
MoDOT	9.5" HMA over rubblized or 8.5" HMA over saw/crack/seat	10.5" UBCOL with 2" HMA bond breaker	8" UBCOL with 1" HMA bond breaker or 12" HMA over rubblized PCC
TxDOT	9.5" HMA over cracked and seated PCCP	11.5" UBCOL with 2" HMA bond breaker	6" bonded PCC OL (special test case)
VDOT	Mill 6" stripped HMA then place 9" new HMA	13" UBCOL	Mill all 10" HMA and replace with 12" HMA
WSDOT	Remove existing HMA over PCC, crack and seat PCC & place 7.5" HMA	10.5" UBCOL	Remove existing HMA over PCC, crack and seat PCC & place 8.4" HMA

The team organized and facilitated one pilot workshop in Washington and two regional workshops in Virginia and Missouri.

Near the end of each workshop, every participant was asked to complete a questionnaire. Overall, the participants viewed the guidelines as valuable and useful. In particular, the resource documentation (which is discussed in the next section) was viewed by attendees as excellent content for pavement designers. All comments received were reviewed and addressed in the final guidelines.

RESOURCE KNOWLEDGE BASE

The knowledge base assembled as part of the guidelines includes six documents developed specifically for this project. In addition, a number of other resources developed under separate research efforts have been included in the knowledge base as references.

- Project Assessment Manual
- Flexible Pavement Best Practices
- Rigid Pavement Best Practices
- Guide Specifications
- Life Cycle Cost Analysis
- Emerging Pavement Technology

A synopsis of each document developed as part of this study is provided below. The complete documents are provided in Appendix E.

Project Assessment Manual

The Project Assessment Manual (PAM) was prepared to help obtain a systematic collection of relevant pavement-related data. Further, such data needs to be organized to maximize the usefulness in pavement decision-making process. To that end, this manual provides an overall assessment scheme (Figure 3.7).

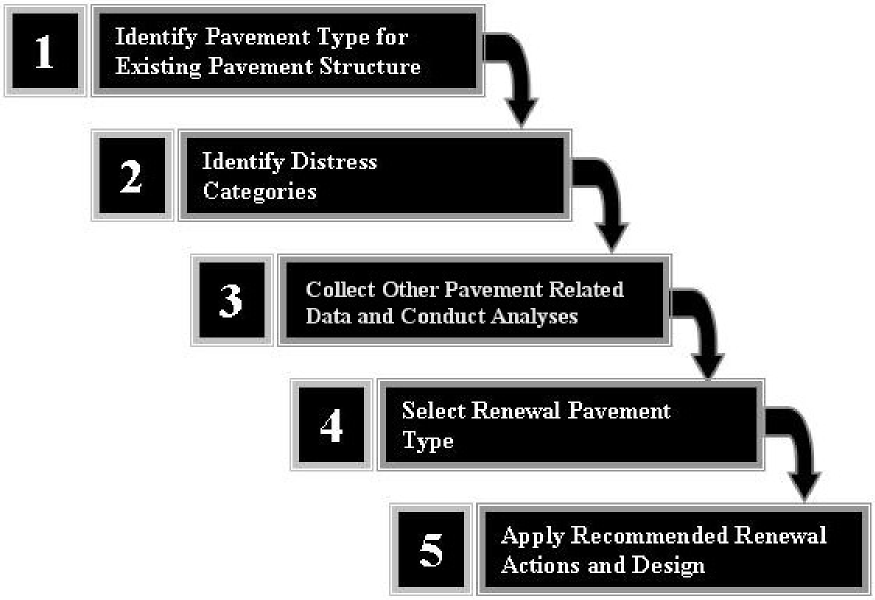


Figure 3.7. Outline of Pavement Assessment Manual Scheme.

The types of data collection in the manual range from basic information such as a distress survey to insights on construction-related traffic impacts. The last section in the PAM provides information on life cycle assessments (environmental accounting). This type of assessment is receiving increasing use and is likely to be more widely applied in the future. The complete PAM can be found in Appendix E-1.

The use of the manual is to compliment the design tools developed by the study. The types of data critical for making pavement-related decisions are described along with methods (analysis tools) for organizing the information for decision-making. It is not assumed that all data categories will be collected or assessed for a specific renewal project.

The following 10 data types are contained in the PAM:

- Pavement distress surveys
- Pavement rut depths and roughness
- Nondestructive testing—Falling Weight Deflectometer
- Ground Penetrating Radar
- Pavement cores
- Dynamic Cone Penetrometer
- Subgrade soil sampling and tests
- Traffic loads for design
- Construction productivity and traffic impacts
- Life cycle assessment (environmental accounting)

Flexible Pavement Best Practices

The Flexible Best Practices document can be found in Appendix E-2. This document provides a collection of best practices for the design and construction of long life flexible pavement alternatives using existing pavements. The intent is to restrict distress such as cracking and rutting to the pavement surface. The document provides an overview of the renewal strategies and the reasoning behind their selection as well as the critical features associated with each strategy, including construction issues.

The document also provides a discussion of HMA construction quality control and ties that discussion to the Guide Specifications also provided in the guidelines. Design issues associated with transitions beneath structures are included as illustrated in Figure 3.8.

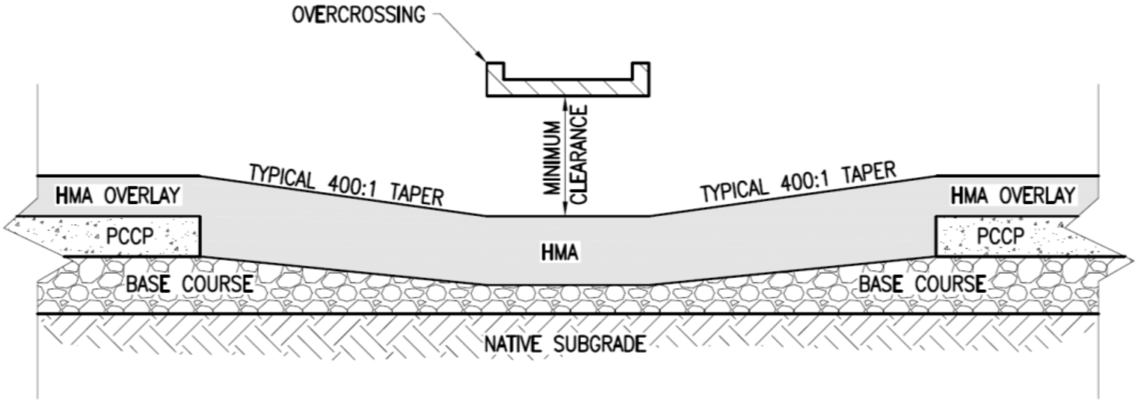


Figure 3.8. Illustration of flexible pavement transitions to overcrossings.

Rigid Pavement Best Practices

The Rigid Best Practices document can be found in Appendix E-3. This document provides recommendations for the design and construction of long life rigid pavement alternatives using existing pavements.

The goal for achieving long life concrete pavements requires an understanding of design and construction factors that affect both short-term and long-term concrete pavement performance. This requires an understanding of how concrete pavements deteriorate and fail as well as what is required to provide long life both from the structural design and construction details.

The rigid pavement approaches using existing pavements are described as well as the supporting information for their selection. Material considerations common to all approaches are discussed. The design and construction for the different long life approaches is presented in some detail along with quality control and assurance needs. Standard practices for added lanes and transitions to adjacent structures are also discussed as illustrated in Figure 3.9.

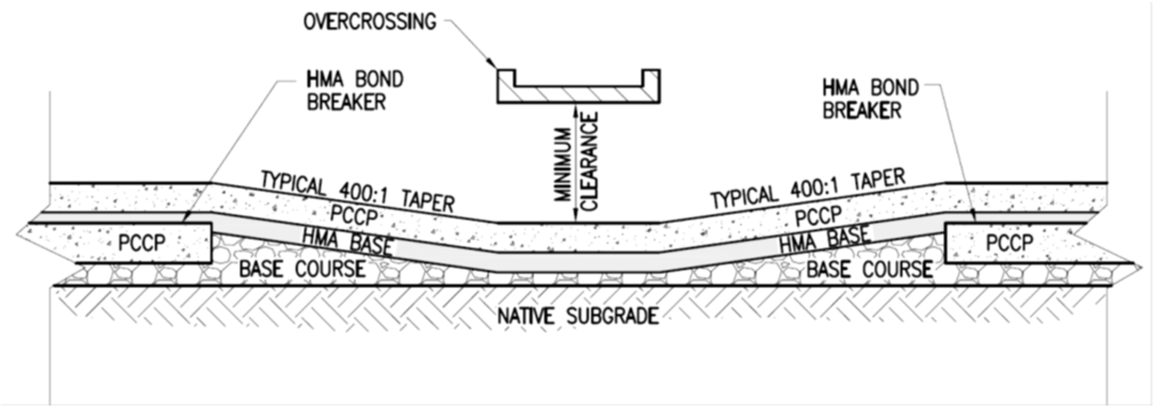


Figure 3.9. Illustration of rigid pavement transitions to overcrossings.

Guide Specifications

The study team used AASHTO Guide Specifications as a starting point in specification development. This was done, in part, due to the fact that there are a wide variety of pavement-oriented specifications developed and maintained by AASHTO committees. Further, AASHTO Guide Specifications provides a common set of terms and structure on which to add components from State specifications.. The approach was to review existing state agency and AASHTO guide specifications, select sensible components (or elements), and place those in lists.

The guide specifications 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 SHA and AASHTO specifications that were used to produce the “elements” in item 2. An illustration of specification elements is shown in Table 3.19 for tack coats—a basic paving process spanning several renewal options. The complete specification documentation can be found in Appendix E-4.

Table 3.19. Specification elements developed from multiple sources for tack coats.

AASHTO Paragraph	R23 Recommendations		Source(s)
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/SY. 2. Many SHAs allow dilution with water up to 50%.	Range generally falls within most state limits
	Application Temperatures	Use manufacturer recommendations.	Study Team

Sources: AASHTO, 2008; Michigan, 2003; Minnesota, 2005; Missouri, 2011; Texas, 2004; Virginia, 2007.

There were four guide specifications that are not contained in the AASHTO Guide Specifications but were felt necessary for this study. These are: Stone Matrix Asphalt (SMA), Open Graded Friction Course (OGFC), Rubblization of PCC, and Saw, Crack, and Seat.

Life Cycle Cost Analysis

These guidelines provide a range of approaches for the design of long life pavements using existing pavements. The determination as to which approach should be selected will depend on how well they meet the engineering requirements of the project and which is the most cost effective. Determining the cost effectiveness of the various approaches requires a life cycle cost analysis. Most public agencies have specific procedures in place and it is expected that those agencies will follow those procedures. Where an agency does not have a specific procedure in place, a general discussion of life cycle cost analysis in included.

The complete Life Cycle Cost Analysis manual can be found in Appendix E-5.

Emerging Pavement Technologies

There are PCC and flexible pavement technologies that are not yet considered to be long life renewal options but may become so in the future. One technology that was reviewed, precast concrete pavement, is likely a long-lasting renewal option at this time. The limitation is that there are too 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:

- Rigid Pavements
    - Ultra Thin CRCP overlays
    - Precast Concrete Pavement
- Flexible or Composite Pavements
    - Resin Modified Pavement (illustrated in Figure 3.10)

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 which should be monitored as they continue to evolve, and which may be or become viable components for long-lasting pavement renewal.

The complete Emerging Pavement Technology document can be found in Appendix E-6.

Resin modified pavement at a truck weigh station on the N-3 near is near Johannesburg, South Africa.

A close-up of the resin modified cement that was placed in open-graded HMA.

Figure 3.10. Resin modified pavement in South Africa.(Photos: Joe Mahoney)

INTERACTIVE APPLICATION

The study resulted in the development of a number of documents and reference tools that provide guidance on scoping and estimating long life renewal strategies for pavements. The following goals and objectives were identified during the study in order to foster broad implementation of the research results:

- Provide a user-friendly means of navigating the large amount of design and best practice information contained within the work product.
- Provide guidance and a method for selecting an appropriate rehabilitation strategy based on information specific to a given project.
- Provide a transparent view of the decision making process as users are selecting the appropriate rehabilitation strategy, design and best practices given their local practices.

To meet these objectives and facilitate accelerated adoption of the research results, a computer-based application to guide users to the applicable research findings was developed to aid implementation—in essence a "scoping tool" for users. The following sections outline the requirements, approach and results of the application development portion of the study.

User Requirements

To best serve the intended audience of the application (scoping tool), it was determined that the following end-user requirements must be met:

- The application must run on any computer (PC/Mac) with commonly installed libraries.
- The application must be distributable on CD-ROM, with option for web distribution in the future.
- No third party licenses or controls to be required by end-users to install or distribute.
- Provide capabilities to periodically update renewal strategies and guidance.
- Provide printable report available to users.
- Minimize application support and maintenance needs.

These requirements were then assessed against a number of different implementation technologies to determine the best approach to the application design.

Application Design

To meet these user requirements, an initial assessment of available technologies to best meet those goals was performed. Several technologies were considered, although ultimately, the Adobe Flash platform and Flex Builder toolkit were selected.

Adobe's Flash Player is currently the world's most pervasive software reaching 99 percent of Internet enabled desktops in markets such as the U.S. and Western Europe and provides a medium for both connected (web) and non-connected (CD-ROM) distribution. The platform also provides users the option of running applications directly via the web, or directly from CD-ROM without need for installation files or impact on the user's computer.

Other technologies considered in implementation included Java, .NET, HTML 5, and Microsoft Office (Excel). While each of these technologies could perform the required function of the scoping tool application, each was unable to meet the user requirements to the same level as Adobe Flash.

Data Structure

The scoping tool was designed to allow subject matter experts the ability to modify the renewal strategy language and recommendations that results from ongoing feedback without having to re-compile the application. To do so, an Extensible Markup Language (XML) data structure was devised to store all of the application logic and workflow information. Screen shots of the major pages in the application follow in Figure 3.11.

Interactive Software Steps

The interactive software developed for this project consists of guiding users through the following five steps:

1. Specify existing and proposed section information
2. Specify existing pavement condition
3. Confirm section design parameters
4. Select renewal strategy
5. Recommended section design

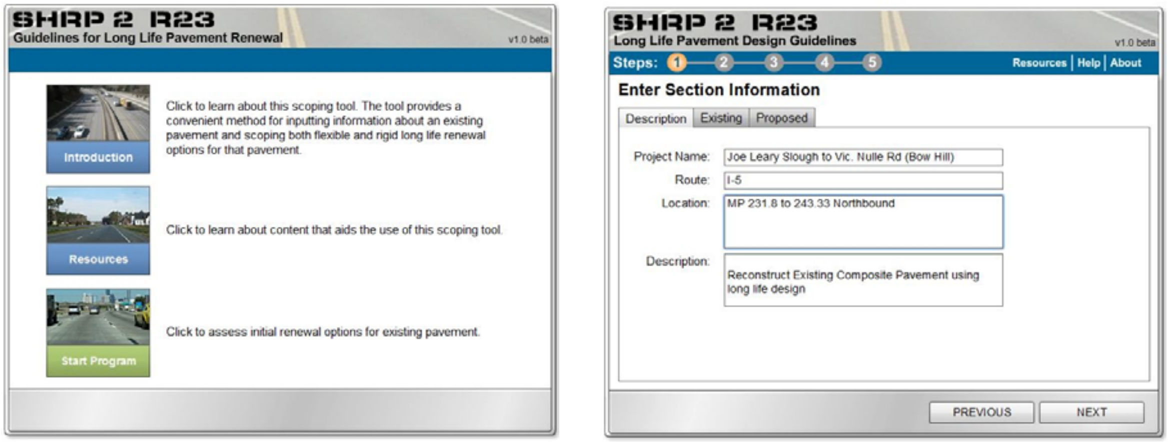
These five steps allow the user to input the parameters needed to obtain feasible renewal options from the decision matrices and thickness design tables discussed previously. In Step 1, the user inputs the existing pavement structure and the design information for the proposed renewal project (i.e., traffic levels, subgrade conditions, geometric constraints). The software uses this information to determine the type of existing pavement being evaluated and selects the appropriate decision matrix from Table 3.9 through Table 3.13 . Design information for the proposed renewal is stored for later use in determining the renewal layer design thickness (described in Step 5).

In Step 2, users input the condition of the existing pavement in terms of key distress types. These distress types are used in the decision matrices to determine feasible renewal alternatives. The presence of certain types of distress (or distress in high quantities and/or severities) precludes some of the renewal strategies from achieving long-life. These alternatives are eliminated by the program during Step 2.

Step 3 provides the user with an overall summary on the existing pavement type/layering, existing condition, and proposed design parameters.

With the existing pavement and proposed design elements confirmed, the user can select the type of renewal (i.e., flexible or rigid) in Step 4. The program utilizes the selection along with the existing conditions stored in Step 2, to determine a list of feasible renewal options, recommended actions/considerations, and a description of the approach. This information is pulled from the decision matrices listed in Tables 3.9-3.13. The user can chose from the list of feasible options and select the existing pavement or base modulus (this information will be used in Step 5).

In Step 5, the software utilizes the thickness design tables listed previously in Table 3.16 through Table 3.17. The software uses the proposed design parameters entered in Step 1 along with the renewal strategy and modulus selected in Step 4 to determine a proposed renewal thickness. In addition, the software provides an overview of the existing pavement, the recommended design, and all of the pertinent design parameters. Also, listed are links to the specific resource documentation for the renewal strategy.



Click to learn about this scoping tool. The tool provides a convenient method for inputting information about an existing pavement and scoping both flexible and rigid long life renewal options for that pavement.

Click to learn about content that aids the use of this scoping tool.

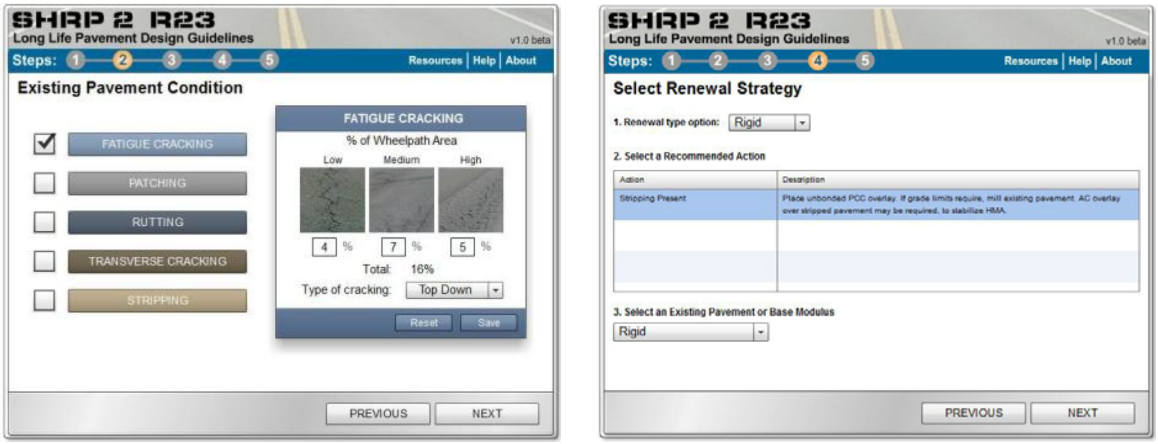
Click to assess initial renewal options for existing pavement.

Enter Section Information

Project Name: Joe Leary Slough to Vic. Nulle Rd (Bow Hill)
Route: I-5
Location: MP 231.8 to 243.33 Northbound
Description: Reconstruct Existing Composite Pavement using long life design

Opening Screen

Enter Section Information



Existing Pavement Condition

☑ FATIGUE CRACKING
☐ PATCHING
☐ RUTTING
☐ TRANSVERSE CRACKING
☐ STRIPPING

FATIGUE CRACKING
% of Wheelpath Area
Low 4 %, Medium 7 %, High 5 %
Total: 16%
Type of cracking: Top Down

Select Renewal Strategy

1. Renewal type option: Rigid

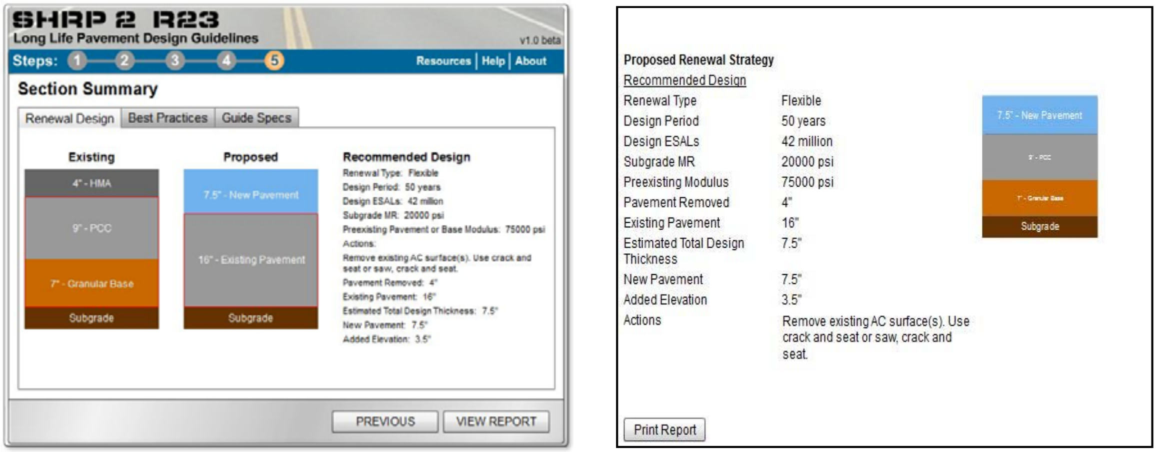
2. Select a Recommended Action

Action	Description
Stripping Present	Place unbonded PCC overlay. If grade limits require, mill existing pavement. AC overlay over stripped pavement may be required, to stabilize HMA.

3. Select an Existing Pavement or Base Modulus
Rigid

Existing Pavement Condition

Selection of Renewal Strategy



Section Summary

Recommended Design
Renewal Type: Flexible
Design Period: 50 years
Design ESALs: 42 million
Subgrade MR: 20000 psi
Preexisting Pavement or Base Modulus: 75000 psi
Actions:
Remove existing AC surface(s). Use crack and seat or saw, crack and seat.
Pavement Removed: 4"
Existing Pavement: 16"
Estimated Total Design Thickness: 7.5"
New Pavement: 7.5"
Added Elevation: 3.5"

Proposed Renewal Strategy

Recommended Design

Renewal Type	Flexible
Design Period	50 years
Design ESALs	42 million
Subgrade MR	20000 psi
Preexisting Modulus	75000 psi
Pavement Removed	4"
Existing Pavement	16"
Estimated Total Design Thickness	7.5"
New Pavement	7.5"
Added Elevation	3.5"
Actions	Remove existing AC surface(s). Use crack and seat or saw, crack and seat.

Section Summary

Proposed Renewal Strategy

Figure 3.11. Screen shots from the Interactive Software.

CHAPTER 4
CONCLUSIONS AND SUGGESTED RESEARCH

There are obvious benefits to using existing pavements in the construction of long life pavements. A more rapid construction process can be achieved as it eliminates the need for removal of material from the project and reduces or eliminates the importation of base aggregates. This type of construction can also facilitate traffic staging through the project. This results in reduced construction duration as well as costs and impact on the traveling public.

The guidelines developed in this project provide a range of approaches for the design and construction of long life pavements using existing pavements. Most agencies used one or more of the approaches identified, but none were found to use all of the approaches identified in this project. A large number of agencies were contacted both nationally and internationally in the development of the guidelines. Some of the agencies that were contacted had tried one or more of the approaches identified and had experienced construction problems which caused them to not consider that approach in future work. In working with the different agencies, it became clear that the details related to the success or failure of these processes must be provided in some form of knowledge base. The old adage that "the devil is in the details" applies fully to the use of existing pavements to construct long life pavements. As such, much of the effort in Phase 2 of this project was devoted to the development of that knowledge base.

The decision matrices that were developed (and refined through the help of the industry and various agencies review) are quite detailed. To facilitate the flow and simplify the use of the matrices, an Adobe flash based program was developed. The program also provides the platform on which to place the knowledge base that supports the decision making process. That knowledge base was separated into six specific documents:

- Project Assessment Manual
- Flexible Best Practices
- Rigid Best Practices
- Guide Specifications
- Life Cycle Cost Analysis
- Emerging Technology

The Project Assessment Manual contains two unique sections. The "Construction Productivity and Traffic Impact" section will be extremely useful because most of the projects considered in these guidelines have a significant traffic staging component to them. Additionally, the "Life Cycle Assessment" section discusses the current approaches to environmental accounting, which is becoming an added consideration in today's highway program.

The guidelines developed under this project provide a single source of current information on all approaches that an agency can reasonably use to design and build long life pavements utilizing existing pavements. The products from this project offer all of the resources in one location. The guidelines are also unique in that they address not only the design approaches but also provide guide specifications that are congruent with those approaches. The material presented will become dated, thus the guidelines should be reviewed in about five years and updated as needed given advances in the industry.

SUGGESTED RESEARCH

The guidelines were produced under a contract that defined long life as pavements that provide 50 years of service. Though this is an admirable goal, most agencies that the project team interviewed do not design pavements for 50 years of service. The more typical design life was for about 30 years. In Europe and United Kingdom, long life pavement designs are for 30 to 40 years. The one comment that the team heard often was "If we designed for 50 years of service this would be a very good resource," implying that the guidelines had limited use. All agencies also had funding issues so the full application of the guidelines was also limited because of the current funding levels.

The design process in the guidelines is not restricted to 50 years. The program allows the user to compute traffic loading ranging from 30 to 50 years. The decision matrix; however, *does not* include some approaches that could provide 30 to 40 years of service. The team clearly felt that if the guidelines were shown as providing guidance for long life pavements (with long life pavements defined as those that provide 30 to 50 years of service), more agencies would use them.

It is recommended that the guidelines be modified to provide design guidance for 30 to 50 year service lives. The bulk of the information contained in the guidelines would not change. The major change would be in the decision matrix to include a number of applications that were eliminated because they would likely provide only 30 to 40 years of service. These would include bonded PCC overlays, as well as HMA overlays of CRCP. There would also be additional material placed in the best practices documents to describe the design and construction of those approaches. The Guide Specifications would be updated to include guide specifications unique to the construction of those added approaches. There would be little actual change in the guidelines, but it is felt that there would be a perception by many agencies that the guidelines were developed for their use not just for those few agencies that designed for 50 years.

To account for the 30 to 40 year design lives, the following actions are proposed:

- Revise decision tables to include bonded PCC overlays and AC overlays of CRCP and add design thickness estimate tables to match added approaches
- Circulate revised decision tables to agencies and industry for review
- Finalize decision tables based on review comments
- Revise best practices documents and guide specifications to account for added options
- Circulate revised documents to agencies and industry for review comments
- Finalize documents based on review comments

- Revise program based on changes
- Conduct a beta test of the revised program with participating agencies
- Prepare addendum to final report to document revisions

Those agencies that participated in Phase 2 would also be asked to work on the revisions to the guidelines. Those agencies would be asked to comment on the revisions and then on the revised program. A workshop or two may be required to help focus the process.

IMPLEMENTATION

Fully Web-Enable the Guidelines

The R23 program application was originally designed for self-contained delivery via CD to minimize the upfront cost and promote rapid development, but ultimately this has limited its functionality as to broader use, support, and maintenance of the program.

- User inputs and results are not stored, meaning users do not have the ability to "load" or "save" user input and application outputs to reproduce guidance, compare results between scenarios, or share with colleagues. Once the application is closed, all inputs and results are lost.
- Documentation is not cross-linked, limiting the full effectiveness of the information provided. For example, cross-linking would allow the reader to move easily for example from sections in the best practices to appropriate sections in either the Pavement Assessment Manual or the Guide Specifications. This will significantly improve access to the information and the utility of the program.
- Application and documentation is not indexed or searchable, limiting search engine recognition and ultimately exposure to potential users.
- Increased long term maintenance and support cost would be reduced due because the current program has disconnected distribution environment and dependency on Adobe AIR and Flash framework compilation for any development updates.

To promote the implementation of the application and research results, it is recommended that further work be done to improve the functionality of the guidelines. Because of funding and time limitations during this project, the documents that reside on the current program were prepared in MS Word and housed on a host server in PDF format for access by the users. They were developed, however, with the view that the content could be reformatted so that there would be cross connections between the various sections within the documents to increase their utility. Based on feedback from the pilot group of users and the experience of the R23 Team, it is recommended that the R23 application be moved into a web-based application with the following elements:

- Develop database and security elements to provide users the ability to load, save, and compare various individual application results within their organization.
- Conversion of static documents like the project assessment manual, guide specifications, and best practices into a content management system with cross-

linked pages to aid in accessibility, reduce maintenance costs, and improve search capabilities of the documentation.

It is assumed that many of the SHRP 2 products will benefit from being placed in a web-based program with the full functionality and ease of access that format provides. The R23 program is clearly one of those products.

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ABBREVIATIONS, ACRONYMS, INITIALISMS, AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt concrete
ACPA	American Concrete Pavement Association
ASTM	American Society for Testing and Materials
CRCP	Continuously reinforced concrete pavements
CRC	Continuously reinforced concrete
CTE	Coefficient of thermal expansion
DOT	Department of Transportation
ESAL	Equivalent Single Axle Load
FWD	Falling Weight Deflectometer
GGBFS	Ground granulated blast furnace slag
GPS	General Pavement Study
HMA	Hot mix asphalt
IRI	International Roughness Index
ITA	Illinois Tollway Authority
JRCP	Jointed Reinforced Concrete Pavement
JPCP	Continuously reinforced concrete pavements
LTPP	Long Term Pavement Performance
MEPDG	Mechanistic-Empirical Pavement Design Guide
MDOT	Michigan Department of Transportation
MnDOT	Minnesota Department of Transportation
NAPA	National Asphalt Pavement Association
PAM	Project Assessment Manual
PCC	Portland cement concrete
SHRP	Strategic Highway Research Program
SHA	State Highway Agency
SPS	Specific Pavement Study
TTI	Texas Transportation Institute
UBCOL	Unbonded concrete overlay
VDOT	Virginia Department of Transportation
TxDOT	Texas Department of Transportation
WsDOT	Washington State Department of Transportation

APPENDIX A

LITERATURE REVIEW

LITERATURE REVIEW

A thorough literature search for information on highway renewal using existing pavements was conducted. The resources utilized for this task include the following:

- The Transportation Research Information Service (TRIS) database.
- The International Transportation Research Documentation (ITRD) database.
- The Transportation Libraries Catalog (TLCat).
- The National Technical Information Service (NTIS) database.
- The Transportation Research In Progress (RIP) database.
- The Online Library Catalog of the University of Illinois at Urbana-Champaign.
- ProQuest's ABI/INFORM Complete® database of periodicals, professional journals, and trade publications.
- The Federal Highway Administration (FHWA)'s National Highway Specifications website (NHSW).
- The Bureau of Transportation Statistics' National Transportation Library (which searches some of the above databases and others, as well as the websites of the Departments of Transportation of all fifty States and the District of Columbia).
- The Virtual Library of publications of the World Road Association (PIARC).
- The United Kingdom's Transport Research Laboratory (TRL) publication database.
- The Netherlands' Foundation Center for Research and Contract Standardization in Civil and Traffic Engineering (CROW) publication database.
- The publication databases of the American Concrete Pavement Association (ACPA), Asphalt Institute (AI), and National Asphalt Paving Association (NAPA).
- The publication databases of the roadway authorities of Austria, Germany, France, Belgium, the Netherlands, the United Kingdom, Canada, Australia, South Africa, and other countries facing demands of heavy traffic on high-volume roads and the need for in-place renewal of existing pavement structures.

In addition, questionnaires were sent to each of the State Highway Agencies. These surveys were followed by a series of phone calls to learn more about State sponsored reports. The State survey portion of the project is described in more detail in the following chapter.

There have been a number of definitions for long-life pavements depending of the location, pavement type, and roadway facility. 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 reconstructions, and needing only periodic surface renewal in response to distresses confined to the top of the pavement. Table A.1 shows typical ranges of service life for reconstruction and various major rehabilitation techniques for each pavement type (Thompson, 1989). These ranges

Table A.1. Typical ranges of service lives for rehabilitation treatments (Hall et al, 2001).

Treatment	**Typical range of service life, years**
Reconstruction:	
Reconstruction in asphalt	15-20
Reconstruction in concrete	20-30
Asphalt pavement rehabilitation	
Structural asphalt overlay of asphalt pavement	18-15
Structural concrete overlay of asphalt pavement	20-30
Surface recycling without overlay	4-8
Nonstructural asphalt overlay of asphalt pavement	4-8
Nonstructural (ultrathin) concrete overlay of asphalt pavement	5-15
Asphalt patching without overlay	4-8
Concrete pavement rehabilitation	
Structural asphalt overlay of concrete pavement	8-15
Asphalt or concrete overlay of fractured concrete slab	15-25
Unbonded concrete overlay of concrete pavement	20-30
Nonstructural asphalt overlay of concrete pavement	4-8
Bonded concrete overlay of concrete pavement	15-25
Restoration without overlay	5-15
Asphalt-overlaid concrete pavement rehabilitation	
Structural asphalt overlay of AC/PCC pavement	8-15
Asphalt or concrete overlay of fractured concrete slab	15-25
Unbonded concrete overlay of AC/PCC pavement	20-30
Surface recycling without overlay	4-8
Nonstructural asphalt overlay of AC/PCC pavement	4-8
Nonstructural (ultrathin) concrete overlay of AC/PCC pavement	5-15

are general estimates only and represent the “conventional wisdom” about the service lives that may reasonably be expected of the different rehabilitation techniques. Based on these estimates the most promising renewal strategies for long-life using existing pavements are:

- Thick AC overlay over existing AC pavement
- PCC overlay over existing AC pavements
- Thick AC overlay over fractured PCC pavement
- Bonded PCC overlay over existing PCC pavement
- Unbonded PCC overlay over existing PCC pavement
- Thick AC or PCC overlay over fractured PCC of AC/PCC composite pavement
- Unbonded PCC overlay over existing AC/PCC composite pavement

The following sections provide details on each rapid renewal strategy along with considerations for long-life pavement based on relevant literature and Agency information.

AC RENEWAL APPROACHES

AC over AC Methods

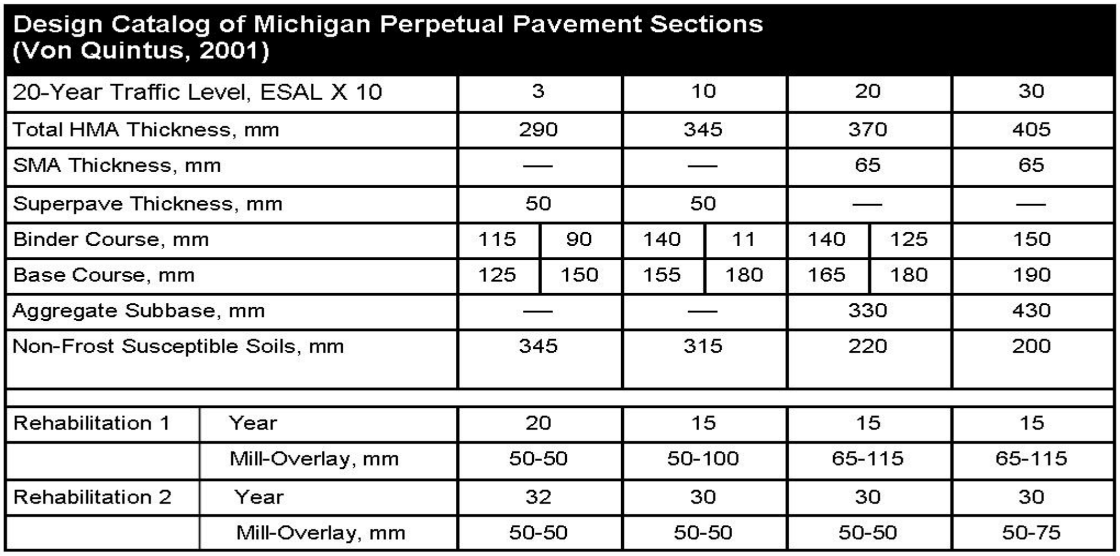
Information was sought on the following potential flexible pavement renewal methods:

- AC over existing AC
- AC over crushed and shaped AC
- AC over reclaimed AC

The three overlay methods listed above have been done by many states and other countries, for conventional rehabilitation purposes, i.e., for rehabilitation design lives typically not exceeding 15 years. Nonetheless, it seems entirely feasible from a conceptual standpoint that a new AC surface of sufficient thickness and durable mix design could be placed on an existing AC-surfaced pavement as a long-life renewal approach. However, to determine the structural and material requirements needed to achieve a true long-life renewal of the existing pavement, it may be necessary to think of the new AC surface as new construction on a high-quality base, rather than as a conventional overlay.

AC over existing AC pavement

The HMA over existing HMA strategy consists of “milling and filling” for the lower levels of traffic to “milling and strengthening” for the higher levels of traffic. Figure A.1 shows the example design cross-sections for long-life performance of HMA pavements developed by Von Quintus for the Michigan APA (APA, 2002). It includes suggested types of HMA mixtures to be placed within the pavement structure. Von Quintus recommends that the asphalt mixture for the HMA base layer be designed to have 3 percent air voids to mitigate bottom-up fatigue cracking. The surface course mixture is a dense graded Superpave in the case of 3 and 10 million ESAL levels, and an SMA in the case of the 20 and 30 million ESAL levels (20-year). The strategies presented in the figure are for planning purposes only.



Design Catalog of Michigan Perpetual Pavement Sections (Von Quintus, 2001)

20-Year Traffic Level, ESAL X 10		3		10		20		30
Total HMA Thickness, mm		290		345		370		405
SMA Thickness, mm		—		—		65		65
Superpave Thickness, mm		50		50		—		—
Binder Course, mm		115	90	140	11	140	125	150
Base Course, mm		125	150	155	180	165	180	190
Aggregate Subbase, mm		—		—		330		430
Non-Frost Susceptible Soils, mm		345		315		220		200
Rehabilitation 1	Year	20		15		15		15
	Mill-Overlay, mm	50-50		50-100		65-115		65-115
Rehabilitation 2	Year	32		30		30		30
	Mill-Overlay, mm	50-50		50-50		50-50		50-75

Figure A.1. Michigan Design Catalog for Long-life HMA Pavements (APA, 2002).

AC over crushed and shaped AC pavement

The HMA over crushed and shaped HMA technique consists of crushing the existing HMA layer and shaping it into a base layer before overlaying it with a new HMA layer (Figure A.2). This strategy is suitable for severely cracked HMA pavements. Marginal base material can be upgraded with admixtures to provide high quality support. To avoid reflection cracking, crack-relieving separator layers or membranes can be used including: (1) Geotextile or fabrics, and (2) stress-relieving or stress-absorbing membrane interlayers (SAMIs). Crushing is usually more economical than hot-mix recycling, unless the asphalt surface is quite thick. When the existing mat is quite thick (over 6 inches), a common procedure is to mill off part of the HMA, then crush the remainder.

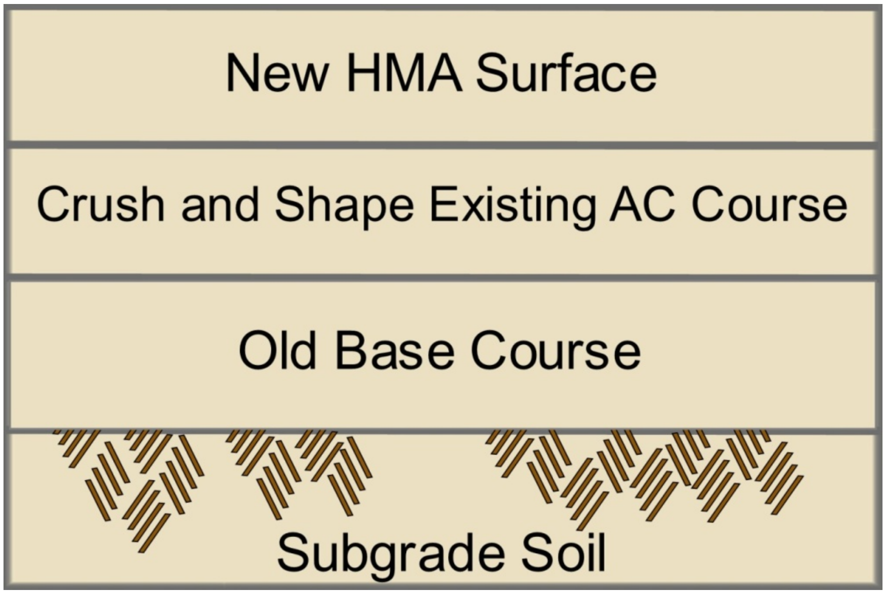


Figure A.2. Schematic of a HMA over a crushed and shaped HMA pavement.

AC over reclaimed AC pavement

An alternative to crushing and shaping is to recycle the HMA layer using hot mix or cold mix in-place recycling techniques. The product is a renewed HMA base layer that will be overlaid with new HMA. Recycling can involve cold mix for lowest layers or hot mix for the upper base layer. The categories of pavement recycling options are shown in Figure A.3 (NHI, 2003). Only the categories applicable to using existing pavement in place are discussed next.

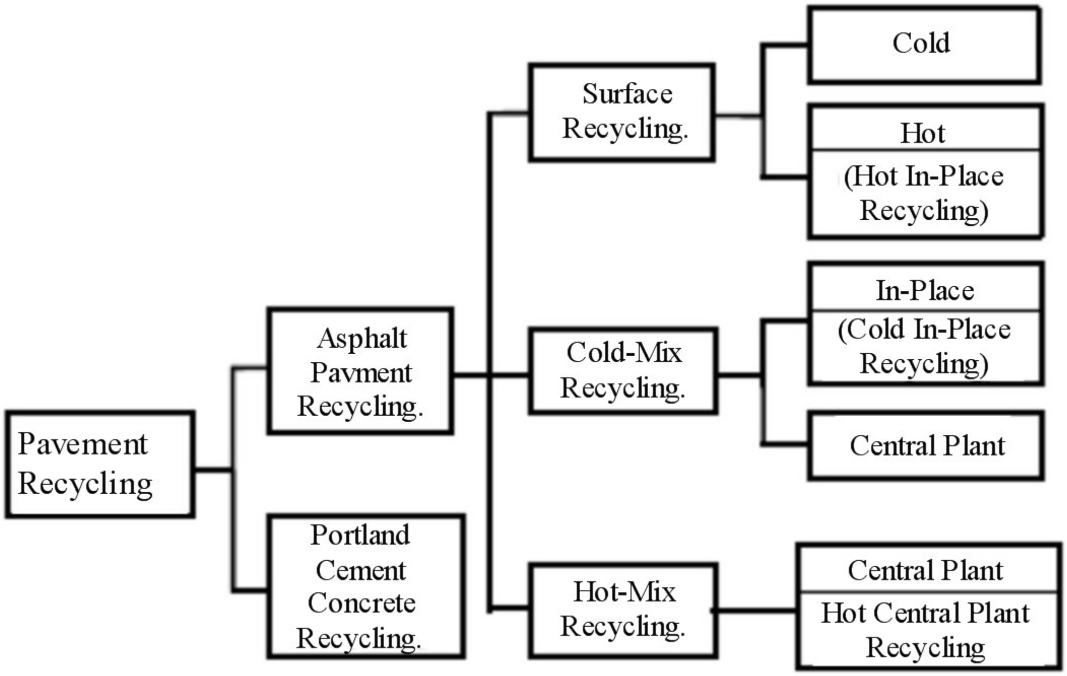


Figure A.3. Categories of pavement recycling options. (NHI, 2003)

Cold-in-Place Recycling (CIPR). This involves the reuse of an asphalt concrete pavement that is either processed in-place with the addition of asphalt emulsions, cutbacks, portland cement, lime and/or other materials as required to achieve desired mix quality, followed by placement and compaction. CIPR is accomplished by a special machine that scarifies the existing surface to a given depth, crushes it in a pug mill, adds asphalt cement, and lays the resultant mix back down, almost in its original location. The Asphalt Recycling and Reclaiming Association (ARRA) differentiate two different CIPR procedures as full depth and partial depth. Partial-depth CIPR involves the recycling of the asphalt bound layers to a depth of 3 to 4 inches. Full-depth CIPR, also termed full-depth reclamation, involves the recycling of the asphalt bound layers and the unbound granular layers in the flexible pavement. The finished product is considered as a base only, and a hot-mix surface course is necessary. CIPR has been performed on all types of roadways with the concentration being on lower volume roadways. However, full-depth reclamation has successfully been conducted on high volume Interstate pavements. This process can directly address structural problems through the production of an improved stabilized layer when full depth reclamation is used. Partial-depth reclamation is limited to correcting only those distresses which are surface problems in the asphalt layer (Hall et al, 2001).

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 CIPR include: (1) Use of an excessive amount of recycling agent; (2) Application of a surface seal prematurely; (3) Recycling only to the depth of an asphalt layer, resulting in de-lamination from the underlying layer; and/or (4) Allowing project to remain open for too long into the winter season (Hall et al, 2001).

Hot-in-Place Recycling (HIPR). ARRA defines three types of hot in-place recycling operations: heater scarification, repaving, and remixing. Each of these is described below (Hall et al, 2001).

Heater scarification involves the following steps: (1) Heating the existing pavement surface to about 110 to 150ºC, using one or more propane-fired radiant heaters; (2) Scarifying the softened surface to a depth of about one half to three quarters of an inch; (3) Applying a liquid rejuvenating agent (if needed); (4) Mixing and leveling the loose mixture with an auger and/or laydown machine; and (5) Compacting with rollers.

Repaving is heater scarification combined with placement of a new asphalt concrete overlay. The process involves the following steps: (1) Heating the existing pavement surface to about 190ºC, using infrared heaters; (2) Scarifying the softened surface to a depth about one half to three quarters of an inch; (3) Applying a liquid rejuvenating agent (if needed); (4) Mixing the loose mixture with an auger; (5) Spreading and screeding the recycled mixture; (6) Placing a new asphalt concrete layer over the recycled mixture; and (7) Compacting with rollers.

Remixing is similar to repaving, but involves mixing mineral aggregate or new asphalt concrete hot-mix into the scarified, rejuvenating material, rather than placing a layer of new asphalt concrete on top. Remixing not only increases the structural capacity of the pavement, as does repaving, but also permits improvement to the gradation or binder properties of the existing asphalt concrete layer. Remixing involves heating and reworking material to a greater depth than in heater scarification and repaving. The steps in the remixing process are the following: (1) Heating the existing pavement surface to about 85 to 105ºC, using one or more propane-fired radiant heaters; (2) Milling the softened surface to a depth of about 1 to 2 inches; (3) Mixing the hot milled material, rejuvenating agent and new asphalt concrete material in a pugmill; (4) Placing the mixture; and (5) Compacting with rollers.

Hot in-place recycling without an accompanying overlay or addition of new asphalt concrete material is estimated to have a service life of about 4 to 8 years. How much hot in-place recycling in conjunction with an overlay or additional asphalt concrete thickness benefits overlay performance has not yet been quantified (Hall et al, 2001), although remixing with a thick HMA overlay provides the best potential of achieving long-life.

AC over PCC Methods

When in-place renewal of an existing PCC pavement is considered, the structural design considerations that must be taken into account to ensure good long-term performance are the adequacy of the subgrade, protection of the subgrade from excessive deformation, limiting strain in the existing PCC, limiting stress and strain in the new AC or PCC surface, and minimizing reflection cracking in the new surface.

While AC overlay is no doubt the most commonly used major rehabilitation method for jointed PCC pavements, the service life of this technique is limited by the rate at which reflection cracks develop and deteriorate to unacceptably rough levels. Thus an AC overlay of a jointed PCC pavement is typically considered a conventional rather than a long-life rehabilitation approach, with an expected service life of about 10 to15 years.

However, exceptions exist: Iowa, for example, has experience with jointed PCC pavements built in the 1930s and 40s, widened with PCC or AC from 18, 20 or 22 feet to 24 feet in the 1970s, and then overlaid over time with a total of 5 or more inches of AC. Now, some 30 years later, these old AC/PCC pavements are being widened again, to 28 or 32 feet, and are being overlaid with PCC (Cable, 2008).

The most promising long-life rigid pavement methods, however, appear to be the following:

- AC over CRCP.
- AC over cracked and seated JPCP.
- AC over rubblized PCC.

AC over existing CRCP pavement

AC overlays of CRC pavements can reasonably be expected to perform much longer than AC overlays of jointed concrete pavements, especially when (a) working cracks and punch-outs in the existing CRCP are repaired with continuously reinforced full-depth concrete, and (b) the existing CRCP does not have D-cracking. Permanent patching of punch-outs and working cracks will delay for many years the occurrence and deterioration of reflection cracks in asphalt overlays of continuously reinforced concrete pavements. Reflection crack control treatments are not necessary for asphalt overlays of continuously reinforced concrete pavements, as long as continuously reinforced concrete repairs are used for deteriorated areas and cracks (Barnett, 1981, Darter, 1982, Hall, 1989).

It has often been suggested that an adequate thickness of AC over a sound CRC pavement may be the perfect application for long life design, which would require nothing more than periodic renewal of the AC surface. However, such rehabilitation projects are not currently typically designed for lives in excess of about 20 years.

The most commonly used approach to structural design of asphalt overlays of concrete pavements and asphalt-overlaid concrete pavements is the structural deficiency approach, exemplified by the 1993 AASHTO Guide procedure. The required AC overlay thickness is determined by multiplying the structural deficiency (D_f, the required concrete thickness for future traffic, minus D_{eff}, the effective thickness of the existing concrete slab) by an adjustment factor, A, that converts the thickness deficiency from inches of concrete to inches of asphalt.

A value of 2.5 has traditionally been used for the adjustment factor A. This value was based on the results of accelerated traffic tests conducted by the Corps of Engineers in the 1950s. The value 2.5 does not represent the best fit of the relationship of concrete thickness deficiency to asphalt overlay thickness in those field tests, but rather a conservative value suggested by the Corps for use in design. However, an A value of 2.5 can lead to excessive overlay thickness for larger concrete thickness deficiencies. A formula for the A factor as a function of the magnitude of the concrete thickness deficiency was developed by Hall (1991) using elastic layer analysis, and is recommended in the 1993 AASHTO Guide in place of a constant A factor.

The NCHRP 1-37A MEPDG procedure's software for design of AC over CRCP allows the user to select some or all of the following performance criteria by which the adequacy of a trial overlay design is judged:

- Longitudinal cracking of the AC overlay
- Thermal cracking of the AC overlay
- Rutting of the AC overlay
- Punch-out damage in the existing CRCP

New pavement models for rutting in AC layers, longitudinal (top-down) cracking in AC, thermal cracking in AC, and punch-outs in CRCP are adapted for use in the prediction of AC overlays of CRCP in the MEPDG methodology. The smoothness parameter used for AC overlays of PCC pavements in the MEPDG methodology is the International Roughness Index (IRI), predicted from an empirical model as a function of the existing pavement's IRI at the time of overlay placement, the time elapsed since placement of the overlay, the average rut depth, and the average spacing of medium- and high-severity transverse cracks.

The viability of AC over CRCP as an in-place renewal option, and the AC overlay thickness and CRCP condition requirements necessary to make it viable, need to be explored in this study in coordination with the work underway on another SHRP2 study, *Project R21 on Composite Pavements.*

AC over cracked and seated pavement

Cracking and seating a plain jointed concrete pavement before overlaying it with AC has been done in the United States as far back as the 1940s. The technique attracted renewed interest beginning in the 1980s, as an approach to reflection crack control (Cable, 2008, Barnett, 1981, Darter, 1982). A great number of crack and seat projects have been built on highways in the US, including test sections in the LTPP SPS-6 (Rigid Pavement Rehabilitation) experiment.

This technique is suitable for JPC pavements. It involves breaking the existing concrete into pieces of sizes about 12–48 in (305–1220 mm), as shown in Figure A.4. In principle, the smaller the cracked piece, the larger the potential for reduction in reflection cracking, and the larger the reduction in the structural strength of the concrete pavement.

Cracking and Seating is done in four major steps: (1) cracking the concrete slab, (2) seating the cracked slab, (3) special treatments, and (4) HMA overlay.

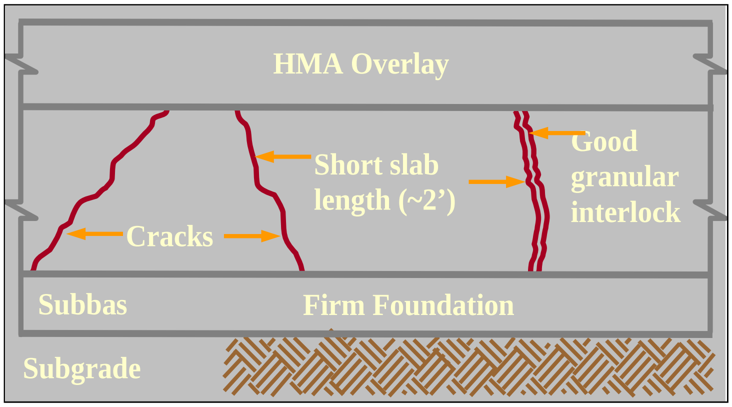
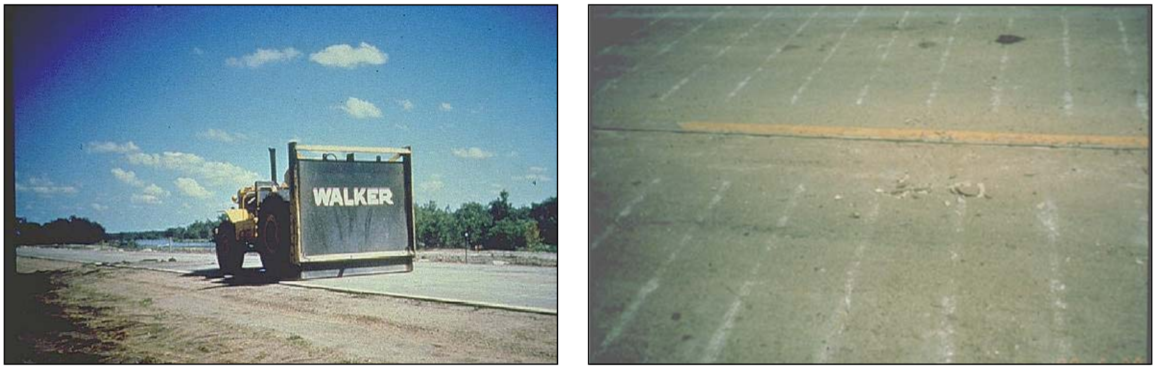


Figure A.4. Schematic of a HMA over a cracked and seated pavement (NHI, 2003).

Cracking of the pavements can be accomplished with a drop hammer, guillotines, modified pile drivers, or whip hammers, with the most commonly used equipment being the drop hammer (

Figure A.5). These are self-propelled units that raise a heavy mass several feet above the pavement and then release the weight, which then falls and strikes the surface of the slabs. Some agencies require cracking in both transverse and longitudinal directions. The resulting pieces should be large enough to retain interlock between aggregates, but also small enough to minimize the joint movement of the unreinforced PCC pavement. Excessive cracking can be detrimental to the PCC pavement.



(a) Guillotine hammer

(b) Fractured Slab with Guillotine Hammer

Figure A.5. Crack of pavement with Guillotine Hammer. (NHI, 2003)

After cracking, the slab is seated using 66–110 kip (30–50 ton) capacity rubber-tired rollers (Figure A.6). Seating of the concrete is done in order to (1) ensure reestablishment of the support between the subbase and the slab by reducing the existing voids, (2) create a relatively uniform grade for supporting paving operations, and (3) locate soft zones in the underlying layers that may need to be removed and/or replaced with more stable material. Excessive rolling may be harmful to the slab.

Figure A.6. Heavy roller used to seat the cracked pavement. (NHI, 2003)

The main concern with break/crack and seat is the reduction in the structural capacity of the pavement. To compensate for the reduction in structural capacity, thickness of the overlay should be increased. Pavement rehabilitated with the crack and seat technique can perform well when the subgrade support is uniform and subgrade modulus is more than 15,000 psi after cracking. Non-destructive testing (NDT) should be used to analyze and design the cracked and seated pavements.

Some studies have suggested that cracking and seating only succeeds in delaying the onset of reflection cracking by a few years (e.g., 5 or fewer), and that once reflection cracking appears, it tends to progress at much the same rate as it does in an AC overlay of an intact PCC pavement (e.g., about a year per inch of overlay thickness in reaching the surface)(Carpenter, 1989). Improvements in slab cracking techniques and the use of greater overlay thicknesses have resulted in better performance from crack and seat on later projects.

The term “breaking and seating,” rather than cracking and seating, is applied to the technique of fracturing a jointed reinforced concrete pavement prior to placement of an AC overlay. In general, breaking and seating has been found to be less effective at reflection crack control than cracking and seating, due to the difficulty of ensuring that the reinforcing steel in the concrete is completely ruptured in the process of breaking the slab.

An example of a successful application of this technique as a long-life HMA pavement is the California Interstate 710 (Figure A.7) in Los Angeles County, known as the Long Beach Freeway, with a design lane traffic of 100 million to 200 million equivalent single axle loads (ESALs) for a 40-year period.

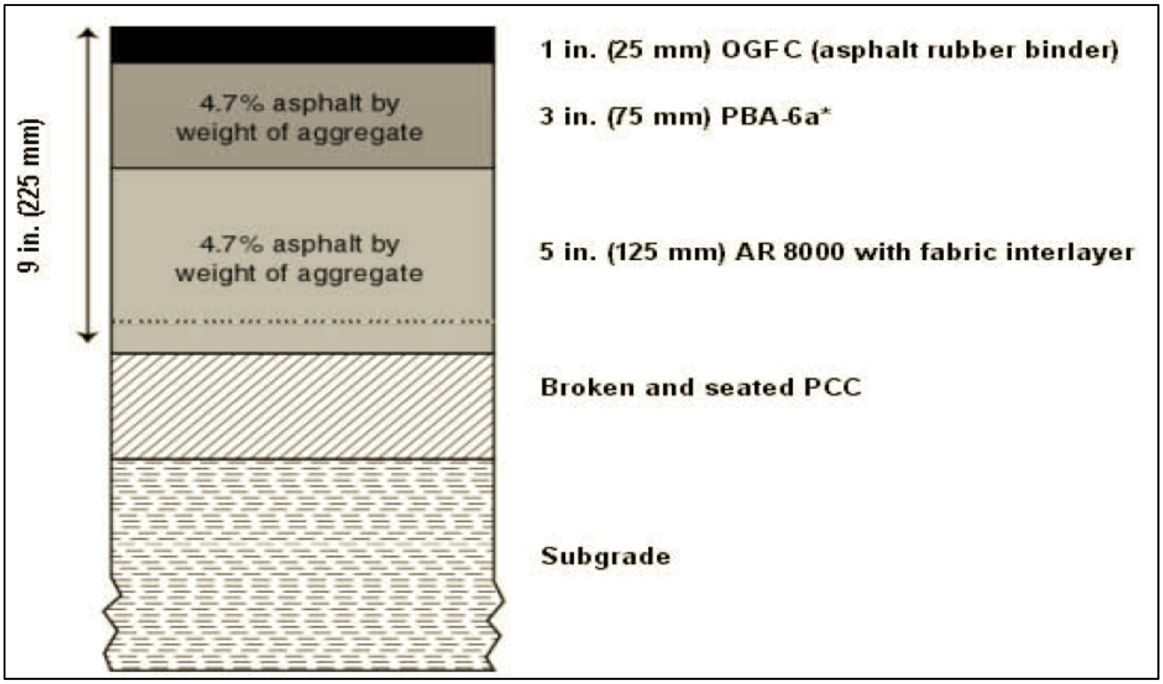


Figure A.7. Cross-sectional design for the I-710 Freeway (TRB, 2001).

AC over rubblized concrete pavement

Rubblization originally developed as an improvement in reflection crack control over cracking and seating. The LTPP SPS-6 experiment includes several rubblized sections built as supplements to the main experimental test sections. At AC overlay thicknesses comparable to those built on crack and seat projects, rubblizing projects typically are expected to provide about 5 to 10 years of additional service life. However, in recent years the two US manufacturers of concrete pavement rubblizing equipment (Antigo and PB4) have both been involved in rubblizing projects in which a much more substantial thickness of AC, e.g., 15 inches or more, has been placed. Such structures are in essence full-depth AC pavements on high-strength granular bases, and thus it appears reasonable to expect that they are viable candidates for long-life in-place renewal projects.

Rubblizing involves breaking the existing concrete pavement into pieces, and thereby destroying any slab action, and overlaying with HMA. The sizes of the broken pieces usually range from 2 in (51 mm) to 6 in (152 mm) (APA 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 CRCP presents some challenges and requires extra care in QC/QA procedures.

A rubblized PCC pavement behaves like a high-quality granular base layer. This loss of structure must be accounted for in the HMA overlay design thickness. A study by NAPA indicated that strength of the rubblized layer is one and a half to three times greater than a high-quality dense graded crushed stone base (NAPA, 1994).

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 maintaining a strong base and/or subgrade soil. Also, poor performance can occur when the underlying soils are saturated. Installation of edge drains prior to rubblization has proven to be successful for this type of condition. If the existing concrete pavement is deteriorated due to poor subgrade support, then rubblization may not be a viable option. Two types of equipment are used in the rubblization process: (1) Resonant Breaker and (2) Multiple-Head Breaker.

The *resonant rubblizer* (Figure A.8) 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 counter-weight situated on top of the beam. The principle on which the resonant breaker operates is that a low amplitude (about 0.5-inch) 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-degrees 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/subbase material and is approximately 1.0 to 1.5-lane-mile/day.

(a) Resonant breaker machine (Photo: Karim Chatti)

(b) Close-up of the sonic shoe (NHI, 2003)

Figure A.8. Resonant frequency pavement breaker.

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 it causes a shut down. Bituminous patches or un-milled overlays can also be problematic, as the shoe penetrates the asphalt causing a large loss in the energy delivered to the concrete. Lastly, the type of base/subbase material, the roadbed/

subgrade soil, and the condition of the concrete pavement being rubblized, all affect the quality of the rubblized product. For example, if the base/subbase materials are softer than the roadbed soil, shear failure may result.

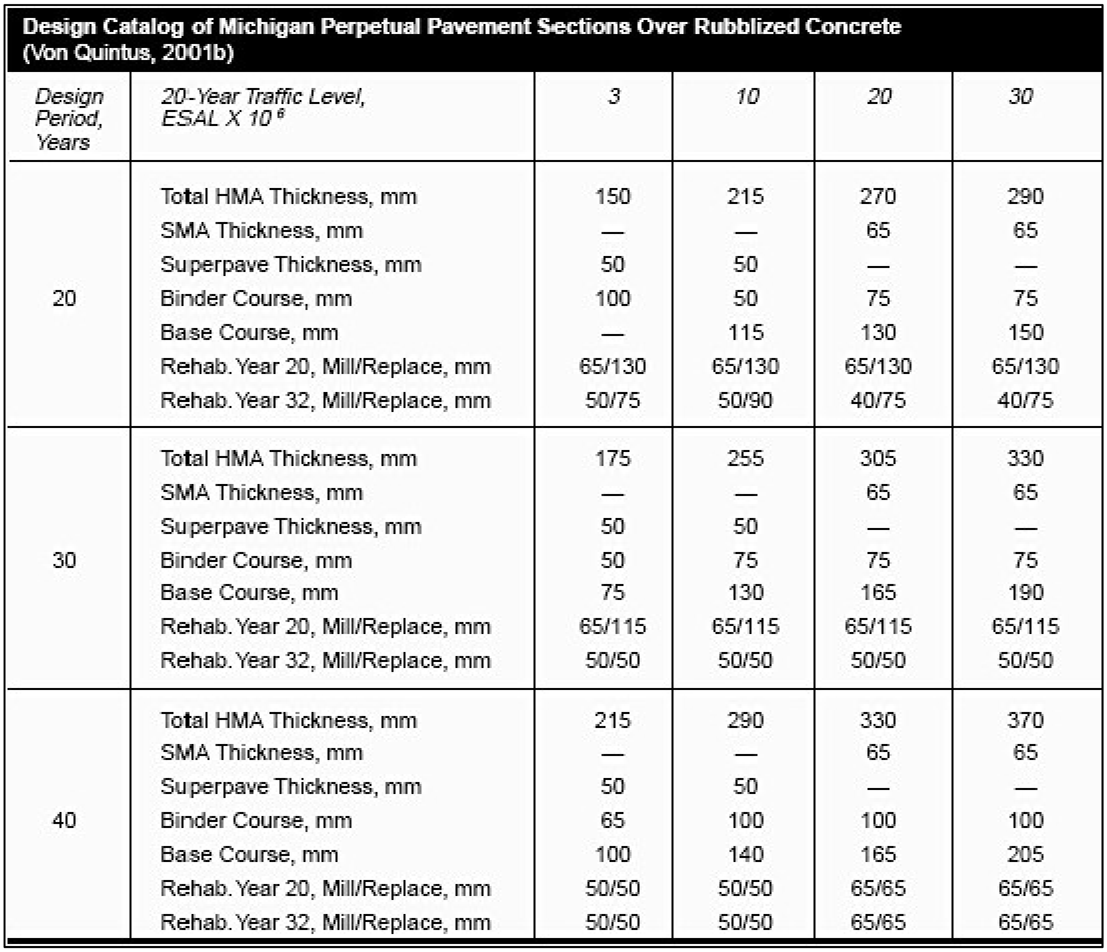
The *multi-head breaker* operation includes multiple drop hammers arranged in two rows on a self-propelled unit and a vibratory grid roller (Figure A.9). The hammers strike the pavement approximately every 4.5 inch. The bottom of the hammer is shaped as to strike the pavement on 1.5-in wide and 8-in long loading strips. The hammers in the first row strike the pavement at an angle of 30 degrees 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/frequency system. By disabling some cylinders, the width of the rubblized area can be varied from 2.5 to 12.67 ft. Typically, a 10-ton vibratory grid roller follows the multi-head breaker to reduce the size of the broken concrete. The rate of production of the multi-head breaker depends on the type of base/subbase material and is about 0.75 to 1 - lane-mile per day. Several variables affect the rubblization process including: speed, height, weight and frequency of the drop hammers. The multi-head 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.

(a) Multi-head breaker

(b) Grid roller

Figure A.9. Multi-head pavement breaker.

Examples of successful application of the rubblization technique as a long-life HMA pavement include: (1) I-440, Raleigh Beltway, North Carolina (ADT>100,000); (2) I-65, Alabama; (3) I-496 near Lansing, Michigan. Figure A.10 shows example design cross-sections for long-life performance of HMA over rubblized concrete pavements developed by Von Quintus for the Michigan APA (APA, 2002).



Design Catalog of Michigan Perpetual Pavement Sections Over Rubblized Concrete (Von Quintus, 2001b)

Design Period, Years	*20-Year Traffic Level, ESAL X 10^6*	*3*	*10*	*20*	*30*
20	Total HMA Thickness, mm	150	215	270	290
	SMA Thickness, mm	—	—	65	65
	Superpave Thickness, mm	50	50	—	—
	Binder Course, mm	100	50	75	75
	Base Course, mm	—	115	130	150
	Rehab. Year 20, Mill/Replace, mm	65/130	65/130	65/130	65/130
	Rehab. Year 32, Mill/Replace, mm	50/75	50/90	40/75	40/75
30	Total HMA Thickness, mm	175	255	305	330
	SMA Thickness, mm	—	—	65	65
	Superpave Thickness, mm	50	50	—	—
	Binder Course, mm	50	75	75	75
	Base Course, mm	75	130	165	190
	Rehab. Year 20, Mill/Replace, mm	65/115	65/115	65/115	65/115
	Rehab. Year 32, Mill/Replace, mm	50/50	50/50	50/50	50/50
40	Total HMA Thickness, mm	215	290	330	370
	SMA Thickness, mm	—	—	65	65
	Superpave Thickness, mm	50	50	—	—
	Binder Course, mm	65	100	100	100
	Base Course, mm	100	140	165	205
	Rehab. Year 20, Mill/Replace, mm	50/50	50/50	65/65	65/65
	Rehab. Year 32, Mill/Replace, mm	50/50	50/50	65/65	65/65

Figure A.10. Michigan Design Catalog for Long-life HMA Pavements over Rubblized Concrete (APA, 2002).

Thompson has demonstrated that a mechanistic-empirical approach to evaluation of the structural capacity of in-service asphalt pavement can be used to determine the required overlay thickness for rubblized concrete pavements (Thompson, 1999). An algorithm to predict the tensile strain at the bottom of the asphalt overlay as a function of a deflection basin parameter, called the Area Under the Pavement Profile (AUPP), has been validated with measurements from instrumented full-depth and conventional flexible pavements. Falling weight deflectometer (FWD) data from rubblized concrete pavements with asphalt concrete overlays were used to develop a relationship between AUPP and an overlay stiffness parameter (Eh^3, where E is the asphalt concrete modulus and h is the asphalt overlay thickness). The estimated strain is an input to an asphalt concrete fatigue model. The asphalt overlay thickness is selected to limit the asphalt concrete tensile strain to an acceptable level.

The NCHRP 1-37A MEPDG procedure's software for design of AC overlay of rubblized PCC allows the user to select some or all of the following performance criteria by which the adequacy of a trial overlay design is judged:

- Rutting
- Alligator cracking
- Longitudinal cracking
- Transverse cracking
- Smoothness

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.

Criteria for Long-life AC Renewal Approaches

For asphalt concrete pavements, achieving long life requires the combination of a rut/wear resistant top layer with a rut resistant intermediate layer and a fatigue resistant base layer as illustrated in Figure A.11 (Newcomb et al, 2001).

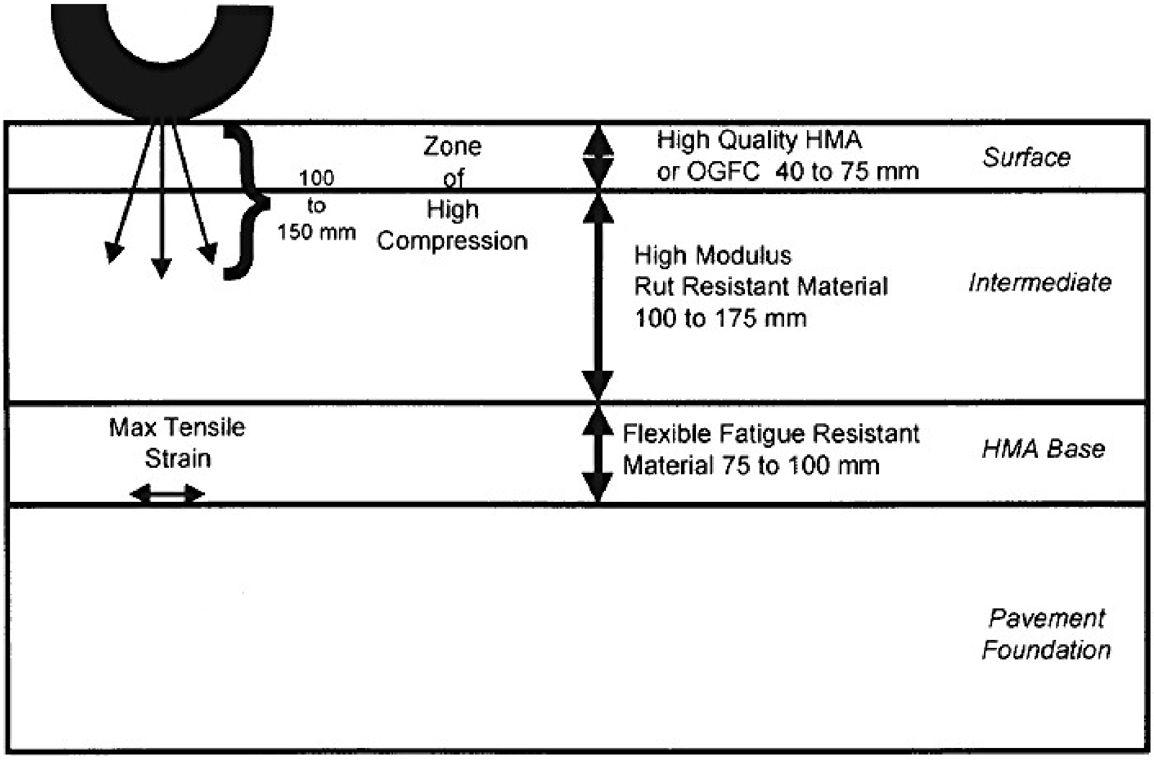


Figure A.11. Long life HMA pavement design concept (Newcomb et al., 2001).

This requires 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. In addition, the pavement foundation must be strong enough to satisfy the limiting strain criteria. Suggested values for the horizontal tensile strain at the bottom of the AC layer and vertical subgrade strain are 65 microstrain and 200 microstrains, respectively. The value for the endurance limit of the tensile strain at the bottom of the AC layer is still debated. Original work by Monismisth and others suggests a value of 65 microstrains (Figure A.12). Others believe that this value is too conservative, and that a higher value (100 to 120 microstrain) should be used to insure that the AC renewal solution is economical.

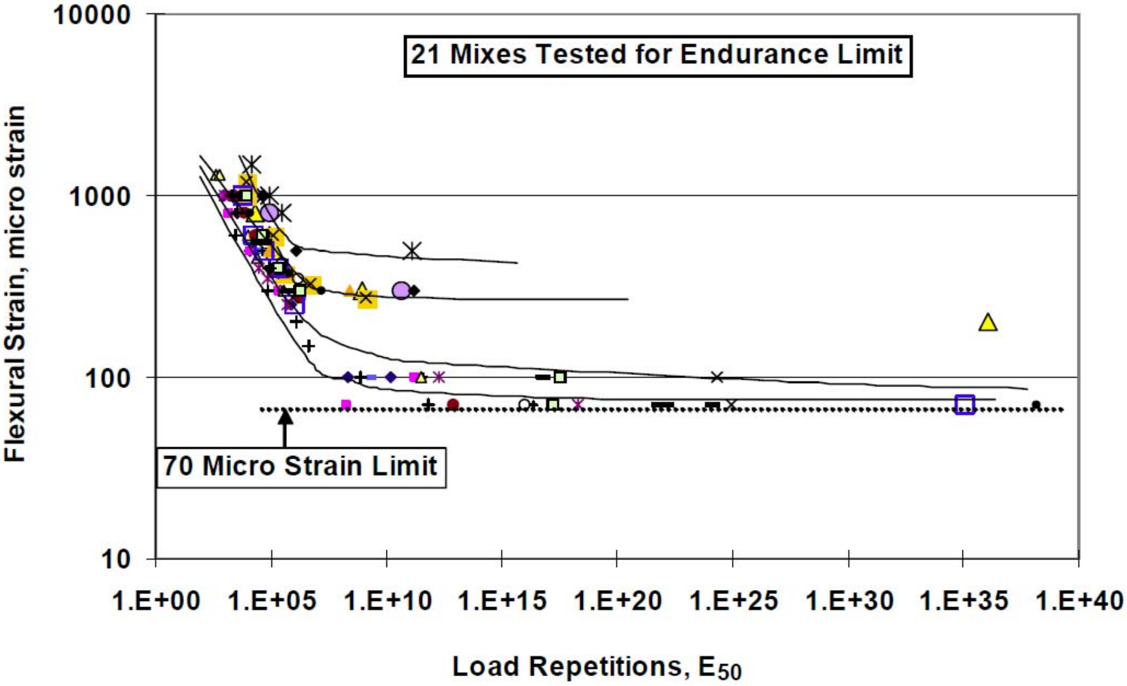


Figure A.12. Endurance fatigue limit for long-life AC pavements (Thompson, 2006).

When applied to existing pavements, a fourth condition is added: The inhibition of reflective cracking. This is true irrespective of the existing pavement type, i.e., distressed HMA or PCC, although experience shows that reflective cracking can be more predominant when the existing pavement is a PCC pavement. 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. Figure A.13 schematically illustrates reflection crack distress in an HMA overlay placed over a joint or crack of an existing PCC slab. Figure A.14 illustrates the mechanism through which the crack develops and propagates in the HMA layer (NAPA, 1994).

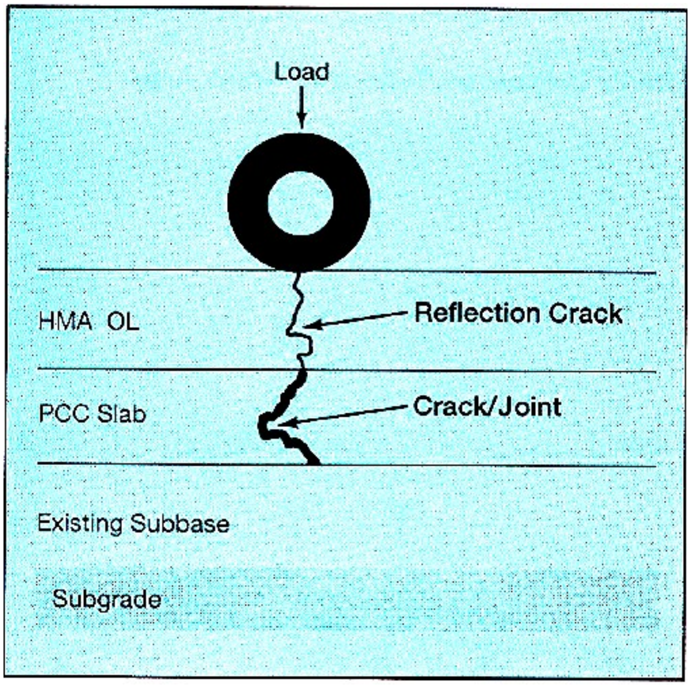


Figure A.13. Schematic representation of a reflection crack (NAPA, 1994).

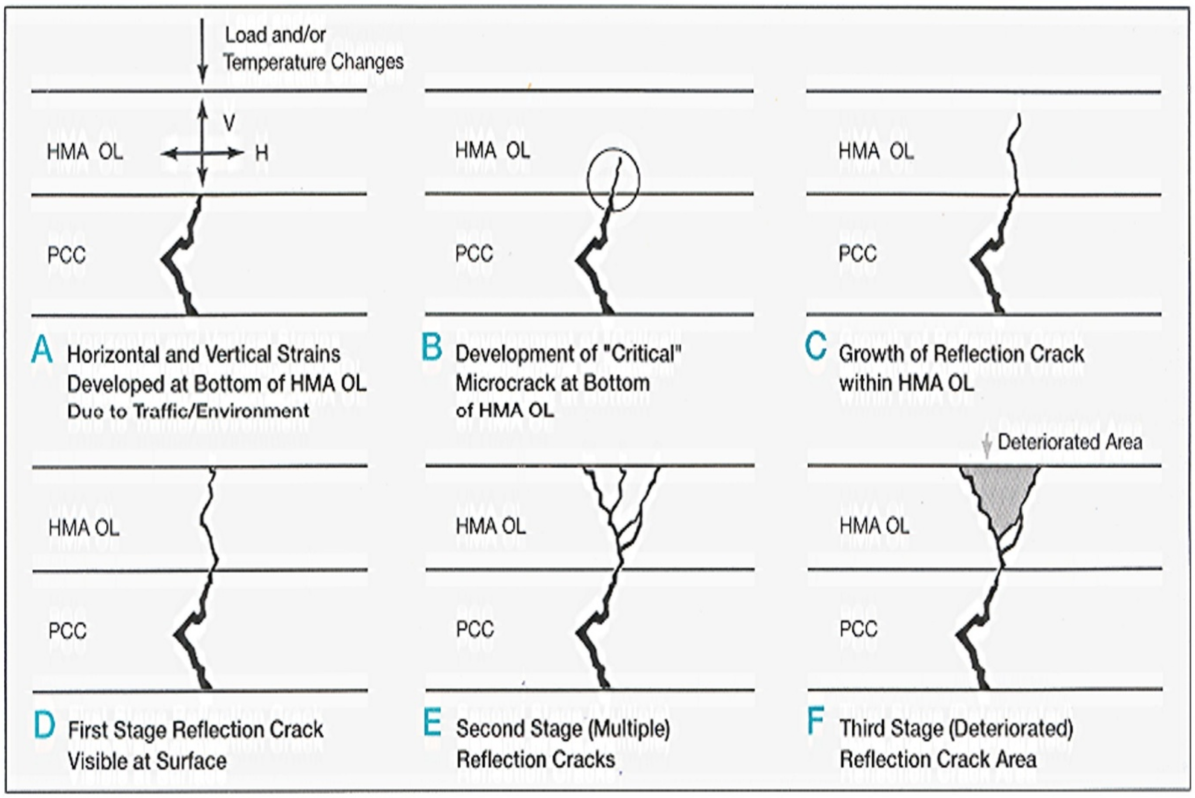


Figure A.14. Growth of a reflection crack (Napa, 1994).

PCC slabs expand and contract with seasonal changes in temperature. This movement causes the development of forces at the bottom of the HMA layer as shown in

"A" within Figure A.14. The combination of forces at the bottom of the HMA overlay will eventually cause the development of a microcrack at the bottom of the HMA overlay as shown in B. With time, this microcrack will grow and eventually reflect upwards to the surface of the HMA overlay as shown in C and D. As temperature and loading cycles continue, multiple cracks will form and eventually result in significant deterioration of the HMA surface as shown E and F. Figure A.15 illustrates a distressed reflection crack area in an HMA overlay over an existing PCC pavement.

Figure A.15. Reflection cracking in HMA overlay over PCC pavement (Martin, 1973).

Existing CRCP pavement is an excellent foundation for a new long-life HMA pavement since reflection cracking is not a problem as long as cracks are of low severity and failed areas (punch-outs and deteriorated cracks) are repaired prior to overlaying. Pavements with D-cracking are not good candidates for HMA overlays without slab fracturing. Studies have shown that the placement of HMA overlay can accelerate D-cracking, and field data showed poor performance of HMA overlays of concrete pavement with D-cracking (Zollinger et al., 2004).

Because the pavement foundation is critical to the construction and performance of a long-life HMA pavement, the question of whether or not an existing pavement can be used in-place largely depends on the quality of the existing foundation. A careful consideration to the existing condition of the pavement foundation must therefore be made. This is in light of the fact that there will be cases where the condition of the existing subgrade does not warrant using the existing pavement in-place (e.g., drainage problems or soft layer underneath existing pavement structure). Several end-result specifications for the foundation layers have been used in Europe (UK, France and Germany), requiring a minimum modulus under FWD loading or imposing a maximum tolerable surface deflection (Newcomb, 2001). The State of Illinois requires a minimum CBR or DCP-based cone index value below which the subgrade soil must be modified (using lime treatment) as shown in Figure A.16.

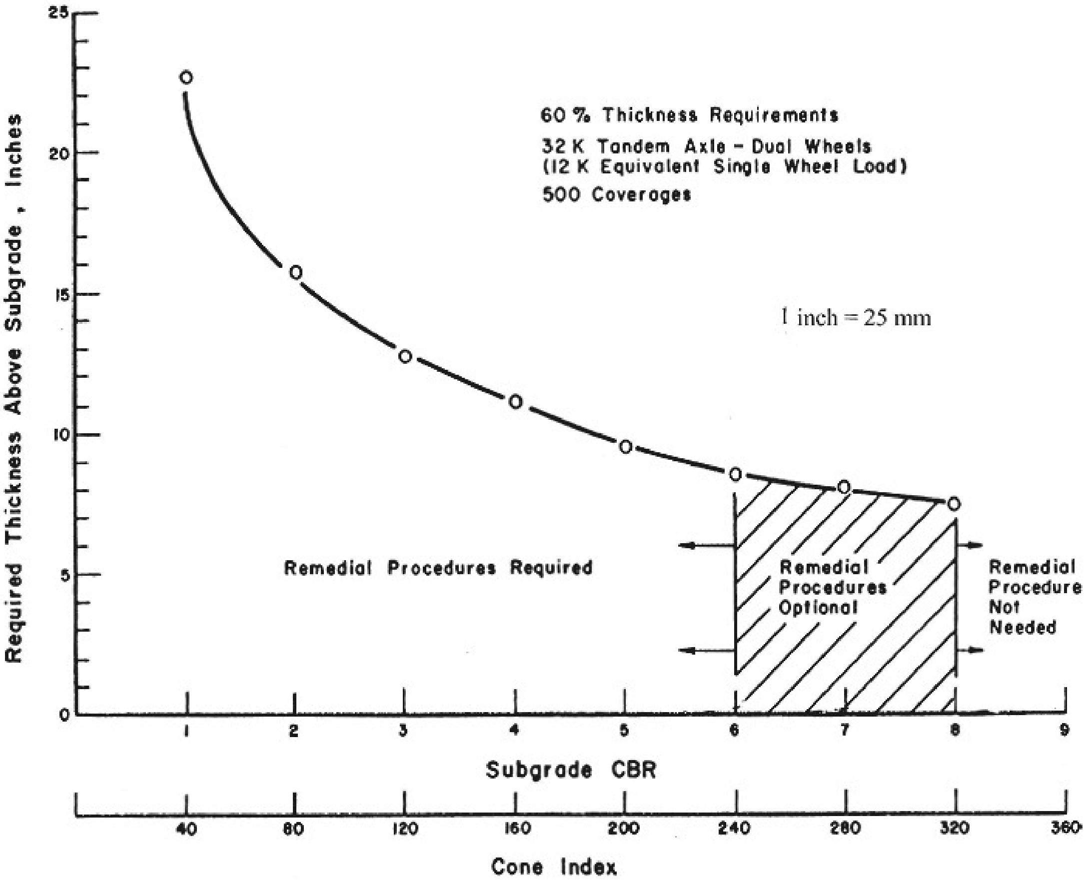


Figure A.16. Illinois granular thickness requirement for foundations (IDOT, 1982).

Overlay Design Approaches for AC Surfaced Pavements

The two most commonly used approaches to structural design of asphalt overlays of asphalt pavements are (1) the structural deficiency approach, exemplified by the 1993 AASHTO procedure (AASHTO, 1993), and (2) the deflection-based approach, exemplified by the Asphalt Institute procedure (AASHTO, 1999). Much less common is the mechanistic approach, in which fatigue and rutting performance are predicted using mechanistic-empirical models (Hall, 2001).

In a mechanistic-empirical approach to design of asphalt overlays of asphalt pavements, performance of the overlay is predicted using mechanistic-empirical distress models. The distresses considered should include at least fatigue cracking, and ideally rutting and thermal cracking as well. The existing pavement layers and foundation are characterized using nondestructive deflection testing and backcalculation of their elastic moduli. Material properties for the overlay are assumed. The overlay thickness that will yield acceptable performance in terms of the distresses considered is determined by iteration. A conceptual overview of the mechanistic-empirical approach to design of asphalt overlays of asphalt pavements is given by Monismith (1992).

The individual tools used in mechanistic-empirical design of asphalt pavements (fatigue models, rutting models, seasonal adjustment, etc.) can be adapted to some extent to design of asphalt overlays. However, there are additional aspects of the problem that need to be considered in order to develop a full design procedure for asphalt overlays of asphalt pavements. Among these are consideration of the extent, type, and quality of pre-overlay repairs, prediction of reflection crack propagation and deterioration (a problem for asphalt overlays of both asphalt and concrete pavements), and calibration of asphalt overlay performance prediction models to the observed performance of asphalt overlays.

Several examples of mechanistic-empirical procedures for design of asphalt pavements exist, such as the Shell procedure (1978), the Asphalt Institute procedure (1981, Shook, 1982), the NCHRP 1-26 procedure (Thompson, 1989), and the MEPDG procedure developed under NCHRP 1-37A (ARA, 2004). Fewer examples exist, however, of mechanistic-empirical procedures for design of asphalt overlays of asphalt pavements.

The NCHRP 1-37A MEPDG procedure's software for design of AC overlay of AC allows the user to select some or all of the following performance criteria by which the adequacy of a trial overlay design is judged:

- Rutting
- Alligator cracking
- Longitudinal cracking
- Transverse cracking
- Smoothness

According to the MEPDG, "the models used for the prediction of structural distresses (i.e., excluding smoothness prediction) in the overlaid pavement are basically the same as those described in Part 3, Chapter 3 [for design of new AC pavements] with some modifications to the rates of distress accumulation in the existing layers."

The smoothness parameter used for AC overlays of AC pavements in the MEPDG methodology is the International Roughness Index (IRI), predicted from an empirical model as a function of the existing pavement's IRI at the time of overlay placement, the time elapsed since placement of the overlay, the percent of the wheel path area with fatigue cracking, the average spacing of medium- and high-severity transverse cracks, the length of medium- and high-severity sealed longitudinal cracks in the wheel path, the percent of the total lane area with medium- and high-severity patches, and the percent of the total lane area with potholes.

Among the few State DOTs that have developed a mechanistic-empirical design procedure for asphalt overlays of asphalt pavements are Washington (Mahoney, 1989), Idaho, and Nevada (NDOT, 1996, Sebaaly, 1996). The Washington State DOT procedure uses a model to predict fatigue as a function of horizontal tensile stress at the bottom of the asphalt overlay and at the bottom of the original asphalt layer, as well as a model to predict rutting as a function of vertical compressive stress at the top of the subgrade. The critical stress locations considered are illustrated in Figure A.17. A flowchart of the Washington State procedure is illustrated in Figure A.18. The overlay thickness required to keep fatigue and rutting below critical levels is determined through a process of iteration.

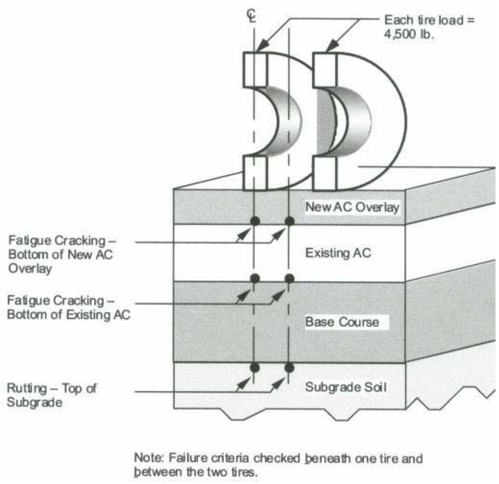


Figure A.17. Critical stress locations considered in Washington State DOT overlay design procedure. (WSDOT, 2004)

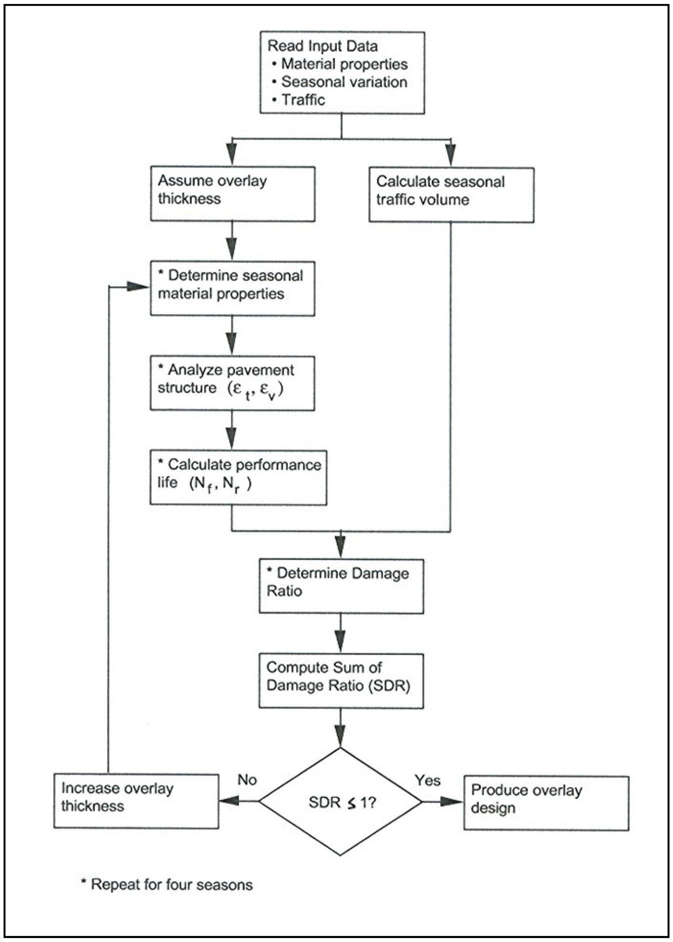
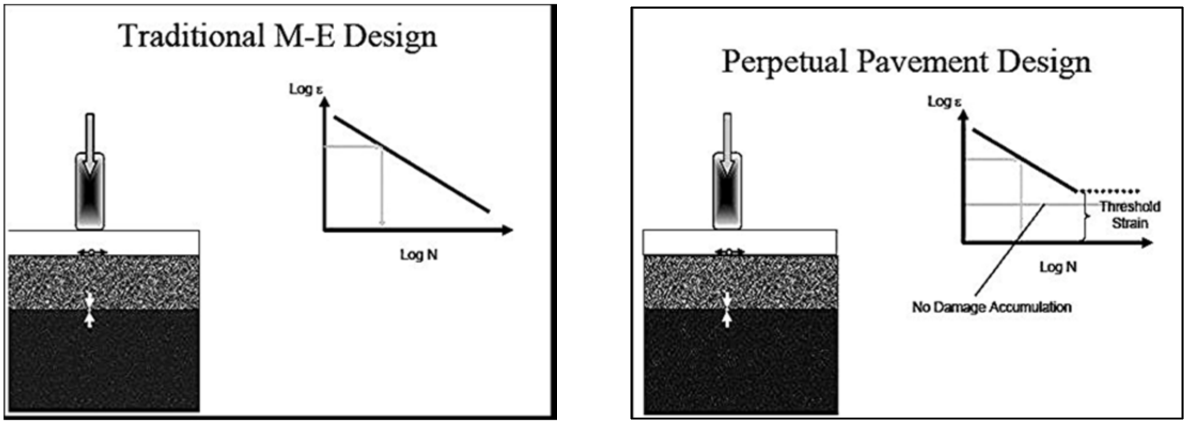


Figure A.18. Washington State DOT overlay design procedure flowchart. (WSDOT, 2004)

(a) Traditional M-E Design

(b) Long-life M-E Design

Figure A.19 compares traditional M-E design to long-life pavement design. The basic concept in designing long-life AC pavements is to use limiting strain criteria (see Figure A-21b):



(a) Traditional M-E Design (b) Long-life M-E Design

Figure A.19. Traditional versus long-life AC pavement design (Timm, 2005)

Structural Design of AC Overlay over Fractured Slab

The approach taken in the 1993 AASHTO Guide to design of asphalt overlays of fractured slabs (both crack and seat and rubblizing) is a structural deficiency approach. The overlay must satisfy the deficiency between the Structural Number (SN_f) required to support traffic over some future design period, and the effective Structural Number (SN_{eff}) of the existing pavement (after fracturing).

Perhaps the most contentious aspect of overlay design for fractured slabs by the structural deficiency approach is what structural coefficient should be assigned to the fractured slab. The 1993 AASHTO Guide recommends the following ranges for structural coefficients for different types of slab fracturing:

Rubblized:	0.14 – 0.30
Crack and seat:	0.20 – 0.35
Break and seat:	0.20 – 0.35

Other recommendations for overlay design for fractured slabs, including recommended ranges of structural coefficients and overlay thickness design tables, have been developed by the National Asphalt Pavement Association. A study done for the American Concrete Pavement Association recommended a range of 0.15 to 0.25, for the structural coefficient of all three types of fractured slabs (Hall, 1999).

A mechanistic procedure for design of AC overlays of cracked and seated concrete pavements was developed by Thompson at the University of Illinois as part of the FHWA/IDOT study "Mechanistic Evaluation of Illinois Flexible Pavement Design Procedures." For a given overlay thickness, the required inputs are the design AC elastic modulus, the subgrade resilient modulus, and the "equivalent modulus" of the cracked and seated concrete.

In the development of the design procedure, the finite element program ILLI-PAVE was used to estimate the asphalt concrete bending strain for a range of overlay thicknesses. Transfer functions for the number of repetitions to failure for a given bending strain were developed for typical IDOT Class I asphalt concrete mixtures (Schutzbach, 1988, 1989). Additional guidance on the use, design, and construction of AC overlays of cracked and seated PCC pavements is given by Thompson in NCHRP Synthesis No. 144 (Thompson, 1989).

Ahlrich has documented the use of FWD testing on intact PCC slabs and testing after cracking and seating and overlaying with AC, in order to determine the "effective modulus" of the cracked and seated PCC layer (Ahlrich, 1989). In field studies conducted by the U.S. Army Corps of Engineers Waterways Experiment Station, at the Rock Island Arsenal in Illinois and Fort Wainwright in Alaska, concrete slabs with an elastic modulus of about 6 million psi were reduced by cracking and seating to a fractured concrete layer with an effective elastic modulus of about 1 to 1.5 million psi. Similar results from analysis of FWD deflections measured on test sections at the LTPP SPS-6 test site on I-57 in Illinois have been reported by Hall (1991).

The NCHRP 1-37A MEPDG procedure's software for design of AC overlay of cracked and seated PCC allows the user to select some or all of the following performance criteria by which the adequacy of a trial overlay design is judged:

- Rutting
- Alligator cracking
- Longitudinal cracking
- Transverse cracking
- Smoothness

In the MEPDG software, the elastic modulus of the cracked and seated PCC is assigned as a function of the crack spacing (i.e., 200 ksi for 12-inch spacing, 250 ksi for 24-inch spacing, and 300 ksi for 36-inch spacing) 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.

Renewal of Rigid Pavements

When in-place renewal of an existing PCC pavement is considered, the structural design considerations that must be taken into account to ensure good long-term performance are the adequacy of the subgrade, protection of the subgrade from excessive deformation, limiting strain in the existing PCC, limiting stress and strain in the new AC or PCC surface, and minimizing reflection cracking in the new surface.

While AC overlay is no doubt the most commonly used major rehabilitation method for jointed PCC pavements, the service life of this technique is limited by the rate at which reflection cracks develop and deteriorate to unacceptably rough levels. Thus an AC overlay of a jointed PCC pavement is typically considered a conventional, rather than a long-life rehabilitation approach, with an expected service life of about 10 to15 years.

However, exceptions exist: Iowa, for example, has experience with jointed PCC pavements built in the 1930s and 40s, widened with PCC or AC from 18, 20 or 22 feet to 24 feet the 1970s, and then overlaid over time with a total of five or more inches of AC. Now, some 30 years later, these old AC/PCC pavements are being widened again, to 28 or 32 feet, and are being overlaid with PCC (Cable, 2008).

The most promising long-life rigid pavement methods, however, appear to be the following:

- AC over CRCP
- AC over cracked and seated JPCP
- AC over rubblized PCC
- Unbonded PCC over PCC
- Bonded PCC over PCC

Definition of Long Life Concrete Pavements

Long-life concrete pavements (LLCPs) have been quite attainable for a long time in the United States, as evidenced by the number of very old pavements that remain in service; however, recent advances in design, construction, and concrete materials technology give us the knowledge and technology needed to consistently achieve what we know to be attainable. A working definition of long-life concrete pavement in the United States is summarized as follows (Tayabji and Lim and 2007):

- 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.

The quest for long-life concrete pavements necessitates a much better understanding of design and construction factors that affect both short-term and long-term concrete pavement performance. Essentially, this requires that there be a better understanding of how concrete pavements deteriorate or fail. Concrete pavements deteriorate over a period of time as a result of distresses that develop due to a combination of traffic and environmental loading. Typical distresses that can develop include the following:

1. **Cracking**: Typically transverse cracking occurs, but longitudinal, random, and corner cracking may also develop due to poor design and construction practices. Cracking is typically referred to as a stress-based distress.
2. **Joint faulting**: Joint faulting may develop with or without outward signs of pumping. Faulting is typically referred to as a deflection-based response. Joint faulting is significantly affected by the type of load transfer provided at transverse joints.
3. **Spalling**: Spalling may develop along joints or cracks and may be caused by poor joint-sawing practices, incompressible materials in joints or cracks, winter snow removal operations, or poor-quality concrete.
4. **Materials-related distress**: The more significant materials- related distresses may include alkali-silica reactivity (ASR) and D-cracking in freezing environments.
5. **Roughness**: The lack of pavement smoothness, or roughness, is affected by the development of various distresses in the concrete pavement, as listed in items 1 through 4 above. The effect of each distress type is additive and results in pavement roughness over a period of time. Some pavement roughness is also built in during construction. Initial pavement smoothness is needed in order that the pavement does not become prematurely rough. Construction specifications typically utilize incentives and disincentives to control new pavement smoothness.
6. **Texture loss**: Although not conventionally considered a distress, texture loss is a significant distress for pavements in high-volume, high-speed applications.

It is realized that it would be impossible or impractical to design and construct concrete pavements that exhibit very little or no distress. Distress development over the pavement's service life is expected. However, the rate of distress development is managed by incorporating sound designs, durable paving materials, and quality construction practices. Generally recognized threshold values in the United States for distresses at the end of the pavement's service life are listed in Table A.2 for jointed plain concrete pavements (JPCPs) and continuously reinforced concrete pavements (CRCPs).

Table A.2. Threshold values for long life concrete pavement distresses.

Distress	Threshold Value
Cracked slabs, % of total slabs (JPCP)	10 to 15
Faulting, mm (in.) (JPCP)	6 to 7 (0.25)
Smoothness (IRI), m/km (in/mi) (JPCP and CRCP)	2.5 to 3.0 (150-180)
Spalling (length and severity) (JPCP and CRCP)	Minimal
Materials-related distress (JPCP and CRCP)	None
Punch-outs, #/km (mi) (CRCP)	10 to 12 (12 to 16)

Unbonded PCC over PCC

An unbonded PCC overlay (sometimes called a separated overlay) contains an interlayer between the existing PCC pavement and the new PCC overlay (Figure A.20). Unbonded overlays of all types (jointed plain, jointed reinforced, and continuously reinforced) can be placed on all types of concrete pavements, including those with existing asphalt overlays. Unbonded concrete pavements are appropriate for pavements with little or no remaining structural life, and/or extensive and severe durability distress. Unbonded concrete overlays require little or no pre-overlay repair, and are thus well suited to badly deteriorated concrete and asphalt-overlaid concrete pavements. An unbonded concrete overlay is an attractive alternative to reconstruction when construction duration is a pressing issue (e.g., for high traffic volumes and/or very poor subgrade conditions).

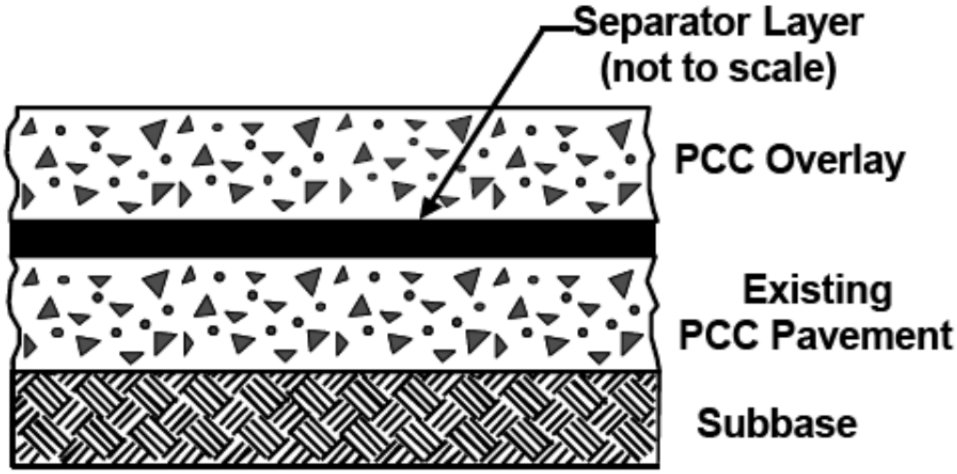


Figure A.20. Typical cross section of an Unbonded PCC overlay (McGhee 1994).

Jointed unbonded PCC overlays of PCC highway pavements have been built in the United States since the 1920s. The first unbonded CRC overlay of an existing jointed PCC highway pavement in the US was constructed in Texas in 1959 (Martin 1973). In subsequent years CRC overlays were placed on hundreds of miles of both asphalt and jointed concrete pavements. Illinois built its first experimental test sections of CRC overlay on jointed reinforced PCC pavement in 1967 (Dhamrait, 1978). Georgia built its first CRC overlay of a jointed plain PCC pavement in 1973 (Tyner, 1981). The first unbonded CRC overlay of an existing CRC highway pavement in the US was constructed on I-59 in Mississippi in 1982 (Crawley, 1982).

There is little doubt that unbonded concrete overlays, be they jointed or CRC, are substantial pavement structures with expected performance characteristics as good as or better than new concrete pavement construction. They are essentially new concrete pavements on high-quality foundations, and the consensus from past field studies is that as long as an adequate separation layer is used, their performance is fairly insensitive to the condition of the overlaid pavement. Thus, they are certainly viable candidates for long-life in-place renewal projects. To date, unbonded overlays have typically been designed for service lives in the range of 20 to 30 years. The PCC overlay thickness design approaches, slab thicknesses, and other design details required to achieve service lives of 40 or 50 years needs to be studied (Hall and Darter, 1993).

Traditionally, unbonded concrete overlays have been designed using some form of the familiar "square root" equation shown below:

$$h_{ol} = \sqrt{h_f^{\,2} - h_{eff}^{\,2}}$$

Where:

h_{ol} = unbonded overlay thickness
h_f = required slab thickness for future traffic
h_{eff} = effective thickness of the existing slab

The square root equation dates back to the Bates Road Test in the 1920s, and its use in unbonded overlay design procedures started in the 1940s (Older, 1924). Full-scale field tests of concrete overlays conducted by the Corps of Engineers in the 1940s and 50s indicated that the square root equation yielded conservative results (Mellinger, 1963).

Although many engineers have the impression that the square root equation (also called the Corps of Engineers equation) for unbonded overlay design is completely empirical, it has a theoretical basis. Several researchers have demonstrated that an overlay slab and a base slab can be represented by an equivalent single slab in a variety of ways, e.g., equivalent surface deflection, equivalent tensile stress in the overlay slab, and equivalent tensile stress in the base slab.

There are, however, some important limitations to the characterization of an unbonded overlay and base slab as an equivalent single slab. The Corps of Engineers square root equation is a simplified form of the equations for stress in either the base slab or overlay slab equivalent to stress in the equivalent single slab. This simplified equation is only valid when the two slabs are equal in thickness and equal in elastic modulus.

Another important limitation to characterizing an unbonded overlay slab and base slab in terms of an equivalent single slab is that it assumes full contact between the overlay and base slabs. They may bend independently, but they must have the same radius of curvature. To whatever extent the overlay slab curls and/or warps to a different shape than the underlying slab, it will experience different, and in some cases, much greater stresses under combined load and curling than the equivalent thickness concept implies.

The third major limitation of the Corps of Engineers equation is the structural deficiency concept itself, namely, the assumption that an overlay satisfies a structural deficiency between a required single slab thickness and an existing slab's effective (i.e., damage-adjusted) thickness. As can be seen by examining the square root equation, the structural deficiency concept implies that for a given required slab thickness for future traffic, a thicker existing pavement will require a thinner unbonded overlay than a thinner existing pavement in the same condition. Conversely, it implies that a given thickness of unbonded overlay will perform better on a thicker existing pavement than on a thinner existing pavement in the same condition. Field observations do not support the implication that unbonded overlay performance is as sensitive to existing pavement thickness as the structural deficiency concept suggests.

One alternative to the Corps of Engineers equation for design of an unbonded overlay is to design the overlay as if it were a new pavement, with the existing pavement structure characterized as a foundation for the new slab. The elastic modulus, modulus of rupture, and load transfer coefficient inputs to the design model are typically the anticipated values for the overlay slab. Two key differences exist between this approach and the Corps of Engineers approach. The first difference is that the existing pavement is not considered to contribute any structural capacity to the total structural capacity of the overlaid pavement. The existing pavement is instead considered a foundation for the new slab. This leads to the second major difference between the two methods. The k-value of the foundation beneath the existing pavement is used to determine the required future slab thickness in the Corps of Engineers method, whereas the new pavement design method requires a k-value beneath the overlay. The major difficulty in application of the new design approach thus lies in selection of an appropriate design k-value (Barenberg, 1981).

Conventional practice in concrete pavement design for many years has been to assign a k value to a granular or stabilized base that is considerably higher than the k-value of the subgrade, and which was a function of the thickness and stiffness of the base layer. This convention is still employed for new concrete pavements in the 1993 AASHTO Guide and Portland Cement Association design procedures. Following this logic, an existing concrete pavement with an asphalt concrete surfacing for a separation layer would be assigned a very high k-value, such as 500 psi/in or more for unbonded overlay design. However, backcalculation results indicate that when an unbonded overlay is designed as a new pavement with the existing pavement as its foundation, it is neither necessary nor appropriate to use an extremely high k-value such as 500 psi/inch or more. A design static k-value in the range of 200 to 400 psi/inch is probably appropriate in most cases. Whenever possible, deflections should be measured on the existing pavement prior to overlay to backcalculate a dynamic k-value for the existing foundation, and to estimate from this a reasonable static k-value for design.

Another issue that should be considered is the effect of curling on performance. If a jointed overlay slab is designed as a new pavement with the existing pavement serving as its foundation, it will experience much higher curling stresses than a conventional concrete pavement on a weaker foundation (Voigt, 1989). These higher curling stresses may be computed using finite element analysis or available equations. However, if the performance model used to determine the required slab thickness was developed for concrete pavements on weak foundations, the detrimental effect of high curling stress will not be adequately reflected in the predicted performance of the overlay. This would be the case if, for example, the 1993 AASHTO design procedure was used to determine the required slab thickness, rather than a fatigue analysis that directly considered the combined effects of load and curling. Either increased slab thickness or reduced joint spacing may be necessary to achieve the performance from the unbonded overlay that is predicted by the model.

Other alternatives to unbonded overlay design involve modeling the overlay and existing slab as either two elastic layers or two plates on a foundation. This is arguably the most realistic of the three design approaches described here, but also the most difficult. The basic approach is the same as for design of the overlay as a new pavement, except that the existing pavement structure is characterized more realistically, not as a uniform foundation, but as a multilayered system. Among the difficulties associated with this approach are the following:

- Characterization of the existing slab, including deciding how (it at all) to account for existing deterioration.
- Identifying the important structural responses (e.g., overlay stress, overlay deflection, original slab stress, etc.).
- Identifying the important performance criteria (e.g., cracking in the original slab and/or cracking in the overlay slab).

In jointed unbonded concrete overlays, the joints should be spaced more closely than they would be in a new pavement on a granular base, and the overlay's transverse joints and the old pavement's transverse joints should be mismatched to improve load transfer across the overlay joints. Mismatching the joints by at least 1 foot is advisable; several agencies specify a mismatch of 3 feet.

According to the American Concrete Pavement Association (ACPA), dowels are not considered necessary for jointed unbonded overlays less than 8 inches thick. For overlays 8 to 9 inches thick, 1.25-inch-diameter dowels are recommended, and for overlays greater than 9 inches thick, 1.5-inch-diameter dowels are recommended. The ACPA also provides guidelines for constructing transitions between unbonded concrete overlays and existing or reconstructed pavement sections.

Additional information on the design and performance of unbonded concrete overlays is provided in NCHRP Synthesis 99, *Resurfacing with Portland Cement Concrete*, NCHRP Synthesis No. 204, *Portland Cement Concrete Resurfacing*, the American Concrete Pavement Association's *Guidelines for Unbonded Concrete Overlays (1990),* the Portland Cement Association's *Guide to Concrete Resurfacing Designs and Selection Criteria (1981),* and NCHRP Report No. 415, *Evaluation of Unbonded Portland Cement Concrete Overlays (1999).*

The performance of unbonded PCC overlays of existing PCC pavements depends significantly upon obtaining effective *separation* between the two layers. Since the unbonded PCC overlays are placed on PCC pavements in a more advanced state of deterioration, distresses from the underlying pavement can potentially reflect through the new overlay and compromise its performance. Typically, a fine graded asphalt surface mixture is used for the separator layer. The thickness of the separator layer is a function of (1) the condition of the existing pavement, and (2) the type of pre overlay repairs. Based on the review of the literature a minimum thickness of 1 inch is recommended for HMA separator layers. Thinner layers erode easily near joints and do not provide adequate isolation of the overlay from underlying PCC pavement. The separator layer is not intended to provide structural enhancement; therefore, the placement of an excessively thick layer should be avoided. Some states DOTs have modified the asphalt mixture because their surface mixes were not stable and were prone to scouring, particularly under heavy truck traffic. In an effort to reduce the scour pore pressure and increase stability, the sand content was reduced and the volume of ⅜ in. (9.5 mm) chip aggregate was increased (Guide to Concrete Overlay Solutions, CP Tech Center). This modified mixture has a reduced unit weight and lower asphalt content.

Other bituminous surface treatments such as slurry seals, cutbacks and emulsions have been used for low volume roads. In Germany lean concrete is used as an interlayer. This is done in conjunction with breaking or fracturing the existing pavement before overlaying the lean concrete interlayer. In addition to this the interlayer is jointed to match the joints of the overlay.

Belgium is the only country outside the United States identified in this review as having reported appreciable experience with unbonded concrete overlays (Hall, 2007). Belgium constructed its first concrete overlay in 1960, over a concrete pavement originally constructed in 1934. The jointed concrete overlay was constructed of 7-inch-thick reinforced concrete slabs. Figure A.21 shows the overlay still in service nearly 45 years later.

Construction of a concrete overlay on the E40/A10 road from Brussels to Ostende in Belgium is shown in Figure A.22. Two mobile concrete plants were used to produce the 2600 cubic yards of concrete a day required for this project. The average paving rate was 3900 ft per day, 24 feet wide. A closer view of the paver is shown in Figure A.23. Due to the very tight schedule for this project, concrete was placed without interruption, 24 hours a day, 7 days a week. As a result, the CRC overlay has no construction joints. A slipform paver was also used to construct the safety barriers on this job, as shown in Figure A.24.

Figure A.21. Belgium's first concrete overlay after 45 years in service.

(Photo: Hall 2007)

Figure A.22. CRC overlay construction on E40/A10 in Belgium. (Photo: Hall 2007)

Figure A.23. CRC overlay paving on E40/A10 in Belgium. (Photo: Hall 2007)

Figure A.24. Slipform paving of the safety barriers on the E40/A10 CRC overlay project. (Photo: Hall 2007)

Bonded PCC over PCC

Bonded PCC overlays of PCC are generally not considered very long-life pavement rehabilitation techniques, because of their sensitivity to the condition of the underlying pavement, and the difficulty of achieving the long-lasting bond necessary for composite bending action. Bonded concrete overlays are not often used, because they perform best on pavements in good to fair condition, that is, pavements that are not in urgent need of rehabilitation.

The *bonded concrete surface* is bonded to the existing concrete pavement to form a monolithic section. This renewal strategy has the potential to increase the structural capacity of an existing concrete pavement or to improve the overall ride quality. The bonded concrete surface is typically 2-5 inches thick. The bonded concrete surface works best when the existing pavement is free of structural distress and in relatively good condition. This rapid renewal strategy is typically attractive when vertical clearances must be met, or in mill and inlay sections, or in conjunction with widening projects. The achievement of an effective bond between the existing pavement and the new surface is critical in ensuring satisfactory performance of the bonded concrete surface. The use of “bonding agents” and “direct placement” are two methods that are practiced for this type of rehabilitation. Figure A.25 shows a cross section of a typical bonded PCC overlay.

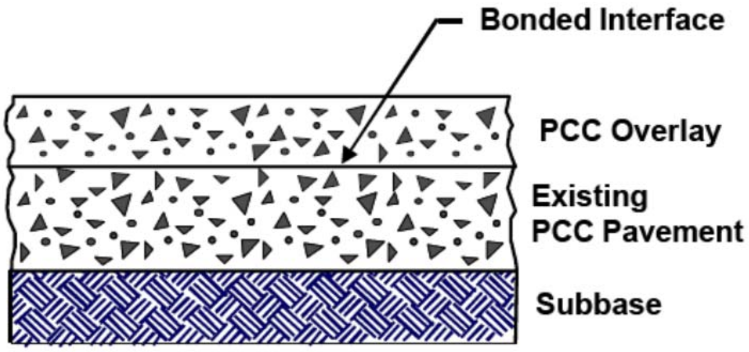


Figure A.25. Typical cross section of a bonded PCC overlay (McGhee, 1994).

The service life of a bonded PCC overlay of a PCC highway pavement is typically estimated at about 15 to 25 years at best. However, in the course of the work done for Task 1 of this study, a bonded PCC overlay recently constructed in Oklahoma was identified as one that is expected by some to be very capable of providing 40 years of service or more. The design and construction details of this project warrant study to gain insight into whether or not, and under what conditions, bonded PCC overlays might be viable candidates for long-life in-place renewal projects. Bonded PCC overlays have also been constructed in many different states, including California, Illinois, Iowa, Louisiana, New York, Pennsylvania, South Dakota, Texas, and Virginia, as well as in the countries of Belgium, Canada, Japan, and Sweden. By far the most common bonded PCC overlay type is JPCP, and these have been placed on existing JPCP, JRCP, and CRCP designs

(Sebaaly, 1996). Some bonded JRCP overlays have been used on existing JPCP and JRCP, although presently they are rarely used. Texas and Virginia have both constructed several bonded overlays on existing CRCP.

For bonded PCC overlays of existing PCC pavements, achieving bond between the two layers is critical monolithic slab behavior. To help achieve this, many state highway agencies place either a cement grout or an epoxy resin on the existing PCC pavement just ahead of the paver. Cement grouts are generally produced in a mobile mixer from a mixture of portland cement and water; the grout should have a maximum *w/c* of 0.62 (ACPA 1990a). Epoxy bonding agents should be applied in accordance with the manufacturer's instructions. Prior to the placement of either type of bonding agent, the pavement surface should have already been prepared and should be dry (ACPA 1990a).

Renewal by Lane Replacement (Inlay) or Lane Addition

When a lane replacement or lane addition is contemplated as an approach to in-place renewal of an existing AC or PCC pavement, the design considerations that must be taken into account to ensure good long-term performance include the adequacy of the foundation, the required thickness and any constraints on it, the method of connection to the adjacent lane, design of transitions, and in the case of widening, geometric considerations such as the availability of horizontal and vertical space for relocating shoulders, slopes, ditches and/or drainage systems, interchanges, and bridges.

Lane Replacement

The evident viability of this technique as a long-life in-place renewal method seems at odds with the relatively little use that it has seen to date in the United States. When a portion of the thickness of an AC lane is milled out and replaced with PCC, it can be considered, and designed and constructed as, a conventional whitetopping overlay. One caution, however, is that in some such applications of concrete inlays with undowelled joints, premature joint faulting has occurred, and has been attributed to the "bathtub effect" of water collecting under the PCC overlay slab. The ACPA recommends that either dowelled jointed PCC or CRC be used when constructing an inlay to replace a portion of the thickness of an AC traffic lane subjected to heavy traffic in one direction and wet climatic conditions. An *inlay* is a renewal option which involves the replacement of all or part of an existing pavement travel lane(s) without significantly raising the surface elevation. Inlays are practical for deteriorated concrete pavements. Single and multilane inlays are common for concrete reconstruction. When a lane replacement or lane addition is contemplated as an approach to in-place renewal of an existing PCC pavement, the design considerations that must be taken into account to ensure good long-term performance include the adequacy of the foundation, the required thickness and any constraints on it, the method of connection to the adjacent lane, design of transitions, and in the case of widening, geometric considerations such as the availability of horizontal and vertical space for relocating shoulders, slopes, ditches and/or drainage systems, interchanges, and bridges.

More information on design and construction of concrete inlays in existing AC or PCC pavements is provided in the ACPA publication *Reconstruction Optimization Through Concrete Inlays* (1993).

Belgium's experience with concrete inlays dates back to 1933 (ERES, 1999). Concrete inlays in Belgium are constructed with either JPCP or CRCP. Figure A.26 is a photo of a CRC inlay being placed on the A10 freeway in Belgium (Caestecker, 2003). The roadway had three lanes in each direction, and the existing pavement was AC over JPCP. Rutting, reflection cracking, and roughness over AC patches in the PCC layer, particularly in the outer lanes, were resulting in steadily increasing annual maintenance costs.

Figure A.26. CRC inlay construction in Belgium. (Photo: Hall 2007)

Lane Addition

While adding new lanes to an existing pavement structure is also clearly a viable option for in-place long-life pavement renewal, it is costly and thus usually is only done when it is essential to increase the capacity of an existing roadway. In the course of the Task 1 work done for this study, several examples of lane addition projects on major highways in the eastern north-south corridor of the US, including some that are currently under construction, were identified. Both the structural design aspects and the construction logistics aspects of such projects need to be studied to identify the requirements for achieving good performance over an extended service life.

Caestecker described an example of this type of work: the replacement of an outer AC shoulder with a fourth traffic lane on a heavily trafficked section of a six-lane highway on the A3 motorway toward Brussels, Belgium (Caestecker, 1993). An important reason that the lane addition option was chosen was that highway noise is a significant environmental concern in Belgium, and designers were confident that a concrete-surfaced lane addition could achieve the capacity increase desired while minimizing the traffic noise generated by the roadway. The new lane was placed with a GOMACO slip form paver, operating at its capacity of 300 to 500 m per day.

Immediately after paving, the surface was sprayed with a retarding agent and covered with plastic sheeting, to be brushed later to achieve the kind of exposed aggregate surface that has become popular in some European countries for both noise control and friction. The bituminous surface material salvaged from the old pavement shoulder was used in the cement-bound base layer of the new shoulder constructed alongside the new traffic lane.

Concrete Overlay Materials Needed for Long-Life

Much of the emphasis in defining the characteristics of in-place pavement renewal options with the potential for service lives in the range of 50 years is necessarily on the structural design of the new material. Decisions regarding PCC mix materials are affected by the type of mixture—conventional or fast track (accelerated)—desired for a specific project. For the purpose of this report, only conventional PCC mixtures will be discussed.

Conventional PCC Mixtures for Overlay Construction

Conventional concrete paving mixtures are typically used in the construction of concrete overlays. As with conventional concrete pavements, an effective mixture design is essential to the performance of a concrete overlay. Each component of the concrete mixture should be carefully selected so that the resulting mixture is dense, relatively impermeable, and resistant to both environmental effects and material related chemical reactions over its service life. As Shilstone points out, thickness is only one of two key components of long-life pavement materials; the other is durability (Shilstone, 2002). For example, in Portland cement concrete, Shilstone identifies the following characteristics as key to long-term durability:

- **Low permeability** – achieved with low total water, well-graded aggregate, good mixture rheology, and high in-place relative density.
- **Freeze-thaw resistance** – achieved with closely spaced small air voids, ultimate compressive strength of 40 MPa (6000 psi) or higher, well-graded aggregate, low permeability, and good curing.
- **Low shrinkage** – achieved with low total water, low cement factor, low water-cement ratio, and minimal use of sharp and elongated particles.
- **Low reactivity** – achieved with proper selection of cement type and aggregates, low permeability to reduce the potential for water penetration, low water-cement ratio, and use of a properly selected pozzolanic material in the mix.

- **Abrasion resistance** – achieved with compressive strength of 40 MPa (6000 psi) or higher, well-graded aggregate, low water content, hard and dense aggregate, and air content appropriate for the exposure conditions.

Most agencies specify a minimum concrete strength requirement for their pavements. Typical values include a 28-day compressive strength of 4,000 psi or a 28-day, third-point flexural strength of 650 psi (these specifications vary amongst State Highway Agencies).

Cementitious Materials

In general, Type I and type II cements are commonly used in concrete mixtures for concrete overlay construction. The standard specification for portland cements used in the United States is presented in AASHTO M 85 (ASTM C 150). There are many references available that provide detailed descriptions of the physical and chemical characteristics of cements [e.g., *Design and Control of Concrete Mixtures* (Kosmatka et al. 2002)] and will not discussed further in this section. Depending on the mix design and strength requirements, cement content is typically in the range of 500–700 lb/yd^3, (226.8–317.5 kg/m^3) although higher content is sometimes used. ACI and PCA provide guidelines for the selection of the appropriate w/cm ratio. A maximum w/cm ratio value of 0.45 is common for pavements in a moist environment that will be subjected to freeze-thaw cycles. However, lower w/cm ratio values are used for concrete resurfacing to minimize drying shrinkage. As with conventional paving, supplementary cementitious materials (SCMs) normally improve durability and can improve construction.

Aggregates

To ensure long life of the overlay, these aggregates should possess adequate strength and physical and chemical stability within the concrete mixture. All aggregates used in the production of PCC mixtures should conform to ASTM C 33. Extensive laboratory testing or demonstrated field performance is often required to ensure the selection of a durable aggregate. For concrete resurfacing of concrete pavements, the types of aggregates in both the original pavement and the overlay should be similar so that the thermal expansion is similar. The coefficient of thermal expansion of concrete significantly influences joint design. It is therefore recommended that the coefficient of thermal expansion of concrete be measured in accordance with AASHTO TP 60. The maximum coarse aggregate size used in concrete mixtures is a function of the pavement thickness or the amount of reinforcing steel. It is recommended that the largest practical maximum coarse aggregate size be used in order to minimize paste requirements, reduce shrinkage, minimize costs, and improve mechanical interlock properties at joints and cracks. Typically, maximum coarse aggregate sizes of ¾–1 in. (1.9–2.5 cm) have been common in the last two decades, however smaller maximum coarse aggregate sizes may be required for concrete (thin) resurfacing. The use of well graded aggregates reduces shrinkage.

Admixtures

Typical admixtures and additives that are commonly used in concrete mixtures include air entrainment (6-7 percent) water reducers, and supplementary cementitious materials (SCMs) such as fly ash and ground granulated blast furnace slag (GGBFS) may also be added to concrete mixtures.

SUMMARY

In this Appendix, various renewal approaches that are applicable to using existing pavements in pace and achieving long-life were described. These include (1) AC overlay over existing AC pavements, (2) AC over crushed and shaped AC, (3) AC over reclaimed AC, (4) AC over CRCP, (5) AC over cracked and seated JPCP, and (6) AC over rubblized PCC. An overview of the criteria required for achieving long-life was also presented, and various overlay design approaches for AC surfaced pavements were outlined.

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APPENDIX B

SYNTHESIS OF LONG-TERM PAVEMENT PERFORMANCE DATA

SYNTHESIS OF LONG-TERM PAVEMENT PERFORMANCE DATA

Data available from the LTPP experiment provides valuable information on the materials, climate, and traffic of test sections with measured performance data. This information was an integral part of the project because it provides an indication of pavement life under various conditions.

AC RENEWAL PROJECTS

The following LTPP experiments were reviewed to determine the pavement life achieved for HMA surfaced pavements:

- GPS 6A: Existing AC Overlay on AC Pavement
- GPS 6B: AC Overlay with Conventional Asphalt Cement on AC Pavement
- GPS 7A: Existing AC Overlay on PCC Pavement
- GPS 7B: AC Overlay with Conventional Asphalt Cement on PCC Pavement
- SPS 5: AC Overlay of AC Pavement
- SPS 6: Rehabilitation of Jointed PCC Pavement

The “LTPP DataPave Online” database (Release 21, January 2007) was used as the primary data source. Layer Inventory information was extracted from the table TST_L05B in IMS module TST (Testing), and has been summarized by:

a) Layer number
b) Layer type
c) Layer description
d) Representative thickness
e) Material type
f) Construction number

“Pavement Age” was calculated at different construction events for each section in a given state as follows: (1) age since initial construction, (2) age at the time of overlay, and (3) age since overlay construction. The initial date was taken as the “Traffic Opening Date” for GPS sections and the “Assigned Date” for SPS sections. Age at the time of overlay was calculated as the difference between “CN Change Date” for SPS5, SPS6, GPS6B and GPS7B sections, or “Major Improvement Date” for GPS6A and GPS7A sections (these sections have been fixed before their “Assign Date”) and the initial date, or the date of any previous fix. The latest “Survey Date” was taken as the end date.

Performance data including “Longitudinal Cracking,” “Alligator (Fatigue) Cracking,” “Transverse Cracking,” “Rut Depth,” and “IRI” were plotted against pavement age. Sections with the longest overlay ages were selected within the experiment and “Traffic Data” (ESALs) corresponding to pavement age was extracted from “TRF_MON_EST_HIST”. In some cases, due to missing “Traffic Monitoring” data, ESAL counts were estimated for the latest reported “Survey Date” by fitting the recorded data and extrapolating.

The following summarizes the main statistics obtained from each experiment, focusing only on: (1) age at last survey, (2) original pavement type, and (3) overlay thickness. Note that the lower overlay ages do not necessarily imply poor performance, since these are ages at the latest survey, and are not tied to any performance criterion. Older overlays merit further investigation.

The next step is to look into the better performing sections to determine potential long-life pavement candidates, if any. To do so, different criteria were considered for selecting sections. Initially, pavements with “Longer Lasting Overlay” were selected within each experiment. Table B.1, Table B.4, Table B.7, Table B.10, Table B.13, and Table B.14 summarize the sections which met this criterion. Except for Asphalt Overlay over CRCP, the outcome of this exercise was not conclusive since overlay age is determined up to the latest survey date and not to the end of its life based on some performance threshold. Therefore, younger overlay structures may potentially live longer. Also, performance of a given pavement structure depends on traffic volume. Therefore, a relatively thick pavement structure which was exposed to low ESALs may not necessarily represent a good-performing pavement.

Next, we selected sections within each experiment which have been subjected to “Heavy Volume of Traffic”. That is, cumulative ESAL counts within an experiment were extrapolated up to the latest survey date. Then, projected ESAL was normalized to pavement age. Within each experiment, sections of higher ESAL count per year were selected and their performance was evaluated. Table B.2, Table B.5, Table B.8, and Table B.11 represent such sections. Some sections have shown an acceptable level of performance after rehabilitation while serving higher traffic volume. Such cross sections can be candidates for perpetual pavement analysis.

The third approach was to select “Good Performing” sections using “Fatigue Cracking” and “Rutting” Performance as the critical distresses. Good-Performing cross sections within each experiment were selected and the number of ESALs was projected up to the latest survey date. These sections were categorized as “Thin,” “Medium,” and “Thick” structures as follows:

- “Thin Structure” refers to any thickness of less than 5 inches for Asphalt Concrete layers and 9 inches for Portland Cement Concrete layers.

- “Medium Structure” refers to any thickness for Asphalt Concrete layers which is greater than 5 inches but less than 9 inches. For Portland Cement Concrete layers, these limits change to 9 inches and 12 inches, respectively.

- “Thick Structure" refers to any Asphalt Concrete layer and Portland Cement Concrete layers with a thickness greater than 9 inches and 12 inches, respectively.

Table B.3, Table B.6, Table B.9, and Table B.12 present such cross sections within the different experiments. Not all the sections were useful since either overlays were not old enough to represent a reasonable trend of performance, or sections were exposed to lower traffic volume.

GPS6A – Existing AC Overlay on AC Pavement

There are a total of 51 sections in 25 states within this experiment. Figure B.6 summarizes the frequency of overlay age within the experiment. Table B.1, Table B.2, and Table B.3 summarize the relevant inventory and performance information on sections with the longest overlay ages, those with high traffic loading, and those with the best performance in cracking and rutting.

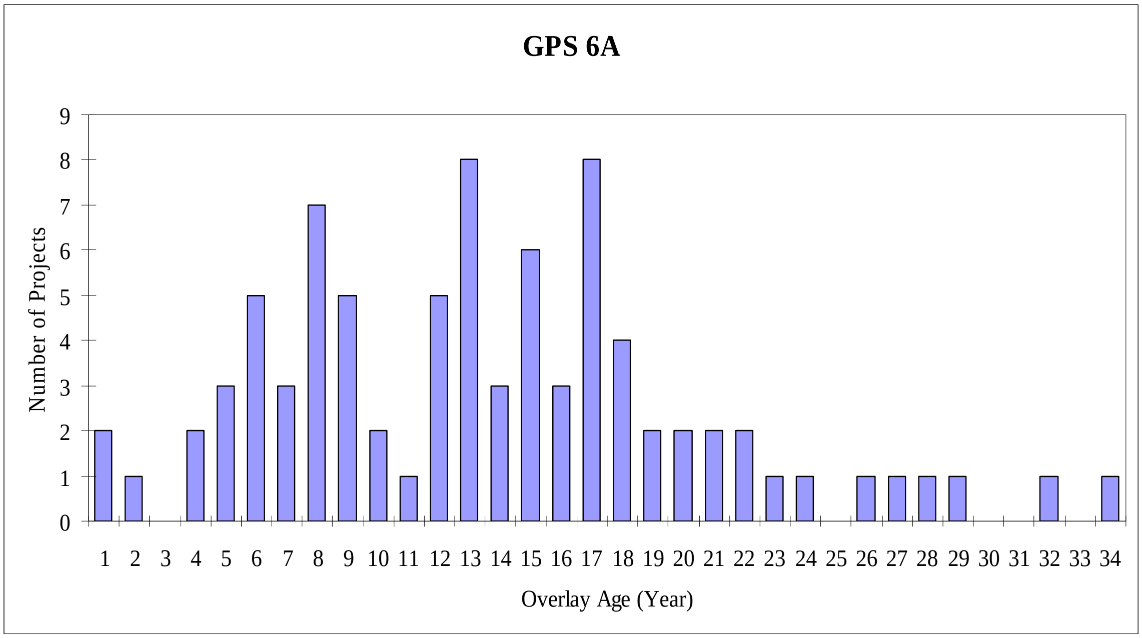


Figure B.1. Frequency of overlay age within experiment GPS-6A.

Summary Interpretation

At first glance, no real trends can be observed in Table B.1. It shows that HMA overlays on existing HMA pavements can perform at an acceptable level for up to 30 or more years. However, the required overlay thickness (obviously) depends on the traffic loading: Section 19-6150 has received only a surface treatment and its cumulative ESAL is only 236,000, clearly indicating that it is a low volume road. Also, it has extensive transverse cracking (at about 5 ft spacing). Section 48-1046 (Texas) has a 10 inch HMA overlay, which is expected for a cumulative traffic of 14.8 million ESALs. It has clearly reached its fatigue end life since it has more than 50 percent cracking. Section 48-6179 has a 4 inch HMA overlay with 1.8 million ESALs. Section 47-6015 was selected for mechanistic analysis. It is 30 years old with an original 8.8 inch HMA layer and a 5.5 inch overlay. It has been subjected to 23.6 million ESALs and the overlay age is 19 years at the latest survey date. It has no fatigue, longitudinal or transverse cracking, only 3 mm of rutting and an IRI of 0.6 m/km.

Table B.1 Summary of sections with the longest overlay age within GPS-6A.

State	SHRP ID	Traffic Open Date	Overlay Const. Date	Overlay Thick. (Inch)	Overlay Age	Traffic (KESAL)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
19	6150	8/1/52	1965	0.4	34	236	0	3.95	102	10	1.830
48	1046	7/1/56	1971	10.1	32	14798	0	52.83	35	8	2.782
48	6179	6/1/65	1975	4.1	29	1806	1.20	0	18	11	1.702

Table B.2 Summary of sections subjected to high volume traffic within GPS-6A.

Experiment	State	SHRP ID	Initial Structure Thickness (Inch)	Overlay Thickness (Inch)	Overlay Age	Traffic (KESAL) per (Year)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
GPS – 6A	4	6053	3.2	4.2	16	1690	No Data	No Data	No Data	13	1.604
	4	6054	7.0	1.4	15	1085	14.3	20.6	23.0	9	1.316
	4	6055	1.8	3.8	No Data	814	No Data	No Data	No Data	5	0.765
	4	6060	3.9	3.4	13	943	0.0	0.0	0.0	55	0.502
	18	6012	14.8	4.0	15	2137	0.0	0.0	0.0	11	2.957
	19	6049	20.4	2.8	26	785	1.0	18.55	25.0	9	2.125
	41	6011	6.1	6.8	22	1547	0.0	0.0	0.0	4	1.183
	47	6015	8.8	5.5	19	986	0.0	0.0	0.0	3	0.588

Table B.3 Summary of good performing sections within GPS-6A.

Experiment	**State**	**SHRP ID**	**Age (Year)**	**Traffic (KESAL)**	**Original Thickness (Inch)**		**Overlay Thickness**		**Overlay Age**
GPS – 6A	30	7075	40	16618	3.40	Thin	3.70 *	3.00	19
								3.50	4
	48	6179	38	2084	1.40	Thin	4.10		29
	56	6029	29	892	1.90	Thin	2.70 *	1.60	21
								1.10	8
	1	6019	20	6874	8.30	Medium	5.00 *	2.80	13
								2.40	8
	47	6015	30	23647	8.80	Medium	5.50		19
	48	6086	29	2771	8.50	Medium	1.50		16
	49	1006	31	5556	9.20	Thick	1.30		15
	56	6031	26	521	11.10	Thick	4.60 *	2.30	14
								2.30	7
	56	6032	28	874	11.70	Thick	4.20 *	2.30	12
								1.00	9
								1.00	0

* Mill and Fill

GPS6B – AC Overlay with Conventional Asphalt Cement on AC Pavement

There are a total of 87 sections in 32 states within this experiment. Figure B.2 summarizes frequency of overlay age within the experiment. Table B.4, Table B.5, and Table B.6 summarize the relevant inventory and performance information on sections with the longest overlay ages, those with high ESAL, and those with the best performance in cracking and rutting.

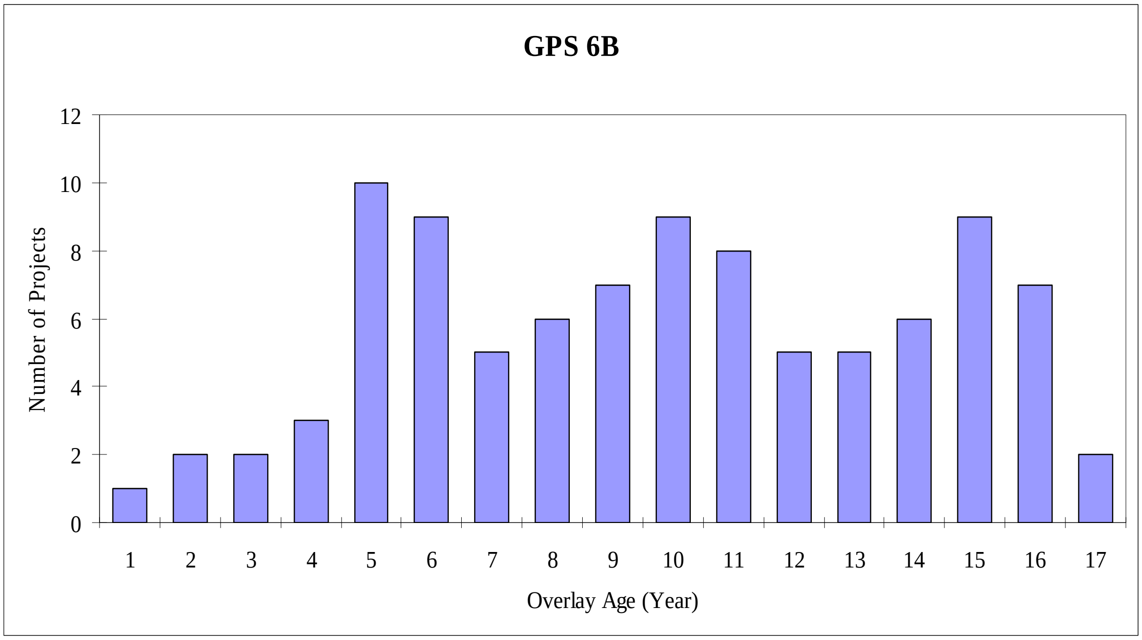


Figure B.2. Frequency of overlay age within experiment GPS-6B.

Summary Interpretation

None of the sections with longest overlay age are promising since they both have fatigue and transverse cracking after 17 years. Two sections subjected to heavy traffic are promising: 18-2008 and 47-3108. They have very little to no cracking, low rutting and IRI values after 11 and 16 years, with 1.275 and 0.861 million ESAL/year, respectively. Two good performing sections are promising: 6-8535 and 47-3108. They have very little to no cracking, low rutting and IRI values after 13 and 16 years, with 16.7 and 28.4 million ESAL, respectively. Section 47-3108 was selected for mechanistic analysis. It is 33 years old with an original 5.5 inch HMA layer and a 2.7 inch overlay. It has been subjected to 28.4 million ESAL and the overlay age is 16 years at the latest survey date. It has no longitudinal cracking, very little fatigue and transverse cracking, 6 mm of rutting and an IRI of 0.78 m/km.

Table B.4. Inventory information of sections with the longest overlay age within GPS-6B.

State	SHRP ID	Traffic Open Date	Overlay Const. Date	Overlay Thick. (Inch)	Overlay Age	Traffic (KESAL)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
47	3109	11/1/78	6/25/89	1.70	17	1808	0	2.30	14	6	1.203
47	3110	8/1/81	6/15/89	1.40	17	2251	0	5.79	71	4	0.758

Table B.5. Inventory information of sections subjected to high volume traffic within GPS-6B.

Experiment	State	SHRP ID	Initial Structure Thickness (Inch)	Overlay Thickness (Inch)	Overlay Age	Traffic (KESAL) per (Year)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
GPS – 6B	18	2008	12.9	2.5	11	1275	0.0	0.0	0.0	1	0.541
	36	1643	2.2	2.9	9	1017	0.0	37.12	19.0	4	1.078
	47	1023	5.3	1.7	11	859	0.0	0.93	20.0	9	1.679
	47	3108	5.5	2.7	16	861	0.0	0.07	2.0	6	0.782

Table B.6. Inventory information of good performing sections within GPS-6B.

Experiment	State	SHRP ID	Age (Year)	Traffic (KESAL)	Original Thickness (Inch)		Overlay Thickness		Overlay Age
GPS – 6B	2	1002	21	772	3.30	Thin	2.00		7
	2	1004	28	3900	3.60	Thin	1.80		14
	6	8534	35	8348	4.80	Thin	5.70		13
	30	7076	19	4412	4.50	Thin	2.40 *	2.40	10
								2.40	3
	30	7088	24	7957	4.60	Thin	2.40 *	2.40	4
								1.70	10
	30	8129	16	1544	3.00	Thin	3.80		1
	40	4086	34	5962	4.30	Thin	3.60		15
	40	4164	26	2455	4.60	Thin	1.00		10
	48	1130	33	1885	2.30	Thin	1.60		13
	56	2017	23	1432	2.40	Thin	1.20		6
	56	2019	19	2289	3.40	Thin	2.70		8
	56	7772	18	811	2.20	Thin	2.40		6
	6	8535	37	16686	6.60	Medium	5.30		13
	23	1028	32	5901	6.60	Medium	1.90		10
	42	1597	23	442	6.40	Medium	6.30		3
	42	1605	32	5165	8.10	Medium	2.70		8
	47	2001	27	9849	6.80	Medium	6.60		15
	47	3108	33	28429	5.50	Medium	2.70		16
	47	9024	29	933	5.10	Medium	1.30		11
	48	1096	25	3017	7.10	Medium	2.00		5
	48	1111	33	3094	6.90	Medium	2.70		6
	48	3835	12	2992	8.50	Medium	5.90		4
	18	1037	23	2046	14.40	Thick	4.30		11
	28	3094	24	6133	10.90	Thick	2.70		16

* Mill and Fill

GPS7A – Existing AC Overlay on PCC Pavement

There are a total number of 30 sections in 19 states within this experiment. Figure B.3 summarizes frequency of overlay age within the experiment. Table B.7, Table B.8 and Table B.9 summarize the relevant inventory and performance information on sections with the longest overlay ages, those with high ESAL, and those with the best performance in cracking and rutting.

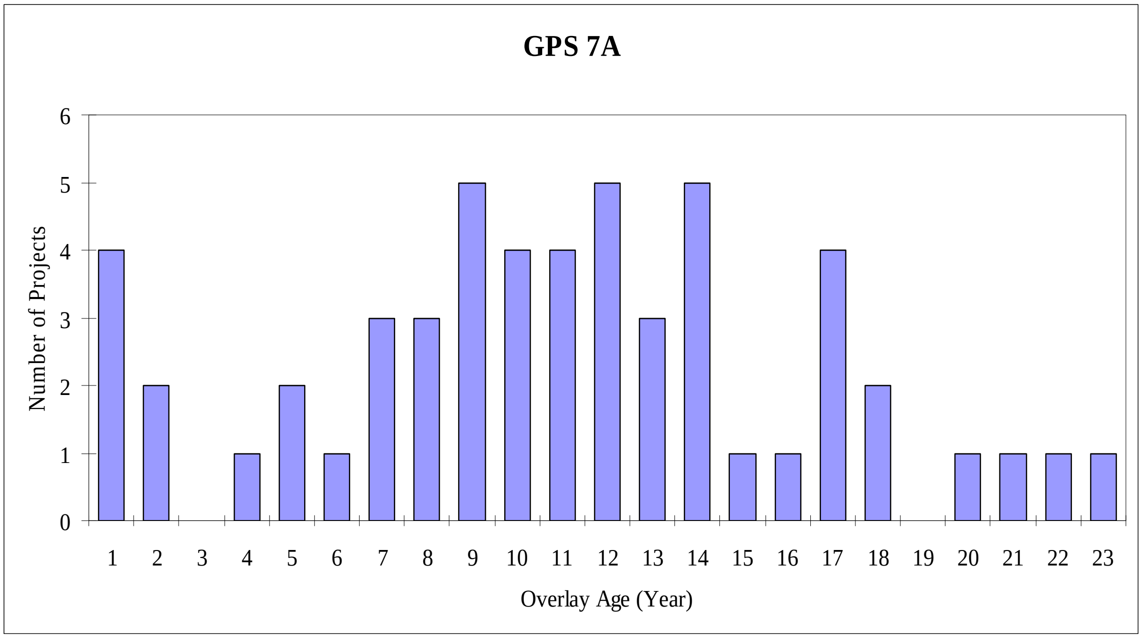


Figure B.3. Frequency of overlay age within experiment GPS-7A.

Summary Interpretation

None of the sections with longest overlay age are promising since they have a high level of transverse cracking after more than 20 years. One section subjected to heavy traffic may be promising: 13-7028. It has no longitudinal or fatigue cracking, but some transverse cracking. It has 7 mm of rutting and an IRI of 1.1 m/km after 12 years, with 16.8 million ESAL. Two good performing sections are promising: 13-7028 and 31-7005. They have very little to no cracking, low rutting and IRI values after 12 years, with 16.7 and 17.8 million ESAL, respectively. Section 13-7028 was selected for mechanistic analysis. It is 17 years old with an original 9 inch concrete slab and a 7 inch overlay. It has been subjected to 16.8 million ESAL and the overlay age is 12 years at the latest survey date. It has no longitudinal or fatigue cracking, but has some transverse cracking, 7 mm of rutting and an IRI of 1.1 m/km.

Table B.7. Inventory information of sections with the longest overlay age within GPS-7A.

State	SHRP ID	Original Pavement Type	Traffic Open Date	Overlay Const. Date	Overlay Thick. (Inch)	Overlay Age	Traffic (KESAL)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
29	7054	JRCP	6/1/57	1973	4.50	21	30246	No Data	No Data	No Data	7	1.011
29	7073	JPCP	6/1/64	1981	2.40	20	2314	0	0	70	3	1.501
41	7019	JRCP	6/1/47	1976	2.10	22	7970	0	0	32	17	1.785
46	7049	JPCP	12/1/54	1980	4.10	23	258	0	0	91	15	4.208

Table B.8. Inventory information of sections subjected to high volume traffic within GPS-7A.

Experiment	State	SHRP ID	Initial Structure Thickness (Inch)	Original Pavement Type	Overlay Thickness (Inch)	Overlay Age	Traffic (KESAL) per (Year)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
GPS – 7A	13	7028	9.1	JPCP	6.0	12	1366	0.0	0.0	27	7	1.118
	17	5453	8.4	CRCP	2.7	13	1059	33.5	1.51	67	3	1.231
	29	7054	10.1	JRCP	4.5	21	829	No Data	No Data	No Data	7	1.011
	39	7021	9.0	JRCP	2.6	14	1521	No Data	No Data	No Data	6	2.444
	41	7018	7.7	JRCP	1.6	14	771	0.0	0.0	9	18	1.542

Table B.9. Inventory information of good performing sections within GPS-7A.

Experiment	State	SHRP ID	Age (Year)	Traffic (KESAL)	Original Thickness (Inch)			Overlay Thickness		Overlay Age
GPS – 7A	44	7401	42	5129	8.20	Thin	JRCP	5.20 *	2.60	17
									3.20	2
	46	7049	48	283	7.40	Thin	JPCP	4.10		23
	13	7028	17	16763	9.10	Medium	JPCP	7.00 *	6.00	12
									2.50	5
	31	7005	44	17824	9.60	Medium	JPCP	5.30 *	4.50	12
									2.00	10
	31	7050	42	21857	9.00	Medium	JRCP	4.50 *	3.40	10
									1.50	8
									3.00	0
									1.40	1

* Mill and Fill

GPS7B – AC Overlay with Conventional Asphalt Cement on PCC Pavement

There are a total number of 43 sections in 18 states within this experiment. Figure B.4 summarizes frequency of overlay age within the experiment. Table B.10, Table B.11, and Table B.12 summarize the relevant inventory and performance information on sections with the longest overlay ages, those with high ESAL, and those with the best performance in cracking and rutting.

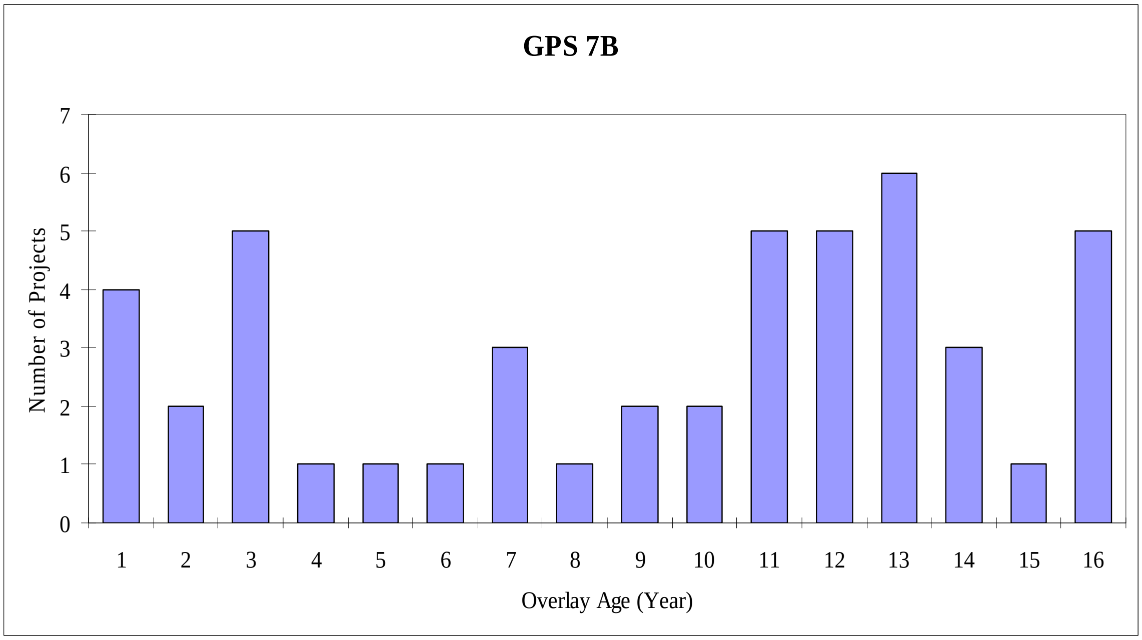


Figure B.4. Frequency of overlay age within experiment GPS-7B.

Summary Interpretation

One section with longest overlay age may be promising: 42-1617. It has a 4.7 inch AC overlay over a CRCP pavement. It has no longitudinal or transverse cracking and very little fatigue cracking and an IRI of 0.93 m/km, but has 9 mm rutting and moderate traffic of 20 million ESAL. Two sections subjected to very heavy traffic (4.5 to 5.2 million ESAL per year) are promising: 18-5022 and 18-5518. They have 4 and 4.8 inch AC overlay over 9.8 and 9.3 inch CRCP pavements, respectively. They have no or very little cracking and less than 0.25 inch rutting and an IRI of about 1 m/km. Out of the three good performing sections, only one is promising since the other two have moderate traffic. Section18-5022 is a 34 year old CRCP pavement with a 4 inch AC overlay that is 13 years old. It has been subjected to very heavy traffic of more than 177 million ESAL. It has been selected for further mechanistic analysis.

Table B.10. Inventory information of sections with the longest overlay age within GPS-7B.

State	SHRP ID	Original Pavement Type	Traffic Open Date	Overlay Const. Date	Overlay Thick. (Inch)	Overlay Age	Traffic (KESAL)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
19	9126	JRCP	1/1/65	6/16/89	5.20	15	28144	1.60	1.27	32	4	1.793
29	5473	JRCP	10/1/60	5/27/89	1.80	15	40541	0	0	19	4	1.148
39	5010	CRCP	7/1/75	6/1/90	2.80	15	7632	0	8.95	20	6	1.063
42	1613	JRCP	6/1/90	6/4/90	3.70	15	14123	0	0.50	17	4	0.952
42	1614	JRCP	6/1/95	7/1/89	4.40	16	7569	2.50	2.12	16	12	2.190
42	1617	CRCP	6/1/72	8/13/90	4.70	15	20308	0	0.95	0	9	0.934

Table B.11. Inventory information of sections subjected to high volume traffic within GPS-7B.

Experiment	State	SHRP ID	Initial Structure Thickness (Inch)	Original Pavement Type	Overlay Thickness (Inch)	Overlay Age	Traffic (KESAL) per (Year)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
GPS – 7B	9	5001	8.2	CRCP	4.7	8	946	0.0	8.2	9.0	9	1.311
	18	3003	10.2	JPCP	4.5	11	1439	0.0	0.0	26.0	4	1.327
	18	5022	9.8	CRCP	4.0	13	5170	0.0	0.0	0.0	6	1.011
	18	5518	9.3	CRCP	4.80	11	4495	0.0	0.23	5.0	7	0.996
	29	5473	7.9	JRCP	1.8	15	1107	0.0	0.0	19.0	4	1.148
	42	1613	10.2	JRCP	3.7	15	1027	0.0	0.5	17.0	4	0.952
	54	4004	9.9	JRCP	5.7	7	1263	0.0	0.0	12.0	3	1.301

Table B.12. Inventory information of good performing sections within GPS-7B.

Experiment	State	SHRP ID	Age (Year)	Traffic (KESAL)	Original Thickness (Inch)			Overlay Thickness	Overlay Age
GPS – 7B	39	3013	35	3720	8.30	Thin	JRCP	3.70	11
	18	5022	34	176836	9.80	Medium	CRCP	4.00	13
	29	5483	32	6746	9.00	Medium	JRCP	3.00	13

SPS5 – AC Overlay of AC Pavement

There are a total number of 29 sections in 16 states within this experiment. Figure B.5 summarizes frequency of overlay age within the experiment. Table B.13 summarizes the relevant inventory and performance information on the sections with the longest overlay ages. None of the sections are promising for long-life.

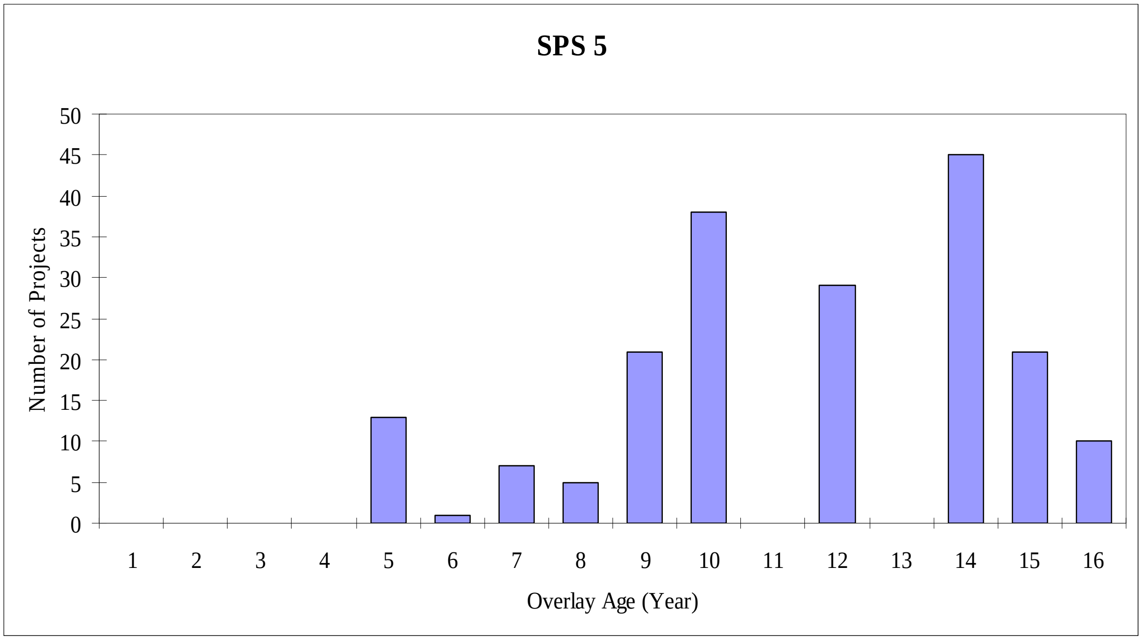


Figure B.5. Frequency of overlay age within experiment SPS-5.

Table B.13. Information of sections with the longest overlay age within SPS-5.

State	SHRP ID	Assign Date	Overlay Const. Date	Overlay Thick. (Inch)	Overlay Age	Traffic (KESAL)	Long. Cracking (meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
4	502	1/1/87	4/20/90	2.70	16	6386	0	54.72	0	11	3.663
4	503	1/1/87	4/20/90	4.70	16	6386	91.30	1.74	70	7	1.916
4	504	1/1/87	4/20/90	4.80	16	6386	0	0.04	17	3	1.495
4	505	1/1/87	4/20/90	2.80	16	6386	1.60	57.39	96	6	1.890
4	506	1/1/87	4/20/90	5.20	16	6386	1.70	0.43	17	4	1.538
4	507	1/1/87	4/20/90	6.80	16	6386	0	0	6	7	1.441
4	508	1/1/87	4/20/90	6.50	16	6386	13.70	0	55	7	1.272
4	509	1/1/87	4/20/90	3.90	16	6386	62.50	8.20	96	9	3.671
4	559	1/1/87	4/20/90	6.00	16	6386	1.90	0	51	4	1.505
4	560	1/1/87	4/20/90	2.20	16	6386	7.40	32.58	58	3	1.859

SPS6 – Rehabilitation of Jointed PCC Pavement

There are a total number of 30 sections in 13 states within this experiment. Figure B.6 summarizes frequency of overlay age within the experiment. Table B.14 summarizes the relevant inventory and performance information on the sections with the longest overlay ages. The best performing section (17-663) is 16 years old with an 8 inch AC overlay over a JRCP pavement. It has no longitudinal or transverse cracking, only 2 mm of rutting, but about 5 percent fatigue cracking. Therefore, it does not promise to be a long-life pavement.

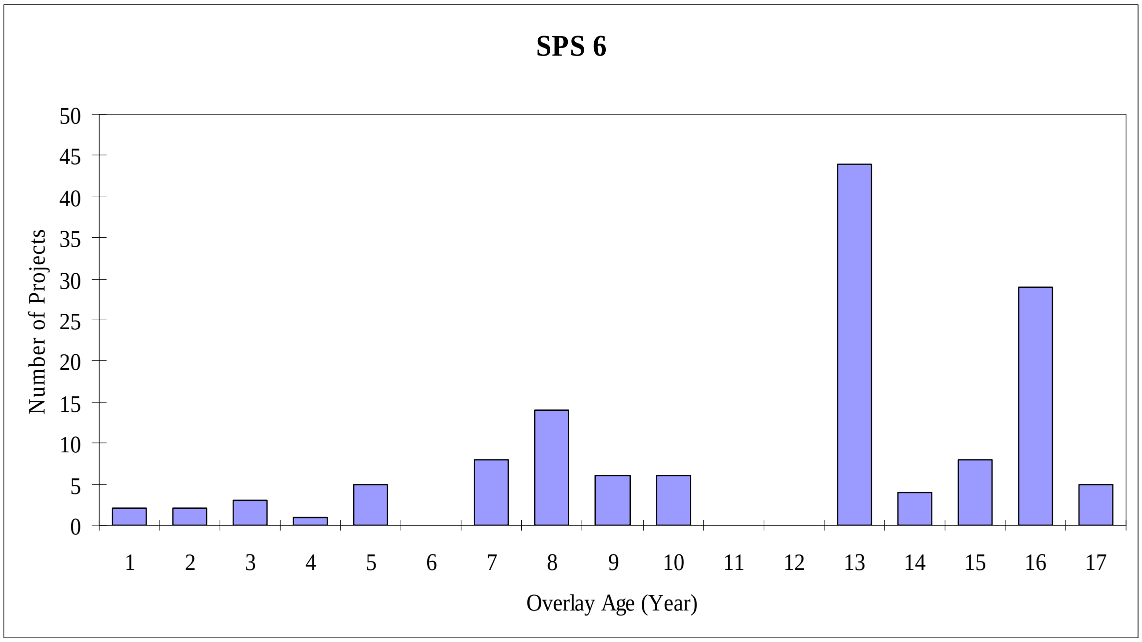


Figure B.6. Frequency of overlay age within experiment SPS-6.

Rubblized Sections in the SPS-6 Experiment

The Strategic Highway Research Program (SHRP), during the planning of the Long Term Pavement Performance (LTPP) experiments, recognized an increasing interest in rubblizing PCC slabs to reduce the occurrence of reflection cracks in HMA overlays. This repair strategy was included in the LTPP Special Pavement Study (SPS) experiment defined as SPS-6. However, only a few of these SPS-6 projects actually included the rubblization process. Those projects with rubblization test sections included Alabama, Arizona, Illinois, Michigan, Missouri, Oklahoma, and Pennsylvania, which are listed in Table B.15. Some of the rubblized test sections had construction related problems – soft foundations and non-uniform particle size distribution throughout the PCC slab thickness.

Table B.14. Information of sections with the longest overlay age within SPS-6.

State	SHRP ID	Original Pavement Type	Assign Date	Overlay Const. Date	Overlay Thick. (Inch)	Overlay Age	Traffic (KESAL)	Long. Cracking (Meter)	Fatigue Cracking (%)	No. of Trans. Cracking	Rut Depth (mm)	IRI (m/Km)
17	603	JRCP	1/1/87	5/24/90	3.70	16	6747	0	14.78	0	2	1.569
17	659	JRCP	1/1/87	6/1/90	3.30	16	6747	0	12.25	15	2	1.885
17	662	JRCP	1/1/87	6/1/90	3.50	16	6747	0	27.04	0	2	1.844
17	663	JRCP	1/1/87	6/1/90	8.0	16	6747	0	4.65	0	2	1.128
17	664	JRCP	1/1/87	6/1/90	6.0	16	6747	0	21.89	0	3	1.369

Table B.15. LTPP SPS-6 Projects with rubblized test sections (Von Quintus et al., 2007).

Project – Agency	Rehabilitation Date	Test Section Identification	HMA Overlay Thickness, mm.	Comment
Alabama	6-98	0661	102	Badger Breaker Machine (Model MHB); particles down to 3 inches in size.
		0662	203	
		0663	241	
Arizona	10-90	0616	140	
		0619	140	
Illinois	6-90	0663	152	High frequency breaking unit; less than 6 inches in size; edge drains placed.
		0664	206	
Michigan	5-90	0659	178	
Missouri	8-92	0661	290	Edge drains placed.
		0662	185	
		0663	292	No edge drains placed.
		0664	175	
Oklahoma	8-92	0607	114	Resonant Frequency Breaker; surface – 2 to 3 inches in size; bottom – up to 8 inches in size; edge drains placed.
		0608	201	
Pennsylvania	9-92	0660	241	Edge drains placed.
		0661	330	

The 2005 LTPP database was reviewed by Von Quintus et al. (2007) to determine the current performance trends of these sections. The load related cracking is still considered minimal and the IRI values are low. In general, the thicker the overlay – the lower amount of cracking, with the exception for Longitudinal Cracking Outside the Wheel Path. The predominant distress exhibited along these test sections is longitudinal cracking outside the wheel path area. The sections without edge drains or those with rubblized pieces less than 2 inches in size have the higher levels of cracking.

PCC RENEWAL PROJECTS

GPS 9 – Unbonded Concrete Overlays

The Long Term Pavement Performance (LTPP) General Pavement Study experiment GPS-9 includes unbonded JPCP, JRCP, or CRCP overlays with a thickness of 125 mm (5 inches) or more placed over an existing JPCP, JRCP or CRCP. An interlayer used to prevent bonding of the two slabs was required. The overlaid concrete pavement may rest on a base or subbase or directly on the subgrade.

Information about GPS-9 experiment (Unbonded PCC Overlay on PCC Pavement) was extracted from the LTPP DataPave Online (Release 21.0). There were 26 sections located in 14 states within this experiment. The continuously reinforced concrete pavement overlays are presented separately from jointed pavements (JRCP and JPCP) since the performance criteria are not entirely identical. Furthermore, JRCP overlays were not considered since state DOTs do not use JRC pavement systems anymore. After the removal of JRC overlays, 14 JPCP and 4 CRCP overlays were considered in the subsequent sections of this chapter.

Summary of Inventory Information

The original construction date for the GPS-9 sections ranged from the early 1950s to the mid-1970s. The location of the various LTPP test sections is shown in Figure B.7.

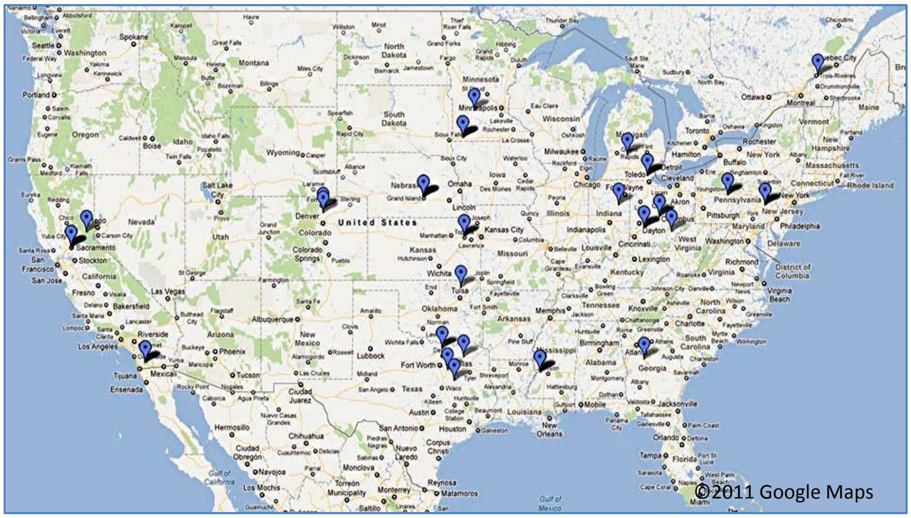


Figure B.7. Locations of GPS-9 sections (Google Maps).

Table B. 16 summarizes the maintenance events during the life of the overlay since being part of the LTPP experiment. The blank cells under the construction change reason indicate no maintenance.

The overlay thicknesses of the various test sections range from 5ft 8in. to 10ft 5in. The distribution of unbonded overlay thickness is shown in Figure B.8. Unbonded concrete overlay projects are typically 4ft 11in. thick, depending upon the level of traffic. Figure B.9 shows the distribution of the traffic (in ESALs) carried by the various test sections until the de-assign date.

Table B. 16. Construction events.

Section ID	Const. No.	LTPP Assign Date	Construction Change Reason
6-9048	1	7/1/1988	
	2	4/18/2001	PCC Slab Replacement
6-9049	1	6/26/1988	
6-9107	1	7/1/1988	
8-9019	1	7/20/1988	
8-9020	1	7/20/1988	
13-4118	1	1/1/1987	
18-9020	1	1/1/1987	
20-9037	1	1/1/1987	
	2	9/15/1992	Full Depth Patching of PCC Pavement Other Than at Joint
27-9075	1	1/1/1987	
28-7012	1	1/1/1987	
	2	3/15/1993	Asphalt Concrete Overlay, Portland Cement Concrete Overlay
31-6701	1	8/1/1988	
	2	1/15/2000	Crack Sealing, Lane-Shoulder Longitudinal Joint Sealing
	3	2/28/2002	Crack Sealing, Transverse Joint Sealing
	4	3/9/2004	Partial Depth Patching of PCC Pavement Other Than at Joint
	5	2/15/2005	Partial Depth Patching of PCC Pavement Other Than at Joint
40-4155	1	1/1/1987	
42-1627	1	12/1/1988	
48-3569	1	1/1/1987	
48-3845	1	7/1/1989	
48-9167	1	12/31/1987	
48-9355	1	12/31/1988	
89-9018	1	7/1/1988	

Notes: Section ID is the unique number given to each test section in the LTPP program. Const. No. is the construction number. Each test section began in the LTPP program at construction number 1. Sequential numbers are added any time maintenance or rehabilitation activities occur on the test section. LTPP Assign Date represents the date that the test section entered the LTPP program (construction number 1) or the date maintenance and rehabilitation was completed on the test section (construction numbers greater than 1).

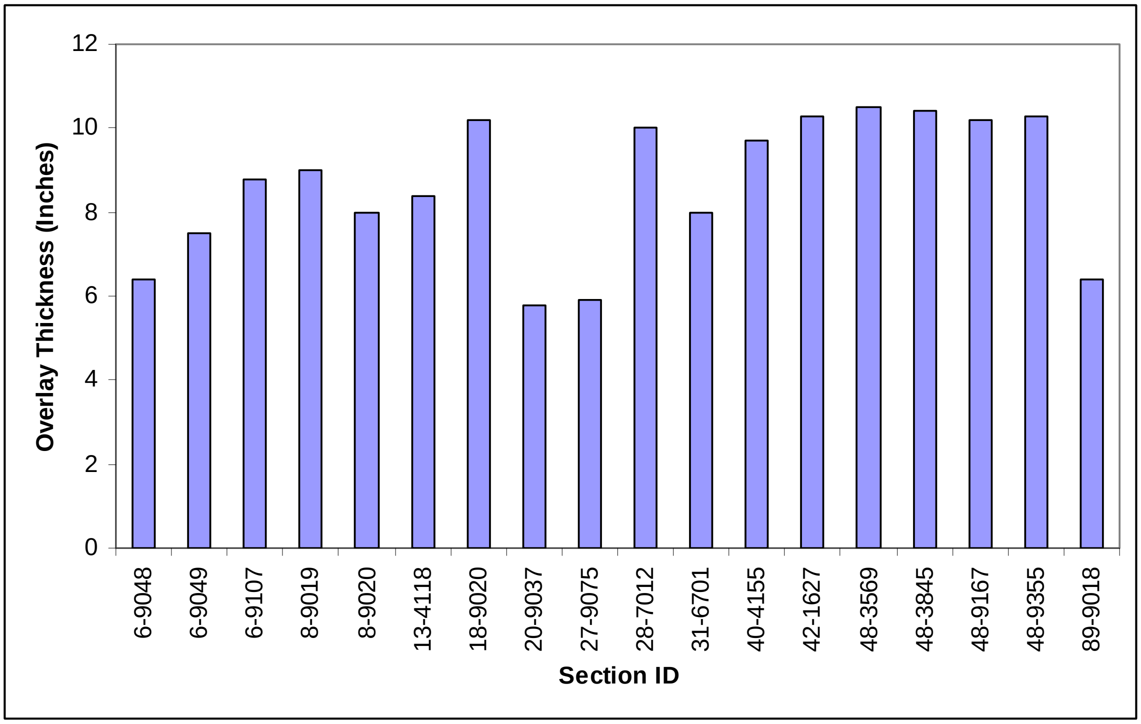


Figure B.8. Overlay thicknesses.

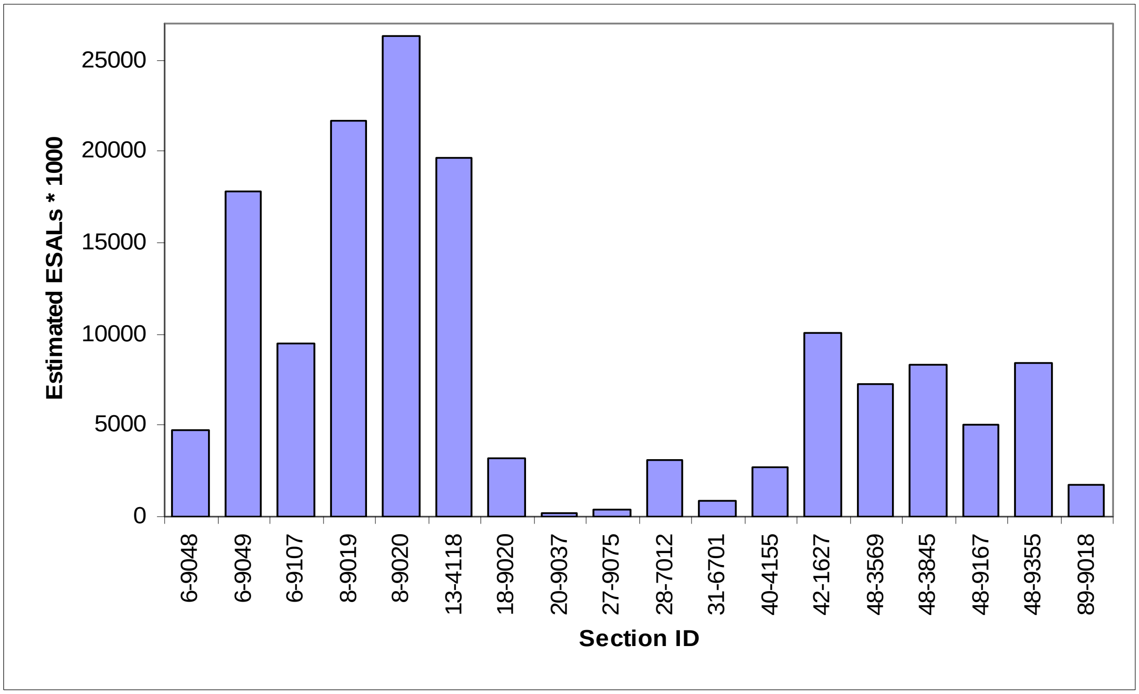


Figure B.9. Traffic after inclusion in LTPP program.

The distribution of separator layer types commonly used in the LTPP test sections is shown in Figure B.10. Typically, a fine graded asphalt surface mixture is used for the separator layer. The thickness of the separator layer is a function of (1) the condition of the existing pavement; and (2) the type of pre overlay repairs.

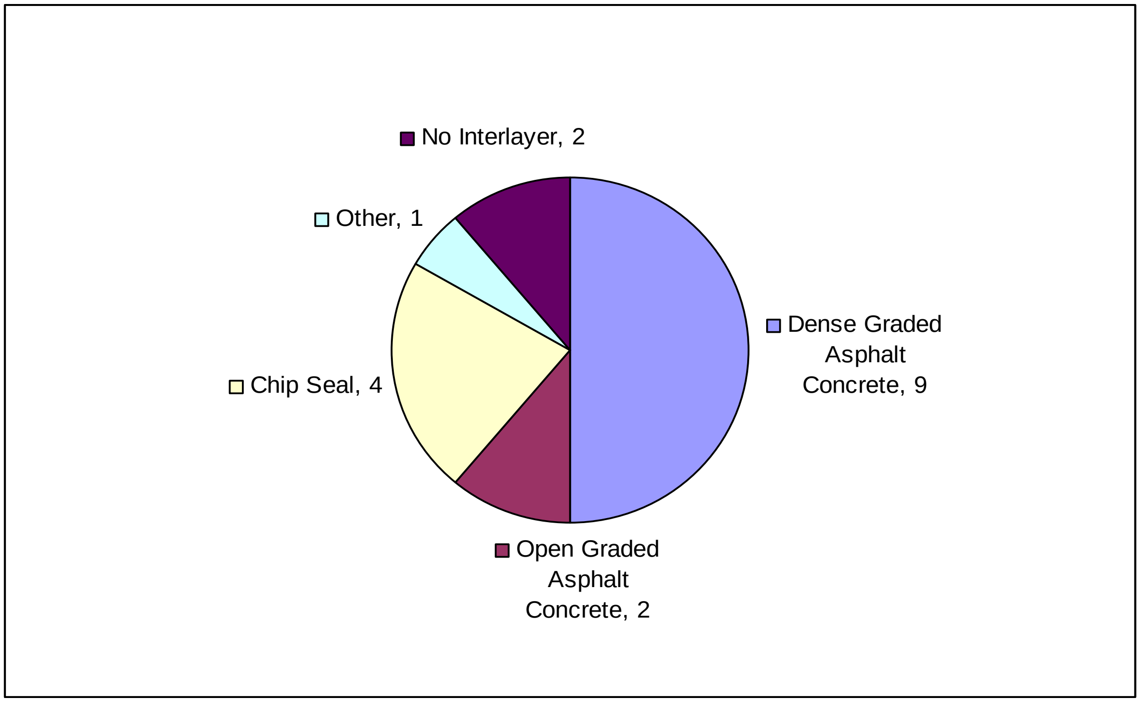


Figure B.10. Types of interlayer.

Figure B.11 shows the distribution of the thickness of the HMA separator layers used in the various sections. According to the review of the literature a minimum thickness of 1 inch is recommended for HMA separator layers. Thinner layers erode easily near joints and do not provide adequate isolation of the overlay from underlying PCC pavement.

Figure B.12 shows a couple of cross sections with thick asphalt interlayer. Section 18-9020 is actually consisted of two interlayers while section 48-9167 has one interlayer.

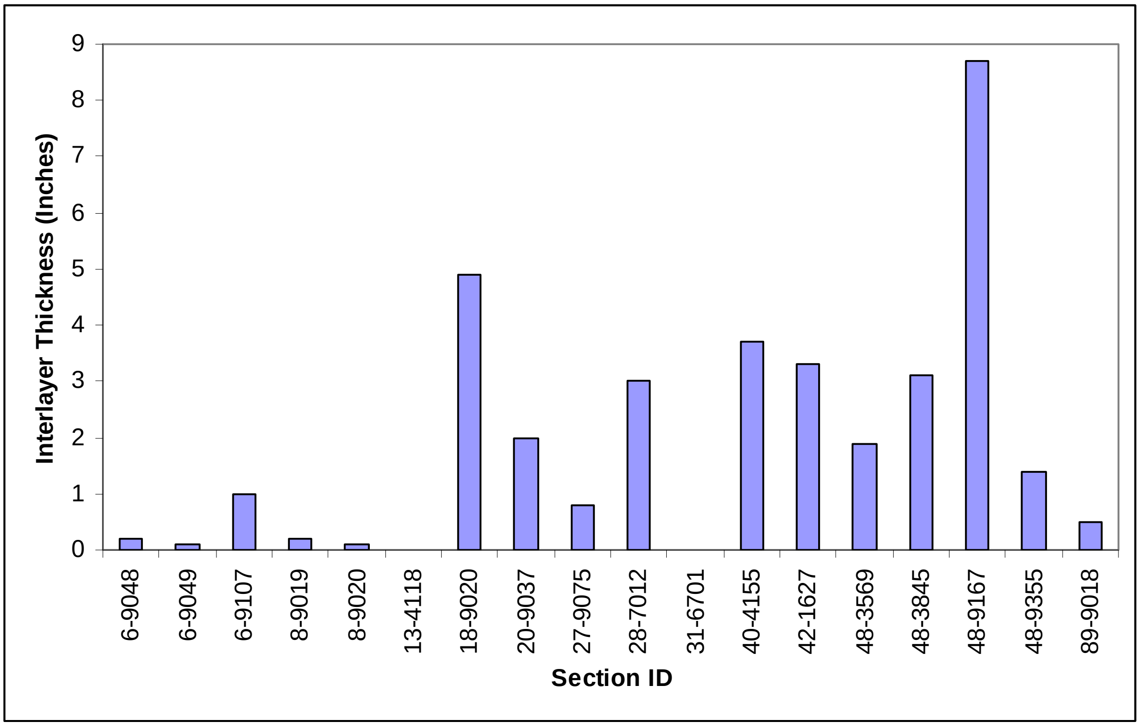
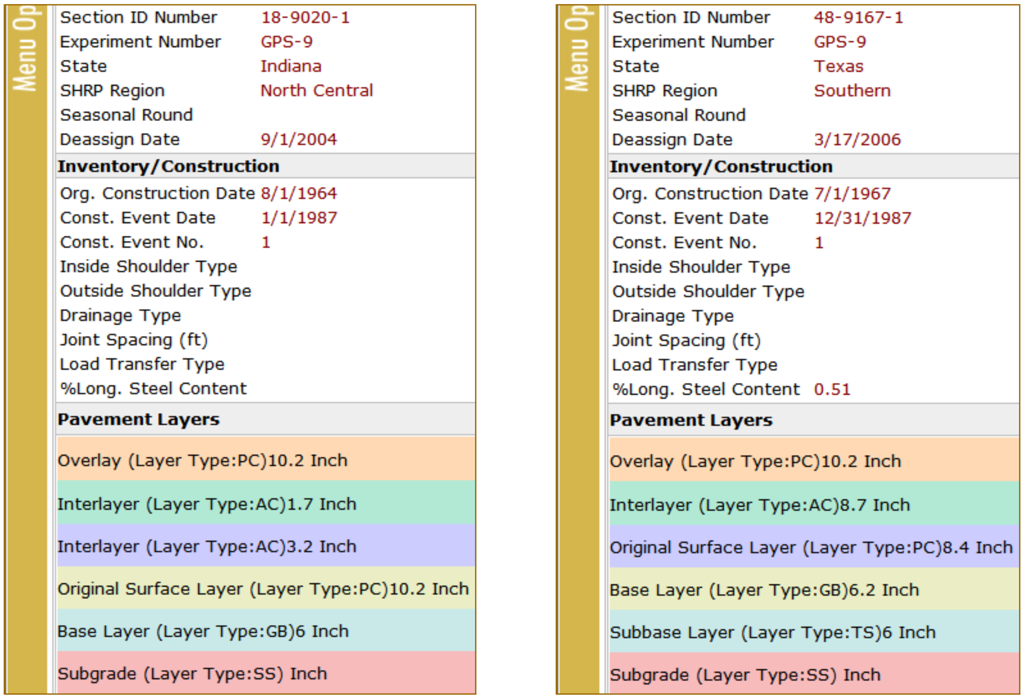


Figure B.11. Interlayer thicknesses.



Section ID Number	18-9020-1
Experiment Number	GPS-9
State	Indiana
SHRP Region	North Central
Seasonal Round	
Deassign Date	9/1/2004
Inventory/Construction	
Org. Construction Date	8/1/1964
Const. Event Date	1/1/1987
Const. Event No.	1
Inside Shoulder Type	
Outside Shoulder Type	
Drainage Type	
Joint Spacing (ft)	
Load Transfer Type	
%Long. Steel Content	
Pavement Layers	

Overlay (Layer Type:PC)10.2 Inch
Interlayer (Layer Type:AC)1.7 Inch
Interlayer (Layer Type:AC)3.2 Inch
Original Surface Layer (Layer Type:PC)10.2 Inch
Base Layer (Layer Type:GB)6 Inch
Subgrade (Layer Type:SS) Inch

Section ID Number	48-9167-1
Experiment Number	GPS-9
State	Texas
SHRP Region	Southern
Seasonal Round	
Deassign Date	3/17/2006
Inventory/Construction	
Org. Construction Date	7/1/1967
Const. Event Date	12/31/1987
Const. Event No.	1
Inside Shoulder Type	
Outside Shoulder Type	
Drainage Type	
Joint Spacing (ft)	
Load Transfer Type	
%Long. Steel Content	0.51
Pavement Layers	

Overlay (Layer Type:PC)10.2 Inch
Interlayer (Layer Type:AC)8.7 Inch
Original Surface Layer (Layer Type:PC)8.4 Inch
Base Layer (Layer Type:GB)6.2 Inch
Subbase Layer (Layer Type:TS)6 Inch
Subgrade (Layer Type:SS) Inch

Figure B.12. Cross sections with thick interlayer.

Figure B.13 shows the distribution of transverse joint spacing. Based on the review of the literature, it is recommended to limit the joint spacing to 21 times the slab thickness. According to that rule of thumb, the transverse joint spacing of the GPS 9 sections should range between 10ft to 18ft (as shown by the dashed horizontal lines in Figure B-7). In general, the risk of premature cracking on unbonded PCC overlays can be minimized by limiting the joint spacing to 15ft, even for very thick overlays.

Figure B.14 shows the distribution of various load transfer mechanisms used across transverse joints in the GPS 9 test sections. Joint performance in unbonded concrete overlays is enhanced due to the presence of the underlying pavement as “sleeper” slabs. The load transfer across joints from the underlying slab can be maximized by mismatching joints. However, the use of doweled joints is highly recommended for overlays subjected to heavy truck traffic to avoid corner breaks and minimize joint faulting.

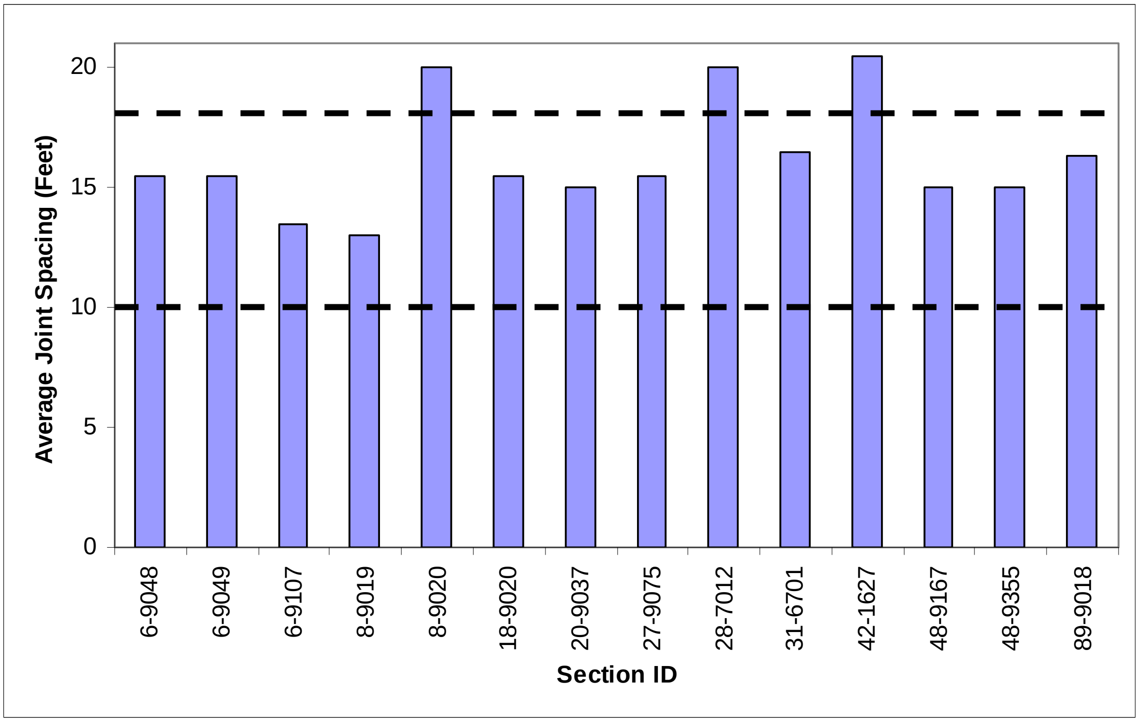


Figure B.13. JPCP average joint spacing.

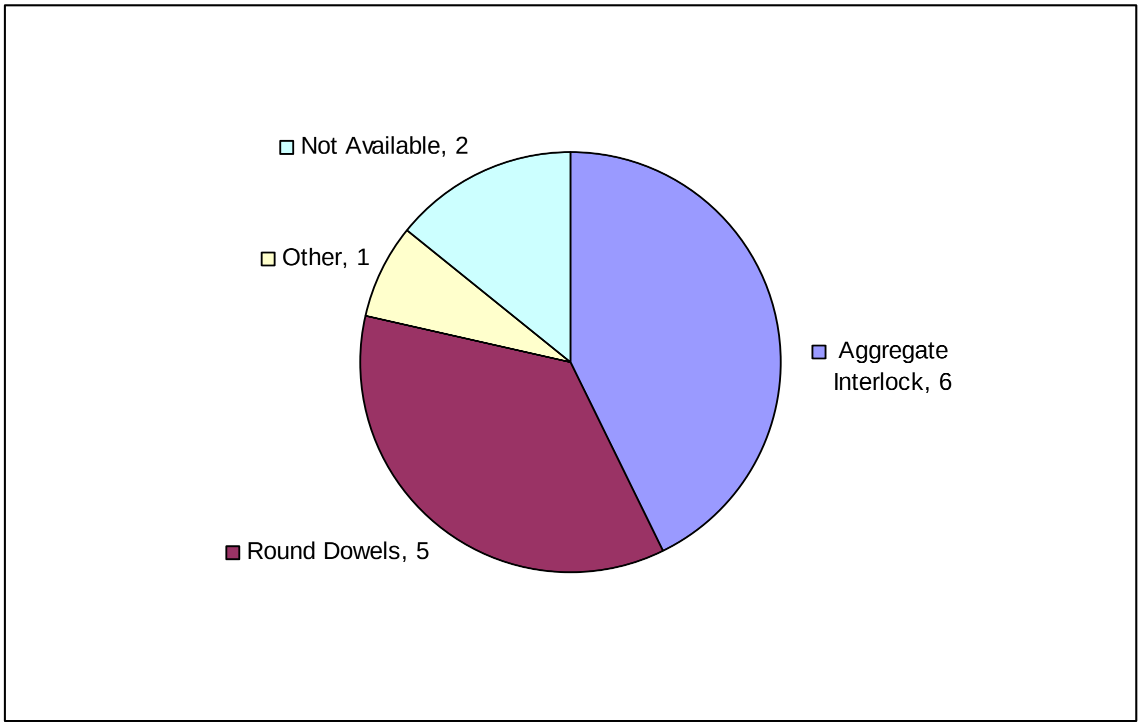


Figure B.14. JPCP load transfer mechanisms.

Summary of Overall Field Performance

The pavement performance criteria selected for the summary include transverse cracking, IRI (and PSI), joint and crack faulting magnitude (JPCP), and punch-outs (only for CRCP). The performance trends presented in this section are based on measurements documented in the latest year.

It should be noted that some of the figures in subsequent sections show the nominal performance of the sections that might include confounding (interaction) effects of two or more factors.

Transverse Cracking. The box and whisker plot shown in Figure B.15 (for jointed concrete overlays) and B-20 (for CRC overlays) shows the distribution of percent cracking for the test sections. The box-and-whisker plots presented here display data as follows: the median is represented by the horizontal line inside the box. The top and bottom of the box represent the third quartile (75th percentile) and the first quartile (25th percentile), respectively. The distance between these two is the interquartile range (IQR). In these plots, whiskers are drawn to the minimum and maximum observations.

Figure B.16 shows the magnitude of cracking as a function of overlay thickness for the jointed concrete pavements. Sections 6-9048 and 20-9037 with 28 and 14 cracks respectively, are among the sections presented in the first category (5.1”–6.5”). Sections 6-9049 and 31-6701 with 26 and 7 cracks respectively, are among the sections represented in the second category (6.6”-8.0”). As expected, the thicker overlays (>9”) are exhibiting fewer transverse cracks. It is worth noting that 11 of the 14 jointed concrete pavement overlays have exhibited little or no cracking in 18 years of service. These test sections exhibit the promise of long life performance. Figure B.18 and Figure B.19 show the percent cracking as a function overlay thickness for the CRC pavements, and average crack spacing as a function of overlay thickness, respectively. Figure B.20 and Figure B.21 summarize the number of punch-outs in the CRC overlays.

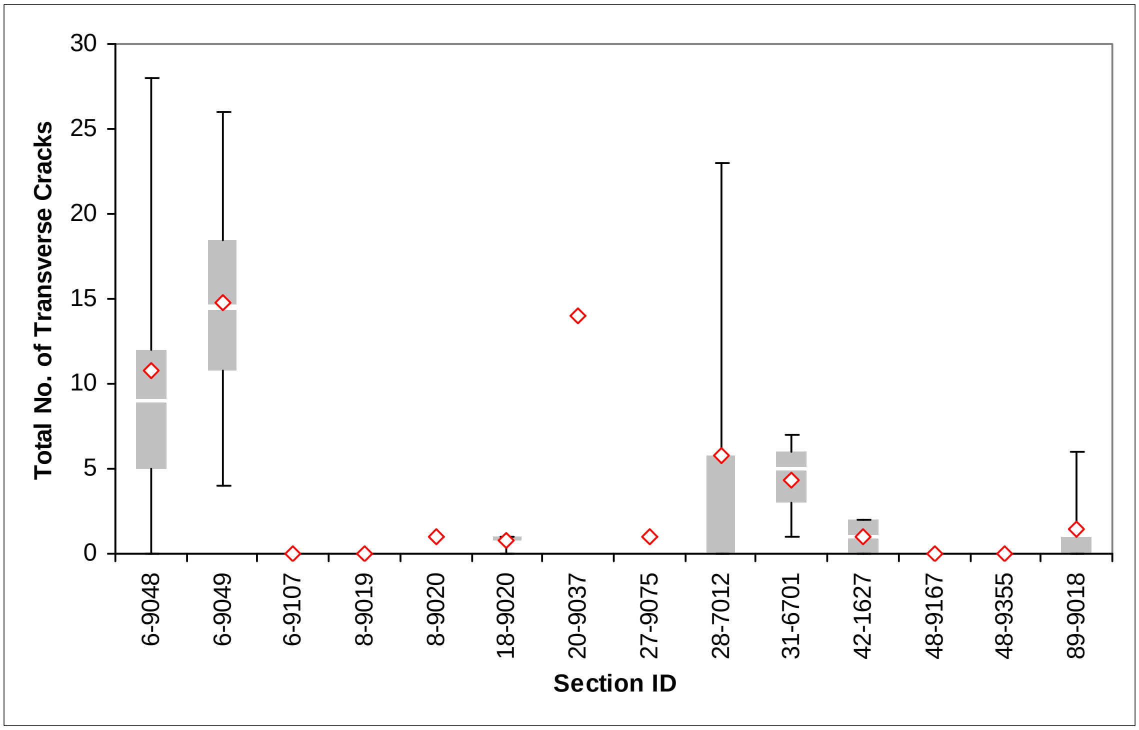


Figure B.15. Distribution of number of transverse cracks for JPCP sections.

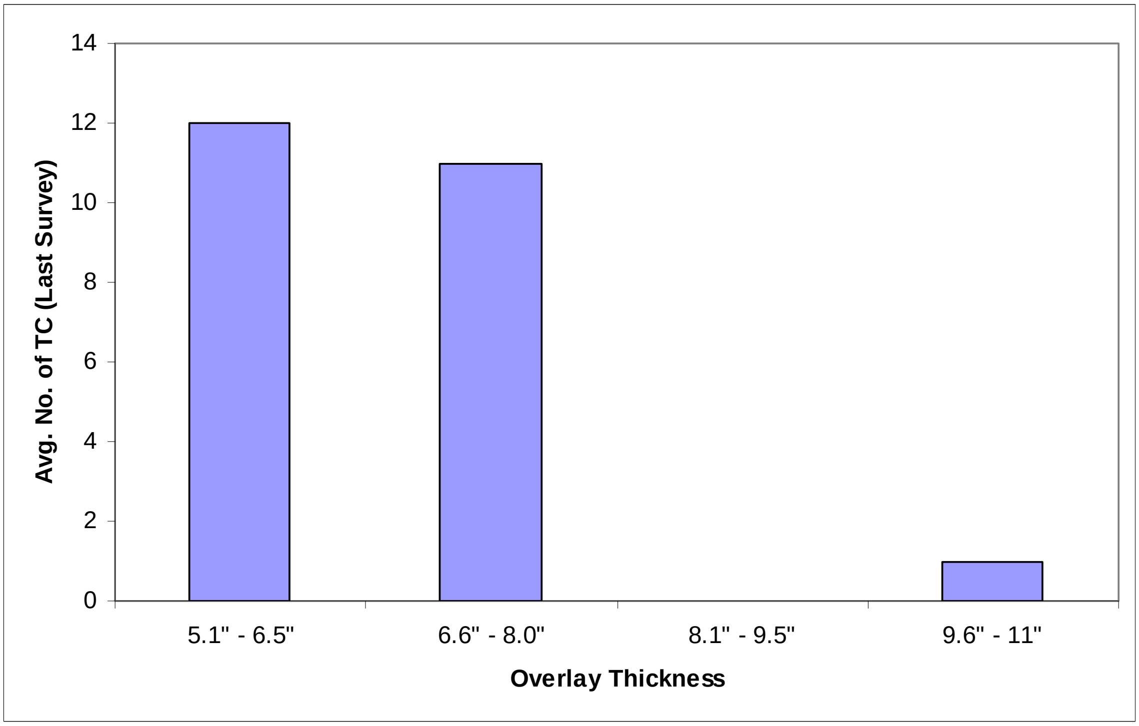


Figure B.16. JPCP Overlay thickness versus average number of transverse cracks.

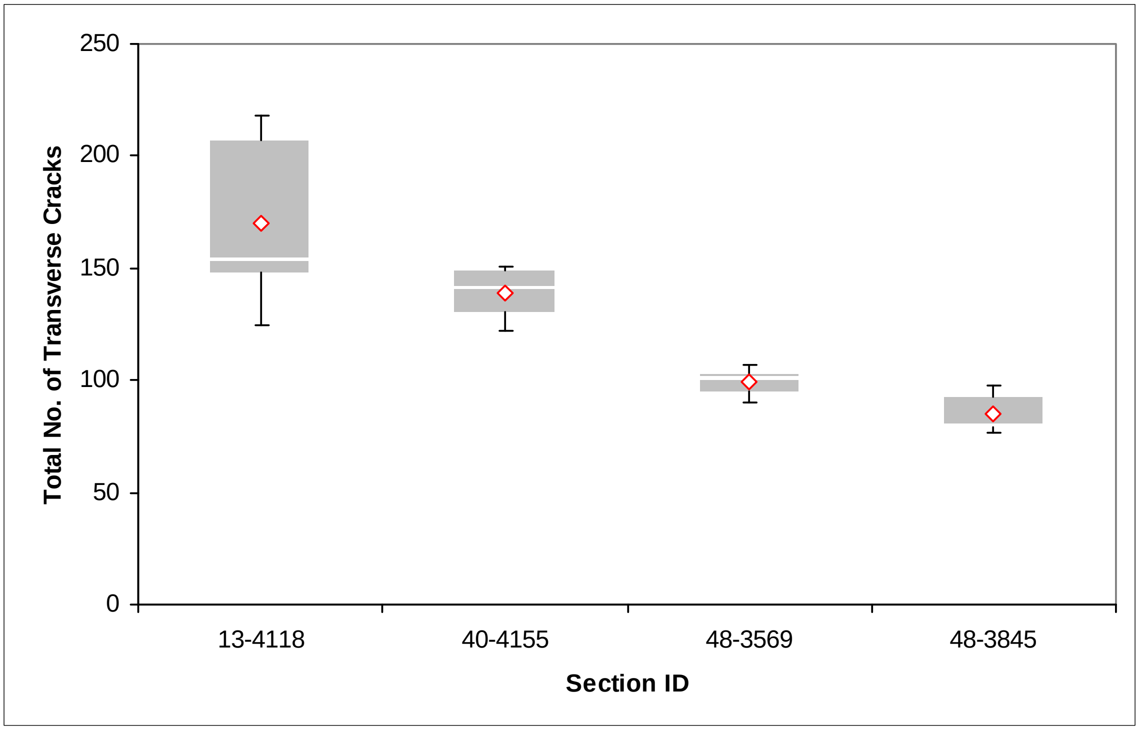


Figure B.17. Distribution of number of transverse cracks for CRCP sections.

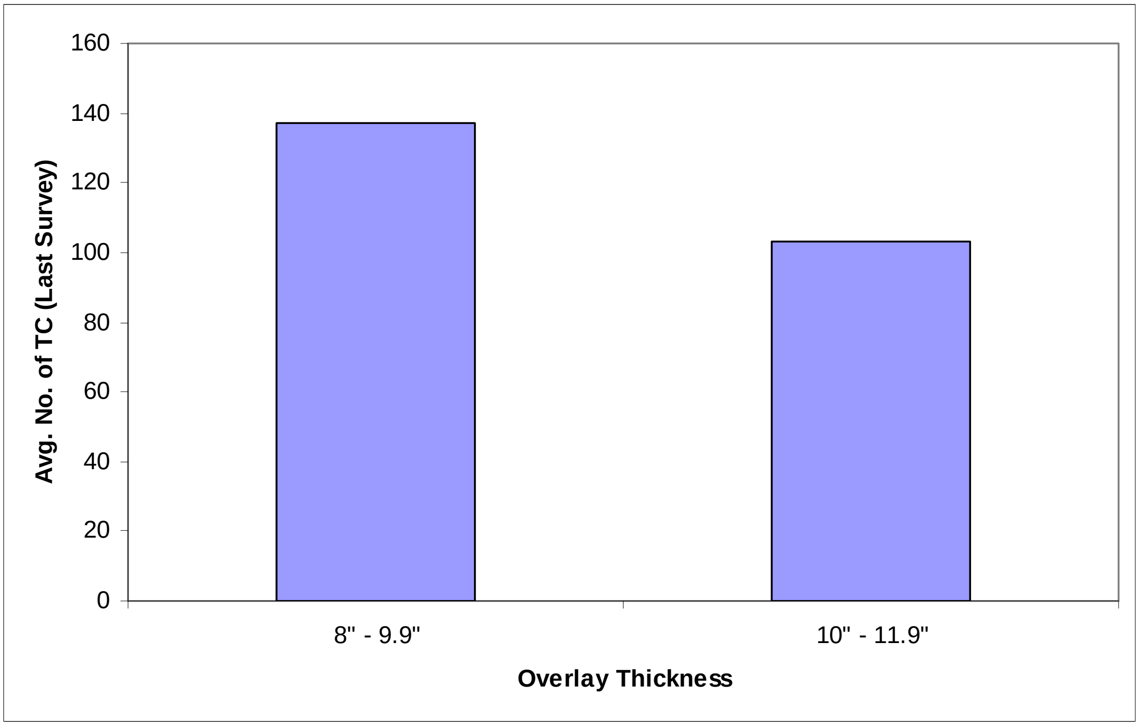


Figure B.18. CRCP Overlay thickness versus average number of transverse cracks.

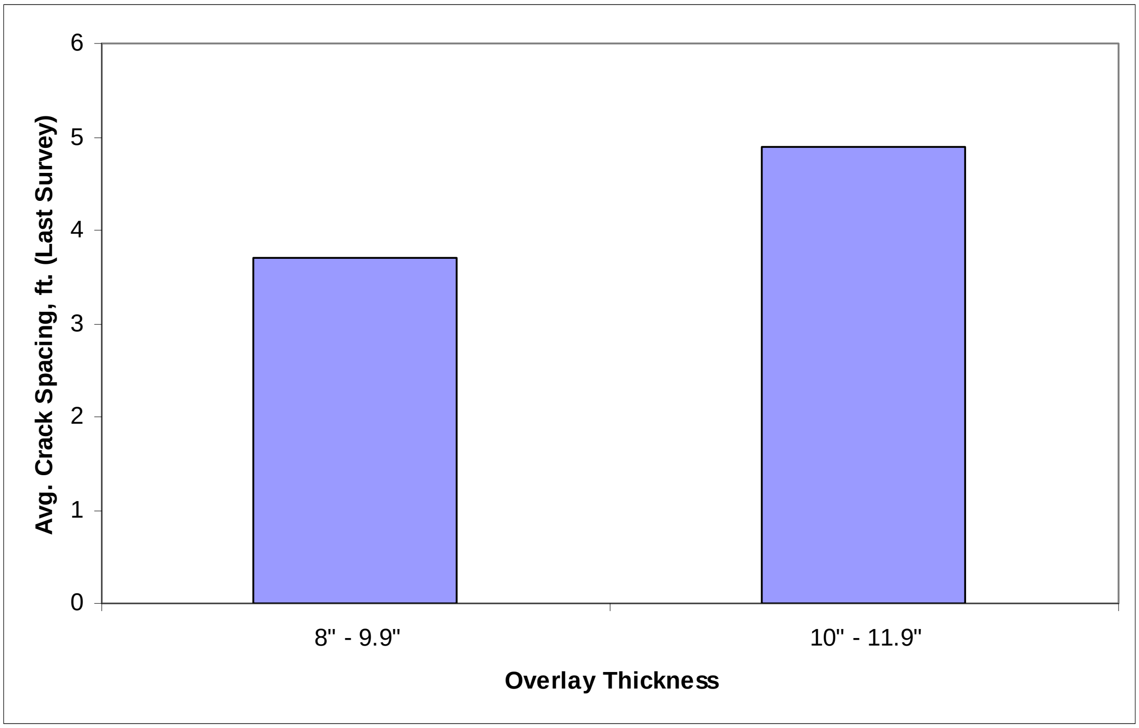


Figure B.19. CRCP overlay thickness versus average crack spacing.

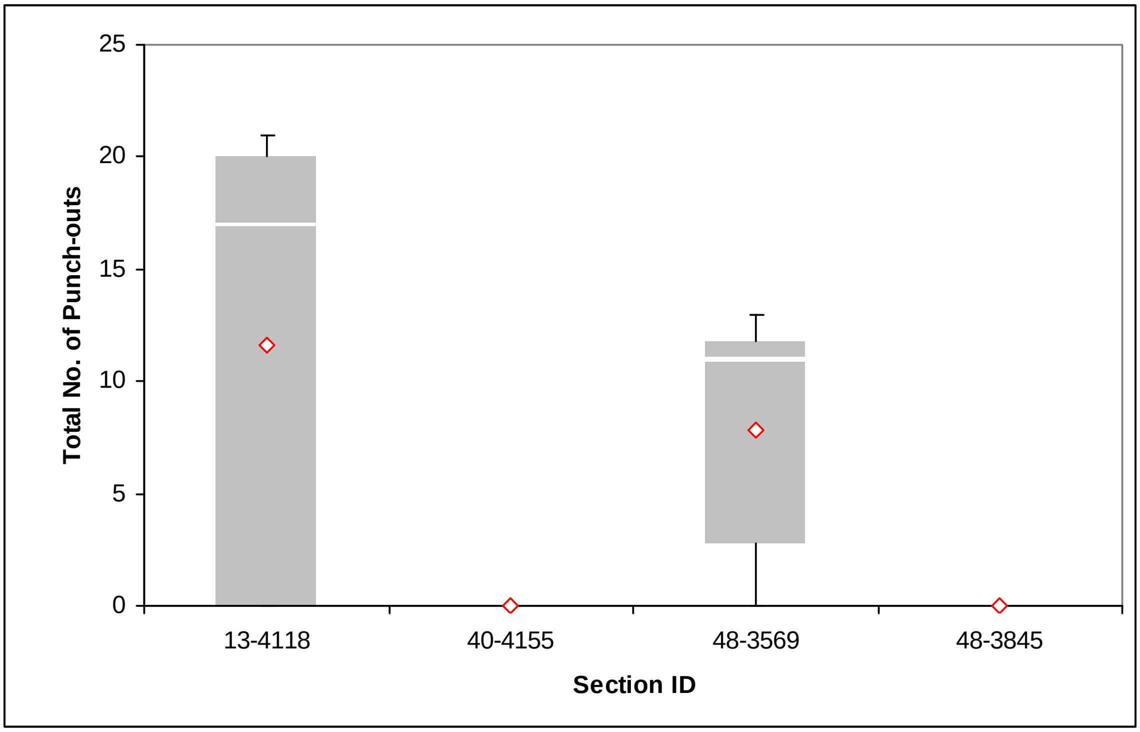


Figure B.20. Distribution of number of punch-outs for CRCP sections.

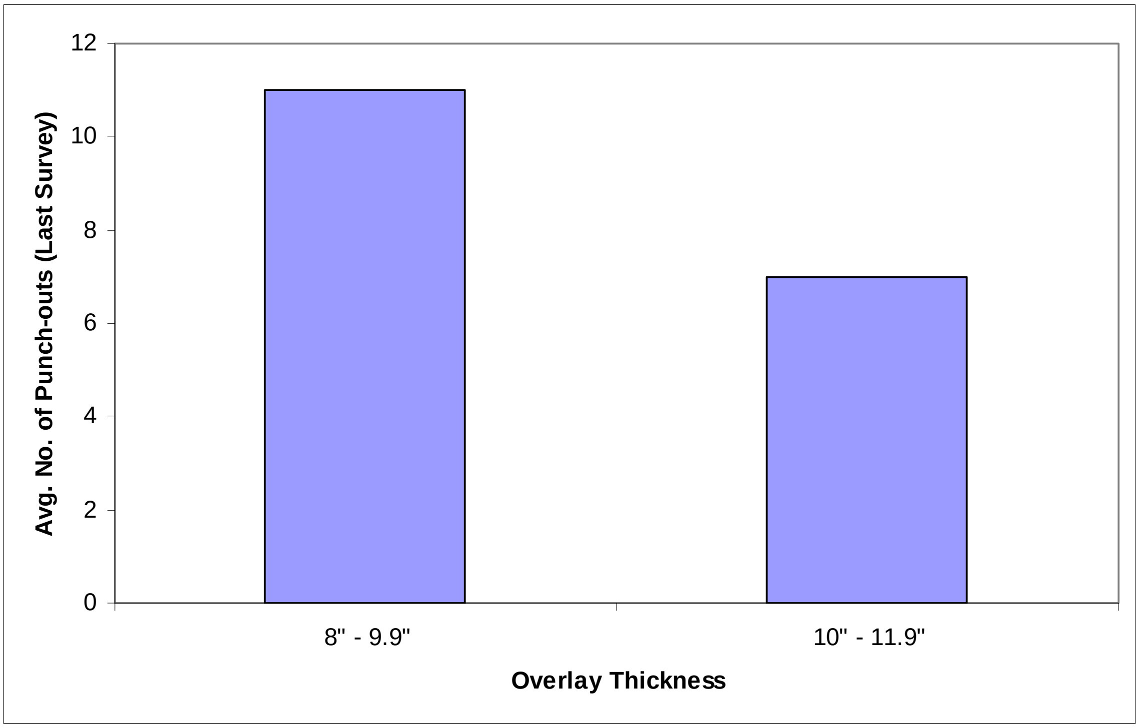


Figure B.21. CRCP overlay thickness versus average number of punch-outs.

International Roughness Index (IRI). Figure B.22 through Figure B.24 illustrate the progression of IRI and PSI for the various GPS 9 sections and the impact of overlay thickness on ride quality.

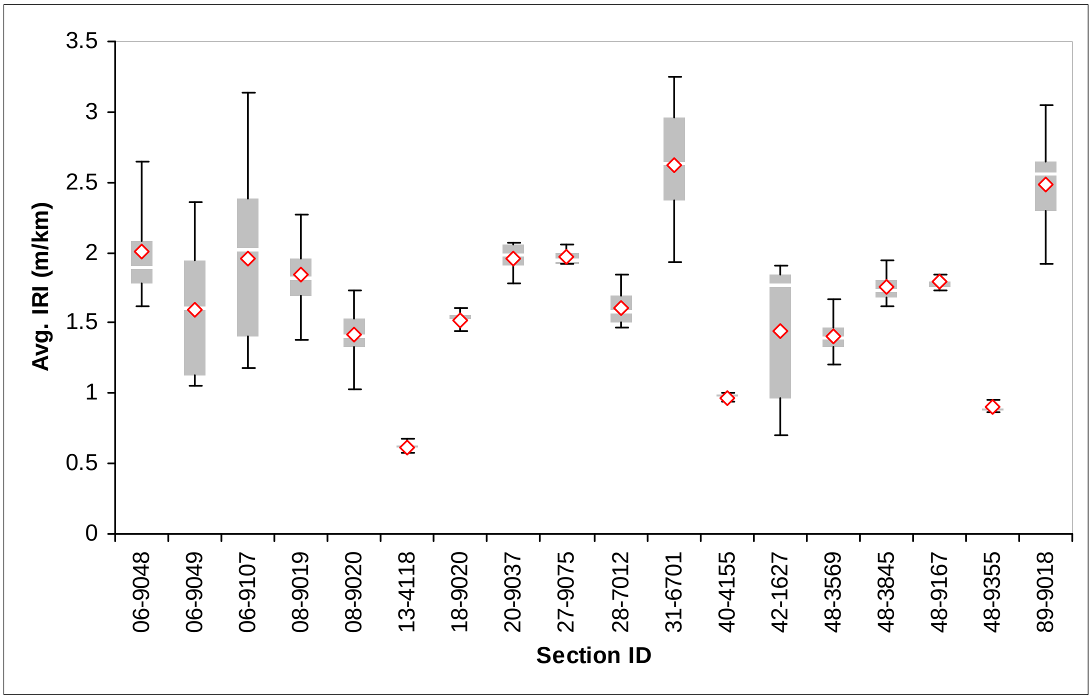


Figure B.22. Distribution of average IRI.

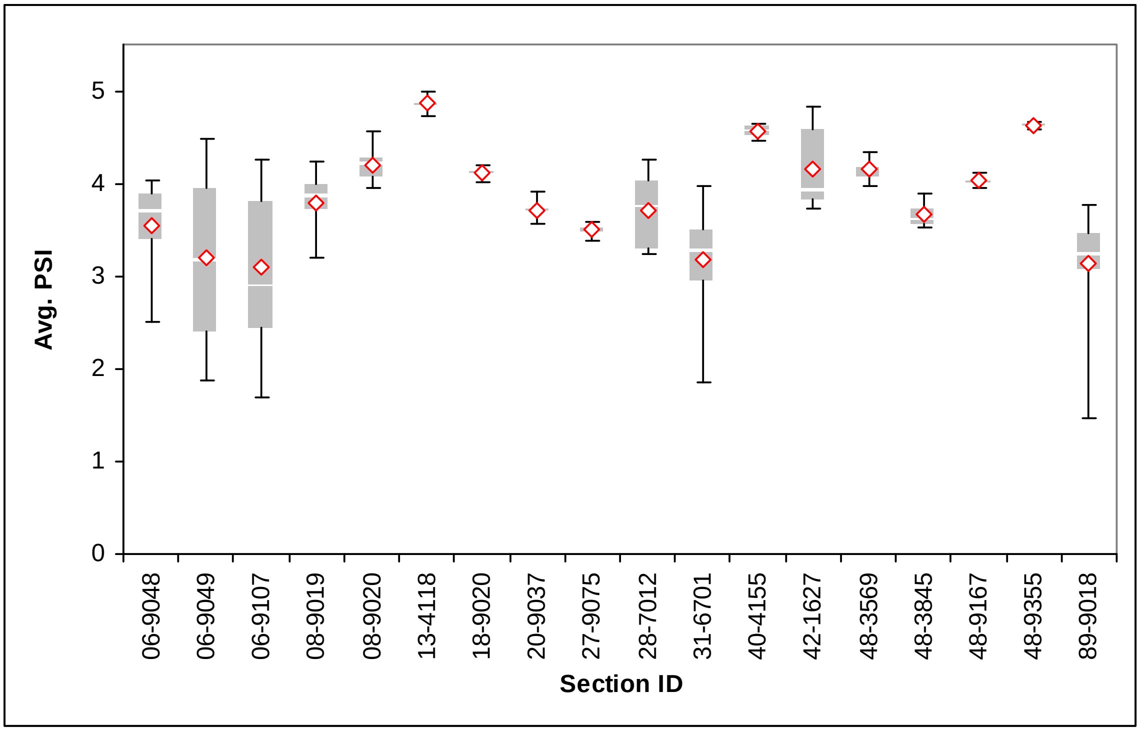


Figure B.23. Distribution of average PSI.

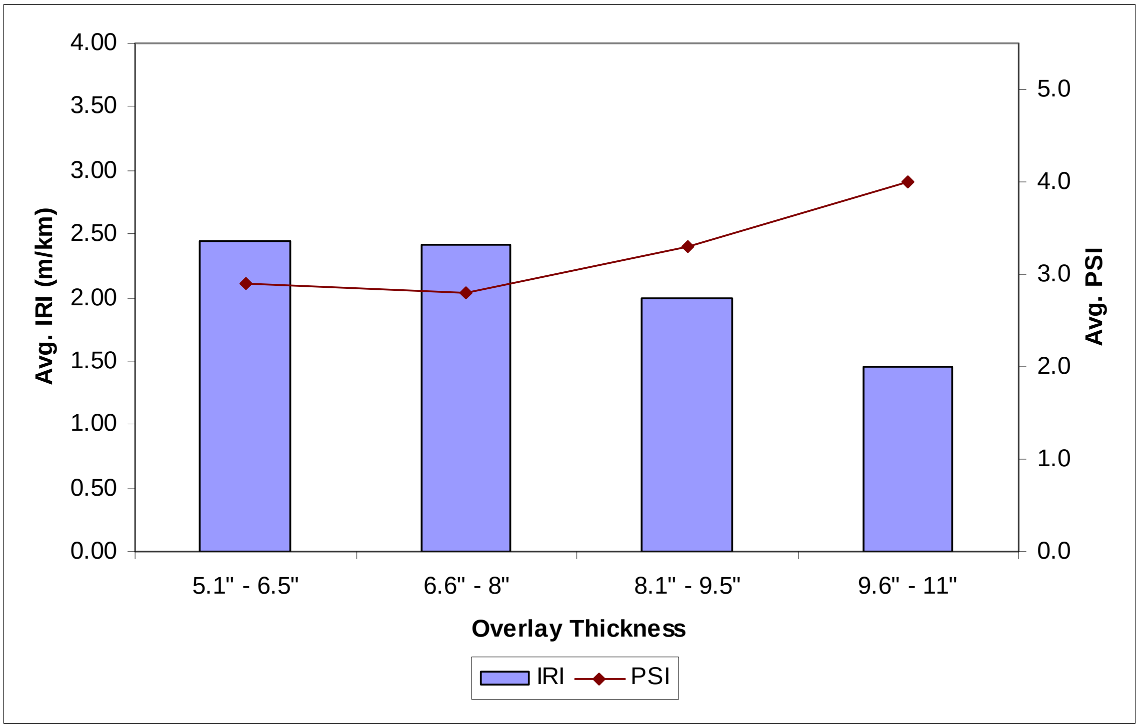


Figure B.24. Overlay thickness versus average IRI and average PSI.

Joint and Crack Faulting. Figure B.25 and Figure B.26 summarize the amount of joint and crack faulting for the various jointed concrete pavement overlays. In Figure B.26, all the sections that belong to the last category (overlay thicknesses of 9.6”-11.0”) are doweled pavements. All the sections belonging to the first three categories are without dowels except for one section (section 89-9018, with overlay thickness of 6ft 4in., is a doweled pavement). It should be noted that the overall magnitude of the faulting is below 0.25in. (the threshold considered for long life pavements) and therefore does not appear to be an issue at this point.

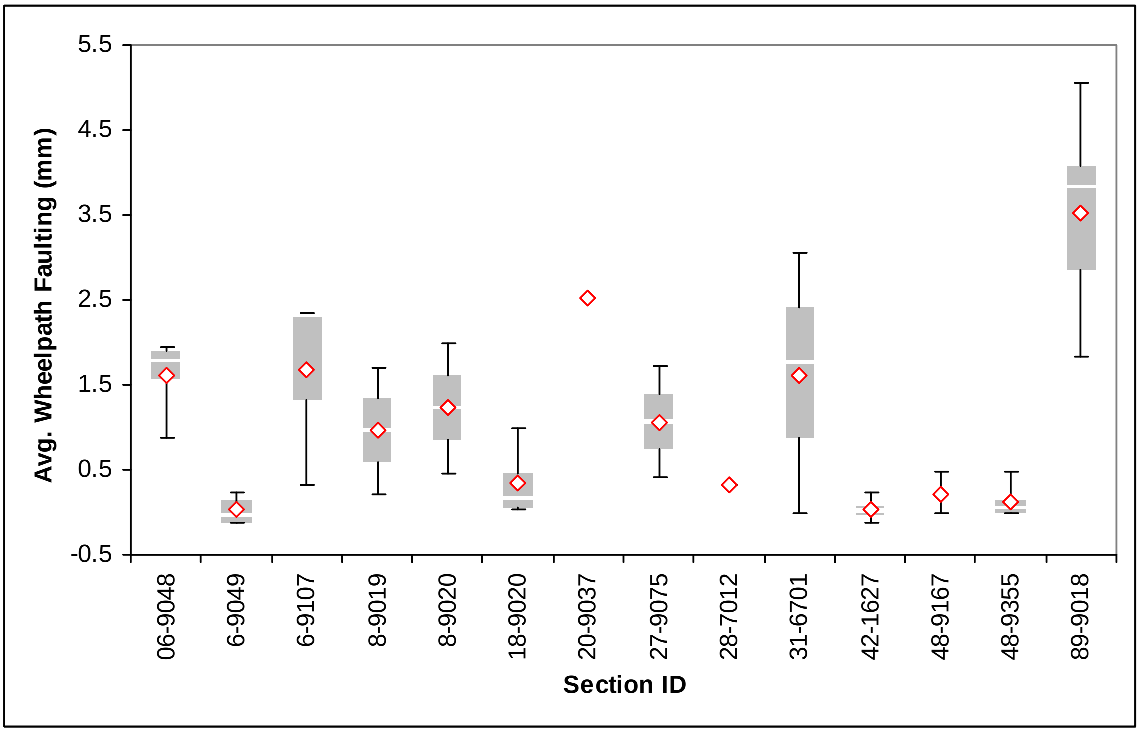


Figure B.25. Distribution of average wheel path faulting.

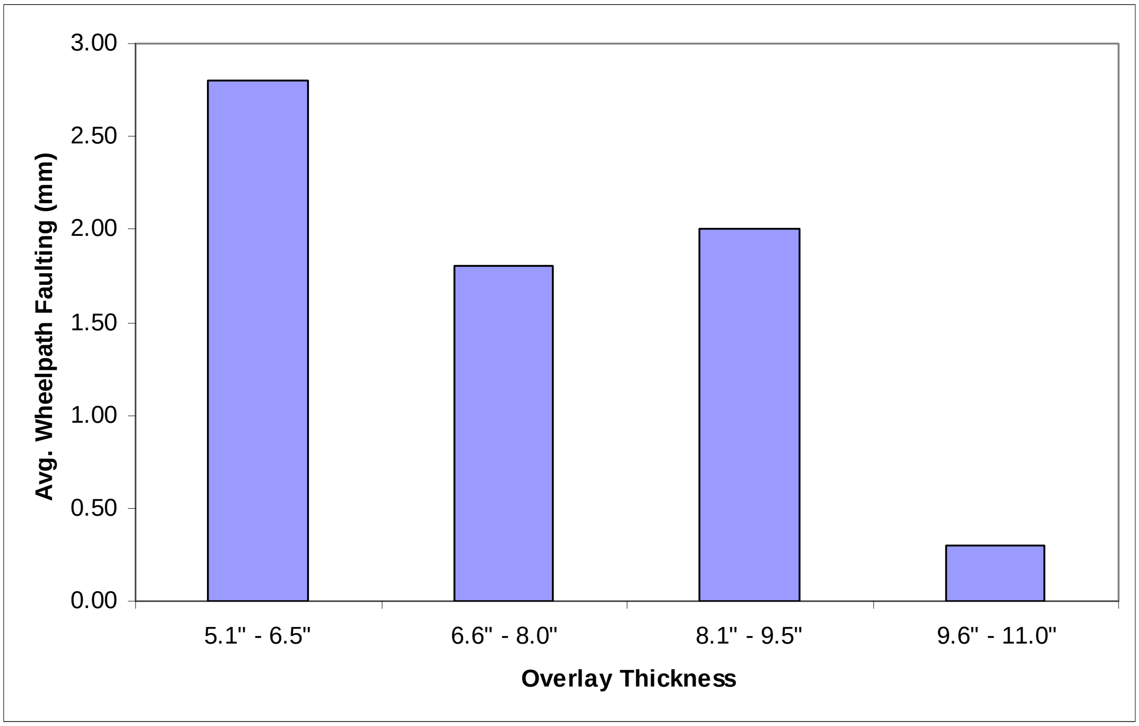


Figure B.26. Overlay thickness versus average wheel path faulting.

Impact of Interlayer Design on Performance. Figure B.27 and Figure B.28 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 B.29 shows that thicker interlayers tend to be associated with less joint faulting.

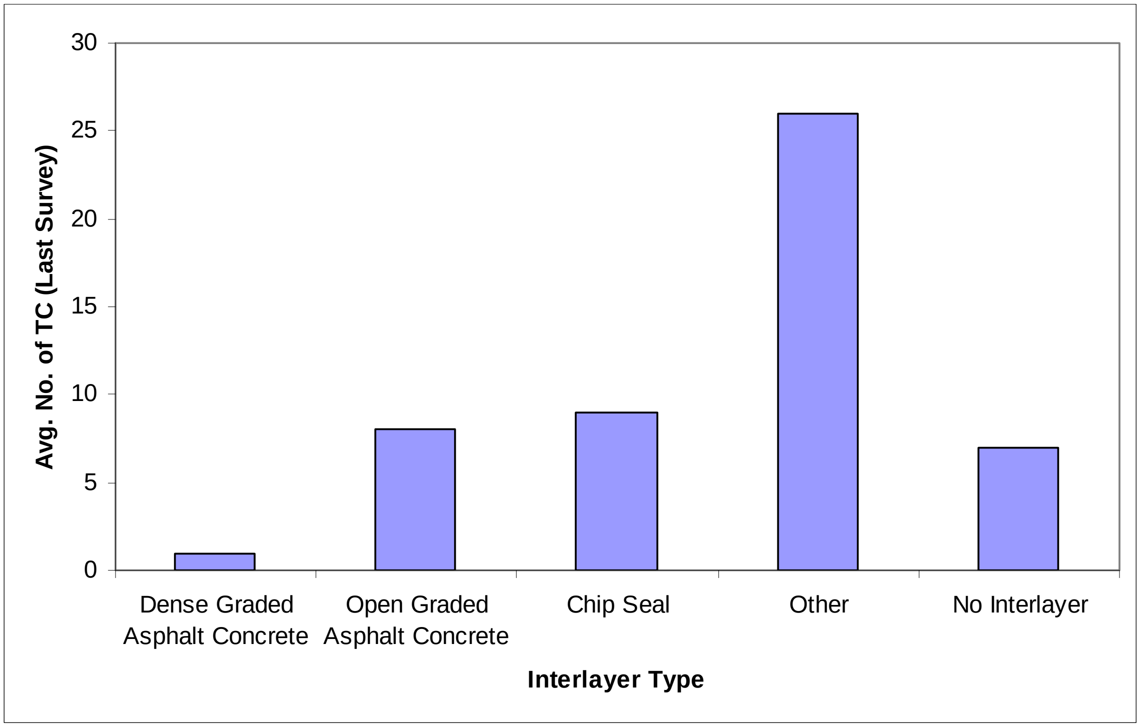


Figure B.27. JPCP interlayer type versus average number of transverse cracks.

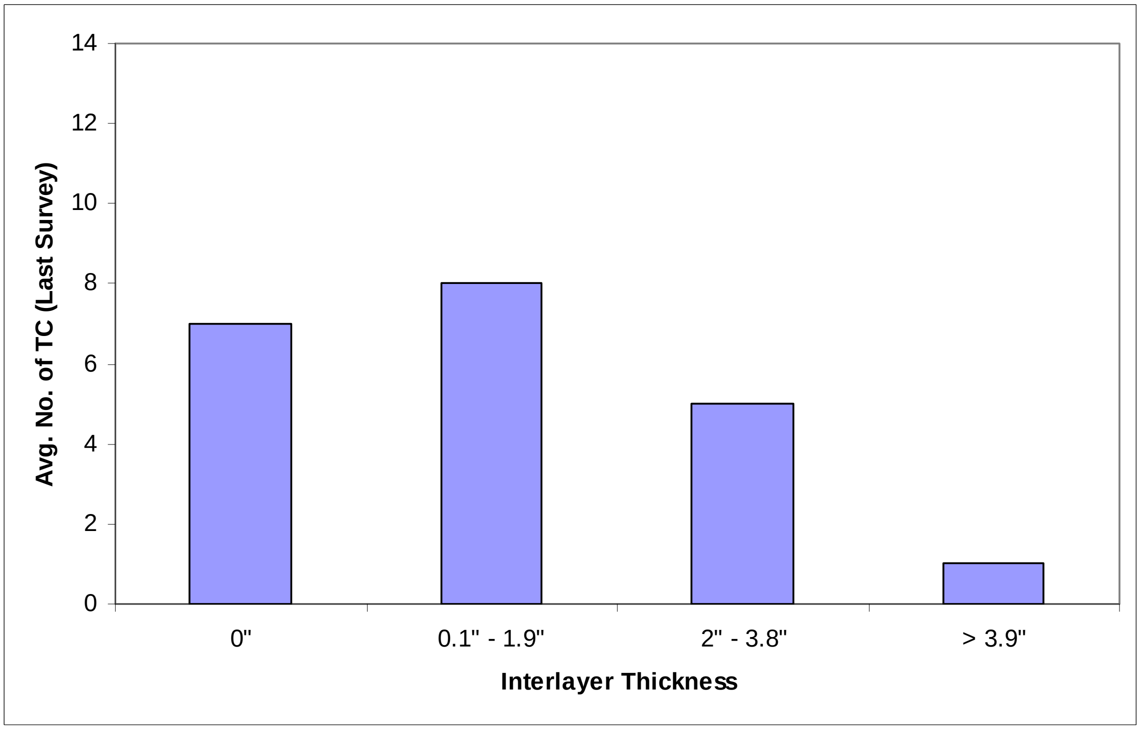


Figure B.28. JPCP interlayer thickness versus average number of transverse cracks.

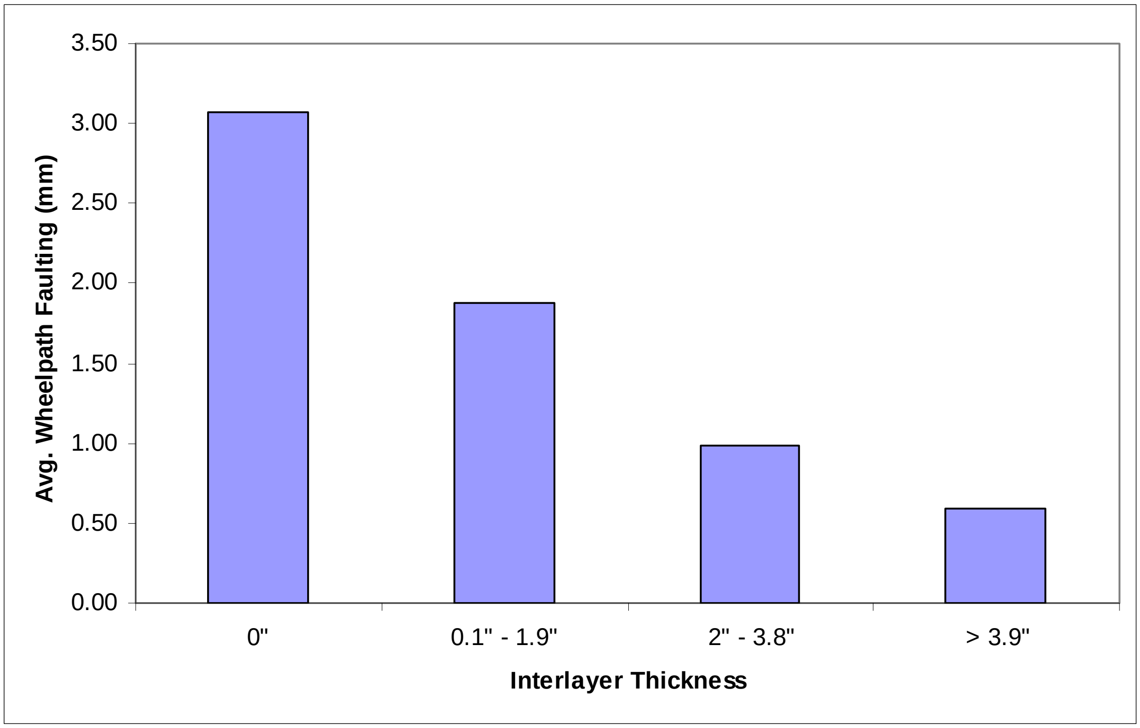


Figure B.29. JPCP interlayer thickness versus average wheel path faulting.

Impact of Load Transfer Mechanism on Performance. Figure B.30 through Figure B.32 illustrate the impacts of dowels and increased pavement thickness on all pavement performance on all pavement performance measures. In these figures, all the sections that belong to the first category (aggregate interlock) have overlay thickness of 9 inches or less. All the sections belonging to the second category (doweled) have overlay thickness of 10 inches or more except for one section (section 89-9018 has overlay thickness of 6.4 inches).

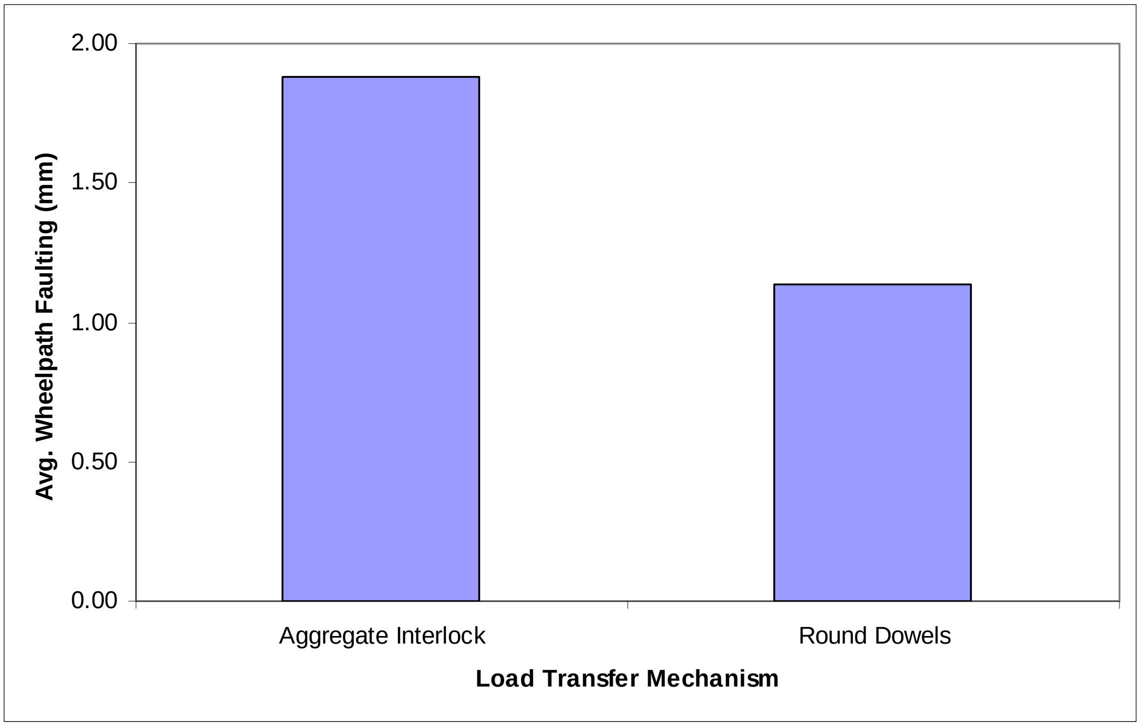


Figure B.30. JPCP load transfer mechanism versus average wheel path faulting.

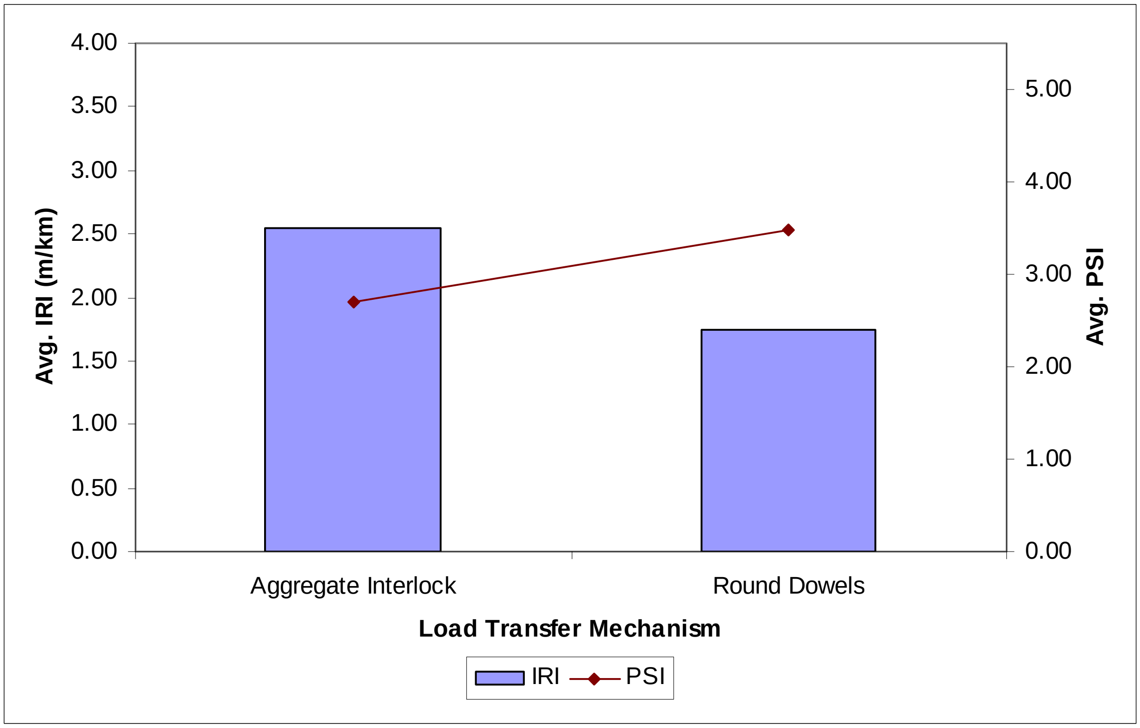


Figure B.31. JPCP load transfer mechanism versus average IRI and average PSI.

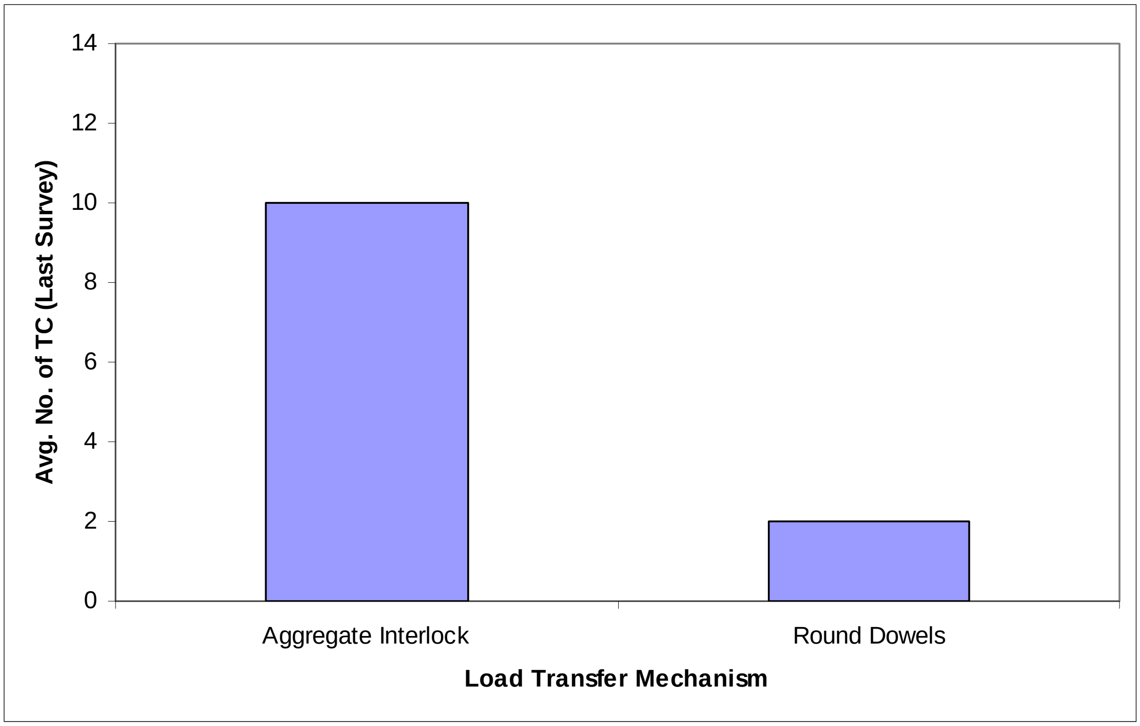


Figure B.32. JPCP load transfer mechanism versus average number of transverse cracks.

SPS7 – Bonded Concrete Overlays

Information about the SPS-7 experiment (Bonded PCC Overlay on PCC Pavement) was extracted from the LTPP DataPave Online (Release 21.0). There were 39 sections located in four states within this experiment. CRCP and PCP (plain concrete pavements only used for SPS-7 overlays of CRCP) were separated from jointed pavements (JPCP), since the performance criteria are not entirely identical. PCP overlays are bonded overlays of CRCP where additional steel was not included in the overlay. CRCP overlays are concrete overlays to which steel was added so that the resulting pavement has two layers of steel. Furthermore, control sections with no overlays (sections ending with 0701) were not considered. Another section that was not considered (29-0759) had an asphalt concrete overlay. After the removal of these sections, data from 18 CRCPs, 9 JPCPs, and eight PCPs are presented in the subsequent sections of this chapter.

Summary of Inventory Information

The location of the various SPS-7 test sections is shown in Figure B.33. Table B.17 summarizes the maintenance events during the life of the overlay since being part of the LTPP experiment. The blank cells under the “construction change reason” indicate no maintenance.

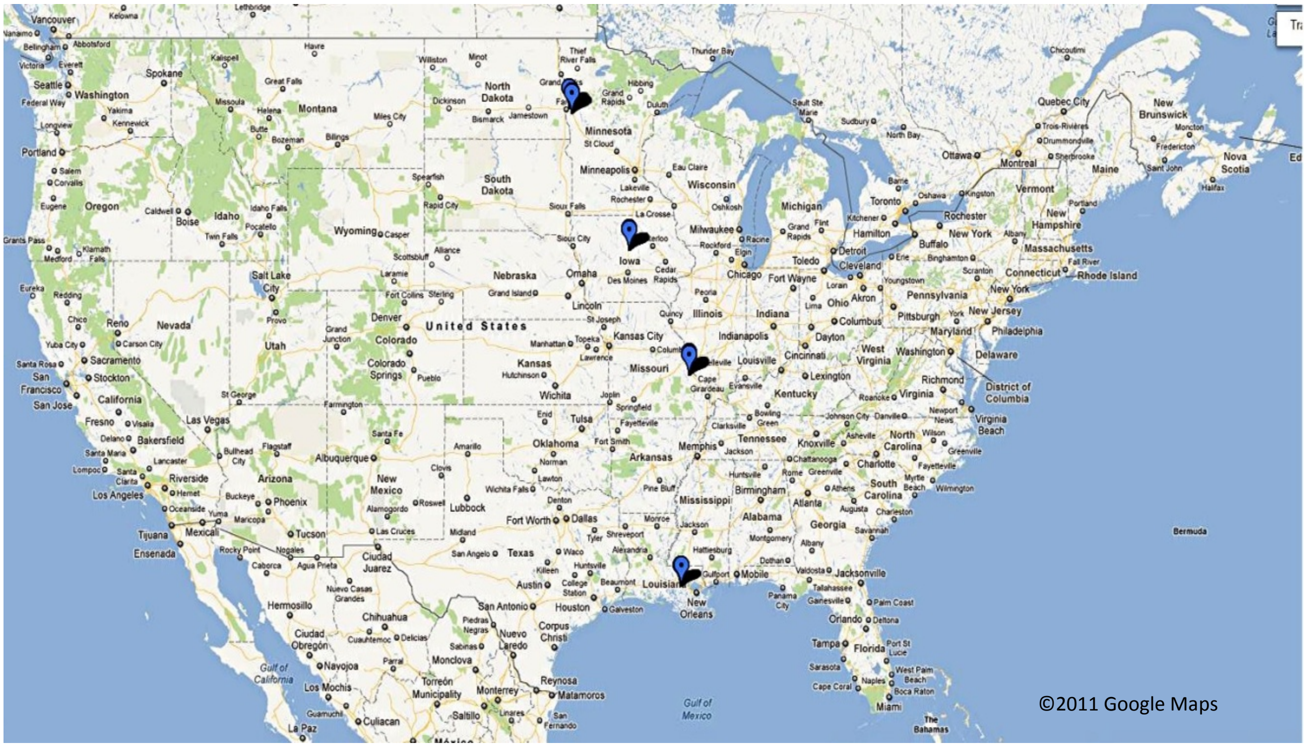


Figure B.33. Locations of SPS-7 sections (Google Maps).

Table B.17. SPS-7 Construction events.

Section ID	Const. No	Const. Assign Date	Construction Change Reason
19-0702	1	1/1/1992	
	2	4/3/1992	Full Depth Transverse Joint Repair Patch, Partial depth patching of PCC pavements at joints
	3	7/14/1992	Lane-Shoulder Longitudinal Joint Sealing, Grinding Surface, Portland Cement Concrete Overlay
19-0703	1	1/1/1992	
	2	4/4/1992	Full Depth Transverse Joint Repair Patch, Partial depth patching of PCC pavements at joints
	3	7/10/1992	Lane-Shoulder Longitudinal Joint Sealing, Grinding Surface, Portland Cement Concrete Overlay
19-0704	1	1/1/1992	
	2	4/6/1992	Full Depth Transverse Joint Repair Patch, Partial depth patching of PCC pavements at joints
	3	7/10/1992	Lane-Shoulder Longitudinal Joint Sealing, Portland Cement Concrete Overlay
19-0705	1	1/1/1992	
	2	4/6/1992	Full Depth Transverse Joint Repair Patch, Partial depth patching of PCC pavements at joints
	3	7/10/1992	Lane-Shoulder Longitudinal Joint Sealing, Portland Cement Concrete Overlay
19-0706	1	1/1/1992	
	2	4/6/1992	Partial depth patching of PCC pavements at joints
	3	7/10/1992	Lane-Shoulder Longitudinal Joint Sealing, Portland Cement Concrete Overlay
19-0707	1	1/1/1992	
	2	4/7/1992	Full Depth Transverse Joint Repair Patch
	3	7/10/1992	Lane-Shoulder Longitudinal Joint Sealing, Portland Cement Concrete Overlay
19-0708	1	1/1/1992	
	2	7/10/1992	Lane-Shoulder Longitudinal Joint Sealing, Grinding Surface, Portland Cement Concrete Overlay
19-0709	1	1/1/1992	
	2	4/6/1992	Partial depth patching of PCC pavements at joints
	3	7/10/1992	Lane-Shoulder Longitudinal Joint Sealing, Grinding Surface, Portland Cement Concrete Overlay

Table B.17 Continued.

Section ID	Const. No	Const. Assign Date	Construction Change Reason
19-0759	1	1/1/1992	
	2	8/6/1993	Portland Cement Concrete Overlay
22-0702	1	1/1/1987	
	2	4/7/1992	Full Depth Patching of PCC Pavement Other Than at Joint, Grinding Surface, Portland Cement Concrete Overlay, PCC Shoulder Restoration
22-0703	1	1/1/1987	
	2	4/10/1992	Grinding Surface, Portland Cement Concrete Overlay, PCC Shoulder Restoration
22-0704	1	1/1/1987	
	2	4/21/1992	Portland Cement Concrete Overlay, PCC Shoulder Restoration
22-0705	1	1/1/1987	
	2	4/21/1992	Portland Cement Concrete Overlay, PCC Shoulder Restoration
22-0706	1	1/1/1987	
	2	4/22/1992	Portland Cement Concrete Overlay, PCC Shoulder Restoration
22-0707	1	1/1/1987	
	2	4/21/1992	Portland Cement Concrete Overlay, PCC Shoulder Restoration
22-0708	1	1/1/1987	
	2	4/9/1992	Grinding Surface, Portland Cement Concrete Overlay, PCC Shoulder Restoration
22-0709	1	1/1/1987	
	2	4/9/1992	Grinding Surface, Portland Cement Concrete Overlay, PCC Shoulder Restoration
27-0702	1	1/1/1987	
	2	9/10/1990	Grinding Surface, Portland Cement Concrete Overlay, Longitudinal Subdrains
27-0703	1	1/1/1987	
	2	9/10/1990	Grinding Surface, Portland Cement Concrete Overlay, Longitudinal Subdrains
27-0704	1	1/1/1987	
	2	9/10/1990	Portland Cement Concrete Overlay, Longitudinal Subdrains
27-0705	1	1/1/1987	
	2	9/10/1990	Portland Cement Concrete Overlay, Longitudinal Subdrains

Table B.17 Continued.

Section ID	Const. No	Const. Assign Date	Construction Change Reason
27-0706	1	1/1/1987	
	2	9/10/1990	Portland Cement Concrete Overlay, Longitudinal Subdrains
27-0707	1	1/1/1987	
	2	9/10/1990	Portland Cement Concrete Overlay, Longitudinal Subdrains
27-0708	1	1/1/1987	
	2	9/10/1990	Grinding Surface, Portland Cement Concrete Overlay, Longitudinal Subdrains
	3	9/1/1998	Partial Depth Patching of PCC Pavement Other Than at Joint
	4	7/1/2001	Partial Depth Patching of PCC Pavement Other Than at Joint
27-0709	1	1/1/1987	
	2	9/10/1990	Grinding Surface, Portland Cement Concrete Overlay, Longitudinal Subdrains
27-0759	1	1/1/1987	
	2	9/10/1990	Portland Cement Concrete Overlay, Longitudinal Subdrains
29-0702	1	1/1/1987	
	2	6/18/1990	AC Shoulder Restoration, Grinding Surface, Portland Cement Concrete Overlay
29-0703	1	1/1/1987	
	2	6/15/1990	Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing, Full Depth Transverse Joint Repair Patch, AC Shoulder Restoration, Grinding Surface, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
29-0704	1	1/1/1987	
	2	6/26/1990	AC Shoulder Restoration, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
29-0705	1	1/1/1987	
	2	6/28/1990	AC Shoulder Restoration, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing

Table B.17 Continued.

Section ID	Const. No	Const. Assign Date	Construction Change Reason
29-0706	1	1/1/1987	
	2	6/29/1990	AC Shoulder Restoration, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
27-0708	1	1/1/1987	
	2	9/10/1990	Grinding Surface, Portland Cement Concrete Overlay, Longitudinal Subdrains
	3	9/1/1998	Partial Depth Patching of PCC Pavement Other Than at Joint
	4	7/1/2001	Partial Depth Patching of PCC Pavement Other Than at Joint
27-0709	1	1/1/1987	
	2	9/10/1990	Grinding Surface, Portland Cement Concrete Overlay, Longitudinal Subdrains
27-0759	1	1/1/1987	
	2	9/10/1990	Portland Cement Concrete Overlay, Longitudinal Subdrains
29-0702	1	1/1/1987	
	2	6/18/1990	AC Shoulder Restoration, Grinding Surface, Portland Cement Concrete Overlay
29-0703	1	1/1/1987	
	2	6/15/1990	Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing, Full Depth Transverse Joint Repair Patch, AC Shoulder Restoration, Grinding Surface, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
29-0704	1	1/1/1987	
	2	6/26/1990	AC Shoulder Restoration, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
29-0705	1	1/1/1987	
	2	6/28/1990	AC Shoulder Restoration, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
29-0706	1	1/1/1987	Table C-16 Continued
	2	6/29/1990	AC Shoulder Restoration, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing

Table B.17 Continued.

Section ID	Const. No	Const. Assign Date	Construction Change Reason
29-0707	1	1/1/1987	
	2	6/29/1990	AC Shoulder Restoration, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
29-0708	1	1/1/1987	
	2	6/19/1990	AC Shoulder Restoration, Grinding Surface, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
29-0709	1	1/1/1987	
	2	6/19/1990	AC Shoulder Restoration, Grinding Surface, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing
29-0760	1	1/1/1987	
	2	6/11/1990	Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing, Full Depth Transverse Joint Repair Patch, AC Shoulder Restoration, Grinding Surface, Portland Cement Concrete Overlay
	3	2/15/2000	Crack Sealing, Transverse Joint Sealing, Lane-Shoulder Longitudinal Joint Sealing

Figure B.34 shows the distribution of overlay age. It should be noted that all SPS-7 sections have been taken out of the LTPP study and additional data is not available. The distribution of the overlay age shown in Figure B.34 is until the de-assign date. The overlay thicknesses of the various test sections range from 3.1 to 6.5 inches. The distribution of bonded overlay thickness is shown in Figure B.35 and Figure B.36. All the JPCP overlays had transverse joint spacing of 20 feet. The distribution of bonding agent types commonly used is shown in Figure B.37.

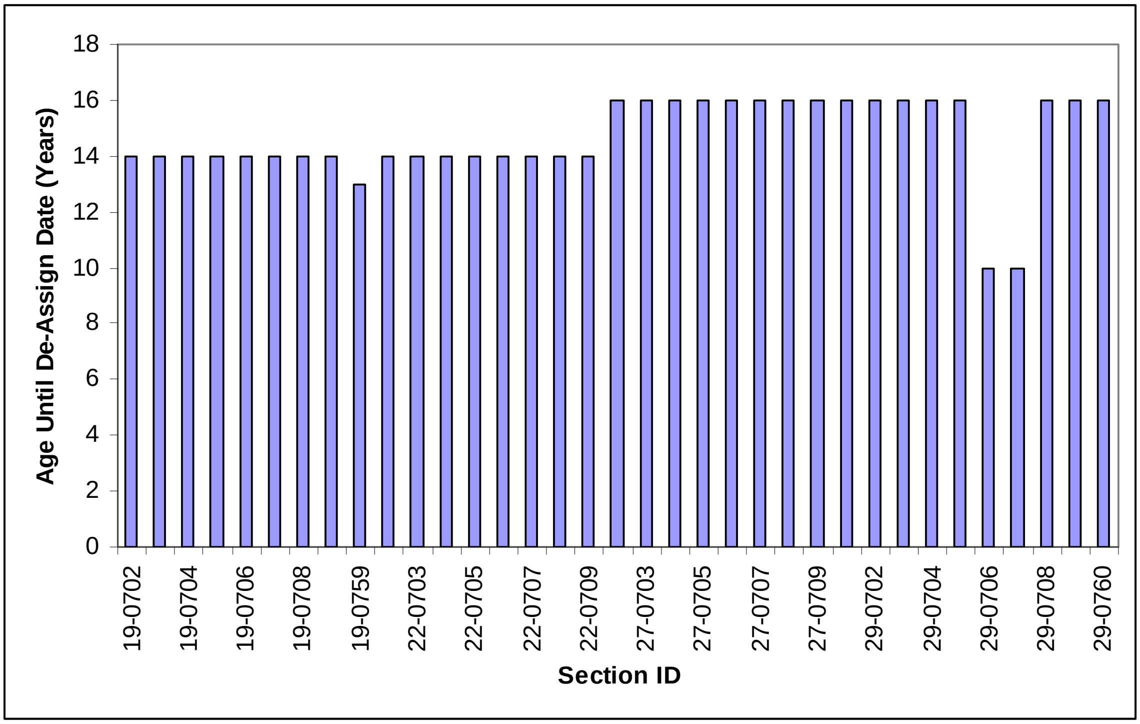


Figure B.34. Overlay age until de-assign date.

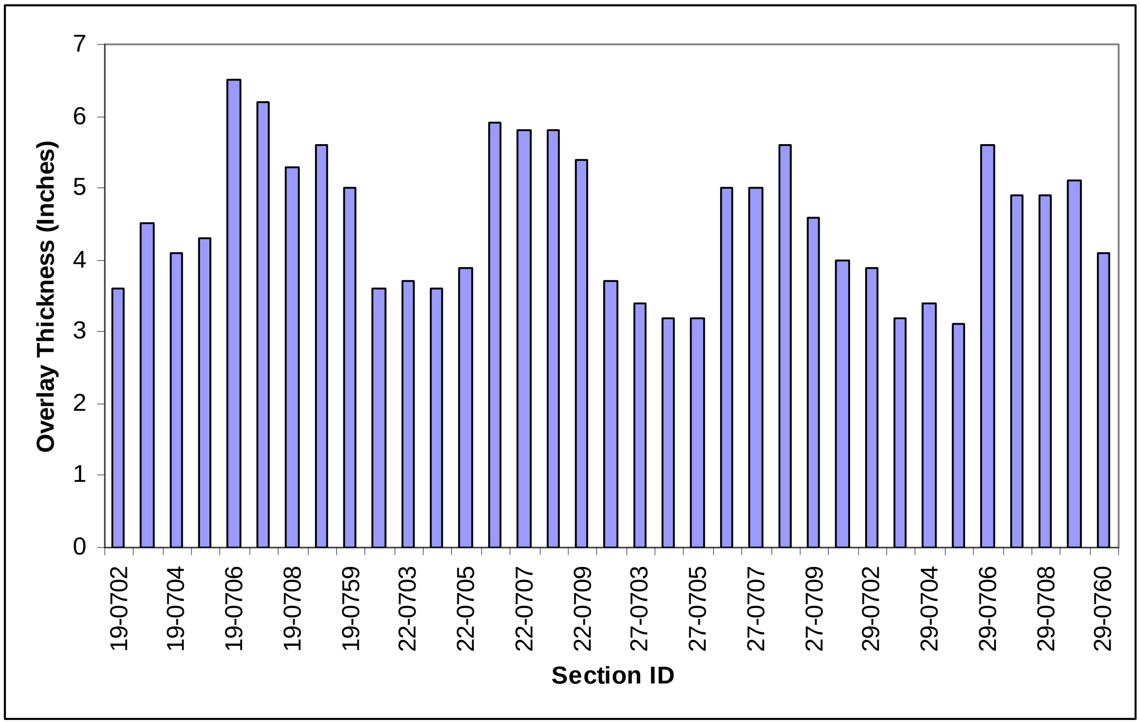


Figure B.35. Overlay thicknesses.

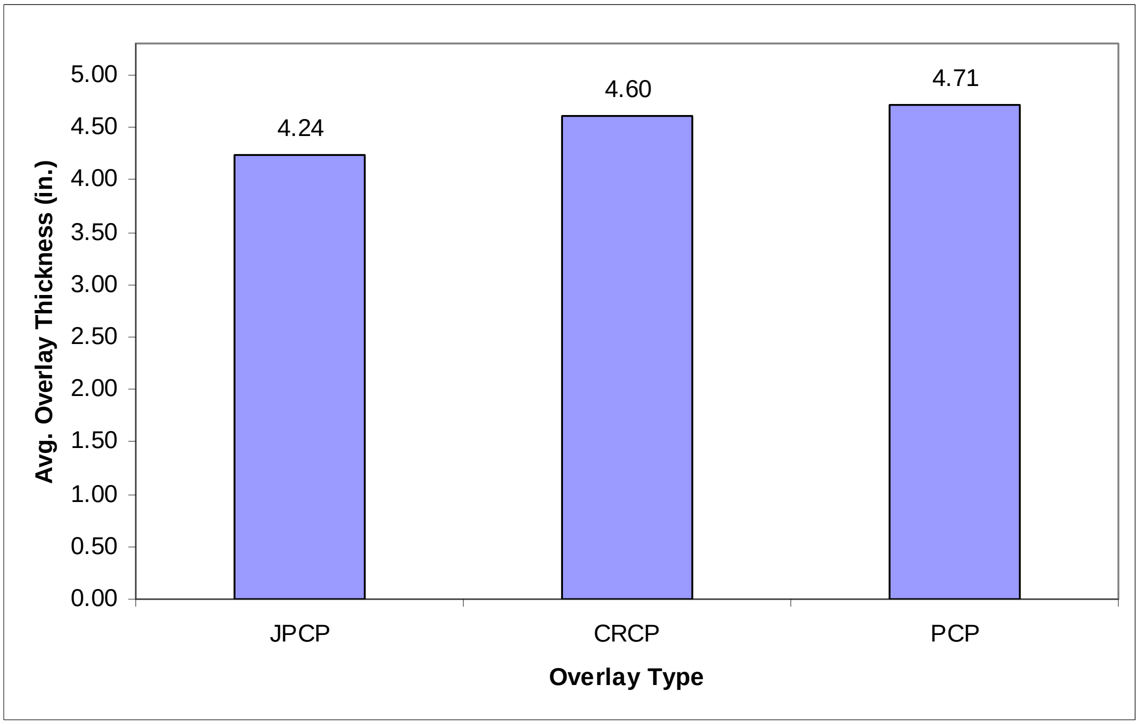


Figure B.36. Average overlay thickness for SPS-7 overlay types.

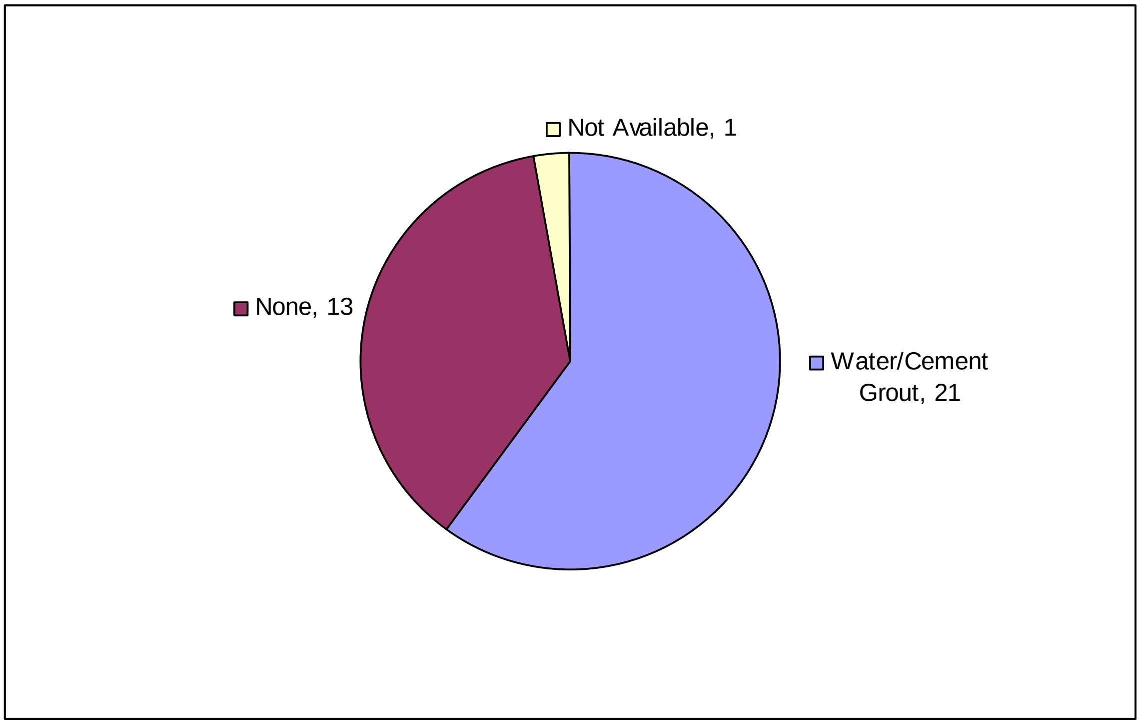


Figure B.37. Types of bonding agents.

Figure B.38 shows the distribution of the surface preparation methods used to create a bond in the various sections. The impact of the various surface preparations to create a bond on pavement performance is negligible.

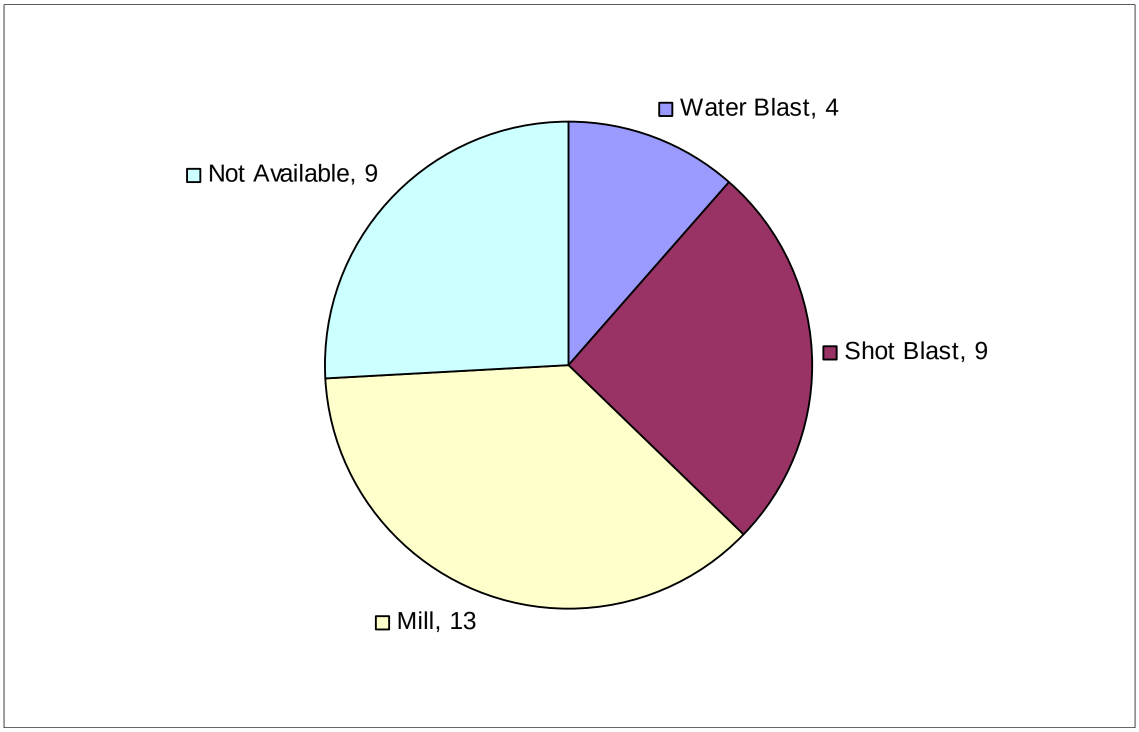


Figure B.38. Surface preparation methods.

Summary of Overall Field Performance

The pavement performance criteria selected for the summary include transverse cracking, IRI (and PSI), joint and crack faulting magnitude (JPCP), and punch-outs (for CRCP and PCP). The performance trends presented in this section are based on measurements documented before the test section was taken out of the LTPP study.

Transverse Cracking. Figure B.39 (box and whisker plot) shows the distribution of percent cracking across the JPCP sections. The box-and-whisker plots presented here display data as follows: the median is represented by the horizontal line inside the box. The top and bottom of the box represent the third quartile (75th percentile) and the first quartile (25th percentile), respectively. The distance between these two is the interquartile range (IQR). In these plots, whiskers are drawn to the minimum and maximum observations.

Figure B.40 shows the magnitude of percent cracking as a function of overlay thickness for the joint concrete pavements.

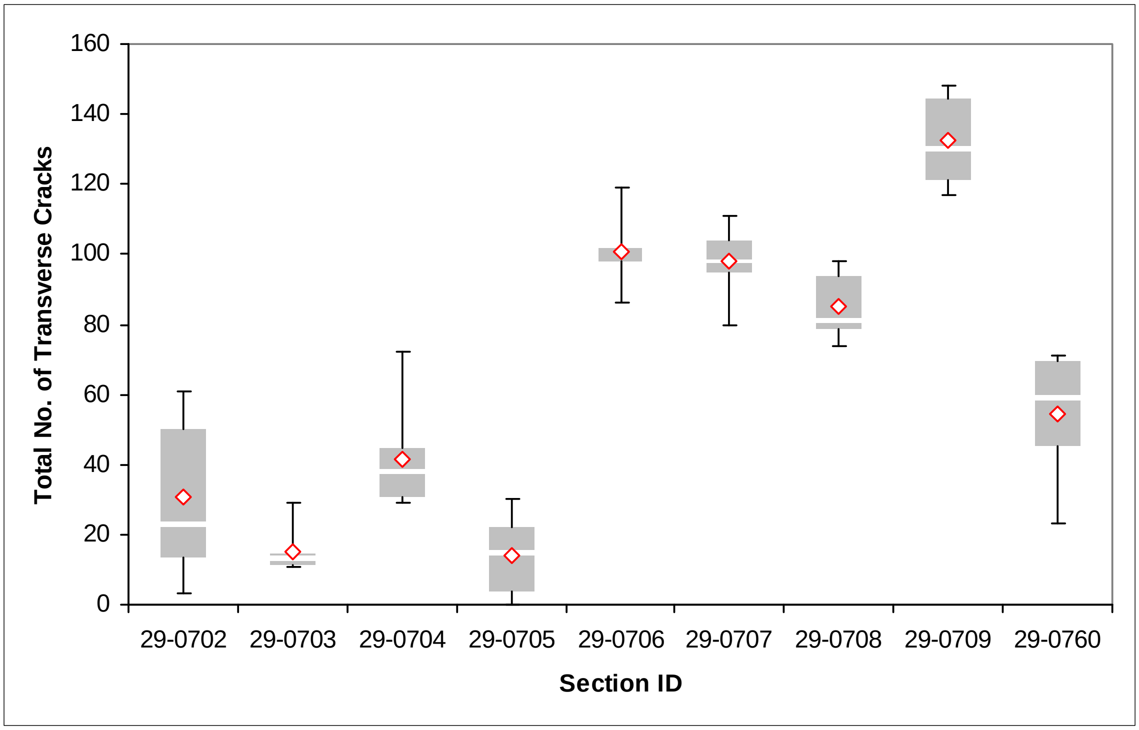


Figure B.39. Distribution of number of transverse cracks for JPCP sections.

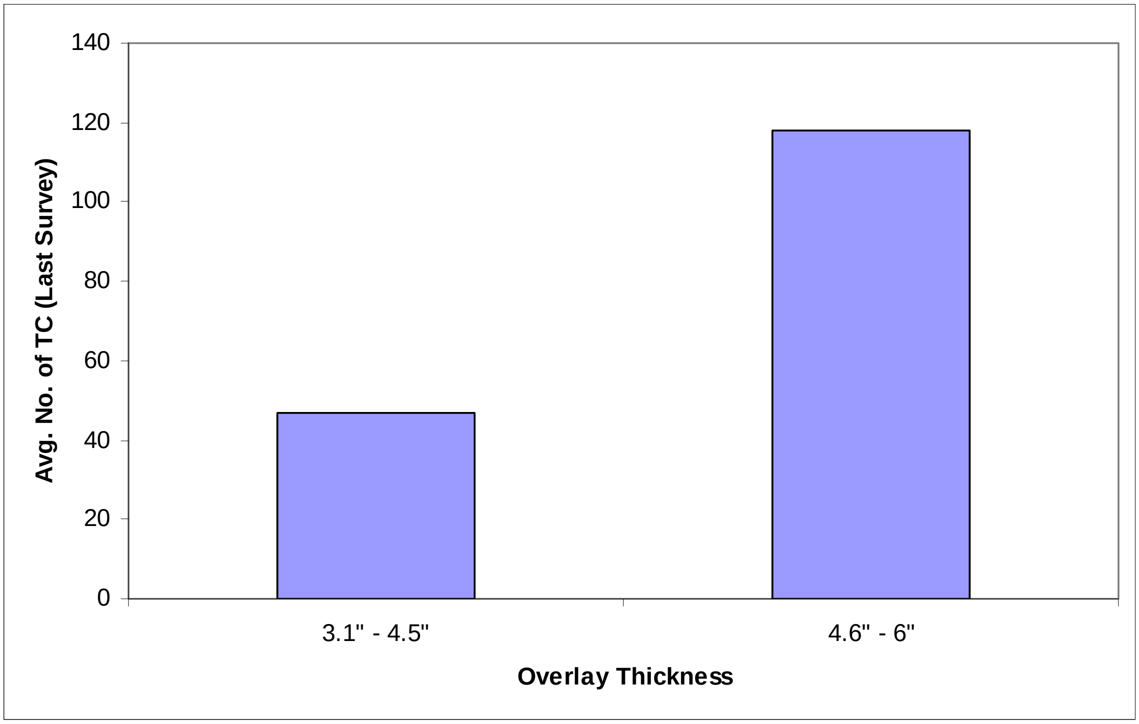


Figure B.40. JPCP overlay thickness versus No. of transverse cracks.

Figure B.41 (box and whisker plot) shows the distribution of percent cracking across the PCP and CRCP sections. Figure B.42 shows the percent cracking as a function of overlay thickness for the CRC and PC pavements.

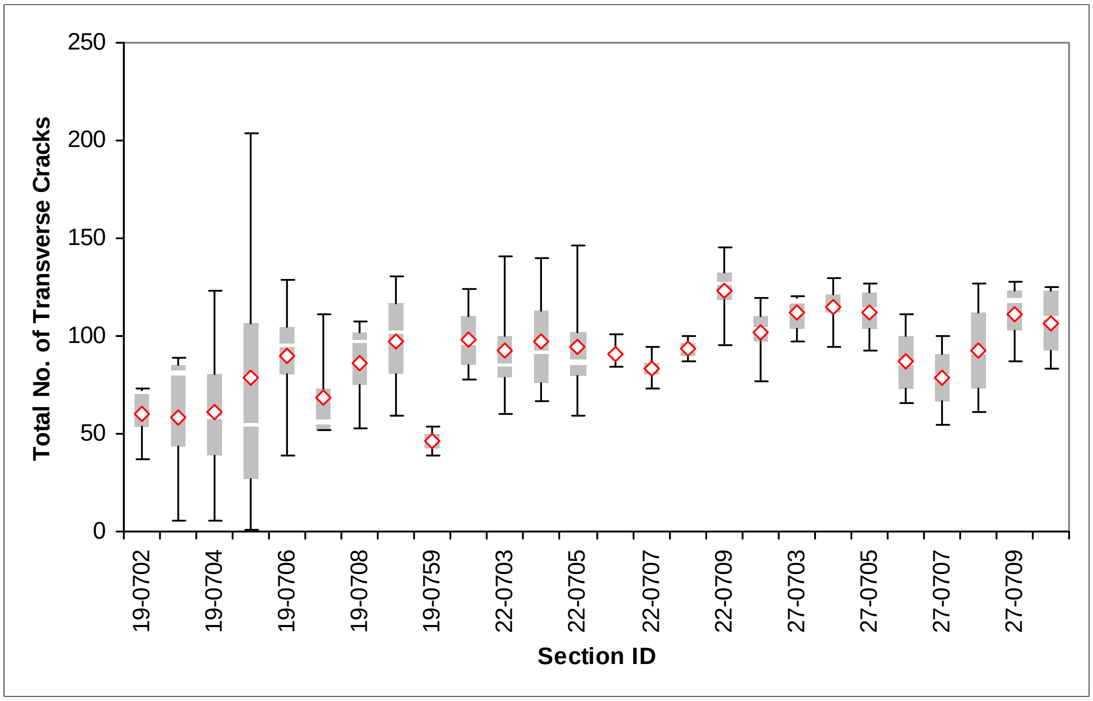


Figure B.41. Distribution of number of transverse cracks for CRCP and PCP sections.

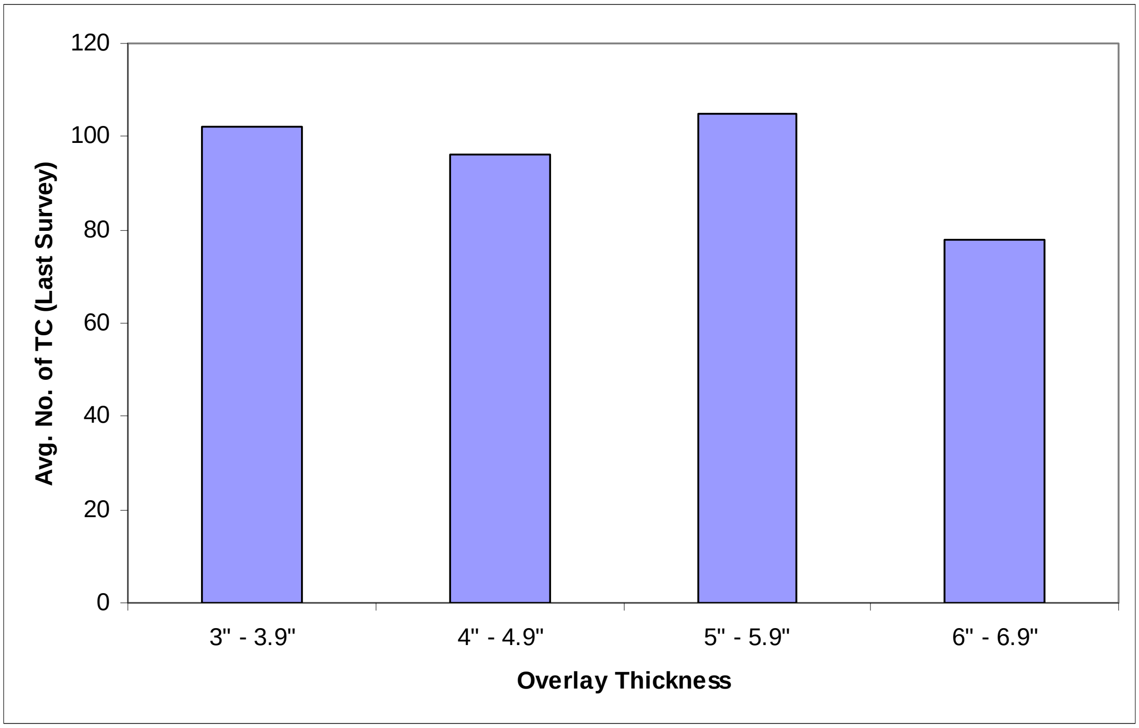


Figure B.42. CRCP and PCP overlay thickness versus number of transverse cracks.

Figure B.43 shows the distribution of the number of punch-outs for PCP and CRCP sections. Figure B.44 shows the number of punch-outs as a function of overlay thickness for the CRC and PC pavements.

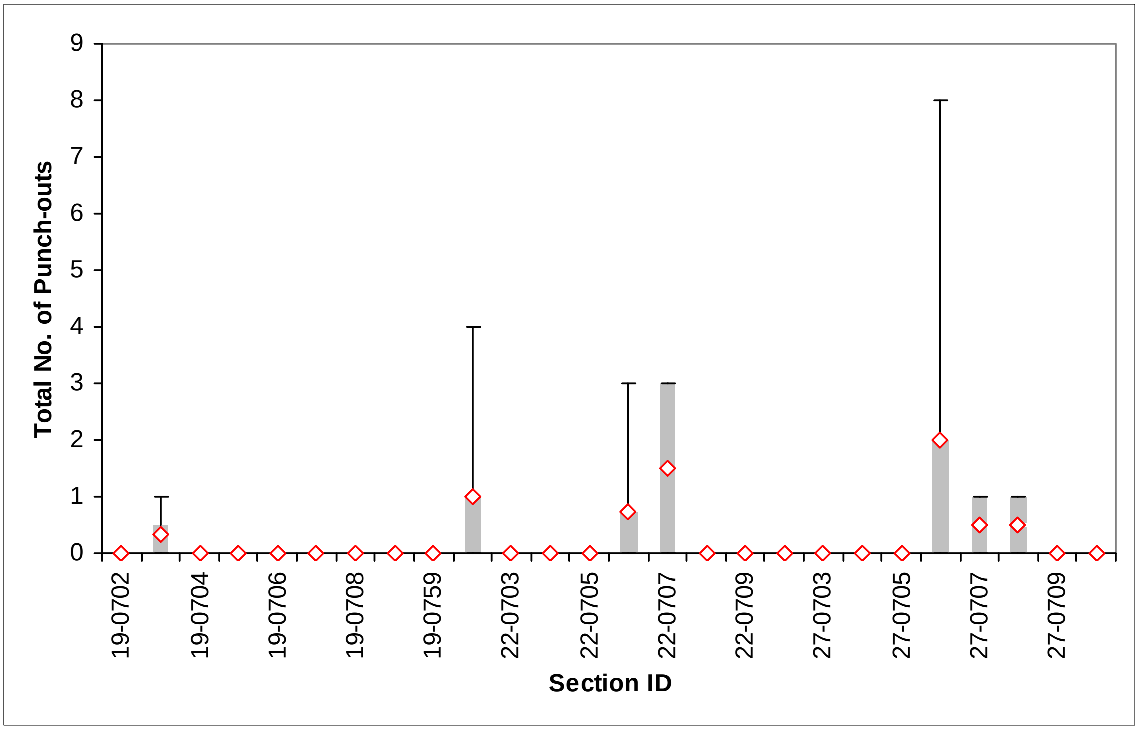


Figure B.43. Distribution of number of punch-outs for CRCP and PCP sections.

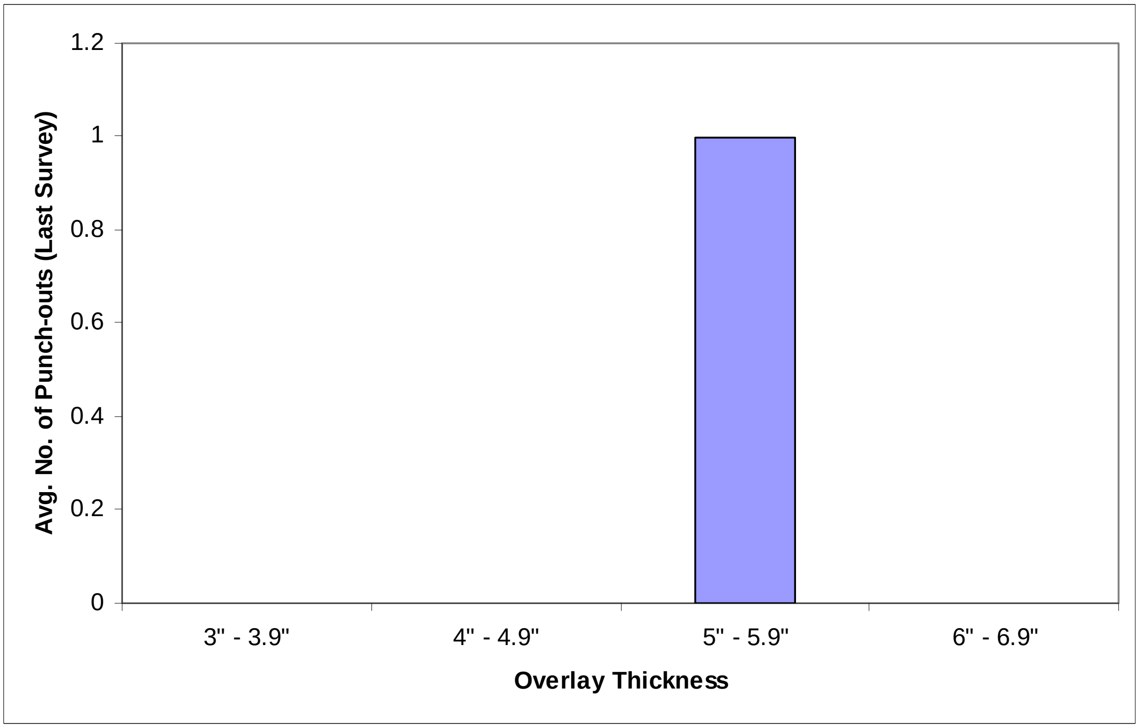


Figure B.44. CRCP and PCP overlay thickness versus number of punch-outs.

International Roughness Index (IRI). Figure B.45 through Figure B.47 illustrate the progression of IRI and PSI for the various SPS 7 sections and the impact of overlay thickness on ride quality.

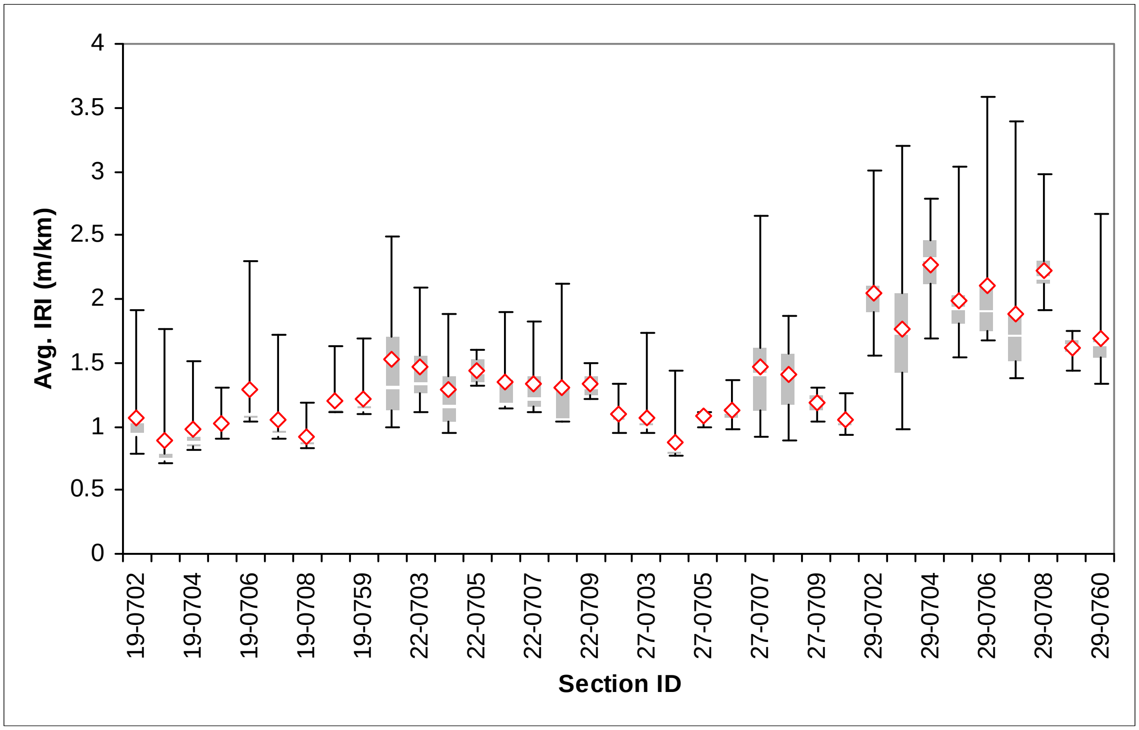


Figure B.45. Distribution of average IRI.

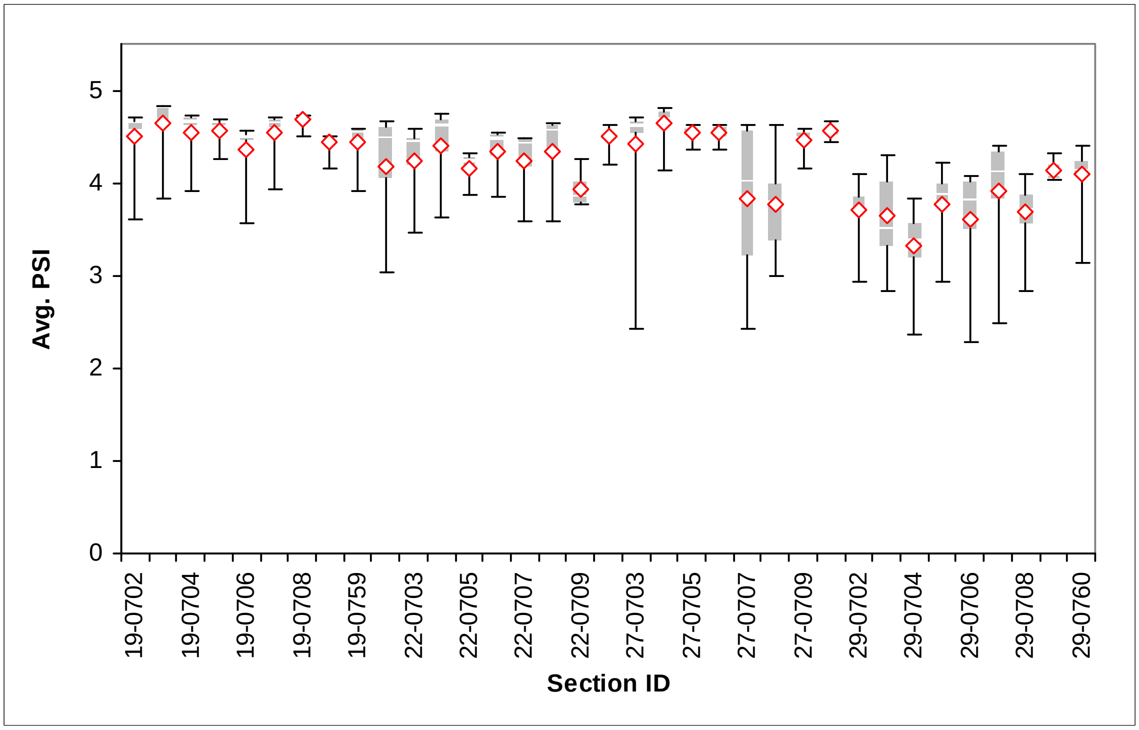


Figure B.46. Distribution of average PSI.

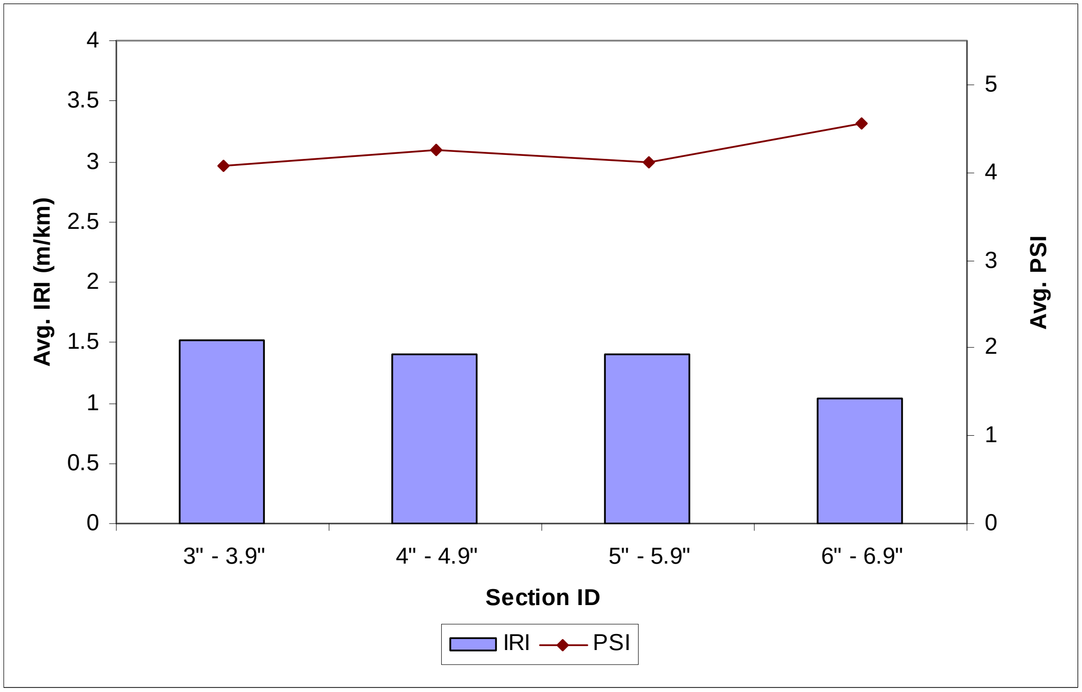


Figure B.47. Overlay thickness versus average IRI and average PSI.

SUMMARY

AC Renewal Projects

Four sections were selected for mechanistic analysis using the ME-PDG and PerRoad software for performance prediction. They all met the criteria of “Heavy Traffic,” “Good Performance,” and promise to be “long-life” pavements. Those sections are provided in Table B.18.

Table B.18. Summary of AC LTPP section analyzed.

Experiment	State	SHRP ID	Age (Yr)	Traffic (KESAL)	Original Thickness (Inch)			Overlay Thickness		Overlay Age
GPS – 6A	47	6015	30	23647	8.8	Medium		5.5		19
GPS – 6B	47	3108	33	28429	5.5	Medium		2.7		16
GPS – 7A	13	7028	17	16763	9.1	Medium	JPCP	7.0	6.0	12
									2.5	5
GPS – 7B	18	5022	34	176836	9.8	Medium	CRCP	4.0		13

PCC Renewal Projects

In this Appendix the performance of 18 GPS-9 sections and 35 SPS 7 sections has been summarized. A significant fraction of the GPS-9 test sections have a potential for long life performance (50 plus years). Unfortunately, this cannot be verified based on field observations because all sections have been de-assigned and no further data collection is planned. Therefore, the potential for long life was predicted using the M-E PDG software.

APPENDIX C

DEVELOPMENT OF RIGID AND FLEXIBLE RENEWAL DECISION MATRICES

DEVELOPMENT OF RIGID AND FLEXIBLE RENEWAL DECISION MATRICES

The decision tables developed for this project were based on extensive literature review, pavement design analysis and in working with a number of engineers both from State DOT's as well as industry. The guidelines were developed to help designers in selecting either a rigid or flexible reconstruction approach that can reasonably be expected to provide long life pavement performance. For this project, long life performance was defined as providing 50 years or service without major structural deterioration. It is anticipated that any approach selected will require some form of rehabilitation or resurfacing during the service life of the pavement. The final selection of the most appropriate design should be based on a life cycle cost analysis of the various approaches including all rehabilitation or resurfacing costs over the life of the pavements.

The development of the decision matrices followed more or less the standard process where team members laid out an outline of the decision process on a blackboard in outline form. The outline had the basic form seen on the tables with pavement type, distress present and potential renewal approaches for those conditions. The outline was circulated to the full team and modified as additional considerations were added. The outline was presented at the kickoff meetings then circulated among the participating Agencies for comment, and again adjustments were made to the outline. To make the process clearer the decision matrix was put in a set of tables. The tables were then circulated again to the full R23 Team and the participating Agencies who provided more comment (most likely because the tables were easier to follow than the outlines). The tables were then used to build an interactive flash based program that would simplify using the decision matrix. In building the logic for the interactive program, a few more decision points were added based on the more rigorous nature of that process. After the program was developed, it was put through a series of trials on a wide range of potential applications and the decision tables were adjusted again based on errors or omissions found in that process. The interactive program and the decision tables were again presented to the participating Agencies for review and comment and final adjustments were made to the program and the tables presented in this appendix.

General guidance was also developed on layer thickness that would be required to provide long life pavement renewal. A set of tables were developed as described in Appendix D. Since these design tables are linked to the actions included in the decision tables, a fifth column was added to the decision tables. The information in this column called "design resources" states the specific thickness design table to be used for those specific sets of conditions and renewal approach. These specific thickness design tables can be found in Appendix D. Where there was a clear repetition of actions and design resources Rules were used to reduce the verbiage in the columns.

The decision tables have been incorporated into the interactive program to help users develop a list of feasible approaches based on existing site conditions. Table C. 1 provides details on the decision matrix for existing flexible pavements. Table C. 2 and Table C. 3 provide details on the decision matrix for existing JPCP and JRCP. Table C. 4

describes the decision process for existing CRCP while Table C. 5 details renewal alternatives for existing composite Pavements. Three rules are commonly referenced in Table C. 2 through Table C. 5 under the “Design Resources” column. Each rule is described below:

Rule 1: Rubblization of existing PCC followed by application of AC overlay from Tables D.37 through D.39 (Appendix D). Rubblization guidelines include the following:

- If the subgrade MR < 6,000 psi or CBR < 4%, do not rubblize, thus defaulting to crack and seat only.
- If the subgrade MR ≥ 6,000 psi but < 10,000 psi, consult the TTI rubblization guidelines as to whether rubblization is viable (Sebesta and Scullion, 2006).
- If the subgrade MR ≥ 10,000 psi, then rubblization is a viable option.

The selection of the AC thickness is based on a drop-down menu of subgrade moduli = 5,000 psi, 10,000 psi, or 20,000 psi. The existing pavement shall be characterized by one of four possible moduli: 30,000 psi, 50,000 psi, 75,000 psi, or 100,000 psi. It is recommended that an existing pavement modulus = 50,000 psi be used to reflect rubblized PCC.

Rule 2: Crack and seat existing PCC followed by application of AC overlay from Tables D.37 through D.39 (Appendix D). The selection of the AC thickness is based on a drop-down menu of subgrade moduli = 5,000 psi, 10,000 psi, or 20,000 psi. The existing pavement shall be characterized by one of four possible moduli: 30,000 psi, 50,000 psi, 75,000 psi, or 100,000 psi. It is recommended that an existing pavement modulus = 75,000 psi be used to reflect crack and seated PCC.

Rule 3: Use Table D.22 (Appendix D) for thickness determination of an unbonded PCC overlay and place on a 2 inch thick AC bond breaker. The unbonded PCC overlay thickness is independent of subgrade support conditions.

Table C. 1. Feasible Renewal Alternatives for Existing Flexible Pavements.

Distress Category	Specific Distress Description	Distress present?	Renewal Pavement Type Option	Action	Design Resources
Environ-mental Cracking	Transverse or Block Cracking	Yes	Flexible	Pulverize pavement structure full-depth followed by a thick AC overlay.	Pulverize and use residual material as untreated base (50 ksi). Apply AC thickness from Tables D.37- D.39.
					Pulverize and treat residual material with emulsion or foamed asphalt resulting in a treated base (100 ksi). Apply AC thickness from Tables D.37- D.39.
			Rigid	No mitigation required, place an unbonded PCC overlay.	Use Table D.22 for thickness determination of an unbonded PCC overlay.
		No	--	Continue to Materials Caused Distress.	--
Materials Caused Distress	Stripping	Yes	Flexible	If stripping is found through all layers, pulverize pavement structure full-depth followed by a thick AC overlay.	Pulverize and use residual material as untreated base (50 ksi). Apply AC thickness from Tables D.37- D.39.
					Pulverize and treat residual material with emulsion or foamed asphalt resulting in a treated base (100 ksi). Apply AC thickness from Tables D.37- D.39.
				If stripping is found in specific layers, remove AC to maximum depth of stripping followed by a thick AC overlay.	Use Tables D.37- D.39 with 30 ksi base and the subgrade M_R to determine total depth of AC thickness then subtract remaining AC thickness to determine overlay thickness.
			Rigid	Place unbonded PCC overlay. If grade limits require, mill existing pavement. AC overlay over stripped pavement may be required to stabilize HMA.	Use Table D.22 for thickness determination of an unbonded PCC overlay.
		No	--	Continue to Full Depth Fatigue Cracking.	--
Full Depth Fatigue Cracking	Longitudinal or Alligator Cracking in Wheel paths	Yes	Flexible	<15% fatigue cracking: patch and repair, moderate thickness AC overlay.	Use Tables D.37- D.39 with 30 ksi base for AC overlay thickness, then subtract existing AC thickness to determine overlay thickness.
				>15% fatigue cracking: pulverize pavement structure full-depth followed by a thick AC overlay.	Pulverize and use residual material as untreated base. Apply AC thickness from Tables D.37- D.39 with 50 ksi base.
					Pulverize and treat residual material with emulsion or foamed asphalt resulting in a treated base. Apply AC thickness from Tables D.37- D.39 with 100 ksi base.
			Rigid	Patch severely cracked areas, place an unbonded PCC overlay. Profile elevation may require milling existing AC pavement.	Use Table D.22 for thickness determination of an unbonded PCC overlay.
		No	--	Continue to Top Down Cracking.	--
Top Down Cracking	Longitudinal or Alligator Cracking in Wheel paths	Yes	Flexible	< 15% patch and overlay	Use Tables D.37- D.39.with 30 ksi base and the subgrade M_R to determine total depth of AC thickness, then subtract the thickness milled out to eliminate the top down cracking (unless indicated the assumed depth is 2 inches). Where patching only, subtract existing depth to calculate overlay.
				>15% Mill down to bottom of cracking followed by a moderate thickness AC overlay.	
			Rigid	Place an unbonded PCC overlay.	Use Table D.22 for thickness determination of an unbonded PCC overlay.

Table C. 2. Feasible Renewal Alternatives for Existing JPCP and JRCP Pavements.

Distress Category	Specific Distress Description	Distress Present?	Renewal Pavement Type Option	Action	Design Resources
Materials Caused Distress	D-Cracking with Light Severity	Yes	Flexible option for JPCP	Rubblization or crack and seat JPCP followed by a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Apply Rule 2.
			Flexible option for JRCP	Rubblization or saw, crack and seat JRCP with a thick overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Saw, crack and seat existing PCC followed by application of AC overlay from Tables D.37-D.39; otherwise, Rule 2 applies.
			Rigid option	Apply 2 inch AC overlay bond breaker followed by an unbonded PCC overlay.	Apply Rule 3 shown below.
		No	--	Continue to next level of D-Cracking.	--
	D-Cracking with Moderate to High Severity	Yes	Flexible option with rubblization if subgrade meets TTI guidelines	Rubblize followed by a thick AC overlay. For rubblization, apply TTI guidelines.	Apply Rule 1.
			Flexible option if does not meet TTI guidelines for rubblization	Do not use the existing pavement, requires all new pavement.	--
			Rigid option	Full depth patch and apply 2 inch AC overlay bond breaker followed by an unbonded overlay.	Apply Rule 3.
		No	--	Continue to ASR.	--
	Alkali-Silica Reactivity (ASR)	Yes	Flexible option	Rubblize followed by thick AC overlay. For rubblization, apply TTI guidelines.	Apply Rule 1.
			Rigid option	Patch plus 2 inch AC bond breaker followed by unbonded PCC overlay.	Apply Rule 3.
		No	--	Continue to Pavement Cracking.	--
Pavement Cracking	% Multiple Cracked Panels	Yes	Flexible option for low to moderate multiple cracked panels (1 to 10% of panels)	Rubblization or crack and seat JPCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
			Rigid option for low to moderate multiple cracked panels (1 to 10% of panels)	Place a 2 inch AC bond breaker followed by an unbonded PCC overlay.	Apple Rule 3.
			Flexible option for moderate to high multiple cracked panels (>10% of panels)	If subgrade meets or exceeds TTI criteria, apply rubblization followed by a thick AC overlay.	Apply Rule 1.
				If subgrade does not meet TTI criteria, options include crack and seat or do not use existing pavement.	Apply Rule 2
			Rigid option for moderate to high multiple cracked panels (>10% of panels)	Replace rocking or shattered slabs followed by a 2 inch AC overlay bond breaker followed by an unbonded PCC overlay.	Apply Rule 3.
		No	Continue to Joint Faulting (Table 2b)		

Table C. 3. Feasible Renewal Alternatives for Existing JPCP and JRCP Pavements (continued).

Distress Category	Specific Distress Description	Distress Present?	Renewal Pavement Type Option	Action	Design Resources
Joint Faulting	--	Yes	Flexible option for low faulting (< 0.25 inches)	Rubblization or crack and seat JPCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Apply Rule 2.
				Rubblization or saw, break and seat JRCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Saw, crack and seat existing PCC followed by application of AC overlay from Tables D.37-D.39; otherwise, Rule 2 applies.
			Rigid option for low faulting (< 0.25 inches)	Place a 2 inch AC overly followed by an unbonded PCC overlay.	Apply Rule 3.
		Yes	Flexible option for high faulting (> 0.25 inches)	Rubblization or crack and seat JPCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Apply Rule 2.
				Rubblization or saw, break and seat JRCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
					Saw, crack and seat existing PCC followed by application of AC overlay from Tables D.37-D.39; otherwise, Rule 2 applies.
			Rigid option for high faulting (> 0.25 inches)	Place a 2 inch AC overlay followed by an unbonded PCC overlay. If joint deflections > 40 mils (0.040 inches), then consider crack and seat JPCP or saw, break and seat JRCP to stabilize slabs.	Apply Rule 3.
		No	--	Continue to Pumping.	--
Pumping	--	Yes	Flexible	Crack and seat JPCP with a thick AC overlay if the drainage can be improved.	Apply Rule 2.
				Saw, crack and seat JRCP with a thick AC overlay if the drainage can be improved.	Saw, crack and seat existing PCC followed by application of AC overlay from Tables D.37-D.39; otherwise, Rule 2 applies.
				If drainage cannot be improved, then AC based renewal should not be used.	--
			Rigid	If joint deflections > 40 mils (0.040 inches), consider crack and seat followed by a 2 inch AC bond breaker followed by an unbonded PCC overlay. Drainage must be improved.	Apply Rule 3.
		No	--	--	--

Table C. 4. Feasible Renewal Alternatives for Existing CRCP Pavements.

Distress Category	Specific Distress Description	Distress Present?	Renewal Pavement Type Option	Action	Design Resources
Punchouts	--	Yes	Flexible option with ≤5 punchouts per mile	Repair all punchouts; place thick AC overlay to achieve a longer service life.	Apply AC overlay from Tables D37-D.39. The selection of the AC thickness is based on a drop-down menu of subgrade moduli = 5,000 psi, 10,000 psi, or 20,000 psi. The existing pavement shall be characterized by one of four possible moduli to select from: 30,000 psi, 50,000 psi, 75,000 psi, or 100,000 psi.
			Rigid option with ≤5 punchouts per mile	Repair major punchouts if slab load support in question. Follow repairs with a 2 inch AC bond breaker followed by an unbonded PCC overlay.	Apply Rule 3.
			Flexible option with >5 punchouts per mile	Rubblization of CRCP with a thick AC overlay. For rubblization, apply TTI guidelines. (Sebesta and Scullion, 2006)	Apply Rule 1.
			Rigid option with >5 punchouts per mile	Repair major punchouts if slab load support in question. Follow repairs with a 2 inch AC bond breaker followed by an unbonded PCC overlay.	Apply Rule 3.
		No	--	--	--

Table C. 5. Feasible Renewal Alternatives for Existing Composite Pavements.

Distress Category	Specific Distress Description	Distress Present?	Renewal Pavement Type Option	Action	Design Resources
Surface course in fair to poor condition	Can be a range of distress types. For the underlying PCC, these are mostly cracking related.	Yes	Flexible Option	Remove existing AC surface(s). Apply rubblization if meets TTI criteria.	Apply **Rule** 1.
				Remove existing AC surface(s). Use crack and seat or saw, crack and seat.	Following crack and seat or saw, crack and seat of existing PCC pavement, apply **Rule** 2.
			Rigid option	Place unbonded PCC overlay. If grade limits require, mill existing AC pavement.	Apply **Rule** 3.
Surface course in very poor condition	Can be a range of distress types. For the underlying PCC, these can include severe D-cracking, ASR, etc.	Yes	Flexible option	Remove and replace existing pavement structure.	--
			Rigid option	Place unbonded PCC overlay. If grade limits require, mill existing AC pavement.	Apply **Rule** 3.

APPENDIX D

DEVELOPMENT OF RIGID AND FLEXIBLE THICKNESS DESIGN TABLES

DEVELOPMENT OF RIGID AND FLEXIBLE RENEWAL THICKNESS DESIGN TABLES

RIGID RENEWAL THICKNESS DESIGN TABLE DEVELOPMENT

The rigid pavement “overlay” designs contained in the interactive software and design guidelines were developed by two separate design procedures—namely AASHTO 93 and the MEPDG v1.1 (released September 2009). Initial thicknesses were developed by use of AASHTO 93 as two layer systems. These layer thicknesses allowed the study team to assemble the initial logic flow for development of the R23 design guidelines. The eventual goal was to model the required PCC thickness as three layer systems. The MEPDG software was selected for this task due to its versatility and focus on long-lasting pavement design.

MEPDG

The MEPDG has numerous features and inputs that need to be addressed. The MEPDG has three levels of inputs and for this assessment Level 3 was used. Some of the required decisions and inputs are:

1. There are three major input types for the MEDPG: (1) Traffic, (2) Climate, and (3) Structure.
2. One pavement type was analyzed via the MEPDG which was **JPCP** with three distress/performance types: (1) joint faulting, (2) transverse cracking, and (3) IRI. The MEDPG inputs that follow are for JPCP only.
3. General Information required to define the analysis period and type of design
    a. Design life = **50 years**
    b. Construction month = **June**
    c. Traffic opening month = **July**
    d. Pavement type: **JPCP**
    e. Shoulder condition: **No tied shoulder**.
4. Climate
    a. Data used to interpolate for **Baltimore, Maryland** (Table D. 1)

Table D. 1. Location information for climate data.

	BALT-WASH INTL ARPT	RON-REAGAN INTL ARPT	WASH-DULLES INTL ARPT	YORK ARPT	NEW CASTLE CO ARPT	HDRTWN RGNL FLD APRT
Latitude (degrees)	39.1	38.52	38.56	39.55	39.4	39.43
Longitude (degrees)	-76.41	-77.02	-77.27	-76.52	-75.36	-77.44
Elevation (ft)	196	3	309	475	95	737
Dist from given location (mi)	0.0	28.0	44.2	52.7	67.3	67.7

5. Traffic
   a. General inputs for MEPDG (Table D. 2)

Table D. 2. General inputs.

Number of lanes in design direction:	2
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	100
Operational speed (mph):	60

b. Conversion of default load spectra (which was used to calculate performance for the various slab thicknesses) to equivalent ESALs (required for the R23 design guidelines) involved several steps. The following tables provide information on how this was done. The steps include:

i. The overall calculation of ESALs for a design life of 50 years is: ((ESALs/truck)(% of total truck traffic/vehicle class)/10 vehicle classes)(AADT/2)(365)($(1+i)^n$ -1)/i)) = Total ESALs. Where i = truck growth rate and n = 50 years.
ii. ESALs/truck by vehicle class is the key element for converting load spectra to ESALs. Table D. 3 shows a summary of ESALs/truck along with the percent of total truck traffic (from Table 2.4.9 (NCHRP, 2004b)).
iii. Table D. 4 through Table D.6 illustrate the needed information for detailed calculations to estimate ESALs/truck. Table D. 4 is from NCHRP (2004b) and shows the average number of axles per vehicle. Table D. 5 illustrates how default load spectra for Class 4 single axles are converted to ESALs/axle. ESALs/truck is then the sum of ESALs/axle x average number of axles per truck. Table D.6 is a summary of ESALs/axle for the various vehicle classes and axle types.
iv. Table D. 7 illustrates the level of daily truck traffic required to achieve the design ESALs used in the R23 design guidelines.

Table D. 3. Calculation process for converting load spectra to ESALs.

Vehicle Class	ESAL/truck[1]	% of Total Truck Traffic[2]
4	0.67	3.3
5	0.30	34.0
6	0.68	11.7
7	1.34	1.6
8	0.69	9.9
9	1.03	36.2
10	1.06	1.0
11	1.69	1.8
12	1.42	0.2
13	2.18	0.3

[1] ESAL/truck based on Level 3 default values from two sources; (1) Table 2.4.11

from NCHRP (2004b) “Suggested default values for the average number of single, tandem, and tridem axles per truck class, and (2) ESALs/axle calculated from MEPDG default axle load spectra (such as Tables 2.4.9 (single axles) and 2.4.10 (tandem axles) from NCHRP (2004b)). Refer to Table D. 4, Table D. 5, and Table D.6.

[2] Percentages for total truck traffic from Table 2.4.4 (NCHRP, 2004b) for TTC 9 (Intermediate light and single-trailer truck route).

Table D. 4. Average number of Single, tandem, tridem, and quad axles per truck. Based on LTPP data (from NCHRP, 2004b).

Vehicle Classification	Number of Axles per Truck			
	Singles	Tandems	Tridems	Quads
4	1.62	0.39	0	0
5	2.00	0	0	0
6	1.02	0.99	0	0
7	1.00	0.26	0.83	0
8	2.38	0.67	0	0
9	1.13	1.93	0	0
10	1.19	1.09	0.89	0
11	4.29	0.26	0.06	0
12	3.52	1.14	0.06	0
13	2.15	2.13	0.35	0

Table D. 5. Example data for conversion of single axle load distribution. Default values to ESAL/Axle for Vehicle Class 4.

Mean Axle Load (lbs)	ESAL/Axle[1]	Axle %[2]	Mean Axle Load (lbs)	ESAL/Axle[1]	Axle %[2]
3000	0.0008	1.80	22000	2.23	0.66
4000	0.0023	0.96	23000	2.66	0.56
5000	0.006	2.91	24000	3.16	0.37
6000	0.0123	3.99	25000	3.72	0.31
7000	0.0229	6.80	26000	4.35	0.18
8000	0.039	11.45	27000	5.06	0.18
9000	0.0625	11.28	28000	5.85	0.14
10000	0.095	11.04	29000	6.74	0.08
11000	0.139	9.86	30000	7.72	0.05
12000	0.198	8.53	31000	8.80	0.04
13000	0.272	7.32	32000	9.99	0.04
14000	0.366	5.55	33000	11.3	0.04
15000	0.482	4.23	34000	12.7	0.03
16000	0.624	3.11	35000	14.3	0.02
17000	0.80	2.54	36000	16.0	0.02
18000	1.00	1.98	37000	17.8	0.01
19000	1.24	1.53	38000	19.9	0.01

Mean Axle Load (lbs)	ESAL/Axle [1]	Axle %[2]	Mean Axle Load (lbs)	ESAL/Axle [1]	Axle %[2]
20000	1.52	1.19	39000	22.0	0.01
21000	1.85	1.16	40000	24.4	0.01

Σ(ESAL/Axle)(Axle%)[3]

[1] ESAL/Axle approximated with $(\text{Mean Axle Load}/18000)^4$

[2] Axle Percentages from Table 2.4.9 (NCHRP, 2004b)

[3] Σ [(ESAL/Axle)(Axle Percentage)] = 0.35 ESAL/Class 4 Axle

Table D.6. ESAL/Axle for all vehicle classes from default load spectra.

Vehicle Classification	Single Axle	Tandem Axle	Tridem Axle
4	0.35 (see example calculation in Table D. 5)	0.27	0
5	0.15	0.16	0
6	0.29	0.39	0
7	0.66	0.80	0.58
8	0.25	0.15	0
9	0.20	0.42	0
10	0.21	0.56	0.22
11	0.37	0.32	0.10
12	0.29	0.33	0.34
13	0.29	0.62	0.61

Table D. 7. Daily trucks to achieve design ESALs along with Level 3 default load spectra.

Average Annual Daily Trucks to achieve Design ESAL Level with Default Load Spectra (two-way)	ESALs (millions)
500	10
1,250	25
2,500	50
5,000	100
10,000	200

5. Analysis parameters--Performance criteria
   a. Reliability for terminal IRI, transverse cracking, and mean joint faulting = **90%.**
   b. Transverse slab cracking (JPCP, maximum allowable over the design period): Range is given as 10 to 45% of the slab (NCHRP, 2004). **Use 10%.**
   c. Transverse joint faulting (JPCP, upper limit over the design period), Range is given as 0.1 to 0.2 in. (NCHRP, 2004). **Used 0.1 and 0.2 in.**
   d. Smoothness range for terminal IRI is given as 150 to 250 inches/mile (NCHRP, 2004). **Used 170 inches/mile** (or 2.7 m/km which is the FHWA break point from “acceptable” to “not acceptable”). Please refer to Table D. 8 .

Table D. 8. FHWA smoothness criteria.

FHWA Ride Quality Terms	**All Functional Classifications**	
	IRI, m/km (inches/mile)	**PSR Rating**
Good	< 1.5 (95)	Good
Acceptable	≤ 2.7 (170)	Acceptable
Not Acceptable	> 2.7 (170)	Not Acceptable

i. Initial IRI (as-constructed smoothness): Range is given as 50-100 inches/mile (NCHRP, 2004). Use **60 inches/miles** (or about 1.0 m/km).

ii. Terminal IRI = **170 in./mi.**

6. Structure and Materials
   a. PCC/JPCP Properties (Layer 1). See Table D.9 through Table D.12.

Table D.9. General Properties.

General Properties	
PCC material	**JPCP**
Layer thickness (in):	**Varied**
Unit weight (pcf):	**150**
Poisson's ratio	**0.2**

Table D.10. Thermal Properties.

Thermal Properties	
Coefficient of thermal expansion (per F° x 10- 6):	**5.5**
Thermal conductivity (BTU/hr-ft-F°) :	**1.25 (see NCHRP, 2004a)**
Heat capacity (BTU/lb-F°):	**0.28 (see NCHRP, 2004a)**

Table D.11. Mixture Properties.

Mix Properties	
Cement type:	**Type II**
Cementitious material content (lb/yd^3):	**500 and 560[1]**
Water/cement ratio:	**0.42**
Aggregate type:	**Limestone**
PCC zero-stress temperature (F°)	**Derived**
Ultimate shrinkage at 40% R.H (microstrain)	**Derived**
Reversible shrinkage (% of ultimate shrinkage):	**50**
Time to develop 50% of ultimate shrinkage (days):	**35**
Curing method:	**Curing compound**

[1] A range of cementitious contents could be used. For example, Minnesota specifies a minimum cement content of 530 lb/CY, Missouri 560 lb/CY, and WSDOT 564 lb/CY (see R23 specification summary in Appendix E-4). The FHWA (2007) notes that Germany and the Netherlands specify a minimum content of 540 lb/CY. Austria uses 540 lb/CY for fix-form paving and 594 lb/CY for slip-form paving. Thus, 500 lb/CY represents a lower bound and 560 lb/CY is the middle of the range.

Table D.12. Strength Properties

Strength Properties	
Input level:	**Level 3**
28-day PCC modulus of rupture (psi):	**690**
28-day PCC compressive strength (psi):	**n/a**

b. Base Properties (Layer 2). Please refer to Table D.13 through Table D.16.

Table D.13. AC, General Properties.

Layer 2 -- Asphalt concrete	
Material type:	**Asphalt concrete**
General reference temperature (°F)	**70**
Layer thickness (in):	**10**
Poisson's Ratio	**0.35 (user entered)**
Erodibility index	**Erosion Resistant (Class 3)**
PCC-Base Interface	**Full friction contact**
Loss of full friction (age in months)	**361**

Table D.14. AC Volumetric Properties.

HMA Volumetric Properties as Built	
Effective binder content (%):	**11.6**
Air voids (%):	**7**
Total unit weight (pcf):	**150**

Table D.15. AC Mixture Properties.

Asphalt Mix	
Cumulative % Retained 3/4 inch sieve:	**0**
Cumulative % Retained 3/8 inch sieve:	**23**
Cumulative % Retained #4 sieve:	**40**
% Passing #200 sieve:	**6**

Table D.16. AC Binder Properties.

Asphalt Binder	
Option:	**Superpave binder grading**
A	**9.4610 (correlated)**
VTS:	**-3.1340 (correlated)**

c. Layer 3. Please refer to Table D.17 and Table D.18.

Table D.17. Subgrade Type.

Layer 3 -- A-6	
Unbound Material:	**A-6**
Thickness(in):	**12**

Table D.18. Subgrade Strength Properties.

Strength Properties	
Input Level:	**Level 3**
Analysis Type:	**Representative value (User Input Modulus)**
Poisson's ratio:	**0.35**
Coefficient of lateral pressure, Ko:	**0.5**
Modulus (input) (psi):	**5000**
Moisture Content(%):	**-9999**

d. Layer 4—Same as Layer 3 but thickness is semi-infinite.
e. All runs were done without tied shoulders.
f. Surface short-wave absorptivity: Ranges between 0 and 1 with 1 implying that all solar energy is absorbed by the pavement surface. **Use default = 0.85** (recommended by NCHRP (2004)). Ranges provided by the FHWA are included in Table D.19.

Table D.19. Surface Properties.

Material	Surface Shortwave Absorptivity
Weathered asphalt (gray)	0.80-0.90
Fresh asphalt (black)	0.90-0.98
Aged PCC layer	0.70-0.90

- g. JPCP Design Features: Input the following:
  - i. Slab thickness: Varies
  - ii. Permanent curl/warp effective temperature difference: **-10°F** (recommended by NCHRP (2004a)).
- h. Joint Design
  - i. Joint spacing: Fix as **15 ft.**
  - ii. Dowel transverse joints: Dowel diameter is **1.5 inches and dowel spacing should be 12 inches**.

7. Other considerations
   - a. Reliability for performance predictions (Figure D.1).
   - b. Figure D.2 through Figure D. 3 below show that the application of reliability shifts the predicted performance upward (in this case an illustration of slab cracking). Figure source: NCHRP, 2004a.

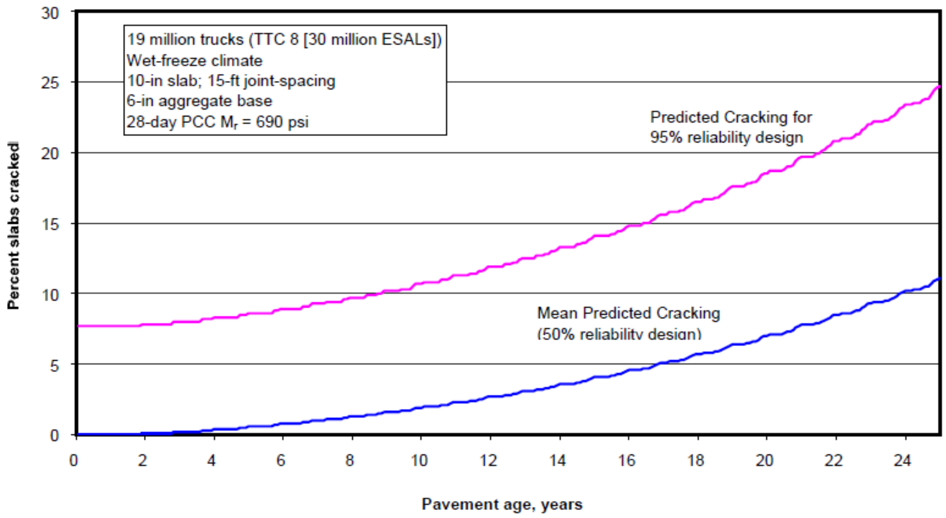


Figure D.1. Slab Cracking.

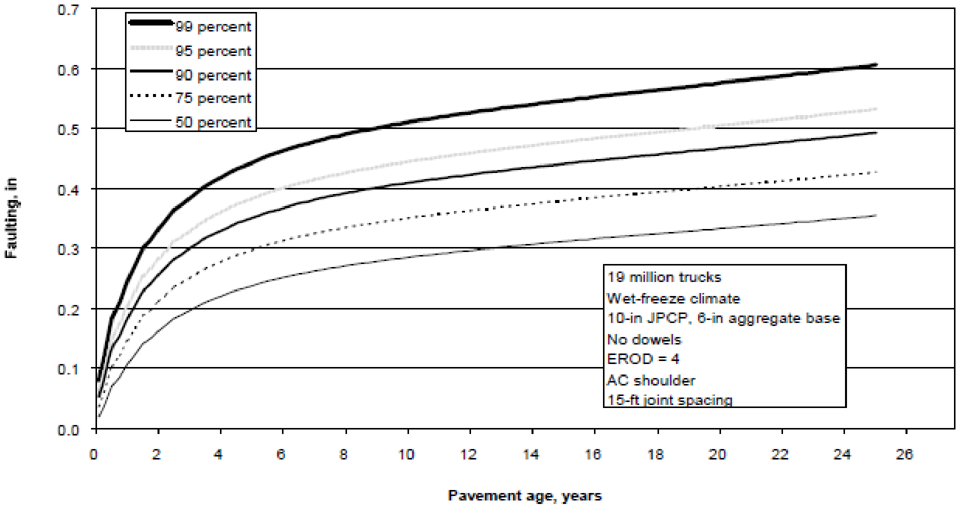


Figure D.2. Joint Faulting for No Dowel Condition.

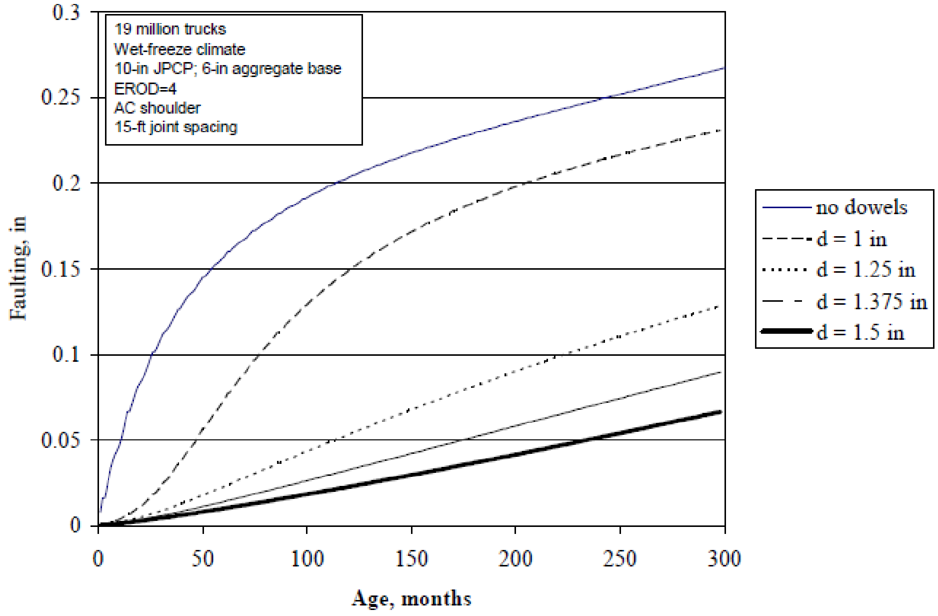


Figure D. 3. Joint Faulting Ranging from a No Dowel Condition up to Dowel Diameter = 1.5 in.

Trial Runs

The MEPDG runs are summarized in Table D. 20 and Table D. 21. The runs in Table D. 20 used a faulting limit of 0.1 in. Subsequently, an additional faulting level equal to 0.2 in. along with a higher cement content was examined due to the extreme slab thickness for weak subgrade (5,000 psi). The results from these additional runs produced the slab thicknesses shown in Table D. 21.

Table D. 20. Initial MEPDG runs with limiting joint faulting set = 0.1 in. and cement content = 500 lb/CY.

Traffic MESAL / AADTT	Performance Criteria	Subgrade Modulus											
		5,000 psi				10,000 psi				20,000 psi			
		PCC Depth	DP[1]	RP[2]	A[3]	PCC Depth	DP[1]	RP[2]	A[3]	PCC Depth	DP[1]	RP[2]	A[3]
10/ 500	Terminal IRI	8.25	82.6	99.88	Pass	7.75	80	99.94	Pass	7.75	80.2	99.94	Pass
	Transverse Cracking		2.1	92.12	Pass		0	99.96	Pass		1.7	93.79	Pass
	Mean Joint Faulting		0.017	99.95	Pass		0.016	99.97	Pass		0.013	99.99	Pass
25/ 1250	Terminal IRI	9.00	89.4	99.45	Pass	8.75	88.2	99.56	Pass	8.50	87.7	99.61	Pass
	Transverse Cracking		1.2	95.89	Pass		1.6	94.45	Pass		2.4	91.14	Pass
	Mean Joint Faulting		0.033	98.87	Pass		0.03	99.3	Pass		0.027	99.55	Pass

Traffic MESAL / AADTT	Performance Criteria	Subgrade Modulus											
		5,000 psi				10,000 psi				20,000 psi			
		PCC Depth	**DP[1]**	**RP[2]**	**A[3]**	**PCC Depth**	**DP[1]**	**RP[2]**	**A[3]**	**PCC Depth**	**DP[1]**	**RP[2]**	**A[3]**
10/ 500	Terminal IRI	8.25	82.6	99.88	Pass	7.75	80	99.94	Pass	7.75	80.2	99.94	Pass
	Transverse Cracking		2.1	92.12	Pass		0	99.96	Pass		1.7	93.79	Pass
	Mean Joint Faulting		0.017	99.95	Pass		0.016	99.97	Pass		0.013	99.99	Pass
50/ 2500	Terminal IRI	9.25	100.3	97.46	Pass	9.25	97.3	98.22	Pass	9.25	96.1	98.48	Pass
	Transverse Cracking		2	92.69	Pass		1.8	93.67	Pass		2.5	90.5	Pass
	Mean Joint Faulting		0.053	91.94	Pass		0.047	94.75	Pass		0.044	96.11	Pass
100/ 5000	Terminal IRI	12.25	99.1	97.77	Pass	12.00	97.9	98.06	Pass	11.50	99.2	97.73	Pass
	Transverse Cracking		0	99.96	Pass		0	99.79	Pass		0.2	99.24	Pass
	Mean Joint Faulting		0.056	90.03	Pass		0.053	91.6	Pass		0.055	90.4	Pass
200/ 10000	Terminal IRI	19.25*	108.4	94.53	Pass	15.5*	97.7	98.07	Pass	15*	98.2	97.95	Pass
	Transverse Cracking		0	99.96	Pass		0	99.96	Pass		0	99.96	Pass
	Mean Joint Faulting		**0.076**	**73.59**	**Fail**		0.055	90.65	Pass		0.056	90.22	Pass

[1]DP: Damage Prediction [2]RP: Reliability Prediction [3]A: Acceptable

*Mean Joint Faulting fails at 19.5 inches which is likely caused by load transfer (dowel) failure

Limiting Values: (1) Terminal IRI = 170 in./mi, (2) Transverse Cracking = 10%, (3) Mean Joint Faulting = 0.1 in.

AADTT = Average Annual Daily Truck Traffic

Table D. 21. Supplemental MEPDG runs with limiting joint faulting = 0.2 and cement content = 560 lb/CY

Traffic MESAL / AADTT	Performance Criteria	DT[1]	Subgrade Modulus											
			5,000 psi				10,000 psi				20,000 psi			
			PCC Depth	DP[2]	RP[3]	A[4]	**PCC Depth**	DP[2]	RP[3]	A[4]	**PCC Depth**	DP[2]	RP[3]	A[4]
200/ 10000 PCC% = 560lb/CY	Terminal IRI	170	11.75	116.9	90.30	Pass	11.25	117.1	90.19	Pass	11.25	116.9	90.30	Pass
	Transverse Cracking	10		0.1	99.72	Pass		0.5	98.26	Pass		1.5	94.93	Pass
	Mean Joint Faulting	0.2		0.089	99.72	Pass		0.089	99.74	Pass		0.087	99.79	Pass

[1]DT: Distress Target (Limiting Value) [2]DP: Damage Prediction [3]RP: Reliability Prediction [4]A: Acceptable

AADTT = Average Annual Daily Truck Traffic

Final Rigid Renewal Design Table

The final slab thicknesses selected for use in the R23 design guidelines are shown in the far right column in Table D. 22. Additional thicknesses are shown for: (1) AASHTO 93, and (2) WSDOT design thicknesses from their Pavement Policy document. The WSDOT pavement design tables were use because WSDOT had just developed those tables based on extensive MEPDG runs calibrated with detailed performance data from their PMS, Thus they were the best indicator of where other States may be in a couple of years using the MEPDG design procedures. The final slab thicknesses are a composite of all of these inputs.

Table D. 22. AASHTO 93, WSDOT, MEPDG and SHRP2 R23 rigid and design results.

ESALs (millions)	AASHTO 93 for k = 500 pci	Design Thicknesses from WSDOT Pavement Policy	Thickness Range for MEPDG for M_R = 5 to 10 ksi[1]	PCC Slab Thickness For R23 Study (inches)
≤ 10	10.0	9.0	7.75-8.25	**9.0**
10-25	11.5	10.0	8.75-9.0	**10.0**
25-50	12.5	11.0	9.25	**11.0**
50-100	14.0	12.0	11.5-12.25	**12.0**
100-200	15.5	13.0	11.25-15.5	**13.0**

[1] For ESALs = 200 million, results generated using both levels of PCC cement content a (500 and 560 lb/CY). Results from all other ESAL levels generated using one cement content (500 lb/CY).

FLEXIBLE RENEWAL THICKNESS DESIGN TABLE DEVELOPMENT

The flexible pavement "overlay" designs contained in the interactive software and design guides were developed by two separate design procedures—namely AASHTO 93 and PerRoad 3.5. The decision was made to exclusively apply PerRoad due to its improved versatility. The software was obtained from http://www.eng.auburn.edu/users/timmdav/Software.html. The newest version is PerRoad 3.5, dated April 2010.

Determine HMA Thicknesses

The pavement structures, as modeled, contained three layers, which were the HMA overlay over an existing processed layer (pulverized HMA, rubblized PCC, or crack and seat PCC) over subgrade.

The layer moduli for the processed layers were of special interest. A range of moduli were determined and summarized in Table D. 23.

Table D. 23. Layer Moduli Properties.

Material	**Description**	**Minimum Modulus (psi)**	**Maximum Modulus (psi)**	**Typical Modulus (psi)**
AC	Asphalt Concrete	50,000	4,000,000	
Cracked AC	Cracked Asphalt Concrete	50,000	500,000	
Pulverized HMA				40,000
PCC	Portland Cement Concrete	2,000,000	7,000,000	4,000,000
Rubb PCCP	Rubblized Concrete	40,000	700,000	150,000
Crack and Seat PCCP	Crack and Seated Concrete	200,000	800,000	200,000
Break and Seat PCCP	Break and Seated Concrete	250,000	2,000,000	
Granular Base	Granular Base	5,000	50,000	
Soil	Soil	3,000	40,000	
Rock	Bedrock	500,000	1,000,000	
Other	User Defined	50	10,000,00	

The ranges associated with each of these are rather wide and were considered in setting up the PerRoad runs. In order to achieve a conservative set of guidelines, the final selection of processed layer moduli were somewhat lower. The four moduli selected were (1) 30 ksi, (2) 50 ksi, (3) 75 ksi, and (4) 100 ksi. These moduli cover the lower end of the expected field moduli for the processed layers.

Three subgrade moduli were selected: (1) 5 ksi, (2) 10 ksi, and (3) 20 ksi. These moduli span the majority of subgrades encountered in the field.

The overall goal is to determine the HMA thickness that will achieve atarget value of either ≥ 10years or ≥ 50 years for D = 0.1 for the given inputs. In PerRoad, D = 0.1 (in lieu of the commonly used D = 1.0 for a damage function) is recommended by the developer of the software, David Timm. It reflects a conservative view for assessing high volume, long-life pavement designs. The 10 year criterion was a way for the study team to match a shorter span of time with the D = 0.1. Additionally:

1. One level of limiting horizontal tensile strain (fatigue endurance limit) at the bottom of the HMA was used: 100με.
2. One processed layer thickness was used which was 10 inches. Earlier work had applied two processed layer thicknesses, but the thinner of these was discarded as unrealistic.
3. The climate (temperatures) that directly influence the stiffness of the HMA were initially based on five cities:
    a. Minneapolis, MN (used in the example runs below with PG 64-34)

b. San Francisco, CA
c. Phoenix, AZ
d. Dallas, TX
e. Baltimore, MD

The results from the initial design runs indicated that the thickness values for San Francisco and Dallas fell within the range of values for the other three cities and did not affect the averages significantly. For that reason, San Francisco and Dallas were eliminated leaving Minneapolis, Phoenix, and Baltimore.

Seasonal temperature characterization was required for each location as shown in Table D. 24.

Table D. 24. Seasonal Properties.

City	**Overall Mean Temperature (°F)**	**Seasonal Duration (months and weeks) and Temperatures (°F)**		
Minneapolis	45°F	**Winter** Nov, Dec, Jan, Feb	17 weeks	21
		Spring Mar, Apr, May	13 weeks	45
		Summer June, July, Aug	13 weeks	70
		Fall Sept, Oct	9 weeks	56
Phoenix	70°F	**Winter** Dec, Jan, Feb	13 weeks	54
		Spring Mar, Apr, May	13 weeks	68
		Summer June, July, Aug, Sept	17 weeks	87
		Fall Oct, Nov	9 weeks	66
Baltimore	56°F	**Winter** Dec, Jan, Feb	13 weeks	35
		Spring Mar, Apr, May	13 weeks	54
		Summer June, July, Aug, Sept	17 weeks	74
		Fall Oct, Nov	9 weeks	53

Sources: (1) Pearce and Smith, "World Weather Guide," and (2) http://www.climatestations.com

Trial Runs

With a criterion for obtaining HMA thicknesses that results in a target value of ≥ 50 years for D = 0.1, selected cases were run. Since D=0.1 seemed extremely conservative it was decided to try HMA thicknesses that result in a value of ≥ 10 years for D = 0.1 as well. Note that ≥ 10 years for D = 0.1 is about the same as ≥ 50 years for D = 0.5 but years were easier to change in the program than D values. Note a damage ratio of D = 1.0 would predict full depth fatigue cracking in 50 years. All PerRoad runs are shown in Table D.25 through Table D.36.

Table D.25. Summary of PerRoad solutions for Subgrade = 5 ksi, Processed Existing Pavement = 30 ksi.

ESALs (millions)	**PerRoad Minneapolis 10 yr D = 0.1**	**PerRoad Phoenix 10 yr D = 0.1**	**PerRoad Baltimore 10 yr D = 0.1**	**PerRoad Baltimore 50 yr D = 0.1**	**R23 Selected Thickness**
≤ 10	10.5	12	10	12	**10**
10-25	12.5	13.5	11	12.5	**11**
25-50	13	14.5	11.5	12.5	**12**
50-100	13.5	15	12	13	**13**
100-200	14	15.5	12.5	13	**14**

Table D.26. Summary of PerRoad solutions for Subgrade = 10 ksi, Processed Existing Pavement = 30 ksi.

ESALs (millions)	**PerRoad Minneapolis 10 yr D = 0.1**	**PerRoad Phoenix 10 yr D = 0.1**	**PerRoad Baltimore 10 yr D = 0.1**	**PerRoad Baltimore 50 yr D = 0.1**	**R23 Selected Thickness**
≤ 10	10	11.5	9.5	11	**10**
10-25	11.5	13	10.5	11.5	**11**
25-50	12	13.5	11	12	**12**
50-100	12.5	14	11.5	12	**12**
100-200	13	14.5	11.5	12.5	**13**

Table D.27. Summary of PerRoad solutions for Subgrade = 20 ksi, Processed Existing Pavement = 30 ksi.

ESALs (millions)	**PerRoad Minneapolis 10 yr D = 0.1**	**PerRoad Phoenix 10 yr D = 0.1**	**PerRoad Baltimore 10 yr D = 0.1**	**PerRoad Baltimore 50 yr D = 0.1**	**R23 Selected Thickness**
≤ 10	9.5	11	9	10.5	**9.5**
10-25	11	12	10	11	**10**
25-50	11.5	13	10.5	11	**11**
50-100	12	13.5	10.5	11.5	**11.5**
100-200	12.5	13.5	11	11.5	**12**

Table D.28. Summary of PerRoad solutions for Subgrade = 5 ksi, Processed Existing Pavement = 50 ksi.

ESALs (millions)	**PerRoad Minneapolis 10 yr D = 0.1**	**PerRoad Phoenix 10 yr D = 0.1**	**PerRoad Baltimore 10 yr D = 0.1**	**PerRoad Baltimore 50 yr D = 0.1**	**R23 Selected Thickness**
≤ 10	9	10	8.5	10	**9**
10-25	10.5	11.5	9..5	10.5	**10**
25-50	11	12	10	11	**11**
50-100	11.5	12.5	10.5	11	**11.5**
100-200	12	13	10.5	11.5	**12**

Table D.29. Summary of PerRoad solutions for Subgrade = 10 ksi, Processed Existing Pavement = 50 ksi.

ESALs (millions)	**PerRoad Minneapolis 10 yr D = 0.1**	**PerRoad Phoenix 10 yr D = 0.1**	**PerRoad Baltimore 10 yr D = 0.1**	**PerRoad Baltimore 50 yr D = 0.1**	**R23 Selected Thickness**
≤ 10	8.5	9.5	7.5	9.5	**8**
10-25	9.5	10.5	9	10	**9**
25-50	10	11.5	9	10	**9.5**
50-100	10.5	12	9.5	10.5	**10**
100-200	11	12	10	10.5	**11**

Table D.30. Summary of PerRoad solutions for Subgrade = 20 ksi, Processed Existing Pavement = 50 ksi.

ESALs (millions)	**PerRoad Minneapolis 10 yr D = 0.1**	**PerRoad Phoenix 10 yr D = 0.1**	**PerRoad Baltimore 10 yr D = 0.1**	**PerRoad Baltimore 50 yr D = 0.1**	**R23 Selected Thickness**
≤ 10	8	9	7	8.5	**7.5**
10-25	9	10	8.5	9	**8.5**
25-50	9.5	10.5	8.5	9	**9**
50-100	10	11	9	9.5	**9.5**
100-200	10.5	11.5	9	9.5	**10**

Table D.31. Summary of PerRoad solutions for Subgrade = 5 ksi, Processed Existing Pavement = 75 ksi.

ESALs (millions)	PerRoad Minneapolis 10 yr D = 0.1	PerRoad Phoenix 10 yr D = 0.1	PerRoad Baltimore 10 yr D = 0.1	PerRoad Baltimore 50 yr D = 0.1	R23 Selected Thickness
≤ 10	7	8	7	8.5	**7.5**
10-25	8.5	9	8	8.5	**8.5**
25-50	9	9.5	8.5	9	**9**
50-100	9.5	10	8.5	9	**9.5**
100-200	10	10.5	9	9.5	**10**

Table D.32. Summary of PerRoad solutions for Subgrade = 10 ksi, Processed Existing Pavement = 75 ksi.

ESALs (millions)	PerRoad Minneapolis 10 yr D = 0.1	PerRoad Phoenix 10 yr D = 0.1	PerRoad Baltimore 10 yr D = 0.1	PerRoad Baltimore 50 yr D = 0.1	R23 Selected Thickness
≤ 10	6.5	7.5	6.5	8	**7**
10-25	8	8.5	7.5	8	**8**
25-50	8.5	9	7.5	8.5	**8.5**
50-100	8.5	9.5	8	8.5	**8.5**
100-200	9	9.5	8	8.5	**9**

Table D.33. Summary of PerRoad solutions for Subgrade = 20 ksi, Processed Existing Pavement = 75 ksi.

ESALs (millions)	PerRoad Minneapolis 10 yr D = 0.1	PerRoad Phoenix 10 yr D = 0.1	PerRoad Baltimore 10 yr D = 0.1	PerRoad Baltimore 50 yr D = 0.1	R23 Selected Thickness
≤ 10	6.5	7	6	7.5	**6.5**
10-25	7.5	8	7	7.5	**7**
25-50	8	8.5	7.5	8	**7.5**
50-100	8	9	7.5	8	**8**
100-200	8.5	9	8	8	**8.5**

Table D.34. Summary of PerRoad solutions for Subgrade = 5 ksi, Processed Existing Pavement = 100 ksi.

ESALs (millions)	PerRoad Minneapolis 10 yr D = 0.1	PerRoad Phoenix 10 yr D = 0.1	PerRoad Baltimore 10 yr D = 0.1	PerRoad Baltimore 50 yr D = 0.1	R23 Selected Thickness
≤ 10	5.5	6	5.5	7	**6**
10-25	6.5	7	6.5	7	**6.5**
25-50	7	7.5	7	7	**7**
50-100	7.5	7.5	7	7.5	**7.5**
100-200	7.5	8	7	7.5	**7.5**

Table D.35. Summary of PerRoad solutions for Subgrade = 10 ksi, Processed Existing Pavement = 100 ksi.

ESALs (millions)	PerRoad Minneapolis 10 yr D = 0.1	PerRoad Phoenix 10 yr D = 0.1	PerRoad Baltimore 10 yr D = 0.1	PerRoad Baltimore 50 yr D = 0.1	R23 Selected Thickness
≤ 10	5.5	6	5	6.5	**6**
10-25	6.5	6.5	6	6.5	**6.5**
25-50	6.5	7	6.5	7	**7**
50-100	7	7.5	6.5	7	**7**
100-200	7	7.5	7	7	**7**

Table D.36. Summary of PerRoad solutions for Subgrade = 20 ksi, Processed Existing Pavement = 100 ksi.

ESALs (millions)	PerRoad Minneapolis 10 yr D = 0.1	PerRoad Phoenix 10 yr D = 0.1	PerRoad Baltimore 10 yr D = 0.1	PerRoad Baltimore 50 yr D = 0.1	R23 Selected Thickness
≤ 10	5	6	5	6	**5.5**
10-25	6	6.5	6	6.5	**6**
25-50	6.5	7	6	6.5	**6.5**
50-100	6.5	7	6.5	6.5	**6.5**
100-200	7	7.5	6.5	7	**7**

Final Design Tables

The final flexible renewal thickness design tables were developed based on the numerous runs made with PerRoads, the MEPDG, and AASHTO 1993 design guidelines. Further refinements were made in consultations with State Highway Agency personnel and industry representatives. Table D.37 through Table D.39 provide details on the final thickness design recommendations.

Table D.37. Final Flexible Renewal thickness design table for Flexible Designs for Subgrade M_R = 5,000 psi.

ESALs (millions)	**Existing Pavement or Base Modulus**			
	30,000 psi	**50,000 psi**	**75,000 psi**	**100,000 psi**
≤10	10.0	9.0	8.0	6.0
10-25	11.0	10.0	8.5	6.5
25-50	12.0	11.0	9.0	7.0
50-100	13.0	11.5	9.5	7.5
100-200	14.0	12.0	10.0	7.5

Table D.38. Final Flexible Renewal thickness design table for Flexible Designs for Subgrade M_R = 10,000 psi.

ESALs (millions)				
	30,000 psi	**50,000 psi**	**75,000 psi**	**100,000 psi**
≤10	10.0	8.0	7.0	6.0
10-25	11.0	9.0	8.0	6.5
25-50	12.0	9.5	8.5	7.0
50-100	12.0	10.0	8.5	7.0
100-200	13.0	11.0	9.0	7.0

Table D.39. Final Flexible Renewal thickness design table for Flexible Designs for Subgrade M_R = 20,000 psi.

ESALs (millions)				
	30,000 psi	**50,000 psi**	**75,000 psi**	**100,000 psi**
≤10	9.5	7.5	6.5	5.5
10-25	10.0	8.5	7.0	6.0
25-50	11.0	9.0	7.5	6.5
50-100	11.5	9.5	8.0	6.5
100-200	12.0	10.0	8.5	7.0

REFERENCES

NCHRP (2004a), “Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures,” Part 3. Design Analysis, Chapter 4. Design of New and Reconstructed Rigid Pavements, National Cooperative Highway Research Program, Transportation Research Board, March 2004.

NCHRP (2004b), “Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures,” Part 2. Design Inputs, Chapter 4. Traffic, National Cooperative Highway Research Program, Transportation Research Board, March 2004.

FHWA (2006), “Geotechnical Aspects of Pavements Reference Manual, Chapter 5.0 Geotechnical Inputs for Pavement Design, 5.5 Thermo-Hydraulic Properties,” NHI-05-037: Geotechnical Aspects of Pavements, Federal Highway Administration, May 2006. http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/05d.cfm

FHWA (2007), “Long-Life Concrete Pavements in Europe and Canada,” Chapter 5 Cement and Concrete, International Programs, Federal Highway Administration, September 10, 2007.

Pearce, E.A., and C.G. Smith, “The World Weather Guide,” Second Edition, Hutchinson Publications, London, 1990.

Appendix E

Resource Documents

The knowledge base assembled as part of the guidelines includes six resource documents developed specifically for this project. In addition, a number of other resources developed under separate research efforts have been reference in this knowledge base. All of the resource documents developed as part of the study are provided in this appendix and are organized as outlined below:

- Pavement Assessment Manual (Appendix E-1)
- Flexible Pavement Best Practices (Appendix E-2)
- Rigid Pavement Best Practices (Appendix E-3)
- Guide Specifications (Appendix E-4)
- Life Cycle Cost Analysis (Appendix E-5)
- Emerging Pavement Technology (Appendix E-6)

Each of these documents is intended to be a stand-alone resource formatted to be used as an independent guideline, manual, or specification recommendation. These are provided in this appendix for documentation purposes without modifying the format to be an integral part of the final report.

APPENDIX E-1

PROJECT ASSESSMENT MANUAL

SHRP2 R23

Using Existing Pavement in Place and Achieving Long Life

Project Assessment Manual

July 28, 2011

Project Assessment Manual Topics

Project Assessment Manual Topics (continued)

Section 1
Introduction

1.1 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 needs 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 a 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.

1.2 How to Use the Manual

The manual is intended to compliment the design tools developed by the SHRP2 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 the SHRP2 R23 project.

1.3 Assessment Data Categories

The following 10 categories are contained in this manual:

- Pavement distress surveys
- Pavement rut depths and roughness
- Nondestructive Testing—Falling Weight Deflectometer
- Ground Penetrating Radar
- Pavement cores
- Dynamic Cone Penetrometer
- Subgrade soil sampling and tests
- Traffic Loads for Design
- Traffic impacts
- Life cycle assessment

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

1.4 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.

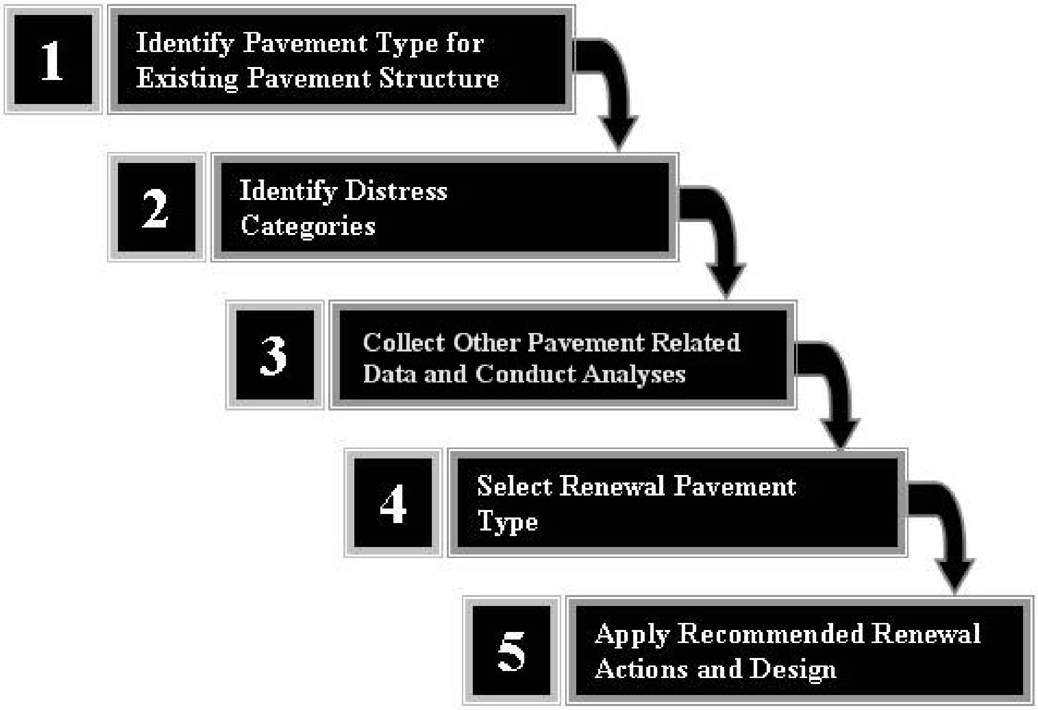


Figure 1.1 Outline of Assessment Scheme.

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).

Section 2
Pavement Distress Survey

2.1 Purpose

This section overviews the use of a pavement distress survey for aiding pavement assessment decisions.

2.2 Measurement Methods

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

(i) Pavement Distress Measurements: ASTM D6433-07 Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys.

(ii) Distress Identification Manual for the Long-Term Pavement Performance Program: FHWA-RD-03-031, June 2003.

(iii) 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 2.1. The majority of distress survey protocols use a subset of fracture, distortion, and/or disintegration.

Upon closer inspection of Table 2.1 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/raveling. Tables 2.2, 2.3, and 2.4 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 other 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 2.1 Distress Groups (after McCullough, 1971).

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

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). Can be a series of interconnected cracks in early stages of development. 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.
2. **Transverse cracking**: Cracks that are predominantly perpendicular to the pavement centerline.
3. **Stripping or raveling**: 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. This study expands the definition to identification of stripping/raveling in the surface layer to include stripping that may be occurring in lower HMA layers in the pavement structure. The depth of stripping can be verified by GPR analyses and/or coring as discussed in Sections 5 and 6 of this manual.

Rigid Pavement Distress for JPCP, JRCP, and CRCP (definitions from or modified after LTPP Distress Manual, Miller and Bellinger, 2003 with the exception of ASR cracking):

1. **Pavement Cracking**: Pavement cracking includes all major types of cracks that can occur in a slab. This can include corner breaks, 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 and transverse cracking are parallel and transverse to the centerline, respectfully.
2. **Joint Faulting**: Joint faulting is the difference in elevation across a joint or crack.
3. **Materials Caused Distress**: (1) D-Cracking: Closely spaced crescent-shaped hairline cracking pattern; occurs adjacent to joints, cracks, or free edges; dark coloring of the cracking pattern and surrounding area; sometimes referred to as durability cracking, and (2) Alkali-Silica Reactivity (ASR) Cracking: Cracking of the PCC that can be easily confused with D-cracking or shrinkage cracking. AASHTO has issued a Provisional Practice—AASHTO Designation PP 65-10—to address ASR. For complete details please reference the Best Practices for Rigid Pavements resource document.
4. **Pumping**: 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. **Punch-outs**: 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. Also includes "Y" cracks that exhibit spalling, breakup, or faulting.

2.2.1 Pavement Distress Data Templates

The templates for specific pavement distress types follow.

Table 2.2 Template for Flexible Pavement Distress—Fatigue Cracking.

Location (milepost)	Depth		Distress		
	HMA (in)	Base (in)	Fatigue Cracking		
			Severity[2]	Extent[1]	Depth of Fatigue Cracks[4] (measured from the pavement surface)
			Low		
			Moderate		
			High		

Notes:

1. Extent of fatigue cracking is based on % of wheelpath areas.
2. Severity of fatigue cracking is low, medium, and high. (1) **Low** = None or only a few connecting cracks; cracks are not spalled or sealed; pumping not evident, (2) **Moderate =** Interconnected cracks forming a complete pattern; cracks may be slightly spalled; cracks may be sealed; 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; pumping may be evident. The severity definitions are from the LTPP Distress Identification Manual (Miller and Bellinger, 2003).
3. Record extent for each level of severity.
4. Depth of fatigue cracks can be full depth or top down cracking. This should be determined by the use of pavement cores.

Low Severity
(Source: Pavement Interactive)

Moderate Severity
(Source: N. Jackson)

High Severity
(Source: Pavement Interactive)

Figure 2.1 Illustrations of Fatigue Cracking Severity Levels.

Table 2.3 Template for Flexible Pavement Distress—Transverse Cracking.

Location (milepost)	Depth		Distress		
	HMA (in)	Base (in)	Transverse Cracking		
			Severity[2]	Extent[1]	Depth of Transverse Cracks (measured from the pavement surface)
			Low		
			Moderat		
			High		

Notes:

1. Extent of transverse cracking is based on the number of cracks per 100 ft.
2. Severity of transverse cracking is low, medium, and 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).
3. Record extent for each level of severity.
4. Depth of fatigue cracks might be full depth of the HMA or top down cracking. This can only be determined by the use of pavement cores.

Moderate Severity
(Source: Pavement Interactive)

Moderate to High Severity
(Source: WSDOT)

High Severity
(Source: Pavement Interactive)

Figure 2.2 Illustrations of Transverse Cracking Severity Levels.

Table 2.4 Template for Flexible Pavement Distress—Stripping/Raveling.

Location (milepost)	**Depth**		**Distress**	
	HMA (in)	**Base (in)**	**Stripping/Raveling**	
			Extent (% of surface area)	**Full depth stripping/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 does not.
Note: Coring/GPR should be used to verify subsurface moisture damage.

Photo source: WSDOT

Figure 2.3 Illustration of Raveling.

Using Table 2.1 again, the most important Jointed Plain Concrete Pavement (JPCP) distress types that initiates PCCP 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 2.5, 2.6, 2.7, and 2.8. Tables 2.9 and 2.10 apply to CRCP and composite pavements.

Table 2.5 Template for Rigid Pavement Distress—JPCP or JRCP—Pavement Cracking.

Location (milepost)	**Depth**			**Distress**	
	PCC Slab (in)	**Base**		**Pavement or Slab Cracking**	
		Type[1]	**Thick (in)**	**% Slabs with Multiple Cracks[2]**	**Comments**

Notes
1. Three types of base underlying PCC: (1) Granular Base, (2) Cement Treated Base, or (3) Asphalt Treated Base.
2. Percentage of slabs with two or more pavement cracks.

Examples of PCC Slab Multiple Cracks
Photo sources: PI and J. Mahoney

Figure 2.4 Illustrations of PCC Slabs with Multiple Cracks.

Table 2.6 Template for Rigid Pavement Distress—JPCP or JRCP—Faulting.

Location (milepost)	Depth			Distress	
	PCC Slab (in)	Base		Faulting	
		Type[1]	Thick (in)	Average Fault Depth (in)	Comments

Note
1. Three types of base underlying PCC: (1) Granular Base, (2) Cement Treated Base, or (3) Asphalt Treated Base.

Average Fault ~ 0.25 to 0.5 in.
(Source: Pavement Interactive)

Average Fault ~ 0.5 in.
(Source: Pavement Interactive)

Figure 2.5 Illustrations of Various Levels of Joint Faulting.

Table 2.7 Template for Rigid Pavement Distress—D-Cracking.

Location (milepost)	Depth			Distress		
	PCC Slab (in)	Base		D-Cracking		
		Type[1]	Thick (in)	Severity[2]	Extent[3]	Comments
				Low		
				Moderate		
				High		

Notes

1. Three types of base underlying PCC: (1) Granular Base, (2) Cement Treated Base, or (3) Asphalt Treated Base.
2. Severity of D-cracking is low, medium (moderate), and 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.
3. Extent is based on the amount of cracks or joints that exhibit D-cracking. This definition of extent is different than used by LTPP.

Low Severity
(Source: PI and C.L. Monismith)

Low Severity
(Source: N. Jackson)

High Severity
(Source: N. Jackson)

Figure 2.6 Illustrations of D-Cracking Severity Levels.

Table 2.8 Template for Rigid Pavement Distress—ASR Cracking.

Location (milepost)	Depth			Distress	
	PCC Slab (in)	Base		ASR Related Cracking	
		Type[1]	Thick (in)	Does ASR Cracking Apply to this Pavement? Yes or No[2]	How was ASR detected or measured?

Notes

1. Three types of base underlying PCC: (1) Granular Base, (2) Cement Treated Base, or (3) Asphalt Treated Base.
2. Severity levels are not applicable for ASR. Either it exists or does not.

Early State of Cracking
(Source: N. Jackson)

Advanced Stage of Cracking
(Source: N. Jackson)

Figure 2.7 Illustrations of ASR Cracking Severity Levels.

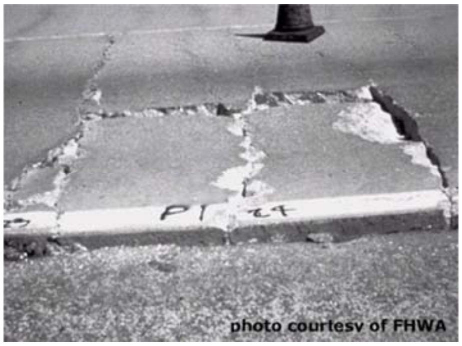
Table 2.9 applies to Continuously Reinforced Concrete Pavement (CRCP). A critical distress for CRCP is punch-outs (which falls under “fracture” in Table 2.1).

Table 2.9 Template for Rigid Pavement Distress—CRCP—Punch-outs.

Location (milepost)	**Depth**			**Distress**	
	PCC Slab (in)	**Base**		**Punch-outs**	
		Type[1]	**Thick (in)**	**No./mile**	**Comments**

Note

1. Three types of base underlying PCC: (1) Granular Base, (2) Cement Treated Base, or (3) Asphalt Treated Base.



Advanced Stage for a Punch-out
(Source: FHWA)

Figure 2.8 Illustration of a CRCP Punch-out.

Table 2.10 Composite Pavement Distress.[1]

Location (milepost)	Depth					Distress[4]	
	HMA Surfacing (in)	PCC				Describe condition of surface course	Comments
		PCC Type[2]	PCC Slab Thick (in.)	Base Type[3]	Base Thick (in)		
						Poor Condition	
						Very Poor Condition	

Notes:

1. Composite pavement definition assumes a flexible (HMA) layer overlies PCC.
2. Three types of PCC pavement: (1) JPCP, (2) JRCP, or (3) CRCP.
3. Three types of base underlying PCC: (1) Granular Base, (2) Cement Treated Base, or (3) Asphalt Treated Base.
4. 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 PCCP distress types can be important and such information collected and used; however, the distress types in the preceding tables were judged as the most critical for pavement renewal decision-making.

2.2.2 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 needed for the renewal design process:

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

2.3 Analysis Tools

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

2.4 References

McCullough, B.F. (1971), "Distress Mechanisms-General," Special Report No. 126, Highway Research Board, National Academy of Sciences, Washington, DC.

Miller, J.S. and Bellinger, W.Y. (2003), “Distress Identification Manual for the Long-Term Pavement Performance Program (Fourth Edition),” Report FHWA-RD-03-031, Office of Infrastructure Research and Development, Federal Highway Administration, McLean, Virginia, June 2003.

Stark, D. (1994), “Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures,” SHRP-C-315, Strategic Highway Research Program, Washington, DC, originally printed in 1994 but updated.
http://leadstates.transportation.org/asr/library/C315/index.stm#fore

Section 3
Pavement Rut Depth and Roughness (Profile)

3.1 Purpose

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

3.2 Measurement Methods

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

(i) Rut Depth Measurements

NCHRP Synthesis 334 (McGhee, 2004) notes that 46 State 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 3.1 helps to define lateral locations of a typical highway lane (from AASHTO, 2001). Figure 3.2 shows how rut depths are measured with automated equipment.

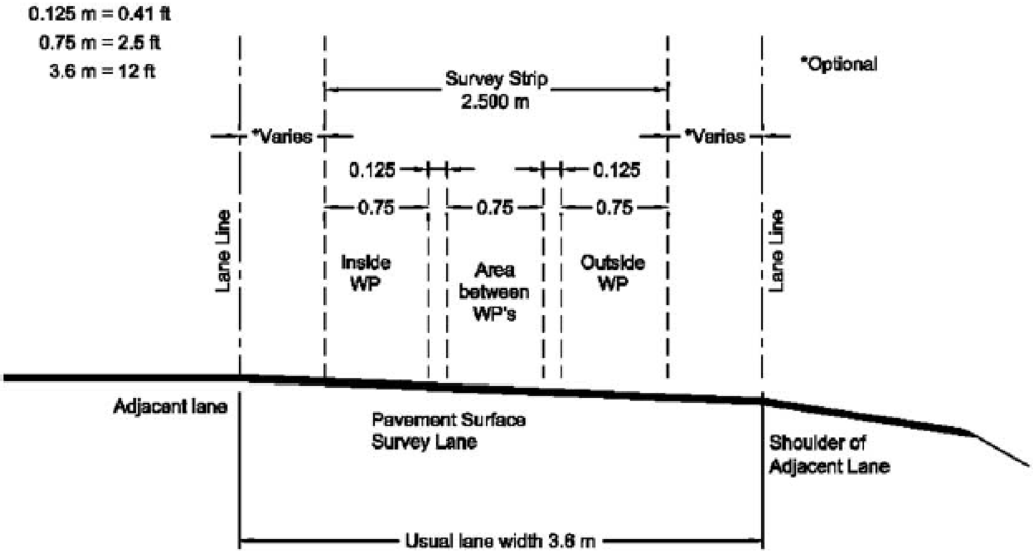


Figure 3.1 Sketch Illustrating wheelpaths and between wheelpaths (from McGhee, 2004 and AASHTO, 2001).

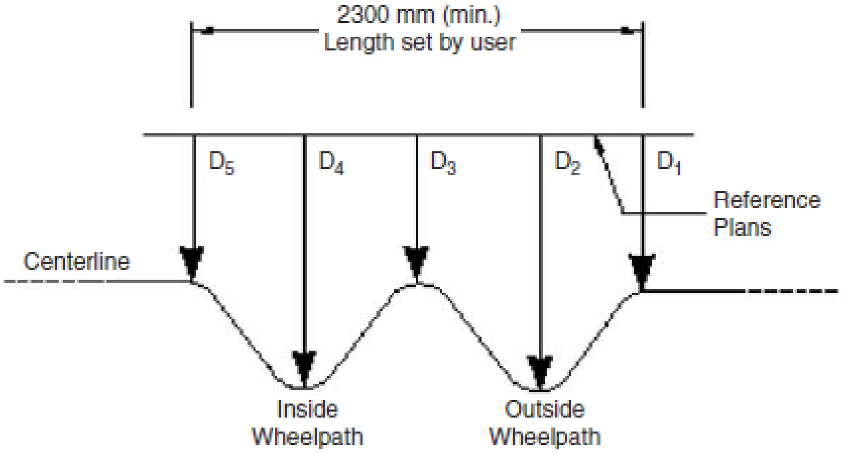


Figure 3.2 Rut Depth Measurements (from McGhee, 2004 and AASHTO, 2000).

(ii) 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." **ASTM E1926-08** (Standard Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements) defines 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 "IRI is reported in either meters per kilometer (m/km) or inches per mile (in/mile)."

3.3 Analysis Tools

Some of the analysis tools available include allowable rut depths and recommended IRI levels, which are shown in Tables 3.1, 3.2, and 3.3.

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

Table 3.1 Typical Maximum Rut Depths.

<table>
<tr><th>Pavement Type</th><th colspan="2">Maximum Rut Depth, inches (mm)</th></tr>
<tr><td>Texas DOT
[concern about hydroplaning]</td><td colspan="2">> 0.2
(5)</td></tr>
<tr><td>Wisconsin Hydroplaning Study
(Start et al, 1998)</td><td colspan="2">0.3
(7.6)</td></tr>
<tr><td>Washington State DOT</td><td colspan="2">0.4
(10)</td></tr>
<tr><td>Fwa (2006)
[based on hydroplaning]</td><td colspan="2">0.5
(12.5)</td></tr>
<tr><td rowspan="3">Shahin (1997)
[from the PAVER Asphalt Distress Manual—Pavement Distress Identification Guide for Asphalt-Surfaced Roads and Parking Lots]</td><td>Low</td><td>0.25 to 0.5
(6 to 13)</td></tr>
<tr><td>Medium</td><td>0.5 to 1.0
(13 to 25)</td></tr>
<tr><td>High</td><td>> 1.0
(> 25)</td></tr>
</table>

The IRI criteria used by the FHWA have evolved as illustrated by review of Tables 3.2 and 3.3. The most detailed breakdown was distributed by FHWA in 1999 and suggests that IRI values of less than 60 inches/mile are quite good while those greater than 170 inches/mile are poor. Interestingly, many newly paved HMA projects typically have IRI values close to the 60 inches/mile value. Eventually, the FHWA simplified their criteria as shown in Table 3.2.

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 inches/mile acceptable as to ride quality (85% acceptable). The paper concluded that that there was no evidence to change federal IRI guides (in essence those shown in Table 3.3).

Table 3.2 FHWA IRI Criteria (from FHWA, 2006).

<table>
<tr><th rowspan="2">Ride Quality Terms</th><th colspan="2">All Functional Classifications</th></tr>
<tr><th>IRI, inches/mi (m/km)</th><th>PSR Rating</th></tr>
<tr><td>Good</td><td>< 95
(< 1.5)</td><td>Good</td></tr>
<tr><td>Acceptable</td><td>≤ 170
(≤ 2.7)</td><td>Acceptable</td></tr>
<tr><td>Not Acceptable</td><td>> 170
(> 2.7)</td><td>Not Acceptable</td></tr>
</table>

Table 3.3 Earlier FHWA IRI Criteria (FHWA, 1999).

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

3.4 References

AASHTO (2000), “Standard Practice for Determining Maximum Rut Depth in Asphalt Pavements,” AASHTO Designation PP38-00, American Association of State Highway and Transportation Officials.

AASHTO (2001), “Standard Practice for Quantifying Cracks in Asphalt Pavement Surfaces,” AASHTO Designation PP44-01, American Association of State Highway and Transportation Officials, April 2001.

FHWA (1999), “1999 Status of the Nation’s Highways, bridges, and Transit: Conditions and Performance,” Report FHWA-PL-99-017, Federal Highway Administration, November 1999.

FHWA (2006), “2006 Status of the Nation’s Highways, Bridges, and Transit: Conditions and Performance,” Federal Highway Administration, http://www.fhwa.dot.gov/policy/2006cpr/chap3.htm

Fwa, T. (2006), The Handbook of Highway Engineering, Taylor and Francis Group, CRC Press.

McGhee, K. (2004), “Automated Pavement Distress Collection Techniques,” Synthesis 334, National Cooperative Highway Research Program, Transportation Research Board.

Shafizadeh, K. and Mannering, F. (2003), “Acceptability of Pavement Roughness on Urban Highways by Driving Public,” Transportation Research Record 1860, Transportation Research Board.

Shahin, M. (1997), “PAVER Distress Manual,” TR 97/104, US Army Construction Engineering Research Laboratories, Champaign, IL, June 1997.

Start, M., Jeong, K., and Berg, W. (1998), “Potential Safety Cost-Effectiveness of Treating Rutted Pavements, Transportation Research Record 1629, Transportation Research Board.

SHRP (1993), “Distress Identification Manual for the Long-Term Pavement Performance Project,” Strategic Highway Research Program, National Research Council.

Texas DOT (2009), “Hydroplaning,” Hydraulic Design Manual, Texas DOT, March 1, 2009.

Section 4
Nondestructive Testing via the Falling Weight Deflectometer

4.1 Purpose

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

4.2 Measurement Method

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

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

(i) ASTM D4694-96: Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device

(ii) ASTM D4695-03: Standard Guide for General Pavement Deflection Measurements

The significant features of ASTM D4694 include: (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 range of 20 to 60 ms with a rise time in 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 percent or ± 160 N (± 36 lb), whichever is greater, (7) the deflection measurements must be accurate to at least ± 2 percent or ± 2 µm (± 0.08 mils), whichever is greater. Note that 0.08 mils = 0.00008 inch and 2 µm = 0.002 mm, and (8) a precision guide in ASTM D4694 notes 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 percent. 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).

(iii) FWD

The FWD is the most widely used FWD in the US. 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 seconds. The standard load plate has a 300 mm (11.8 in.) diameter.

Locations for the seven to nine velocity transducers vary. From 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:

Distance from the center of the load plate (in.)
0
8
12
24
36
48

The Strategic Highway Research Program (SHRP) sensor spacing with the 11.8 in. load plate were:

Distance from the center of the load plate (in.)
0
8
12
18
24
36
60

4.3 Analysis Tools

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

4.3.1 Description of Available Analysis Tools for Flexible Pavements

This subsection will be used to describe three data assessment tools: (1) maximum deflection, (2) the Area Parameter, and (3) a simplified method for calculating 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 themselves 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 4.1).

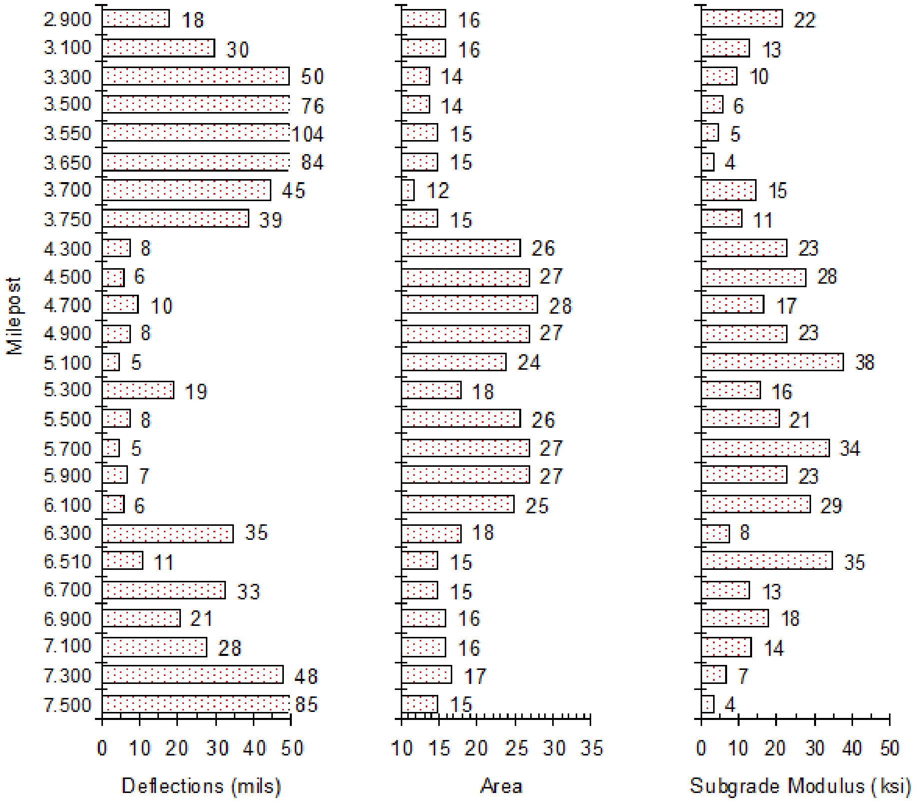


Figure 4.1 Illustrations of FWD Deflection Data Summarized by the Three Types of Data.

The deflection data in Figure 4.1 is “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 4.1 provides general information about conclusions that can be drawn from the FWD parameters of Area and D_0.

Table 4.1 General Information about the Area and D_0.

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

(i) Maximum pavement deflection (D_0)

The maximum pavement deflection can vary widely for different pavement structures and throughout the day as the temperature changes. D_0 ranges can be grouped into the following broad and approximate categories (Table 4.2):

Table 4.2 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

(ii) 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. By normalized, it is meant 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 center of test load,

D_1= surface deflection (mils) at 1 ft,

D_2= surface deflection (mils) at 2 ft, and

D_3= surface 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:
If, $D_0 = D_1 = D_2 = D_3$ then, Area = 6(1 + 2 + 2 + 1)/1=36.0 m = 36.0 in.

All four deflection measurements being equal (or nearly equal) indicates an extremely stiff pavement system (like portland cement concrete 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 of

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

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)/ = 11.1 in. Further, this value of Area 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 (this is not always a bad thing if the subgrade is extremely stiff). Typical Area values are shown in the Table 4.3.

Table 4.3 Typical Area Values.

Pavement Structure	Area Parameter (in.)
PCCP 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

(iii) Subgrade Modulus

An NCHRP study [Darter, et al, 1991], which revised Part III of the AASHTO Pavement Guide, recommended that the following equation be used to solve for subgrade modulus:

$$M_R = P(1 - \mu^2)/(\mu)(D_r)(r) \qquad \text{(Eq. 4.1)}$$

where M_R = <u>backcalculated</u> subgrade resilient modulus (psi),

P = applied load (lbs) from the FWD,

D_r = pavement surface deflection a distance r from the center of the load plate (inches), and

r = distance from center of load plate to D_r (inches).

Using a Poisson's ratio of 0.40, Equation 4.1 reduces to

$$M_R = 0.01114\ (P/D_2) \qquad \text{(Eq. 4.2)}$$

$$M_R = 0.00743\ (P/D_3) \qquad \text{(Eq. 4.3)}$$

$$M_R = 0.00557\ (P/D_4) \qquad \text{(Eq. 4.4)}$$

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

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

$$M_R = 0.01058(P/D_2) \qquad \text{(Eq. 4.5)}$$

$$M_R = 0.00705\ (P/D_3) \qquad \text{(Eq. 4.6)}$$

$$M_R = 0.00529\ (P/D_4) \qquad \text{(Eq. 4.7)}$$

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 (inches),

D = total thickness of pavement layers (inches)

E_P = effective pavement modulus (psi), and

M_R = backcalculated subgrade resilient modulus.

For "thin" pavements, $a_e \simeq$ 15 in. and for "medium" to "thick" pavements, $a_e \simeq$ 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 4.4 below (after Chou et al, 1989):

Table 4.4 Typical Subgrade Moduli.

Material	**Subgrade Moduli and Climate Condition**			
	Dry, psi	**Wet — No Freeze, psi**	**Wet - Freeze**	
			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

4.3.2 Example Analyses of FWD deflection basins for Flexible Pavement

The following deflection basins shown in Table 4.5 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 below:

Table 4.5 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 9000 lb.	**16.59**	**13.24**	**11.34**	**6.58**	**4.18**	**2.99**

The pavement structure at the time of FWD testing was:

- 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.

(i) Requirements

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

(ii) Results

Maximum surface deflection

The maximum surface deflection = 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)

$M_R = P(1-\mu^2)/(\pi)(D_r)(r)$

$= 9000(1-0.45^2)/(\pi)(0.00418)\ (36)$

$\cong 15{,}200$ psi

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

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

Area Parameter

$\text{Area} = 6(D_0 + 2D_{12''} + 2D_{24''} + D_{36''})/D_0$

$= 6(0.01659 + (2)(0.01134) + (2)(0.00658) + 0.00418)/0.01659$

$\cong 20.5$ in.

This Area Parameter is **low** for this thickness of AC. Thus, the Area value suggests a weak pavement structure but not extremely so.

(iii) More Detailed Project Data Example

Table 4.6 below summarizes deflection data that was collected on a portion of an actual project. The project was about 5 miles in length and FWD testing was performed every 250 feet, but only four of FWD locations are shown (these locations were also coring sites). The average pavement temperature at the time the FWD data was 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.

As shown in the Table 4.6, 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 to 5.3 inches with an average of 5.2 inches, which constitutes a “medium” thickness of HMA (refer back to Table 4.2).

The Area values shown in the table suggest weak HMA, but not necessarily extreme weakness due to stripping. Table 4.7 illustrates typical theoretical Area values for various uncracked HMA thicknesses, which aids this type of comparison.

Table 4.6 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**

Table 4.7 Typical Theoretical Area Values for Uncracked HMA.

HMA Thickness (in.)	Approximate Area Parameter (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 4.8.

Table 4.8 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, 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 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

4.3.3 Description of Available Analysis Tools for Rigid Pavements

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

(i) Load Transfer Efficiency

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

Joint performance can be evaluated by calculating load transfer efficiency (LTE) across a joint or crack using measured deflection data. The concept of joint load transfer efficiency is illustrated in Figure 4.2. Load transfer efficiency can be calculated using the following equation:

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

where LTE = load transfer efficiency, percent

Δ_U = the deflection of the unloaded slab, mils

Δ_L = the loaded slab deflection, mils.

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 4.3.

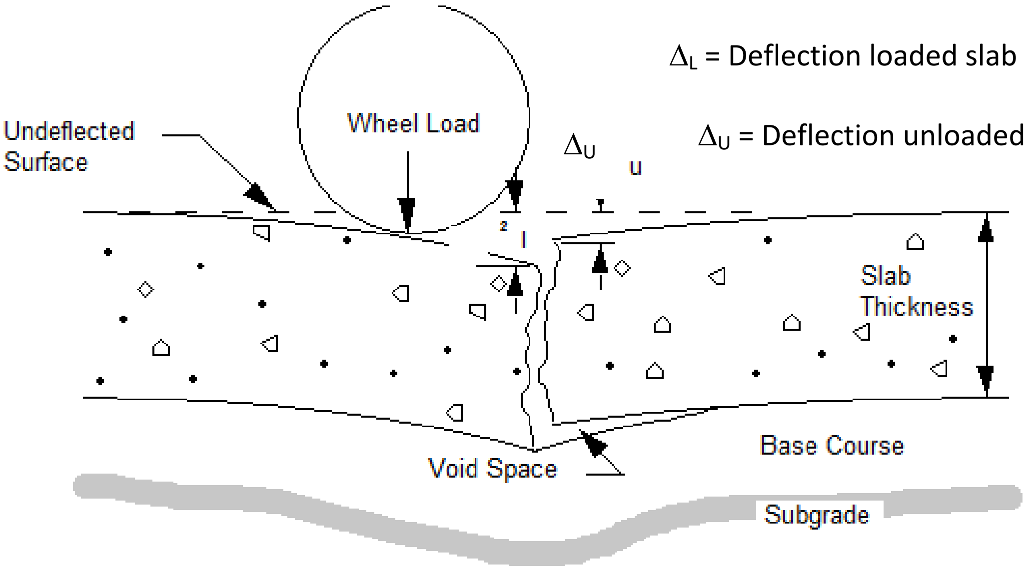


Figure 4.2 Illustration of Joint Load Transfer Efficiency. (NHI, 2003)

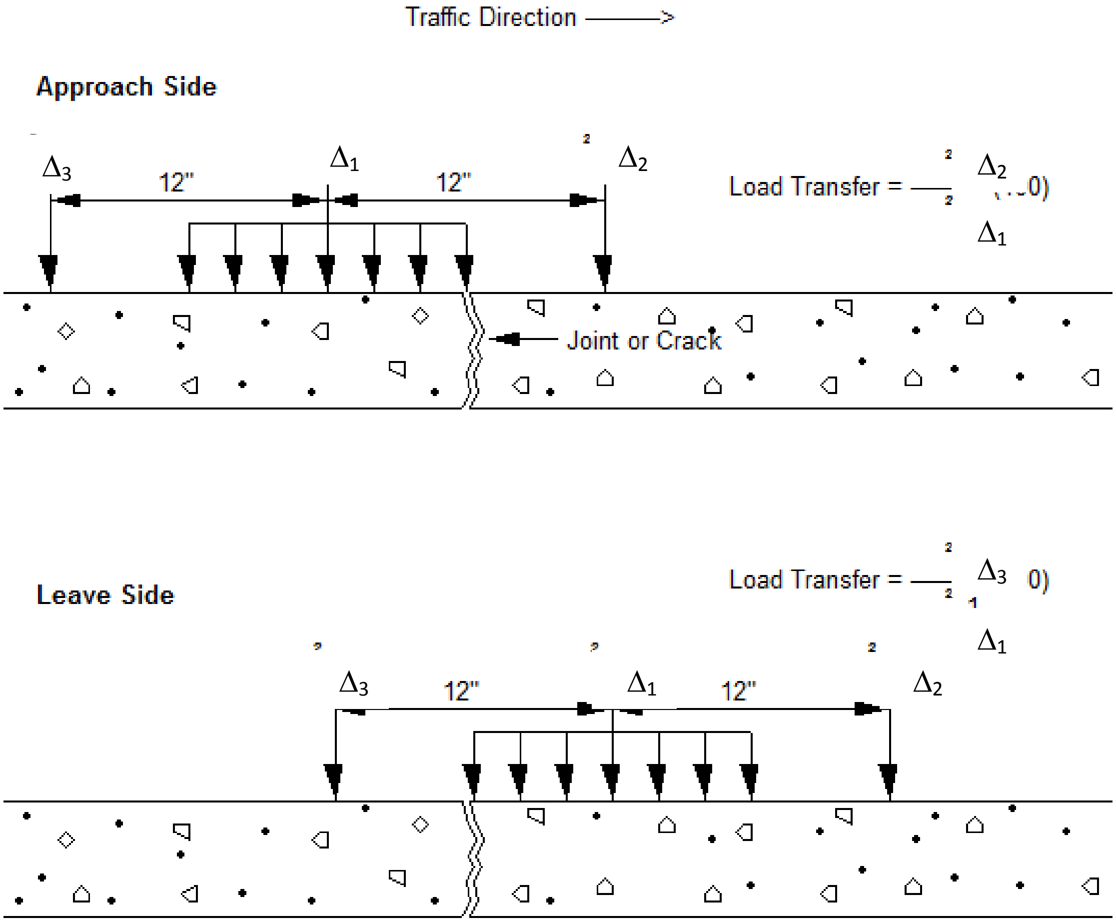


Figure 4.3 Locations of FWD Load Plate and Deflection Sensors for Determining Load Transfer Efficiency. (NHI, 2003))

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

4.3.4 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 US. It is likely that within a specific state, there is a preferred backcalculation software package to use.

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

(i) Number of Layers

Generally, use no more than 3 or 4 layers of unknown moduli in the backcalculation process (preferably, no more than 3 layers). If a three layer system is being evaluated, and questionable results are being produced (extremely weak base moduli, for example), then it is sometimes advantageous to evaluate this pavement structure as a two layer system. Some experienced users have found that there are times when dealing with a highly stress sensitive subgrade, it may be beneficial to consider adding layers to redue 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 not done so 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. For example, 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.

(ii) 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., 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 since one can quickly be attempting to backcalculate too many unknown layer moduli (i.e., greater than 3 or 4). 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/Subbase Course. A “thin” base course beneath “thick” surfacing layers (say HMA or PCC) often result in low base moduli. There are a number of reasons why this can occur. One, 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/subbase layers.

(iii) Subgrade

If unusually high subgrade moduli are calculated, check 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 feet below the pavement surface.

(iv) 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, makes 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 stiff layer at 20, 30, or 50 feet if no stiff layer is otherwise indicated (i.e., use a semi-infinite depth for the subgrade). The depth to 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 stiff layer.

The stiffness (modulus) of the stiff layer can vary. If the stiff layer is due to 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.

(v) Layer Moduli

A few comments about layer moduli are appropriate.

Cracked HMA Moduli. Generally, fatigue cracked HMA (about 10 percent 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 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 to 80°F).

Base and Subbase Moduli. Typical base and subbase moduli are shown in Table 4.9.

Table 4.9 Typical Unstabilized and Stabilized Base and Subbase Moduli.

<table>
<tr><th>Material</th><th colspan="2">Typical Modulus (psi)</th><th>Modulus Range (psi)</th></tr>
<tr><td colspan="4">Unstabilized</td></tr>
<tr><td>Crushed Stone or Gravel Base</td><td colspan="2">35,000</td><td>10,000 to 150,000</td></tr>
<tr><td>Crushed Stone or Grave Subbase</td><td colspan="2">30,000</td><td>10,000 to 100,000</td></tr>
<tr><td>Sand Base</td><td colspan="2">20,000</td><td>5,000 to 80,000</td></tr>
<tr><td>Sand Subbase</td><td colspan="2">15,000</td><td>5.000 to 80,000</td></tr>
<tr><td colspan="4">Stabilized</td></tr>
<tr><th>Material</th><th>Compressive Strength (psi)</th><th>Typical Modulus (psi)</th><th>Modulus Range (psi)</th></tr>
<tr><td rowspan="3">Lime Stabilized</td><td>< 250</td><td>30,000</td><td>5,000 to 100,000</td></tr>
<tr><td>250 to 500</td><td>50,000</td><td>15,000 to 150,000</td></tr>
<tr><td>> 500</td><td>70,000</td><td>20,000 to 200,000</td></tr>
<tr><td rowspan="3">Cement Stabilized</td><td>< 750</td><td>400,000</td><td>100,000 to 1,500,000</td></tr>
<tr><td>750 to 1250</td><td>1,000,000</td><td>200,000 to 3,000,000</td></tr>
<tr><td>> 1250</td><td>1,500,000</td><td>300,000 to 4,000,000</td></tr>
</table>

Subgrade Moduli. Typical subgrade moduli were previously shown in Table 4.5.

(vi) 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 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 on this powerful assessment technique.

A review 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.

4.4 References

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Chou, Y. J., Uzan, J., and Lytton, R. L., "Backcalculation of Layer Moduli from Nondestructive Pavement Deflection Data Using the Expert System Approach," Nondestructive Testing of Pavements and Backcalculation of Moduli , ASTM STP 1026, American Society for Testing and Materials, Philadelphia, 1989, pp. 341 - 354.

Darter, M.I., Elliott, R.P., and Hall, K.T., (1991) "Revision of AASHTO Pavement Overlay Design Procedure," Project 20-7/39, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., September 1991.

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Von Quintus, H.L. and Simpson, A.L. (2002), "Backcalculation of Layer Parameters for LTPP Test Sections, Volume II: Layered Elastic Analysis for Flexible and Rigid Sections," FHWA-RD-01-113, Federal Highway Administration, October 2002.

Yau, A. and Von Quintus, H.L. (2002), "Study of LTPP Laboratory Resilient Modulus Test Data and Response Characteristics," Report No. FHWA-RD-02-051, Federal Highway Administration, October 2002.

Section 5
Ground Penetrating Radar

5.1 Purpose

This section describes Ground Penetrating Radar (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.

5.2 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

(i) Air Coupled GPR systems

A typical commercially available 2.2 GHz air-coupled GPR unit is shown in Figure 5.1. The radar antenna is attached to a fiberglass boom and suspended about 5 feet from the vehicle and 14 inches 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 inches. All GPR systems include a distance measuring system and many of the new systems also have synchronized/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.

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

The disadvantages are a) the limited depth of penetration, b) not ideal for penetrating thick concrete pavements and c) the most popular operating frequency (1GHz) is now subject to FCC restrictions in the U.S.

Figure 5.1 Air Coupled GPR systems for highways. (Photo: Tom Scullion)

(ii) Ground Coupled GPR systems

As shown in Figure 5.2, a whole range of different operating frequencies are 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 to the pavement under test.

The advantage of these systems is their depth of penetration; several of the lower frequency systems can penetrate 20 feet under ideal conditions. The higher frequency systems are superior for many concrete pavement applications such as locating both reinforcing steel and sub-slab defects such as voids or trapped moisture. The disadvantage of these 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 5.2 Ground coupled systems, 1.5 GHz on left, lower frequency antennas with control unit on right. (Photos: Tom Scullion)

5.3 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 electro-magnetic (e-m) wave, and therefore obeys the laws governing reflection and transmission of e-m 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 feet 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 will be 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.

5.3.1 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 5.3, as a graph of volts versus arrival time in nanoseconds. To understand GPR signals, it is important to understand the significance of this plot.

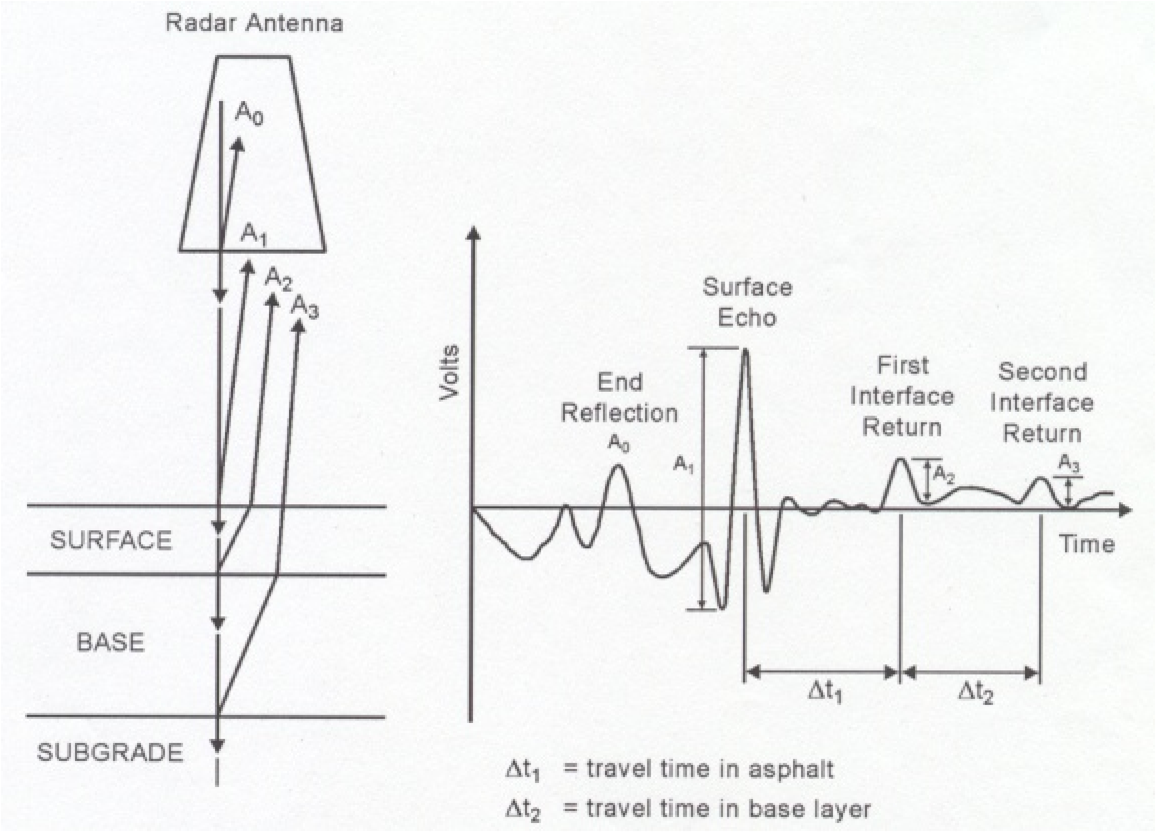


Figure 5.3 Captured GPR reflections from a typical flexible pavement.

The reflection, $A_{0,}$ is known as the end reflection and is internally generated system noise which will be 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 indicate 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 which is most influenced by moisture content and density; it also governs the speed at that 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:

$$\in_a = \left[\frac{1 + A_1 / A_m}{1 - A_1 / A_m} \right]^2 \qquad \text{(Eq 1)}$$

where ε_a = the dielectric of the surface layer,

A_1 = the amplitude of surface reflection, in volts

A_m = the amplitude of reflection from a large metal plate in volts (this represents the 100% reflection case, see Figure 5.1 for the metal plate test)

$$h_1 = \frac{cx\Delta t_1}{\sqrt{\in_a}} \qquad \text{(Eq 2)}$$

Where h_1 = the thickness of the top layer

c = a constant speed of e-m wave in air (5.9 ins/ns two way travel)

Δt_1 = the time delay between peaks A_1 and A_2, (in 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 getting a table of layer properties.

In most GPR projects, several thousand GPR traces like Figure 5.3 are collected. In order 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 below in Figure 5.4.

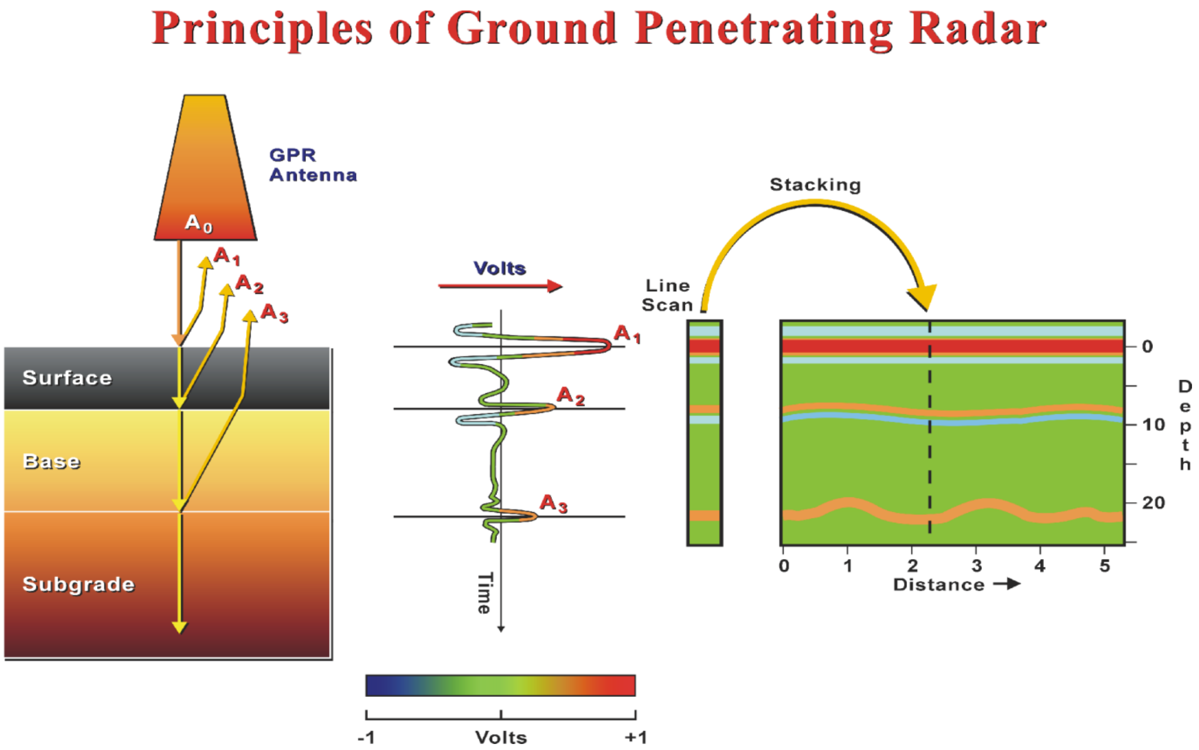


Figure 5.4 Color Coding and stacking individual GPR images. (Credit: Tom Scullion)

The raw GPR image collection is displayed vertically in the middle of Figure 5.4. 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 5.4. 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 3000 ft by 24 inches deep of a thick flexible pavement is shown in Figure 5.5. 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.

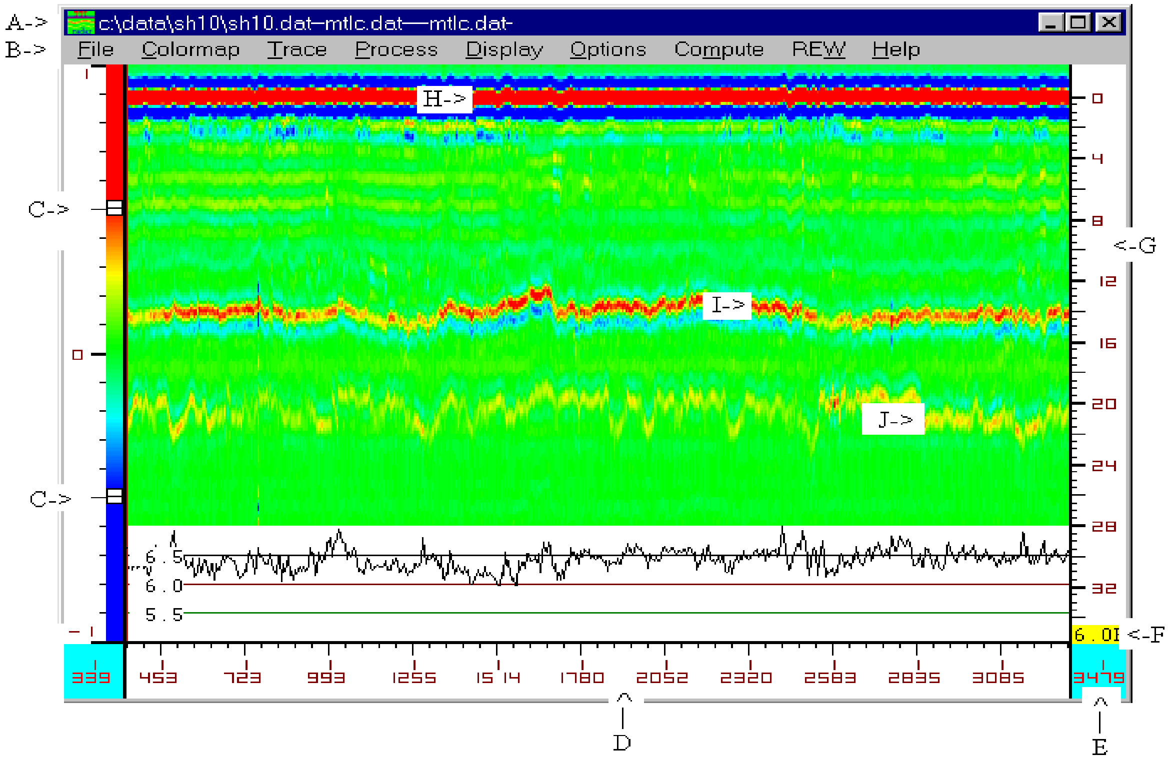


Figure 5.5 Color-Coded GPR Traces.

The labels on Figure 5.5 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 (1mile and 3479 feet), G) depth scale in inches, with the zero (0) being the surface of the pavement, 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 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 5.5. 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/white coding scheme is selected. This is widely used when reviewing data collected with ground coupled GPR systems. An example of the grey scale for the pavement shown in Figure 5.5 is shown below in Figure 5.6.

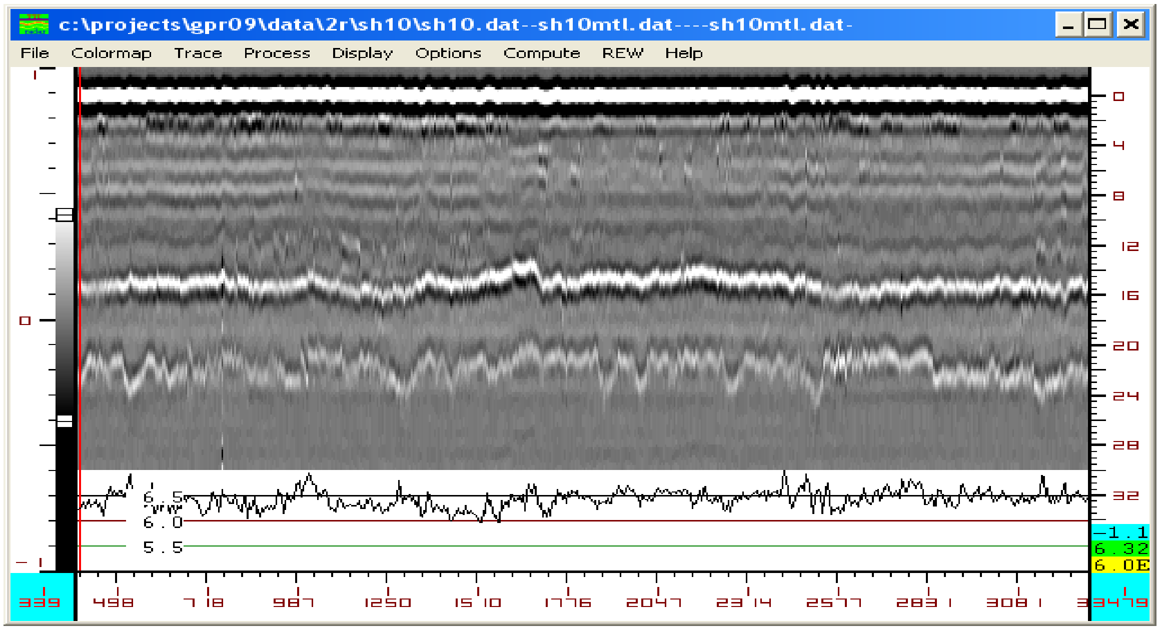


Figure 5.6 Similar data to Figure 5.5 presented as a grey scale.

All of the commercially available software packages produce both a color display of subsurface condition, such as Figures 5.5 and 5.6, together with a table of computed layer thicknesses and dielectrics that is usually exported to Excel. A typical table is shown below in Figure 5.7, where Thick 1 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 5.7 Tabulated thicknesses and dielectric values from GPR data.

5.3.2 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. Lab 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 as described in Section 4 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.

(i) 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 5.8 shows variations in asphalt layer thickness for an FDR candidate. At the sample location the structure was 5 inches of asphalt and 10 inches of granular base. Based on lab test results, the plan was to recycle to a depth of 10 inches blending 50 percent asphalt and 50 percent existing base with 3 percent cement. However, from a review of Figure 5.8, the average 5 inches of HMA has several noticeable exceptions. The first 800 feet only has 3 inches of asphalt, which is not thought to be a concern. However, for about 2000 feet, the total HMA thickness is over 12 inches. From previous experience, the 3 percent cement treatment does not work with 100 percent RAP. In these locations it was necessary to modify the construction plan, wherein 5 inches of the existing HMA was milled and replace with 5 inches 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.

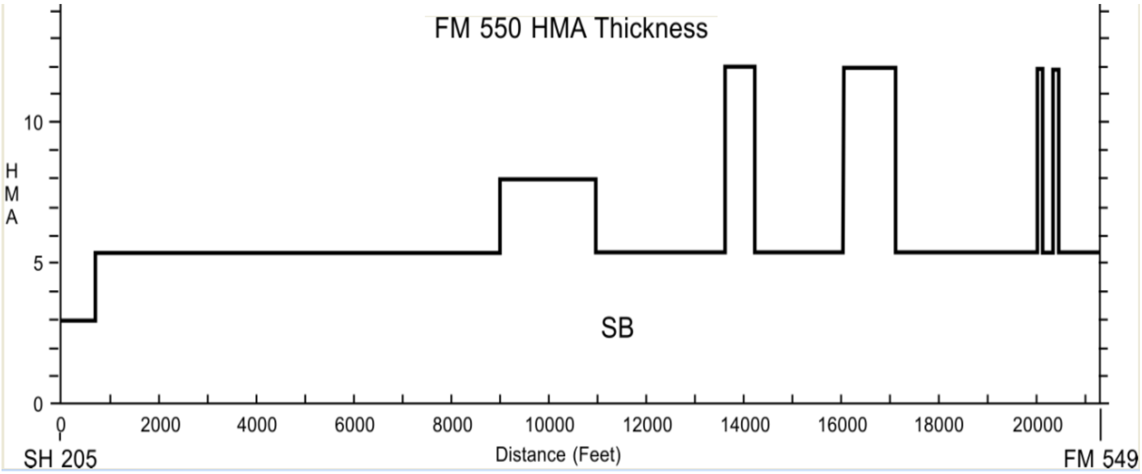


Figure 5.8 Surface thickness variations from GPR profiling on FM 550.

(ii) 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 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 – 3 feet intervals so very precise location of defects is possible. The GPR color coded profile shown in Figure 5.5 is from a thick HMA section with no defects. This should be contrasted with GPR profile shown below in Figure 5.9. This again is a thick HMA section, but in this case there are strong reflections from within the HMA and very strong reflections from the bottom of the layer. The red/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 impact long term performance.

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

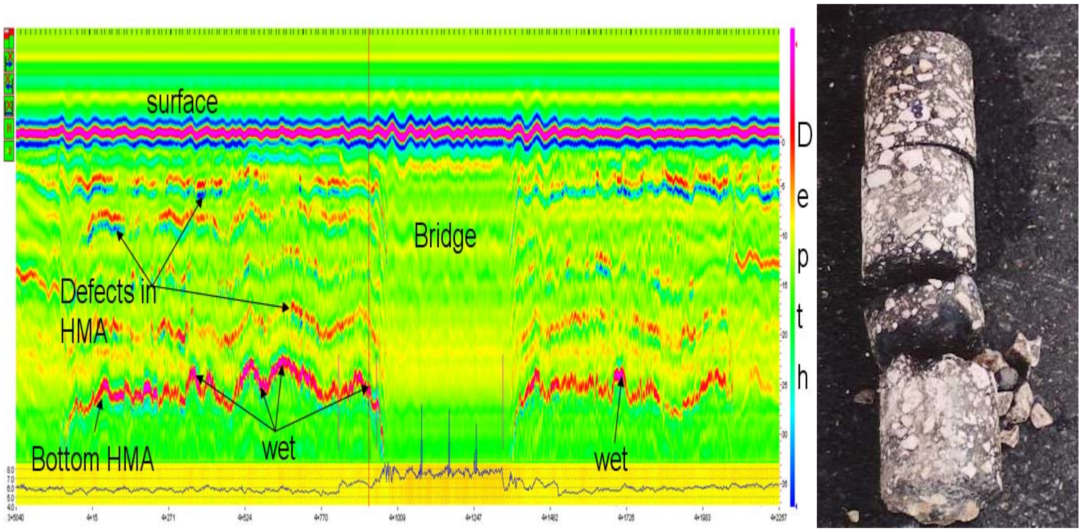


Figure 5.9 Using GPR to identify defects in surface and base layers.

(iii) 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 below in Figure 5.10. This is a 1.8 mile 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.

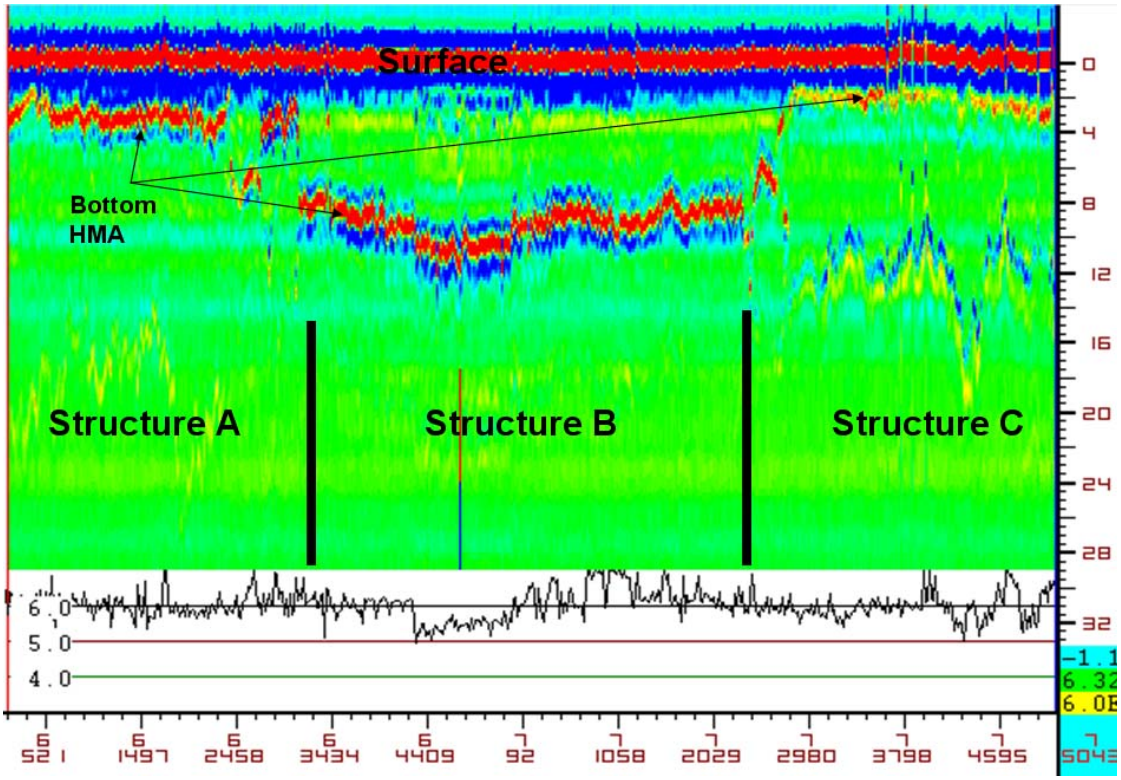


Figure 5.10 Using GPR to map subsurface variability.

5.3.3 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 5.2, can give more focus and better target resolution than air coupled units. Several case studies are shown below.

(i) Rebar detection

The GSSI handbook on Radar Inspection of GPR has some very good examples on rebar detection. The figure below 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 concrete cover. GSSI also claims that the size of the rebar can be determined by the shape of the hyperbola.

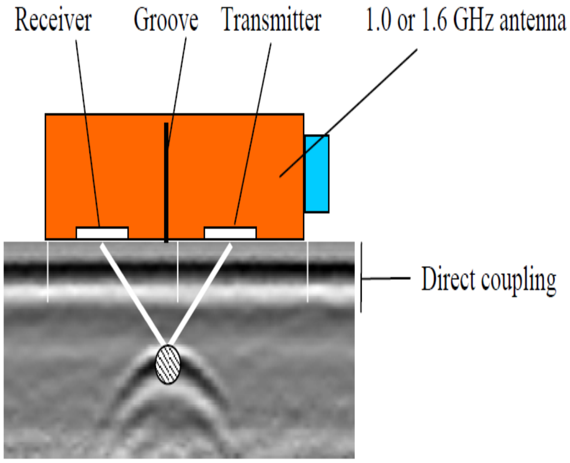


Figure 5.11 Ground Coupled GPR signals from steel in concrete (GSSI).

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 5.12

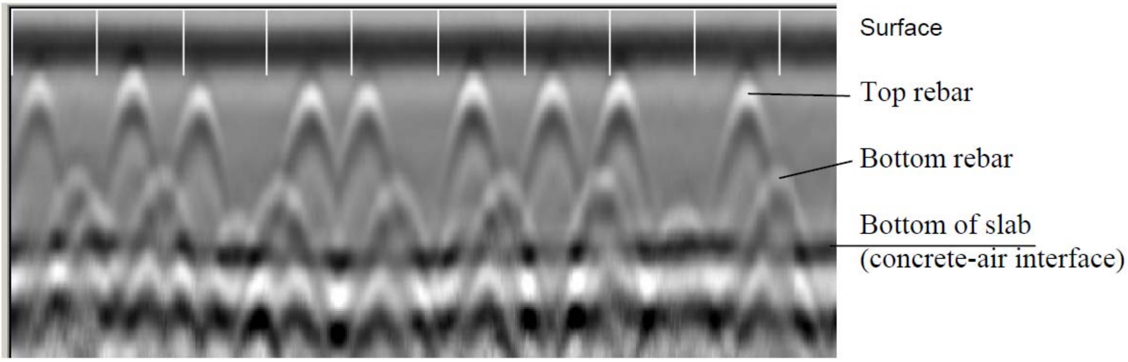


Figure 5.12 Mapping multiple layers of steel in concrete (GSSI).

(ii) 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 of less than 0.75 inches 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 inch PCC slab with water filled voids is shown below in Figure 5.13. The strong reflections (red areas) are locations of trapped water.

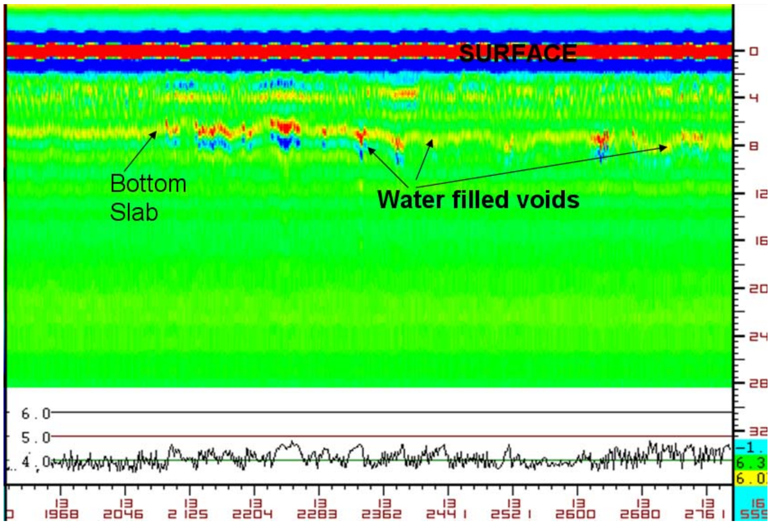


Figure 5.13 Mapping sub-slab water filled voids with GPR.

(iii) Deep Investigations of sub slab conditions with GPR

The 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 5.14 shows the color profile from a 400 MHz ground coupled system. The entire pavement system and changes in pavement support can be observed. The transverse rebar can be seen towards 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.

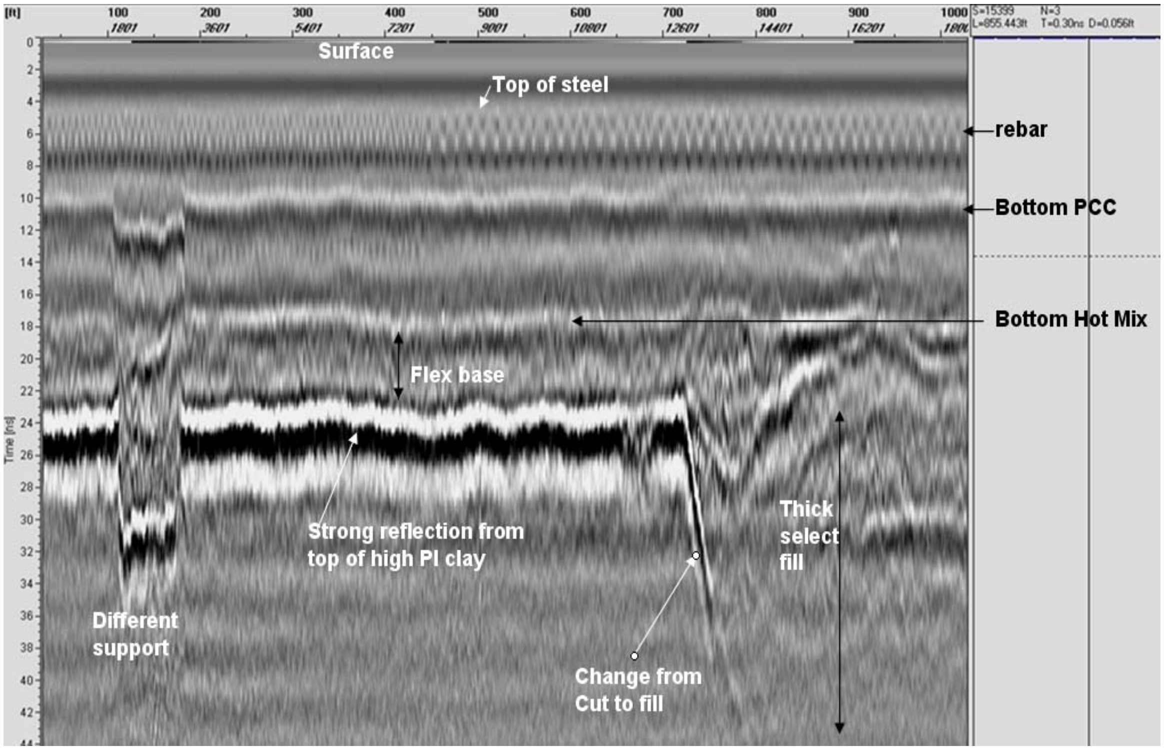


Figure 5.14 Mapping concrete pavements structure with GPR.

5.4 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 these will be discussed in this section. The main factors are:

1)The FCC ban on 1 GHz air coupled systems in 2002 (these units can be purchased in any country worldwide except the US). For the past decade most air coupled GPR systems have been performed with systems built before 2002. Only recently have commercial systems become available such as GSSI's 2.2 GHz as shown in Figure 1.
2)A lack of understanding about what GPR can and cannot do, in many cases the technology was oversold.
3) Inadequate data processing software and 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; these include:

1)Need for GPR hardware specifications
2)Need for data collection software specifications
3)Training/Specifications for data collection activities
4)Specifications/Software for processing and interpreting GPR signals
5)End User training
6)Specifications for output formats and data storage system

Several DOTs have implemented GPR technology in-house (for example FDOT and TxDOT), but most agencies get GPR services from consultant companies. Selecting the best vendor can also be a problem.

(i) 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:

a) The consultant has quality equipment. Ask them to run their equipment against the performance specs (which are available), and
b) 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.

(ii) 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 are:

a) *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 that a thickness measuring tool. It provides information on the quality of existing structures and helps to 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.

b) 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.

c) GPR is a black box which is impossible to understand
Not true. The basics of GPR are very 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.

d) 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.

e) *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.

5.5 References

GSSI Handbook for Radar Inspection of Concrete, Aug 2006, www.geophysical.com

Section 6
Pavement Cores

6.1 Purpose

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

6.2 Measurement Method

This subsection briefly overviews both the frequency of sampling and 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 inch is essential in assuring 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 Section 4) provide the good guidance for determining core locations because substantial changes in the pavement structure can be identified. If GPR data is collected, using the layer profiles in conjunction with FWD Area values also provide very good guidance for developing coring plans. Calibration cores for GPR data collection can also be utilized for other assessments.

Typical core diameters are either 4 or 6 inches. 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.

6.3 Analysis Tools

This subsection will focus on how to organize pavement core data to aid decision-making.

Core data should be organized similarly to the example data shown in Table 6.1 below. Additionally, the location of each core in the lane should be recorded (such as centerline, left wheelpath, between wheelpath, right wheelpath, outside pavement edge).

Table 6.1 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

Section 7
Dynamic Cone Penetrometer

7.1 Purpose

This section overviews the Dynamic Cone Penetrometer (DCP) and how it can be used to aid pavement assessment decisions.

7.2 Measurement Method

This subsection describes the dynamic cone penetrometer device. The standard test method is:

(iii) 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."

A sketch of a standard DCP is shown in Figure 7.1.

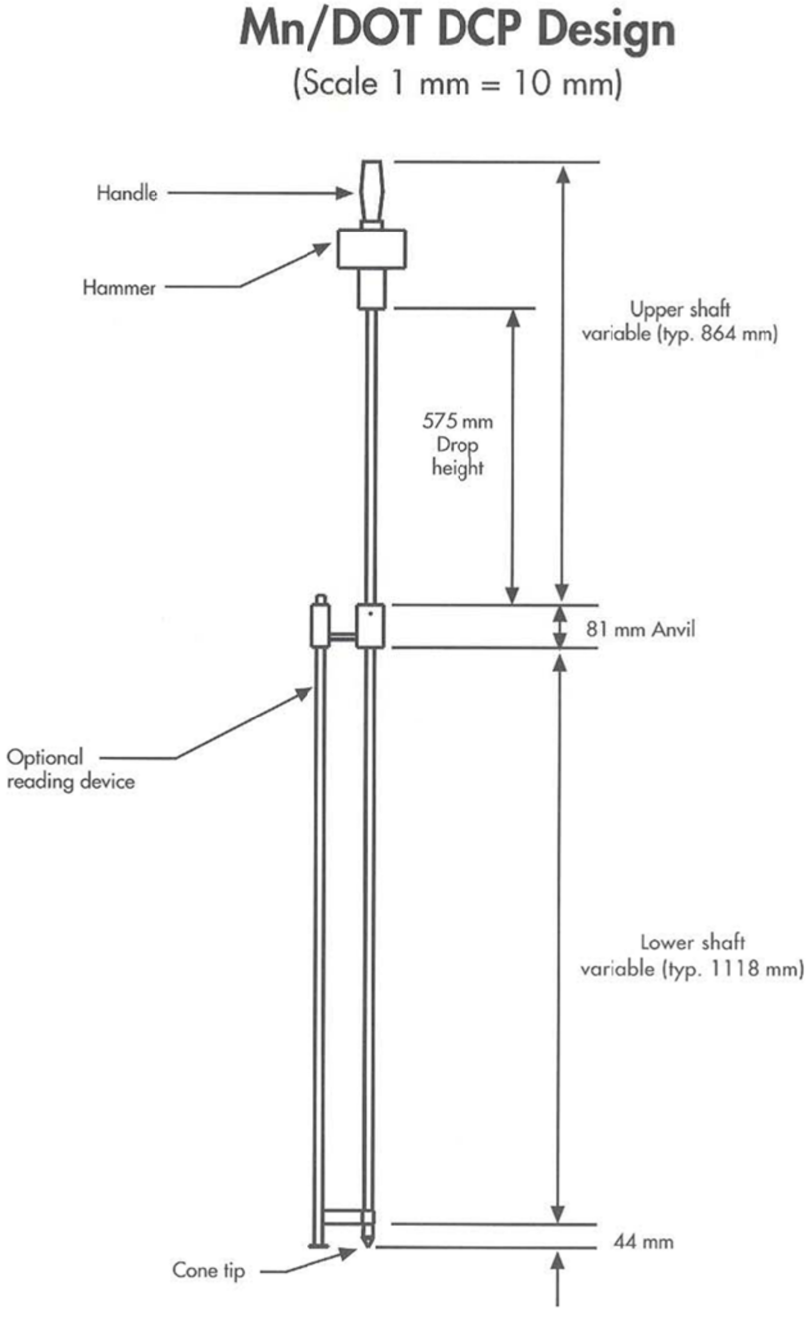


Figure 7.1 Sketch of the Minnesota DOT DCP (from Minnesota DOT, 1993).

7.3 Analysis Tools

DCP test results are typically expressed in terms of 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 inches/hammer blow (Minnesota DOT, 1993).

(i) Basic correlation

A common correlation with DCP data is to estimate the California Bearing Ratio (CBR) of unstabilized materials in a pavement structure. A correlation developed by the U.S. Army Corps of Engineers (Webster et al, 1992) is:

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

where DPI = mm/blow

Table 7.1 shows typical CBR and DPI ranges for three soil types (Minnesota DOT, 1993):

Table 7.1 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

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

(ii) 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 7.2. Following this work, the following DPI limits were recommended for use by MnDOT personnel when analyzing DCP results for rehab studies.

Silty/clay materials: DPI < 25 mm/blow
Select granular materials: DPI < 7 mm/blow
Class 3 Special gradation materials: DPI < 5 mm/blow

Table 7.2 Minnesota DCP Results Following Placement of the Base Course.

Material	**DPI Avg (mm/blow) (Std Dev) 0-12 inches depth**	**DPI Avg (mm/blow) (Std Dev) 12-24 inches depth**	**DPI Avg (mm/blow) (Std Dev) 24-36 inches 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)

[DPI average values were rounded to the nearest whole number.]

(iii) Subgrade stability

The Illinois DOT (1982, 2005) has used the DCP to check the 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 IBV (Immediate Bearing Value) 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 7.2 shows the relationship between unsoaked CBR (actually IBV), DPI, and required thickness of 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.

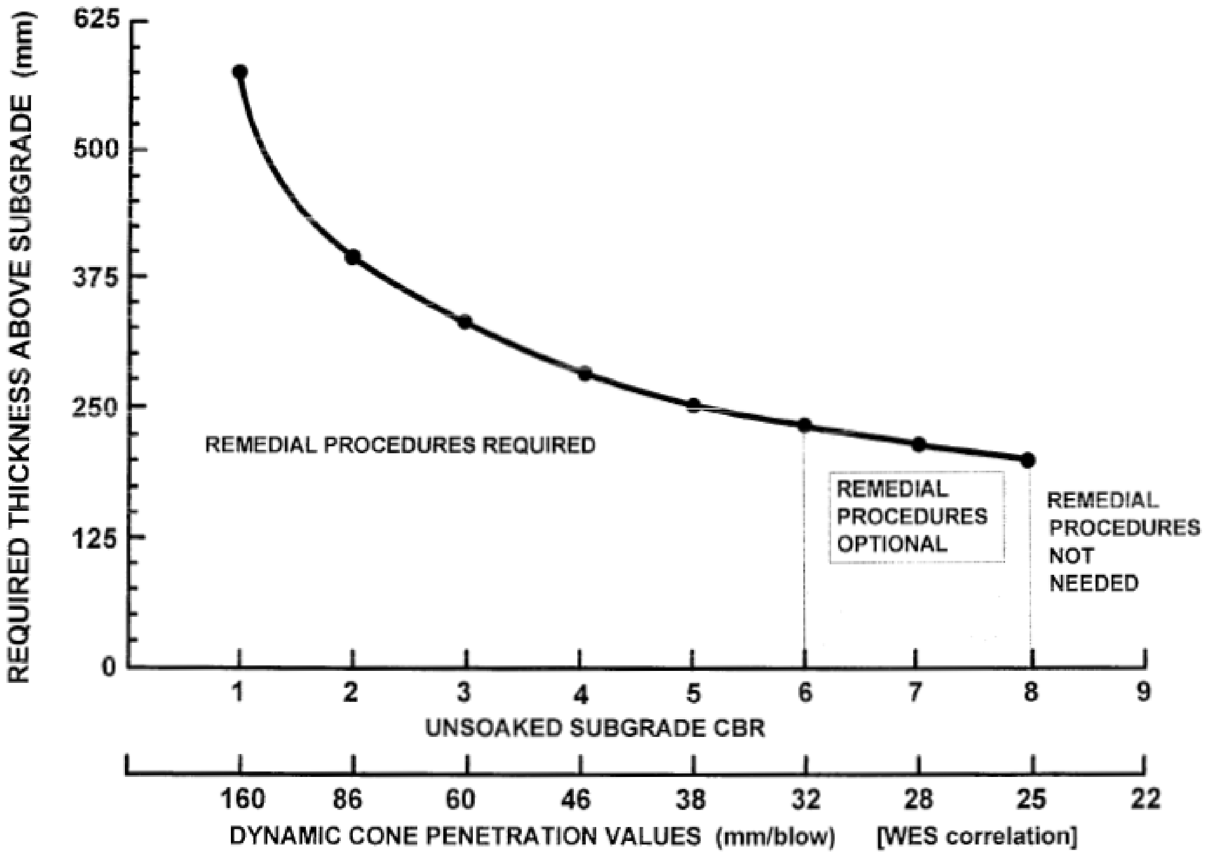


Figure 7.2 DCP Based Thickness Design for Granular Backfill and Subgrade Modification for the Illinois DOT (figure from Burnham, 1997 but checked against Illinois DOT, 2005).

(iv) 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 7.3). The “High Risk” portion of the figure implies the pavement is not a good candidate for rubblization since the supporting base and subgrade is excessively weak. Figure 7.3 is similar to, but modified from, similar guidelines developed for Illinois (Figure 7.4). Figure 7.4 is of interest since data obtained by Sebesta and Scullion (2007) for US 83 in Texas are plotted by total pavement thickness versus DCP derived CBR values.

The DCP—CBR correlation used in Texas is the same as the one described in 6.3(i), which was originally done by the US Army Corps of Engineers.

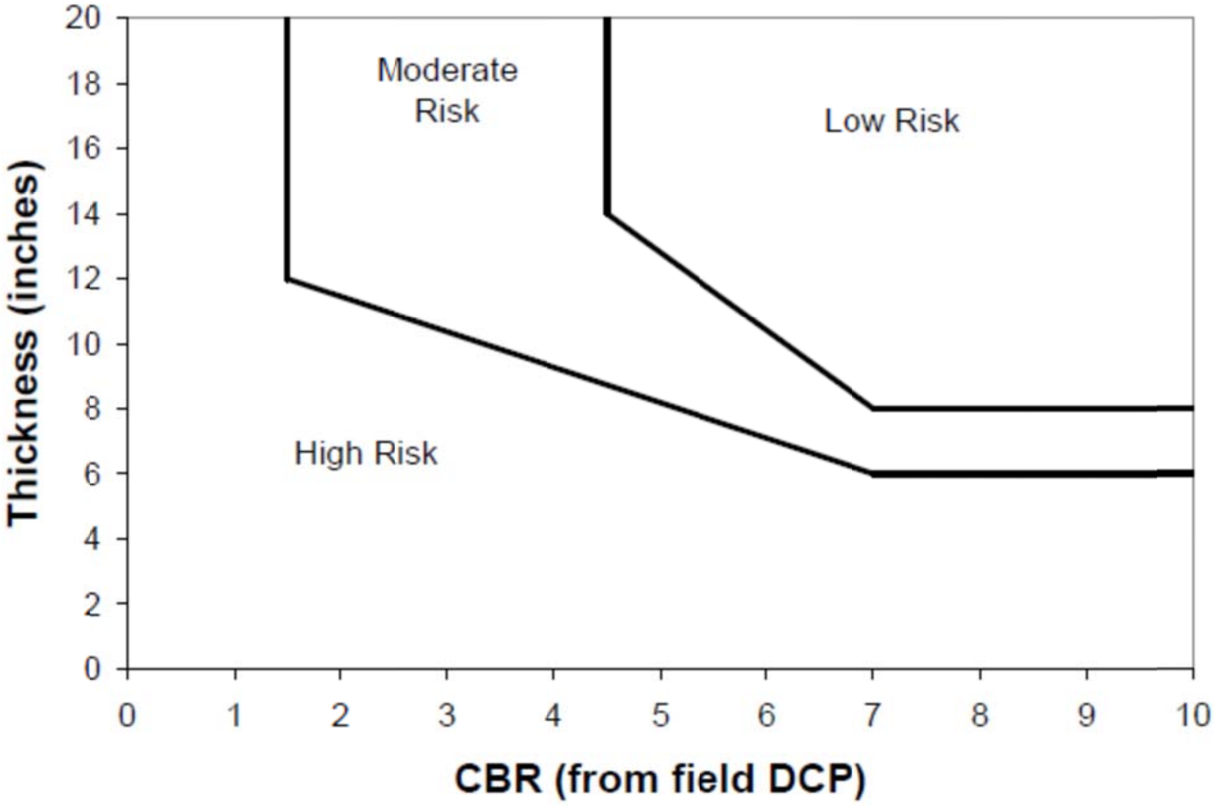


Figure 7.3 Rubblization Selection Chart Developed by TTI
(from Sebesta and Scullion, 2007).

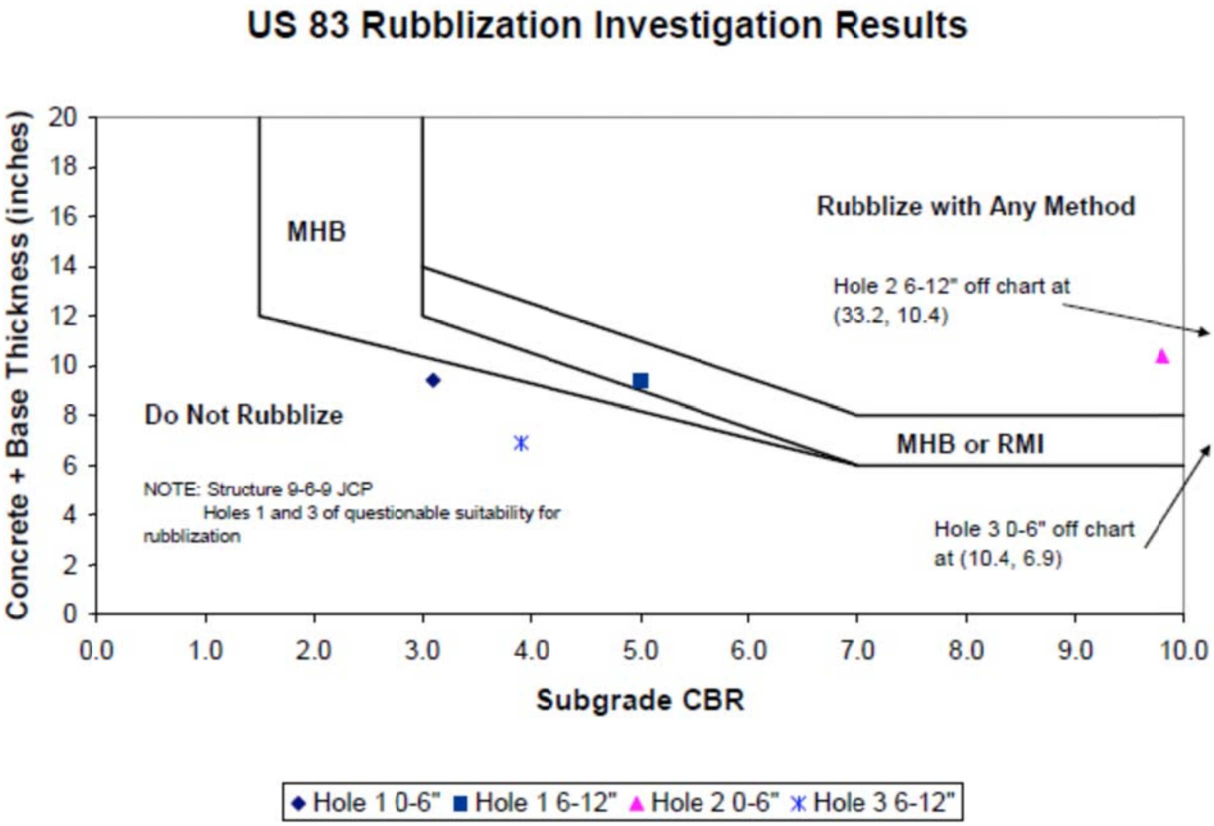


Figure 7.4 Illinois Rubblization Selection Chart with Data from US 83 (Texas)
(From Sebesta and Scullion, 2007; original Illinois DOT criteria from Heckel, 2002).

7.4 References

Burnham, T. (1997), “Application of Dynamic Cone Penetrometer to Minnesota Department of Transportation Pavement Assessment Procedures,” Report No. MN/RC 97/19, Minnesota Department of Transportation, St. Paul, MN, May 1997.

Heckel, L (2002), Rubblizing with Bituminous Concrete Overlay—10 Years Experience in Illinois, Report IL-PRR-137, Illinois Department of Transportation, April 2002. [The report contains an appendix entitled Guidelines for Rubblizing PCC Pavement and Designing a Bituminous Overlay. This appendix contains a similar version of Figure 6.4.]

Illinois DOT (1982), “Subgrade Stability Manual,” Policy Mat-10, Illinois Department of Transportation.

Illinois DOT (2005), “Dynamic Cone Penetrometer,” Pavement Technology Advisory PTA-T4, Bureau of Materials and Physical Research, Illinois Department of Transportation, February 2005.

Minnesota DOT (1993), “User Guide to the Dynamic Cone Penetrometer,” Office of Minnesota Road Research, Minnesota DOT.

Sebesta, S. and Scullion, T. (2007), “Field Evaluations and Guidelines for Rubblization in Texas,” Report No. FHWA/TX-08/0-4687-2, Texas Transportation Institute, December 2007.

Webster, S., Grau, R., and Thomas, W. (1992), “Description and Application of Dual Mass Dynamic Cone Penetrometer,” Instruction Report GL-92-3, Waterways Experiment Station, US Army Corps of Engineers, May 1992.

Section 8
Subgrade Soil Sampling and Tests

8.1 Purpose

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

8.2 Measurement Methods

This subsection will show both the types of tests and frequency of sampling associated with subgrade soils. A summary of these tests is contained in Table 8.1.

Table 8.1 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 (Section 7). Caution is needed since laboratory and field produced CBRs can have quite different moisture conditions—hence 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 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 4). Straightforward methods for estimating M_R are available (Section 4) or backcalculation procedures allow up to 3 pavement layers to be estimated.

8.3 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?

(i) 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.

(ii) Frost Susceptibility

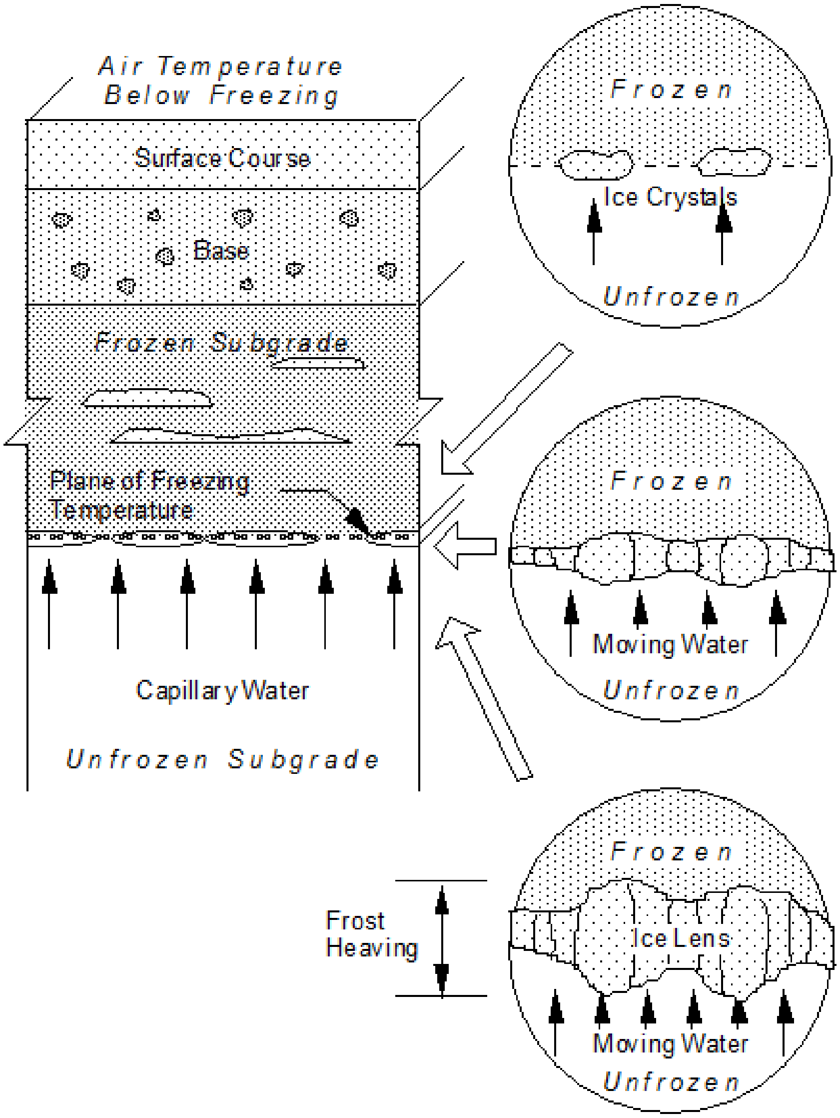
Both sophisticated and very straightforward soils tests are available for estimating the likelihood of subgrade soil frost susceptibility. The basic issue is the potential for the creation of ice lenses under the existing pavement and the resulting loss of support when it all thaws out. When ice lenses 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 8.1.

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 [reference for this content is 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).

Another tool which can aid decisions about the potential frost susceptibility of a subgrade soil is to use the US Army Corps of Engineers classification system for frost design (NCHRP Synthesis 26, 1974), as shown in Table 8.2.



Water in large void space freezes into ice crystals along plane of freezing temperature.

Ice crystals attract water from adjacent voids, which freezes on contact and forms larger crystals.

Crystals continue to grow and join, fed mostly by capillary water, forming ice lens. Vertical pressure exerted by ice lens heaves surface.

Figure 8.1 Formation of Ice Lenses in a Pavement Structure.

Table 8.2 US Army Corps of Engineers Frost Design Soil Classification (after US Army, 1990).

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 to 1.5	GW, GP
Potentially Frost Susceptible Soils			
PFS	(a) Gravels Crushed stone Crushed rock	1.5 to 3	GW, GP
	(b) Sands	3 to 10	SW, SP
S1	Gravelly soils	3 to 6	GW, GP, GW-GM, GP-GM
S2	Sandy soils	3 to 6	SW, SP, SW-SM, SP-SM
F1	Gravelly soils	6 to 10	GM, GW-GM, GP-GM
F1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	(a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	6 to 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, Pl > 12	–	CL, CH
F4	(a) All silts	–	ML, MH
	(b) Very fine silty sands	>15	SM
	(b) Clays, Pl < 12	–	CL, CL-ML
	(c) Varved clays and other fine-grained, banded sediments	–	CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

(iii) 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?

(iv) Groundwater Issues

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

8.4 References

Terzaghi, K. and Peck, R. (1967), "Soil Mechanics in Engineering Practice," John Wiley and Sons, 1967.

US Army (1990), "Design of Aggregate Surfaced Roads and Airfields," Technical Manual TM 5-822-12, Department of the Army, September 1990.

Section 9
Traffic Loads for Design

9.1 Purpose

This section overviews the use of basic traffic information to estimate design loadings for pavement design. The fundamental parameter that will be estimated is Equivalent Single Axle Loads (ESALs). More detailed assessments of traffic loading such as load spectra used in the MEPDG are not needed for use in these guidelines.

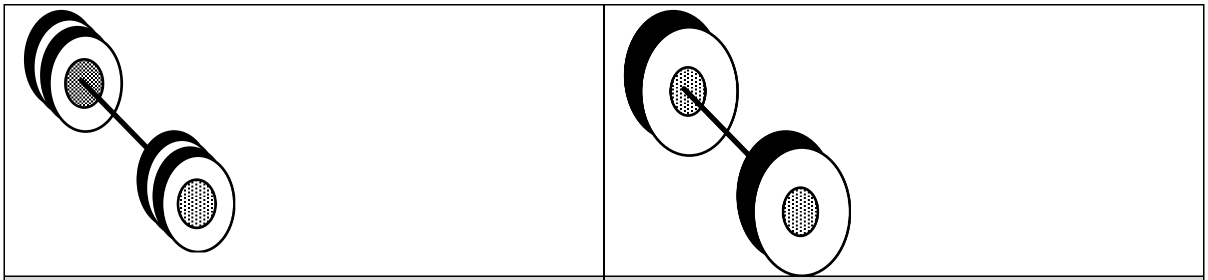
9.2 Measurement Method

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

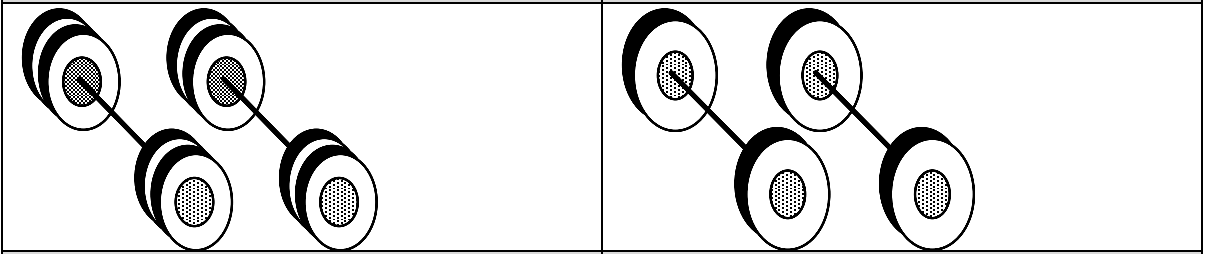
(i) Tire Loads and Terminology

Typical truck and bus axles are shown in Figure 9.1, which illustrates single and tandem axles with either single or dual tires.

States generally have regulations limiting allowable load per inch of tire width. This tire load limitation varies from a high of 800 lbs/inch to a low of 450 lbs/inch. 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.



Single Axle with Dual Tires | **Single Axle with Single Tires**



Tandem Axles with Dual Tires | **Tandem Axles with Single Tires**

Figure 9.1 Illustration of Typical Axle and Tire Configurations.

(ii) Typical Federal and State Axle Load Limits

Typical federal and state axle load limits are:

- Single axles: 20,000 lbs
- Tandem axles: 34,000 lbs
- Total truck gross weight: 80,000 lbs

(iii) FHWA Bridge Formula

A major additional limitation on US 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 9.1 below. The bridge formula is needed since an individual set of bridge design computations cannot be done 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.

$W = 500(NL/(N-1) + 12N + 36)$ Eq. 9.1

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 5 axle truck with a 51 ft. separation from the steer axle to the rear portion of the back tandem. If you wish to know the total vehicle allowable gross weight via the Bridge Formula, then $W = 500(5(51)/(5-1) + 12(5) + 36) = 80{,}000$ lb.

(iv) Repetitions of Wheel Loads and 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."

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, 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 9.1.

Table 9.1 Sample of AASHTO Equivalency Factors.

Axle Type (lbs)	**Axle Load (lbs)**	**ESAL Load Equivalency Factors [from AASHTO, 1993]**
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 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 used by vehicle manufacturers). These classes are shown in Table 9.2.

Table 9.2 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 handle bars 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, motor homes, etc., 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, etc., 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 Less Axle Single Trailer Trucks* — All vehicles with four or less 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 Less Axle Multi-Trailer Trucks* — All vehicles with five or less 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.

A somewhat simplified scheme for summarizing the 13 vehicle classes in Table 9.2 is to group all truck and bus traffic into three groups or units as shown in Table 9.3.

Table 9.3 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)

9.3 Analysis Tools

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

Table 9.4 shows typical ESALs per vehicle according to the groupings in Table 9.3. The ESALs per vehicle were developed by a State DOT and appear to be typical for US truck traffic. They may appear to be low, but the values are averages that include empty backhauls.

Table 9.4 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 (1/2 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 percent).

9.4 References

AASHTO (1993), "AASHTO Guide for Design of Pavement Structures, 1993," American Association of State Highway and Transportation Officials, Washington, DC.

Section 10
Construction Productivity and Traffic Impacts

10.1 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/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.

10.2 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/incident minimization, avoidance of certain public event days that generate high traffic, etc.) 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/efficiency, dump truck number/capacity, paver speed, and materials manufacturing plant productivity. It also involves estimating mobilization/demobilization times, concrete cure time, hot mix asphalt cooling time and traffic control setup time. There may be several estimates of each depending upon the construction scenarios being investigated.

2. **Measure existing traffic**. While 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 multiplying ADT by typical hourly distribution factors for the type of roadway being analyzed.

3. **Estimate the fraction of traffic that will cancel their 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.
    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 four scenarios:
    a. **Partial night closures**: 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 since 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/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 pre-paving work (e.g., PCC panel sawcutting, restriping lanes, milling HMA, etc.).
    b. **Full night closures**: 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 are sometimes required to do such things as set up counterflow traffic on one side of a roadway or accomplish dangerous overhead work such as overpass demolition/placement.
    c. **Partial day closures**: closure 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 reduce construction costs by avoiding overtime pay. However, they may not provide the lowest user costs because mobilization/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**: 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**: closure starting Friday evening after peak hour traffic and ending Monday morning before peak hour traffic. The typical scenario is a 55-hour weekend closure starting at 9 or 10 p.m. Friday night and ending at 4 or 5 a.m. on Monday morning. The long closure time allows for better productivity because mobilization/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:** closures that are maintained continuously over an entire week (168 hours). While 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/demobilization times), three times the productivity of a 1-week continuous closure.

5. **Model traffic using the tool of choice (see 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 while 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 10.1). This table is in 1996 dollars and should be adjusted to current dollars using the Consumer Price Index (CPI). A simple calculator is available from the U.S. Bureau of Labor Statistics (BLS) 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 10.1 Recommended Values of Time (from Walls and Smith 1998).

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

(i) General Guidance

The following general guidance for traffic impacts comes largely from the guidance documents listed in the References portion of 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, etc. (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 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 underway 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 et al. 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) provides 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/ln/hr) 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., semi-permanent 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.

10.3 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:

- **How much detail is needed?** Work zone characteristics, desired outputs and the stage of planning/design/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. While a deterministic model can provide a single number, it is better to provide a reasonable range of answers in order 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, while others can estimate them in much smaller time

increments. Some tools make estimates using one single day's traffic input, while others are able to account for daily, weekly and monthly traffic variations.

(i) 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. 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.

(ii) 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 percent confidence intervals seen are mostly dependent on these delivery rates. In all cases, a 10-mile stretch of two lanes was analyzed (20 lane-miles total). As with all data input values, this length of highway and total lane-miles has some influence on productivity. Tables 10.2 through 10.7 show input parameters use in CA4PRS to generate Figures 10.1 through 10.9. Estimates are given for:

- **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, while concurrent operations require two additional lanes shut down for construction access. Calculations were made for both screed paving (slower) and slipform paving (faster).

  - **Screed paving**. Using fixed forms and a screed, this paving is usually slower. Assumes 7.5 yd^3 agitating mixers arriving at 10 trucks/hr and only one demolition crew.
  - **Slipform paving**. Using a slipform paver, this paving is usually faster. Assumes 8.5 yd^3 end dump trucks arriving at 17 trucks/hr 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 lifts. Lifts are generally 3 inches thick with the exception of the top two lifts, which are either 2 or 1.5 inches 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 a 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 inches thick with the exception of the top two lifts, which are either 2 or 1.5 inches 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 inches thick with the exception of the top two lifts, which are either 2 or 1.5 inches thick. A lift is paved for each lane across before the next lift is paved on any lane.
- **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” without the demolition component.

Table 10.2. CA4PRS Input Values for Remove-and-Replace with PCC.

Input	**Value**	**Distribution/Comments**
Activity Constraints		
Mobilization	1.0 hours	None - Deterministic
Demobilization	2.0 hours	None - Deterministic
Base Paving	none	N/A
Demo-to-PCC Paving Lag Times for Sequential Method	1.0 hours	Triangular (min = 0.5 hrs, max = 1.5 hrs)
Demo-to-PCC Paving Lag Times for Concurrent Method	2.0 hours	Triangular (min = 1.0 hrs, max = 3.0 hrs)
Resource Profile		
Demolition Hauling Truck		
Rated Capacity	18.0 tons	9 yd^3 of a 15 yd^3 truck filled w/2.0 $tons/yd^3$ material
Trucks/hr/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	N/A (no base material)
Batch Plant		
Capacity	500 yd^3/hr	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	N/A
Trucks per Hour	10/hr 13/hr	The first rate is for screed paving and the second is for slipform paving Triangular (min = 8/hr, max = 12/hr) Triangular (min = 15/hr, max = 19/hr)
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 hours	
Working Method	see graphs	

Table 10.3. CA4PRS Input Values for Remove-and-Replace with HMA.

Input	**Value**	**Distribution/Comments**
Activity Constraints		
Mobilization	1.0 hours	None - Deterministic
Demobilization	2.0 hours	None - Deterministic
Base Paving	none	N/A
Demo-to-HMA Paving Lag	1.0 hours	Triangular (min = 0.5 hrs, max = 1.5 hrs)
Half Closure Traffic Switch	0.5 hours	Triangular (min = 0.25 hrs, max = 0.75 hrs)
Resource Profile		
Demolition Hauling Truck		
Rated Capacity	18.0 tons	9 yd^3 of a 15 yd^3 truck filled w/2.0 $tons/yd^3$ material
Trucks/hr/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	N/A (no base material)
Non-Paving Speed	15 mph	
Batch Plant		
Capacity	500 yd^3/hr	None – Deterministic (set high to ensure plant is not the limiting activity)
Number of Plants	1	None - Deterministic
HMA Delivery Truck		
Capacity	18 tons	N/A
Trucks per Hour	12/hr	Triangular (min = 10/hr, max = 14/hr)
Packing Efficiency	1.0	None - Deterministic
Schedule Analysis		
Construction Window	see graphs	
Section Profile	see graphs	Top two lifts are 2 inches each, all other lifts are 3 inches 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	Pre-Paving	Shoulder overlays are not accounted for
Curing Time	12-hours	
Working Method	see graphs	
Cooling Time Analysis	User Spec.	Time calculated in MultiCool and manually entered
Lane Widths		
No. of Lanes	2	
Lane Widths	12 ft each	

Table 10.4. CA4PRS Input Values for Mill-and-Fill with HMA.

Input	**Value**	**Distribution/Comments**
Activity Constraints		
Mobilization	1.0 hours	None - Deterministic
Demobilization	2.0 hours	None - Deterministic
Mill-to-HMA Paving Lag	1.0 hours	Triangular (min = 0.5 hrs, max = 1.5 hrs)
Half Closure Traffic Switch	0.5 hours	Triangular (min = 0.25 hrs, max = 0.75 hrs)
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 w/2.0 $tons/yd^3$ material
Trucks/hr/team	13 trucks	Triangular (min = 11 trucks, max = 15 trucks)
Packing Efficiency	1.0	None - Deterministic
Batch Plant		
Capacity	500 yd^3/hr	None – Deterministic (set high to ensure plant is not the limiting activity)
Number of Plants	1	None - Deterministic
HMA Delivery Truck		
Capacity	18 tons	N/A
Trucks per Hour	12/hr	Triangular (min = 10/hr, max = 14/hr)
Packing Efficiency	1.0	None - Deterministic
Paver	None	N/A (no base material)
Non-Paving Speed	15 mph	
Schedule Analysis		
Construction Window	see graphs	
Section Profile	see graphs	Lifts are between 1.5 and 3 inches; Paver speeds are 0.5 to 0.6 mph
Change in Roadway Elevation	No Change	
Shoulder Overlay	Pre-Paving	Shoulder overlays are not accounted for
Curing Time	12-hours	
Working Method	see graphs	
Cooling Time Analysis	User Spec.	Time calculated in MultiCool and manually entered
Lane Widths		
No. of Lanes	2	
Lane Widths	12 ft each	

Table 10.5. CA4PRS Input Values for Crack, Seat and Overlay.

Input	**Value**	**Distribution/Comments**
Activity Constraints		
Mobilization	3.0 hours	None - Deterministic
Demobilization	2.0 hours	None - Deterministic
Half Closure Traffic Switch	0.5 hours	Triangular (min = 0.25 hrs, max = 0.75 hrs)
Resource Profile		
Paver	None	N/A (no base material)
Non-Paving Speed	15 mph	
Batch Plant		
Capacity	500 yd^3/hr	None – Deterministic (set high to ensure plant is not the limiting activity)
Number of Plants	1	None - Deterministic
HMA Delivery Truck		
Capacity	18 tons	N/A
Trucks per Hour	12/hr	Triangular (min = 10/hr, max = 14/hr)
Packing Efficiency	1.0	None - Deterministic
Schedule Analysis		
Construction Window	see graphs	
Section Profile	see graphs	Top two lifts are 2 inches each, all other lifts are 3 inches 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	Pre-Paving	Shoulder overlays are not accounted for
Curing Time	12-hours	
Working Method	see graphs	
Cooling Time Analysis	User Spec.	Time calculated in MultiCool and manually entered
Lane Widths		
No. of Lanes	2	
Lane Widths	12 ft each	

Table 10.6. CA4PRS Input Values for Unbonded PCC Overlay.

Input	**Value**	**Distribution/Comments**
Activity Constraints		
Mobilization	3.0 hours	None - Deterministic (longer time accounts for surface preparation)
Demobilization	2.0 hours	None - Deterministic
Base Paving	none	N/A
Demo-to-PCC Paving Lag Times for Sequential Method	0 hours	No demolition occurs
Demo-to-PCC Paving Lag Times for Concurrent Method	0 hours	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/hr/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	N/A (no base material)
Batch Plant		
Capacity	500 yd^3/hr	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	N/A
Trucks per Hour	10/hr	Triangular (min = 8/hr, max = 12/hr)
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 hours	
Working Method	see graphs	

Table 10.7. MultiCool Input Parameters for HMA Options.

Input	**Value**
Constant Inputs in All Scenarios	
Start Time	1000, 7/15/2010
Environmental Conditions	
Ambient Air Temp.	60°F
Average Wind Speed	5 mph
Sky Conditions	Clear&Dry
Latitude	38° North
Existing Surface	
Material Type	Granular Base
Moisture Content	Dry
State of Moisture	Unfrozen
Surface Temp.	60°F
Mix Specifications	
Mix Type	Dense Graded
PG Grade	64-22
Delivery Temp.	300°F
Stop Temp.	140°F
Lift Thicknesses	
3 inches of HMA total	2 lifts of 1.5 inches each
6 inches of HMA total	3 lifts of 2 inches each
9 inches of HMA total	3 lifts of 2 inches, 1 lift of 3 inches
12 inches of HMA total	2 lifts of 1.5 inches 3 lifts of 3 inches

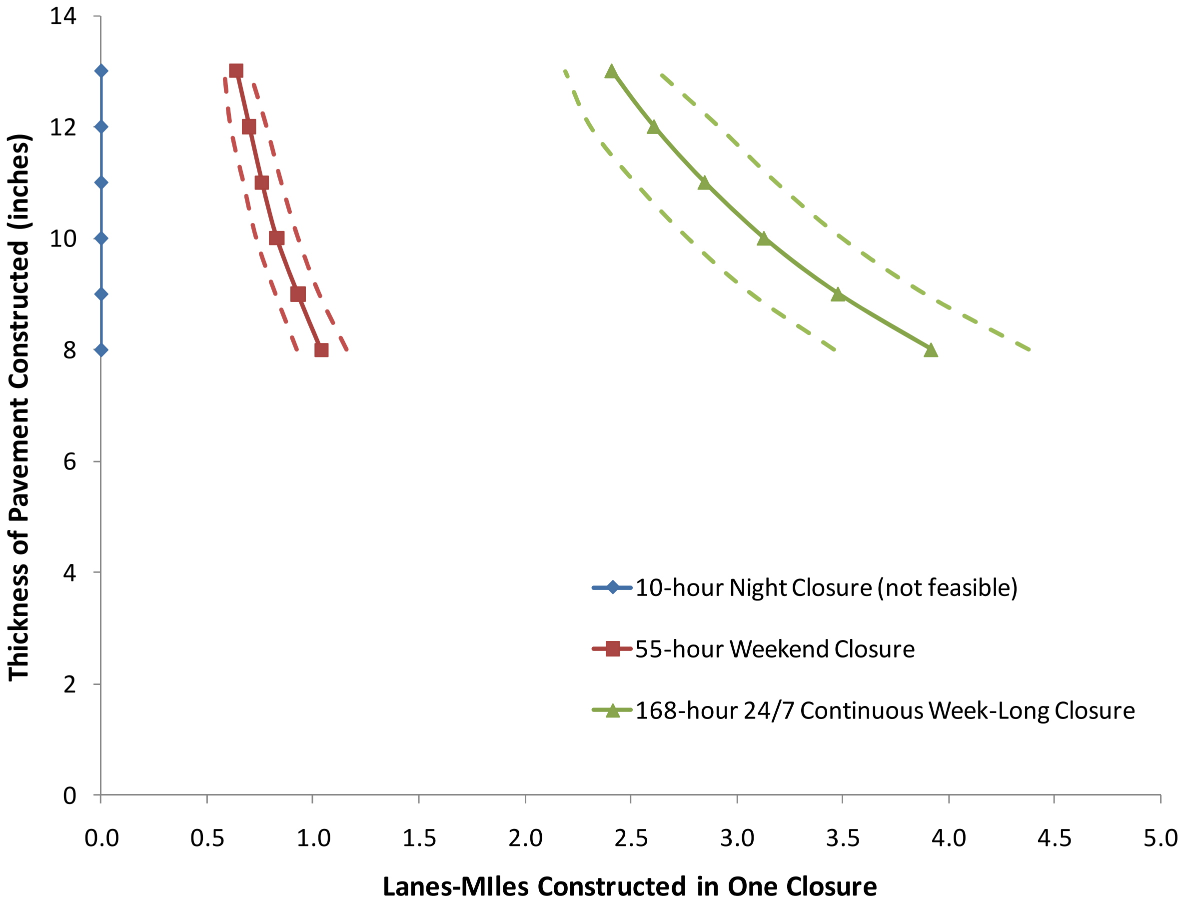


Figure 10.1. Productivity estimates for remove-and-replace with PCC (fixed form) using sequential operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. Note: this option is not feasible using 10-hour night closures.

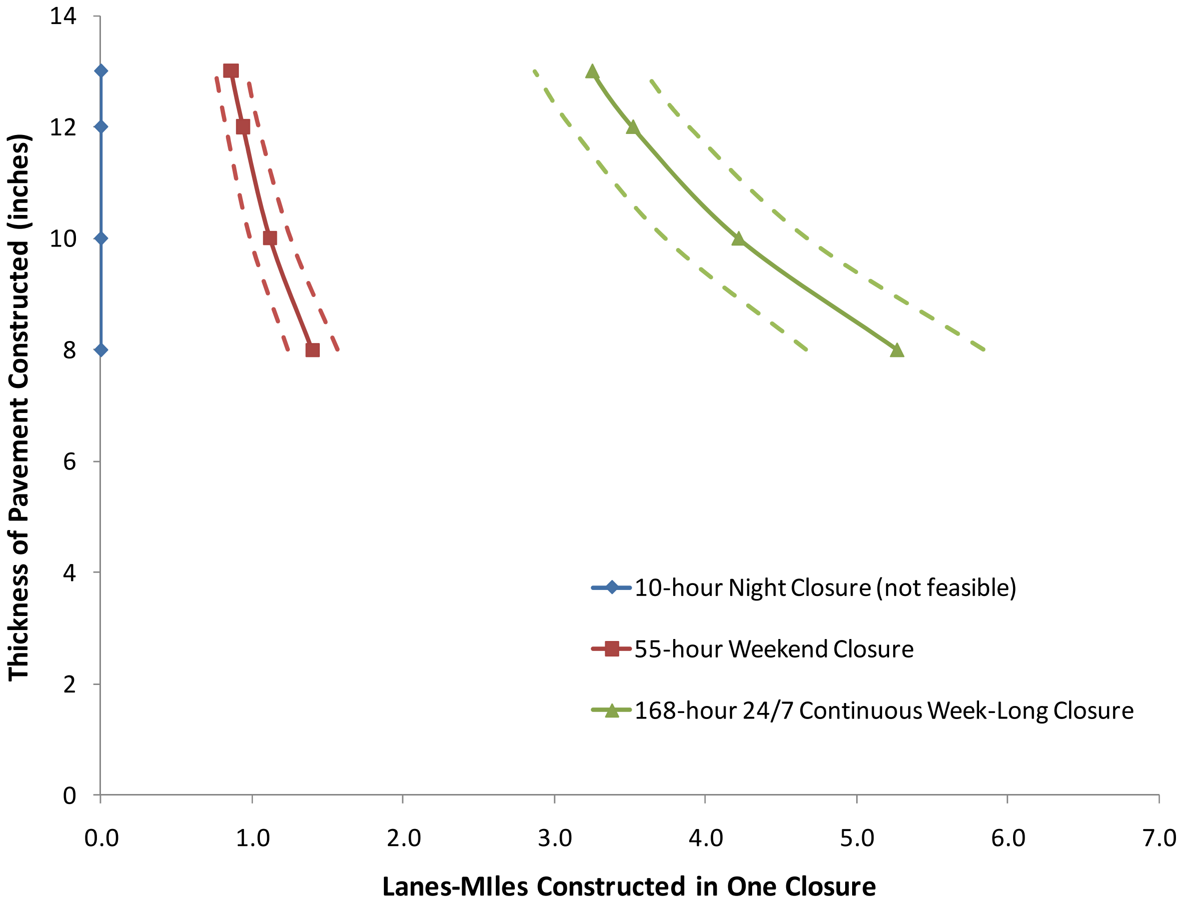


Figure 10.2. Productivity estimates for remove-and-replace with PCC (slipform) using sequential operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. Note: this option is not feasible using 10-hour night closures.

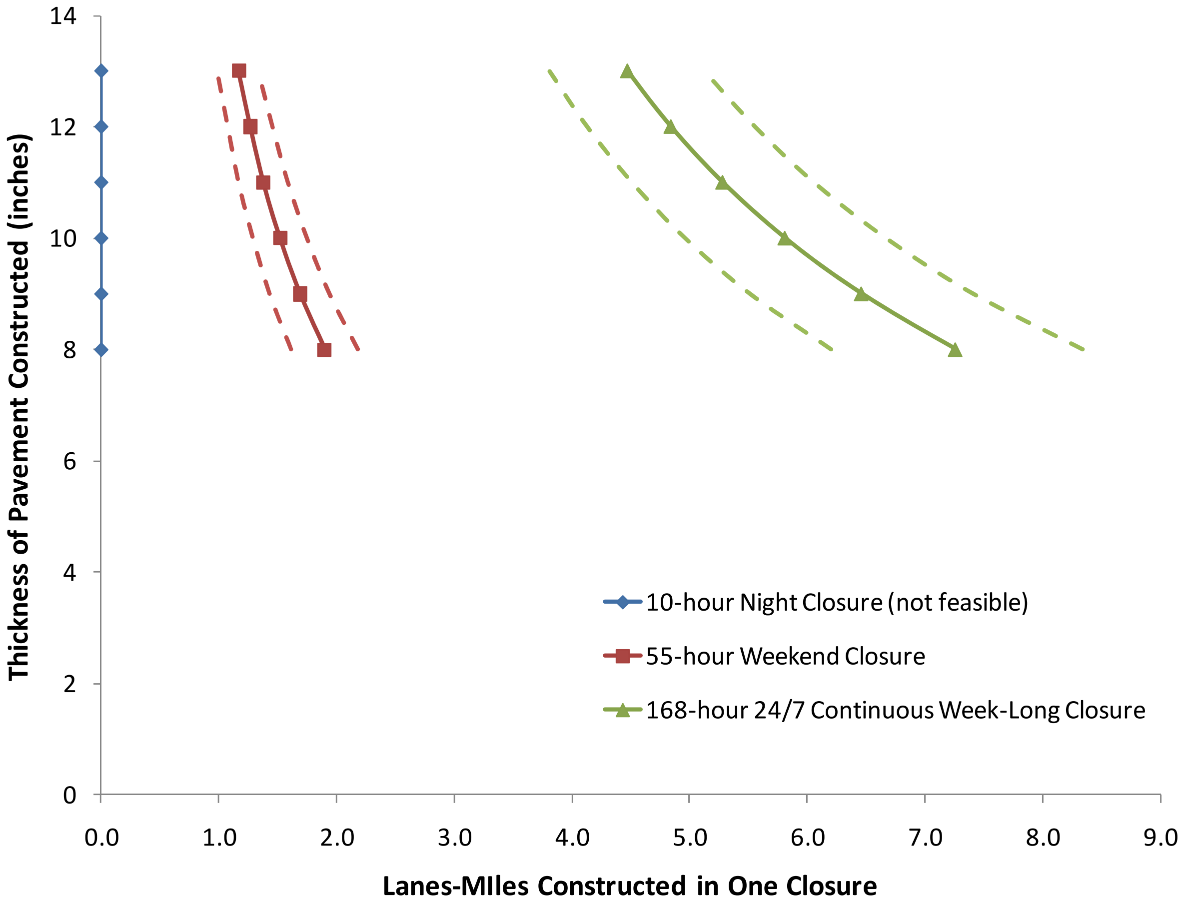


Figure 10.3. Productivity estimates for remove-and-replace with PCC (fixed form) using concurrent operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. Notes: (1) this option is not feasible using 10-hour night closures, (2) doing demolition and paving concurrently results in significantly higher productivities than doing them sequentially.

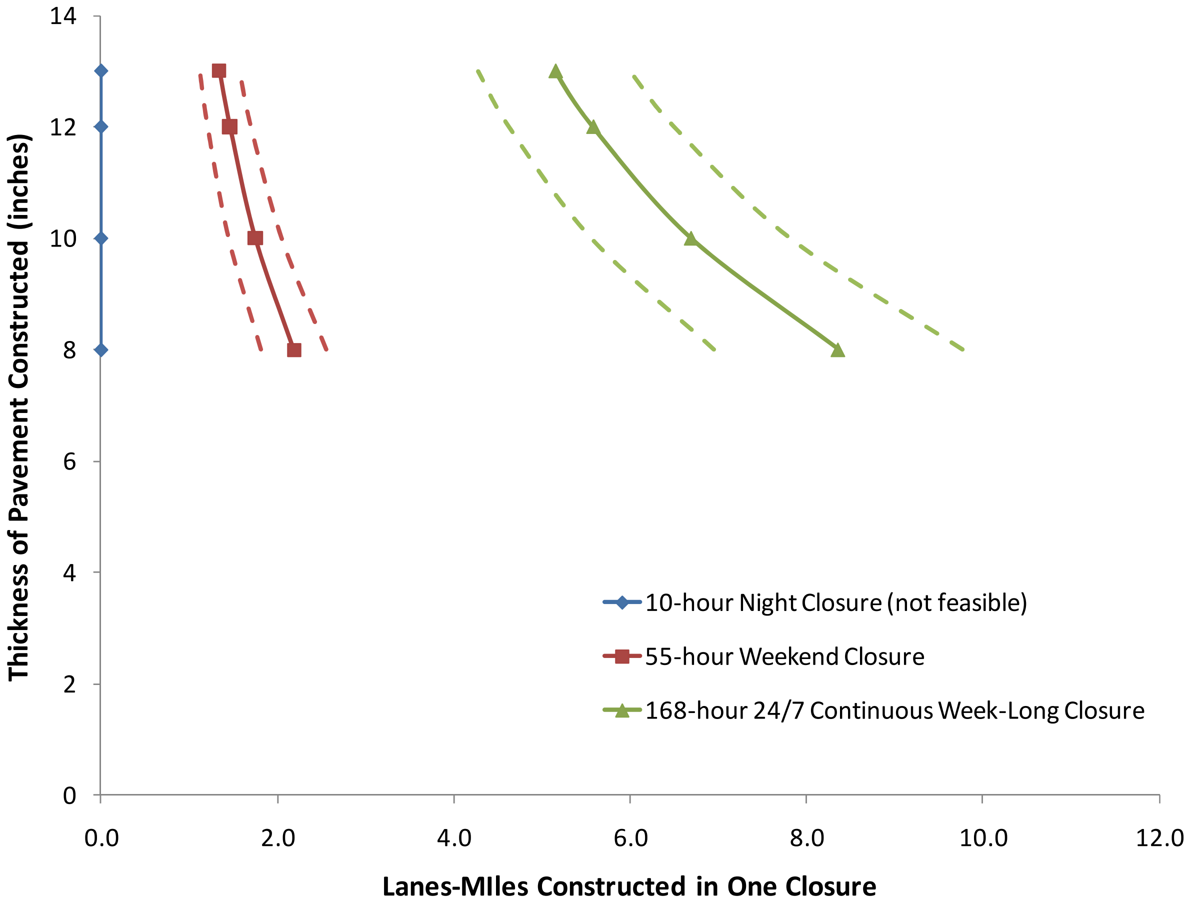


Figure 10.4. Productivity estimates for remove-and-replace with PCC (slipform) using concurrent operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. Notes: (1) this option is not feasible using 10-hour night closures, (2) doing demolition and paving concurrently results in significantly higher productivities than doing them sequentially.

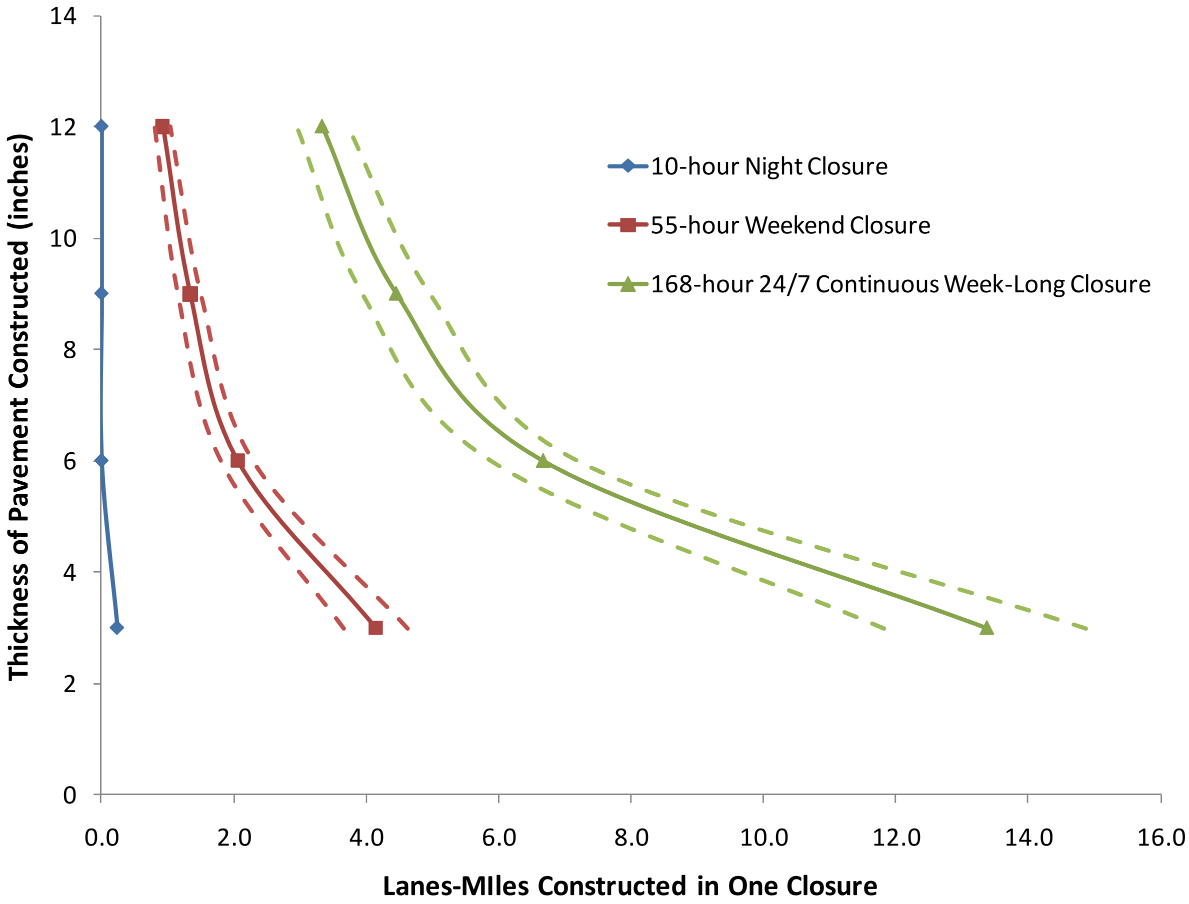


Figure 10.5. Productivity estimates for remove-and-replace with HMA. Solid lines indicate averages and dashed lines indicate 95% confidence intervals.

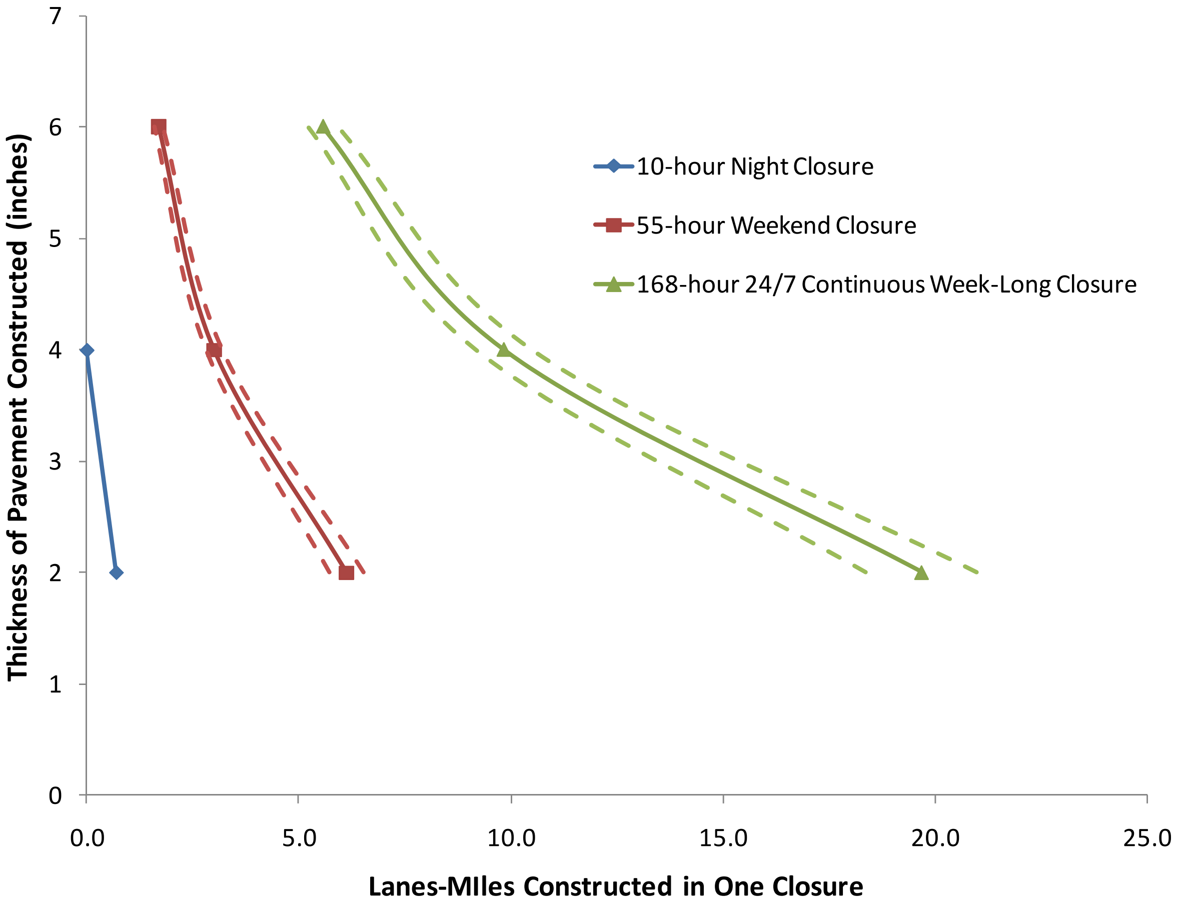


Figure 10.6. Productivity estimates for mill-and-fill with HMA. Solid lines indicate averages and dashed lines indicate 95% confidence intervals.

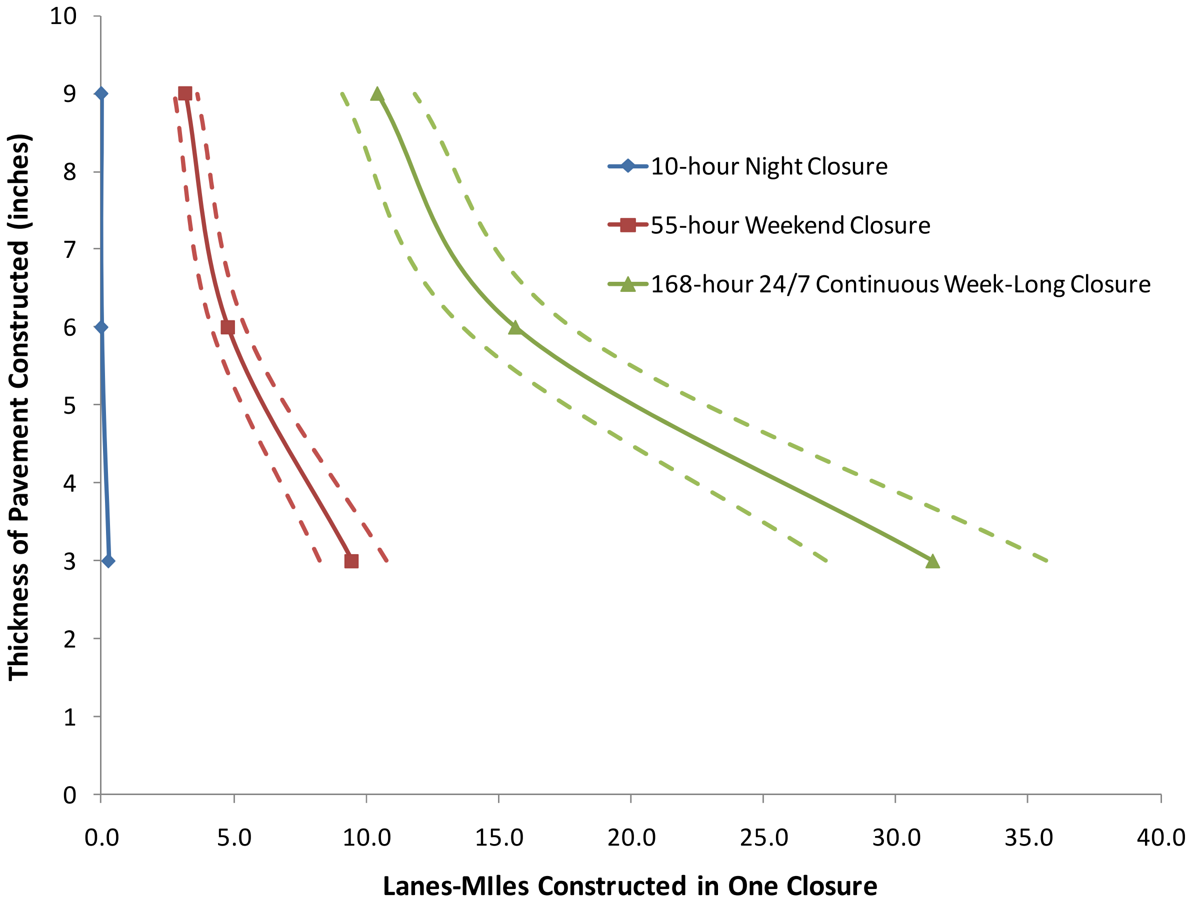


Figure 10.7. Productivity estimates for crack, seat and overlay. Solid lines indicate averages and dashed lines indicate 95% confidence intervals.

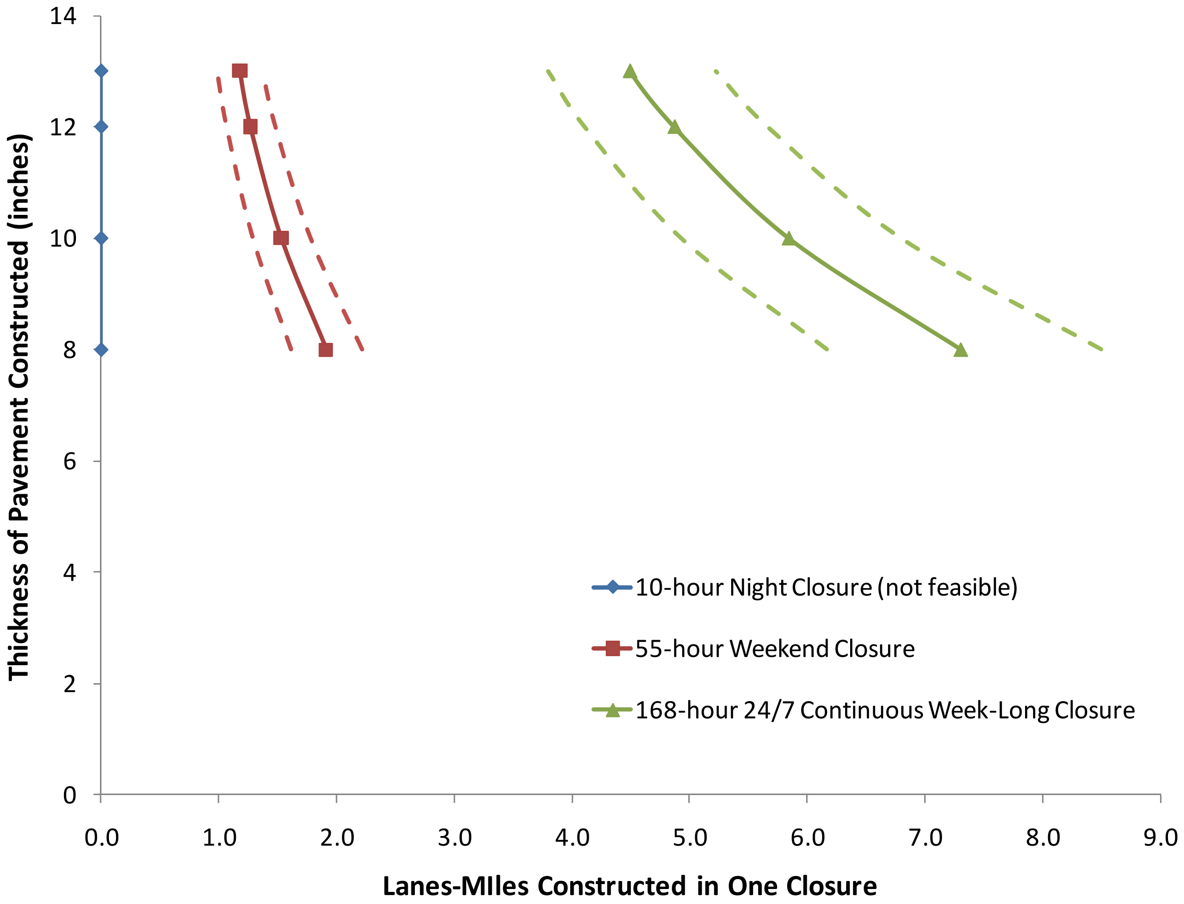


Figure 10.8. Productivity estimates for PCC unbonded overlay. Solid lines indicate averages and dashed lines indicate 95% confidence intervals.

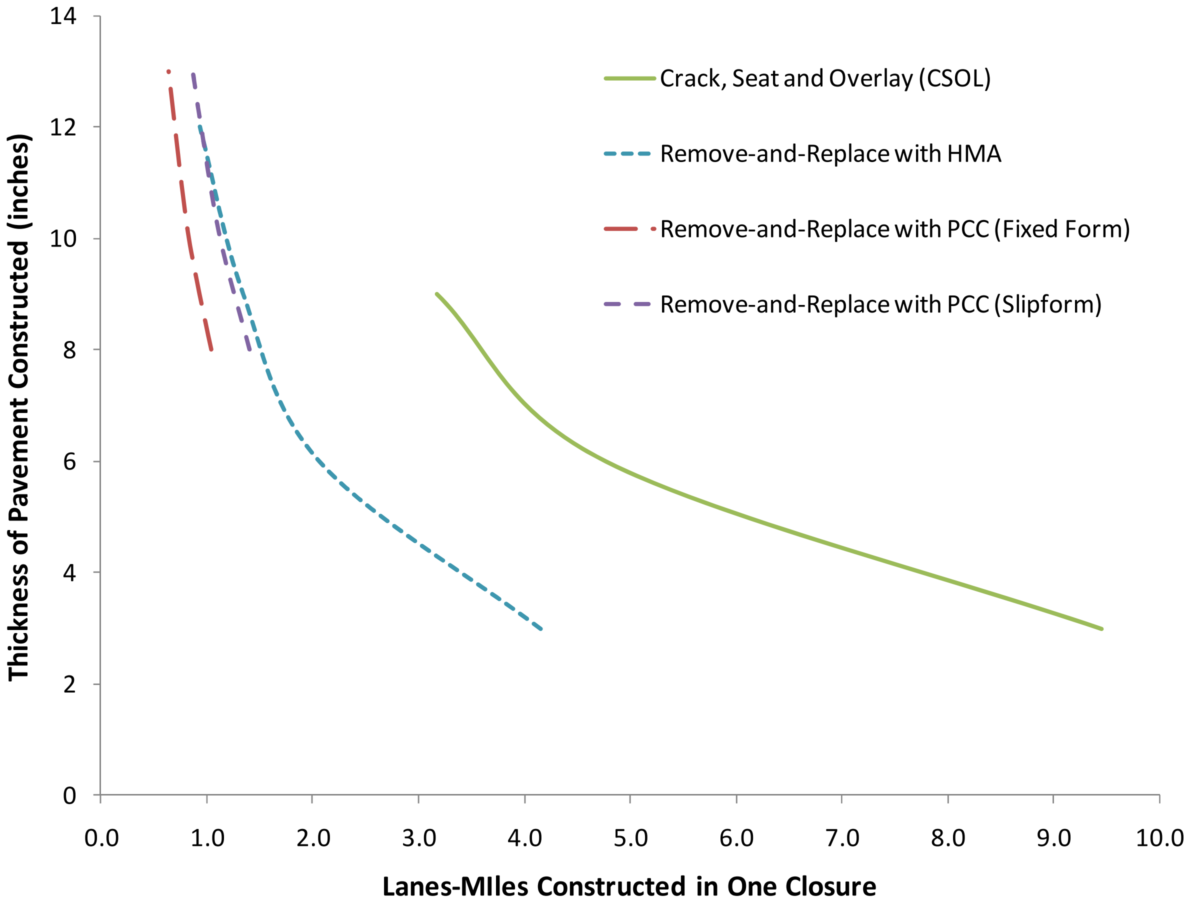


Figure 10.9. A productivity comparison of PCC remove-and-replace (both fixed form and slipform), HMA remove-and-replace and crack, seat and overlay (CSOL).

(iii) Analysis Tools – Traffic Impacts

There are a number of analysis tools available to assist in work zone traffic impacts estimation. The FHWA divides these tools up into six broad categories (Hardy and Wunderlich 2008, and summarized in Table 10.8):

1. **Sketch-planning tools**. 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 *Highway Capacity Manual* (HCM). Such simple estimation tools are often adequate for work zone delay estimation.
2. **Travel demand models**. Forecast future traffic demand based on current conditions, and future predictions of household and employment centers (Alexiadis et al. 2004). Travel demand models are usually used in large regional planning efforts. In work zone analysis, they can help predict region-wide 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**. 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**. Based on the deterministic relationships of traffic speed, flow, and density (Alexiadis et al. 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.
5. **Mesoscopic simulation models**. Represent relative flow of vehicles on a network, but do not model individual lanes or vehicles. These models are between macroscopic and microscopic models in detail. 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 attempting 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.

Table 10.8. 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 micro models	Data intensive
Microscopic	CORSIM, VISSIM, PARAMICS	Can model small details, good communication tool	Data intensive

The most appropriate modeling approach depends upon (Hardy and Wundurlich 2008):

- **Work zone characteristics**. The expected level of impact a work zone will have on travelers including the geographic scale of affected area and complexity of the road network within this area.
- **Transportation management plan strategies**. 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, weekend work, etc.
- **Data availability and quality**. The type, amount, accuracy, and timeliness of available data.
- **Agency resources**. The owner agency's funding, technical staff, and schedule.
- **Work zone performance measures**. The performance measures selected by the owner agency to quantify traffic impacts. Typically this is some form of delay (in minutes or cost) either in total (total delay/cost) or peak (longest queue, longest wait).

Since 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 10.10 and 10.11), 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 average daily traffic (ADT) inputs. It can simulate multiple lane closures over time, model traffic over an entire week (Figure 10.12) and display various traffic impact metrics (Figure 10.13). These capabilities are helpful because they allow QuickZone to show difference in traffic impacts between nights and days, weekends and weekdays, and seasons (e.g., summer vs. 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 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.

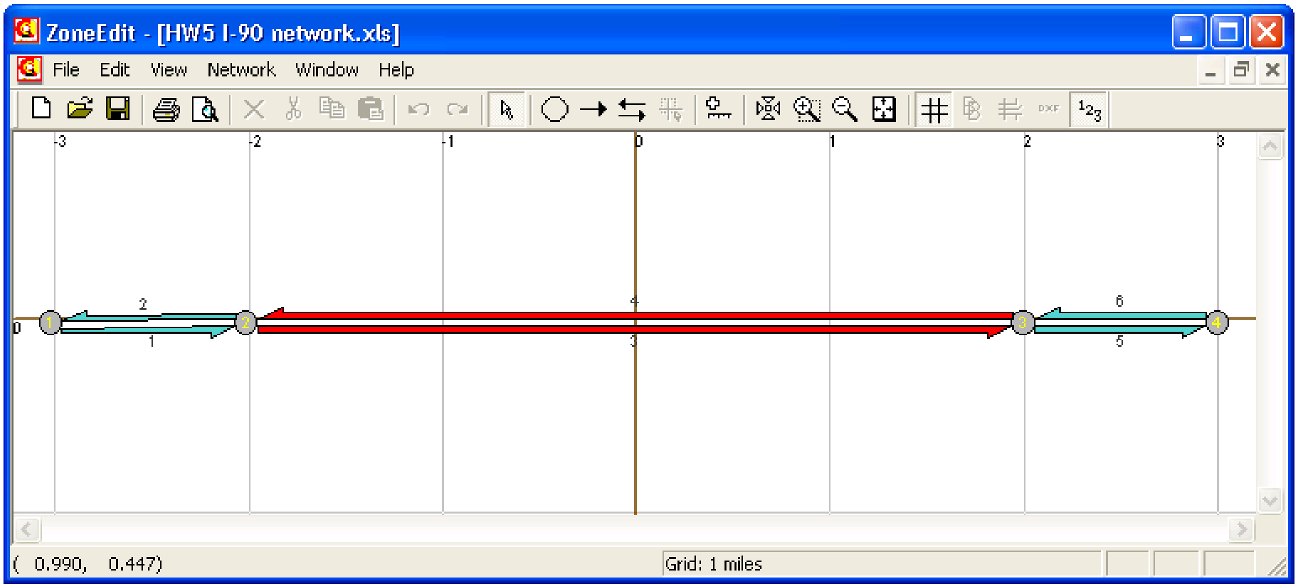


Figure 10.10 A simple network that works quite well in QuickZone.

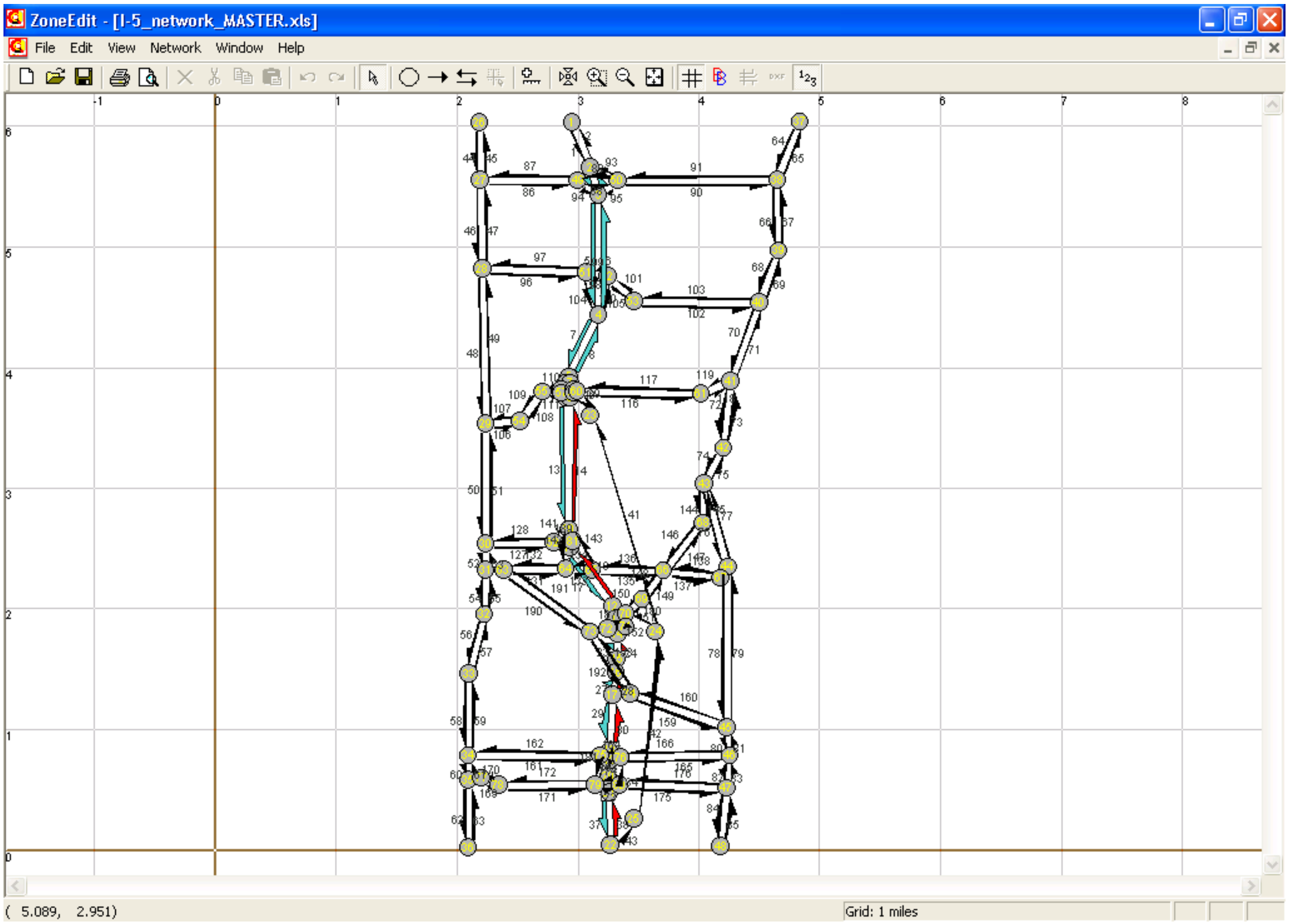


Figure 10.11. A complex network simulation in QuickZone (I-5 in the Seattle, WA area is shown). This network simulation exposed several program bugs, was unwieldy to process and required tedious troubleshooting to make operational. This level of complexity is not recommended.

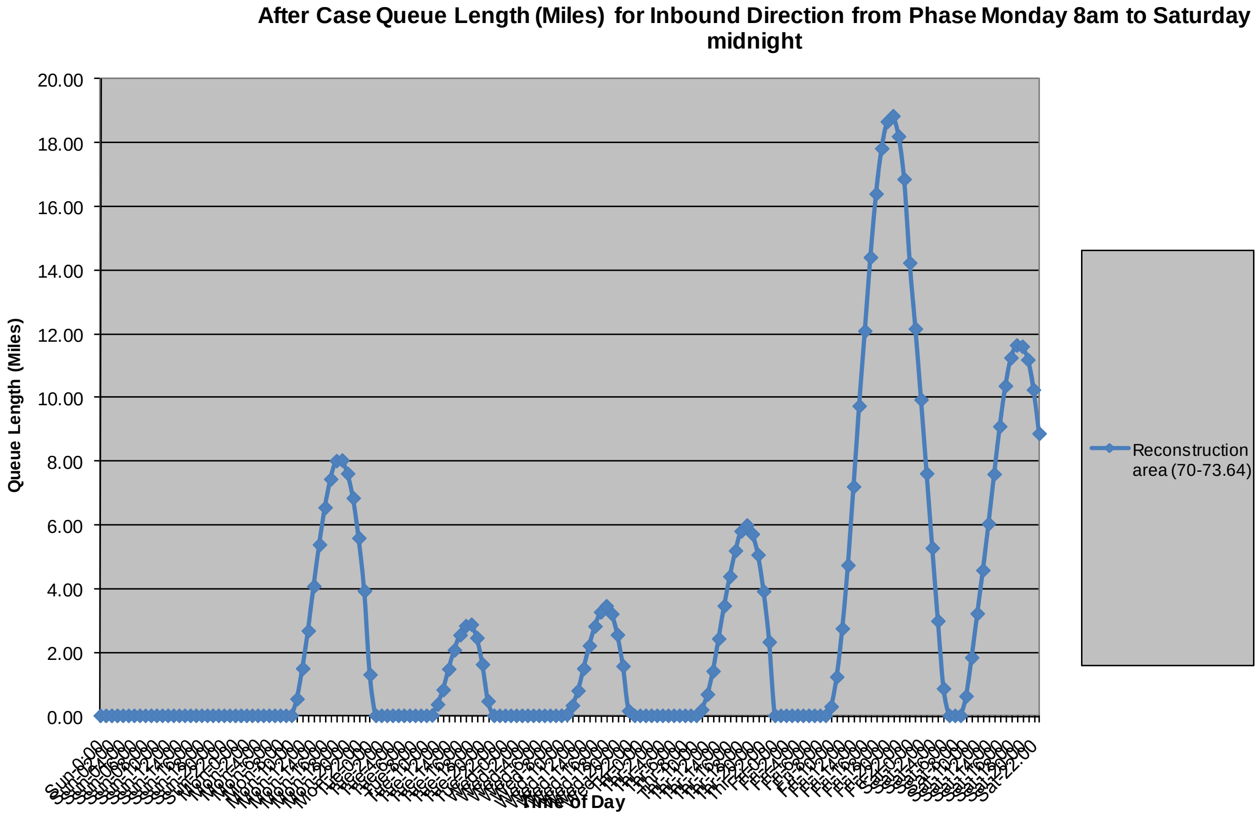


Figure 10.12. Unedited QuickZone 2.0 simulation output chart for a one-week time period. Note that the automatic graph labeling on the horizontal axis is unreadable, however this can be corrected by editing the graph in Excel.

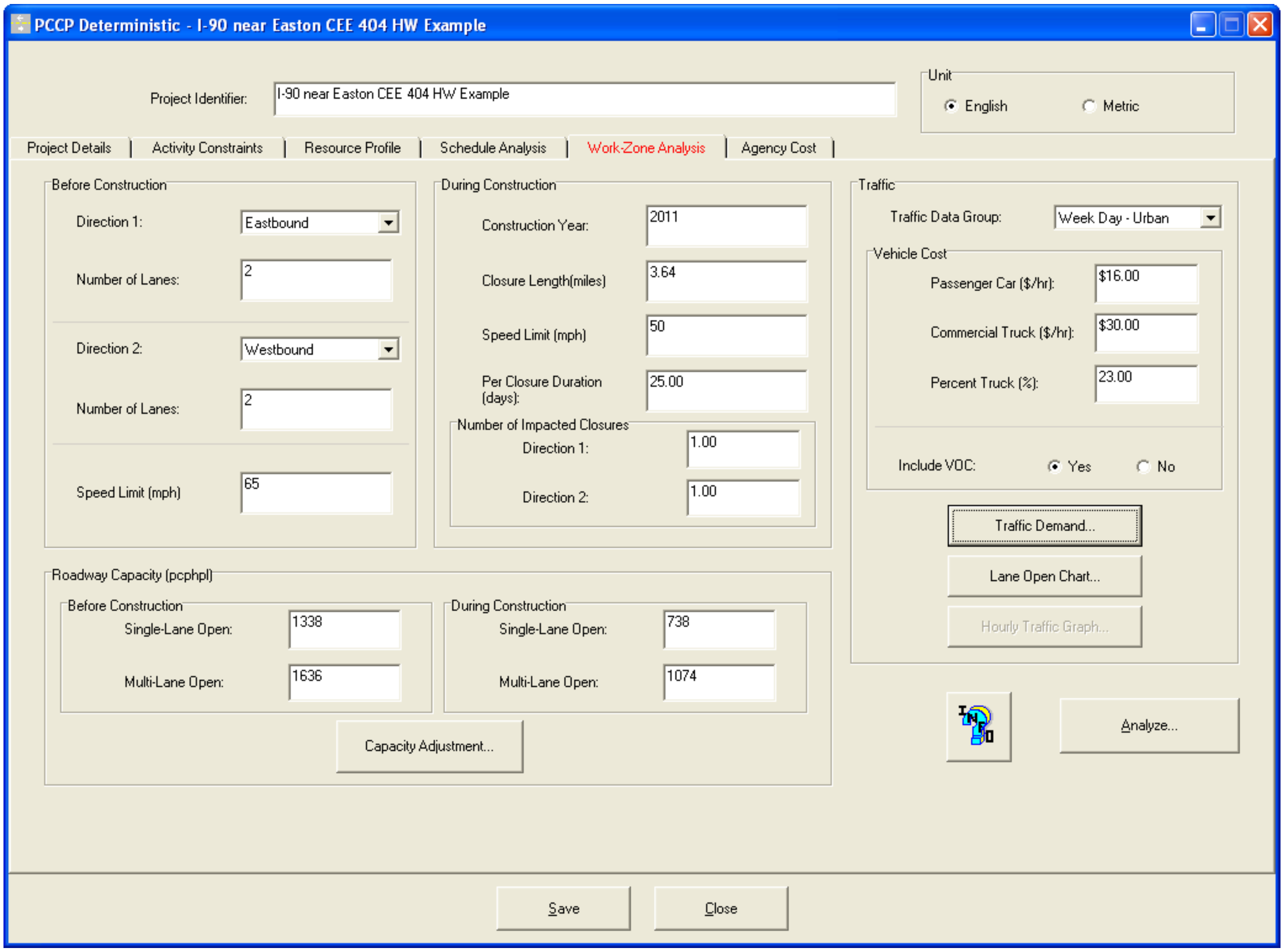
Period with highest delay in After Case

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 10.13. QuickZone 2.0 summary tables showing available traffic impact metrics.

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 hours of traffic through a defined work zone. Work zones are defined by the number of lanes closed, closure duration and work zone capacity (Figure 10.14). Traffic can be entered by hourly count or ADT can be entered and then distributed over 24 hours using hourly factors. CA4PRS can simulate, a one lane closure scenario over a 24-hour period. Longer closures are estimated by multiplying the results of one 24-hour analysis by the total number of closures. The 24-hour 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 vs. weekend or summer vs. fall). Output is similar to that of QuickZone (Figures 10.15 and 10.16). 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.



Project Identifier: I-90 near Easton CEE 404 HW Example

Unit: ● English ○ Metric

Before Construction

Direction 1: Eastbound

Number of Lanes: 2

Direction 2: Westbound

Number of Lanes: 2

Speed Limit (mph): 65

During Construction

Construction Year: 2011

Closure Length(miles): 3.64

Speed Limit (mph): 50

Per Closure Duration (days): 25.00

Number of Impacted Closures

Direction 1: 1.00

Direction 2: 1.00

Traffic

Traffic Data Group: Week Day - Urban

Vehicle Cost

Passenger Car ($/hr): $16.00

Commercial Truck ($/hr): $30.00

Percent Truck (%): 23.00

Include VOC: ● Yes ○ No

Roadway Capacity (pcphpl)

Before Construction

Single-Lane Open: 1338

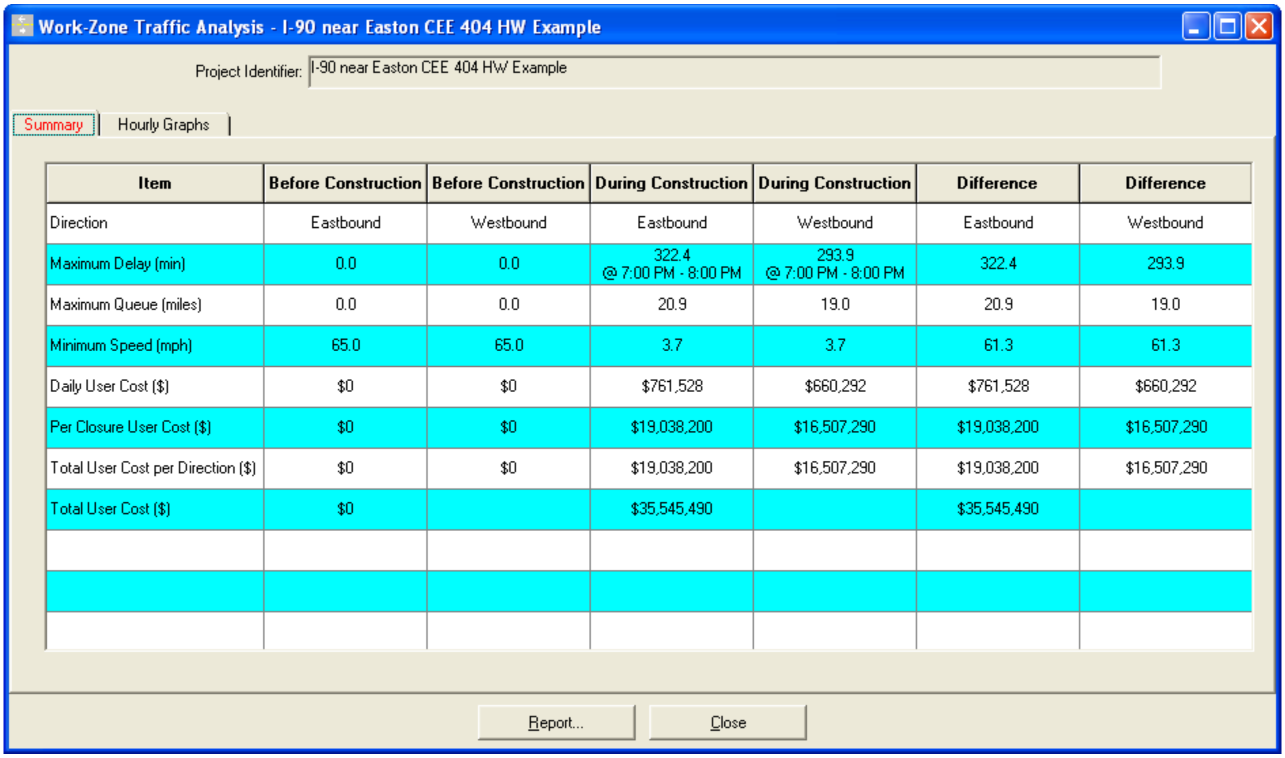
Multi-Lane Open: 1636

During Construction

Single-Lane Open: 738

Multi-Lane Open: 1074

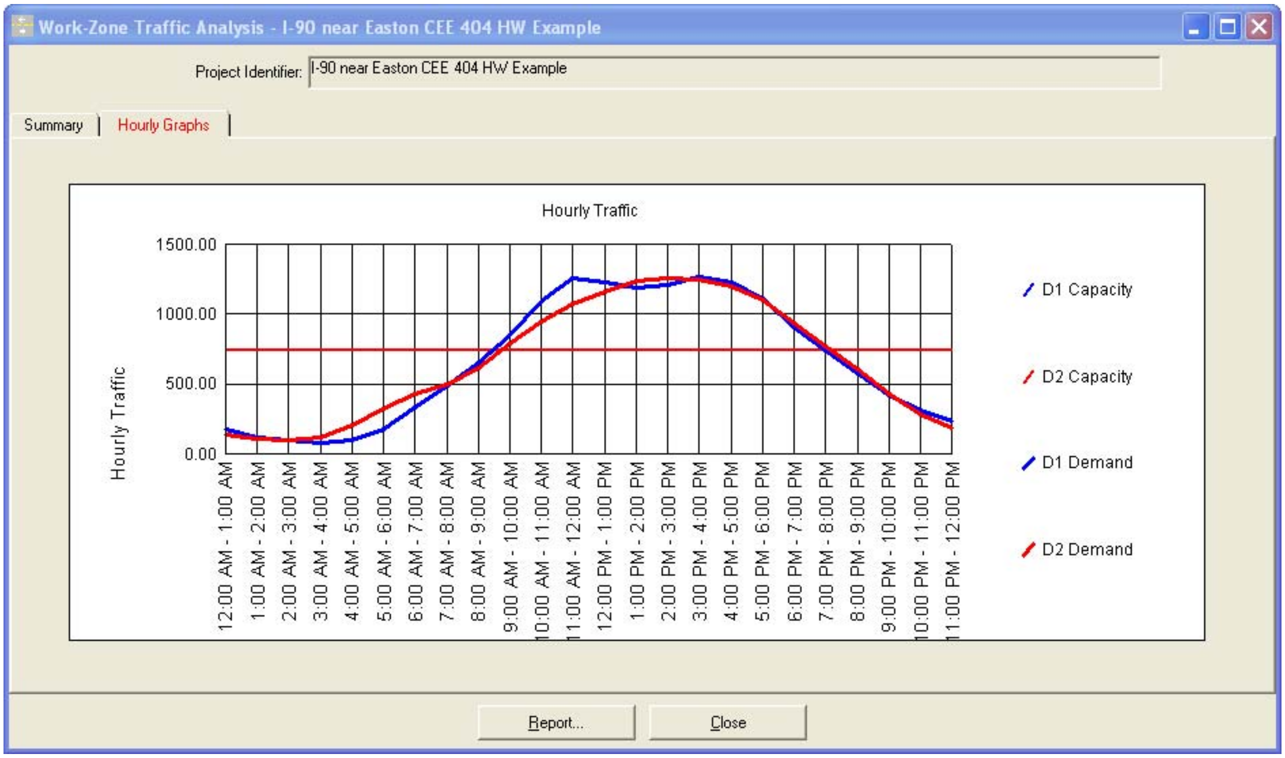
Figure 10.14 CA4PRS Work-Zone Analysis input screen.



Project Identifier: I-90 near Easton CEE 404 HW Example

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 10.15 CA4PRS Work-Zone Analysis summary results screen showing available traffic impact metrics.



Project Identifier: I-90 near Easton CEE 404 HW Example

Figure 10.16 CA4PRS Work-Zone Analysis hourly traffic results graph showing demand vs. capacity.

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Section 11
Life Cycle Assessment (Environmental Accounting)

11.1 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/ 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 U.S. and are increasingly being addressed through the National Environmental Policy Act (NEPA) as recent White House Council on Environmental Quality (CEQ) 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).

11.2 Measurement Methods

A 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 a 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, etc.) 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 phased approach:

1. **Goal and scope**. 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**. Quantify life cycle energy use, emissions, and land and water use for technology use in each life cycle stage.

3. **Impact assessment**. Estimate the impacts of inventory results.
4. **Interpretation**. Investigate the contribution of each life cycle stage, technology use throughout the life cycle, and include 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:

- **Data sources**. Often LCA data come from a select few databases such as the U.S. Life-Cycle Inventory Database (from NREL), ecoinvent, ELCD database, etc. These are generally reviewed for accuracy/errors and can help standardize information for use in LCAs. However, data usually comes 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 hot mix asphalt (HMA) production is not a universal constant, but rather varies depending upon plant type, components and manufacturer, aggregate moisture content, fuel type, amount of reclaimed asphalt (RAP) included, asphalt binder grade, crude oil source, regional electricity mix, etc. While 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, a 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 make-up 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, etc. For example, one of the more comprehensive sources for asphalt refining comes from Eurobitume and was produced in 2000.
- **Data specificity**. While general average data may be more readily available or lead to more consistency between LCAs, it often does not contain the detail needed to distinguish between two alternatives being considered. For instance, the EPA'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 this data is used it cannot distinguish between a mix using all virgin materials and one using 25 percent RAP.
- **Setting boundaries**. A 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 once

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 exist for pavements.

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

(ii) 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.

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, etc.) 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 a 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). EIO-LCA overcomes the subjective boundary issue and data availability issue by basing process 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 other. 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., Department of Commerce) during a certain time period (usually 5 years). This 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 does not contain enough detail to differentiate between two options (e.g., using warm mix asphalt or not).

(i) Typical Values

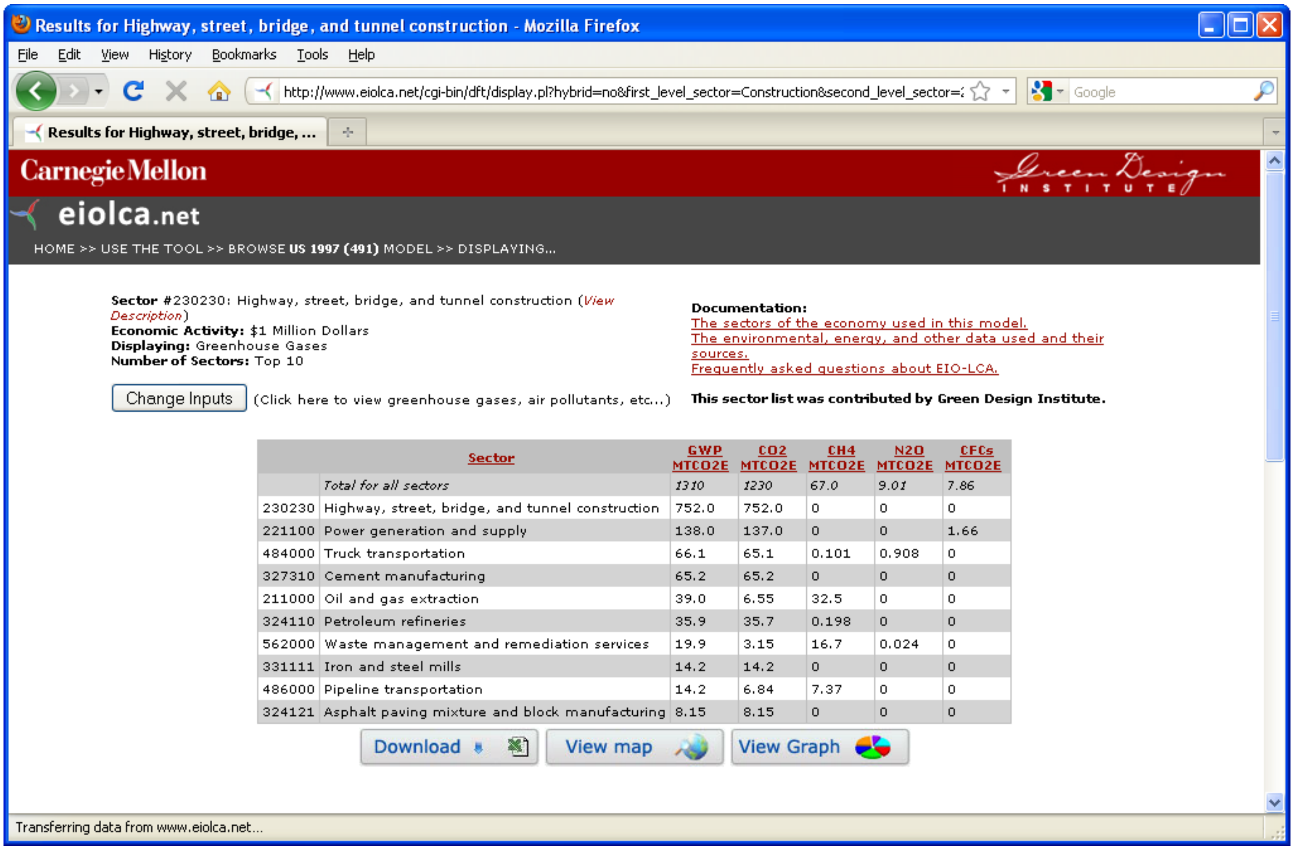
There have been a number of documented pavement LCAs in the past decade or so that can provide valuable information on typical values. Muench (2010) reviewed 12 pavement LCA papers/reports that documented 66 assessments of actual or hypothetical roadways and found:

- **System scope**. Most LCAs tend to address the pavement structure only and not include other road features (e.g., striping, guardrails, etc.). Analysis periods are usually 40-50 years.
- **Relation of roadway construction to traffic use**. A good rule-of-thumb is: the energy expended in initial construction of a new roadway is roughly equivalent to the energy used by traffic on the facility over 1-2 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, de-icing, sanding, drawbridge actions, toll booths, etc. Construction energy ranges from about 25-100 percent 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 3-7 TJ depending upon 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 200-600 tons depending upon the pavement section, maintenance activities, and LCA scope.
- **Contribution of roadway construction components**. The following general statements are reasonable:
    - Materials production accounts for about 60-80 percent of energy use and 60-90 percent of CO_2 emissions.
    - Construction accounts for less than 5 percent of energy use and CO_2 emissions.
    - Transportation associated with construction accounts for about 10-30 percent of energy use and about 10 percent of CO_2 emissions.
    - Maintenance activities account for a broad range of about 5-50 percent of energy and CO_2 emissions.

11.3 Analysis Tools

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

EIO-LCA. 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 11.1). 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).



Sector #230230: Highway, street, bridge, and tunnel construction (*View Description*)
Economic Activity: $1 Million Dollars
Displaying: Greenhouse Gases
Number of Sectors: Top 10

(Click here to view greenhouse gases, air pollutants, etc...)

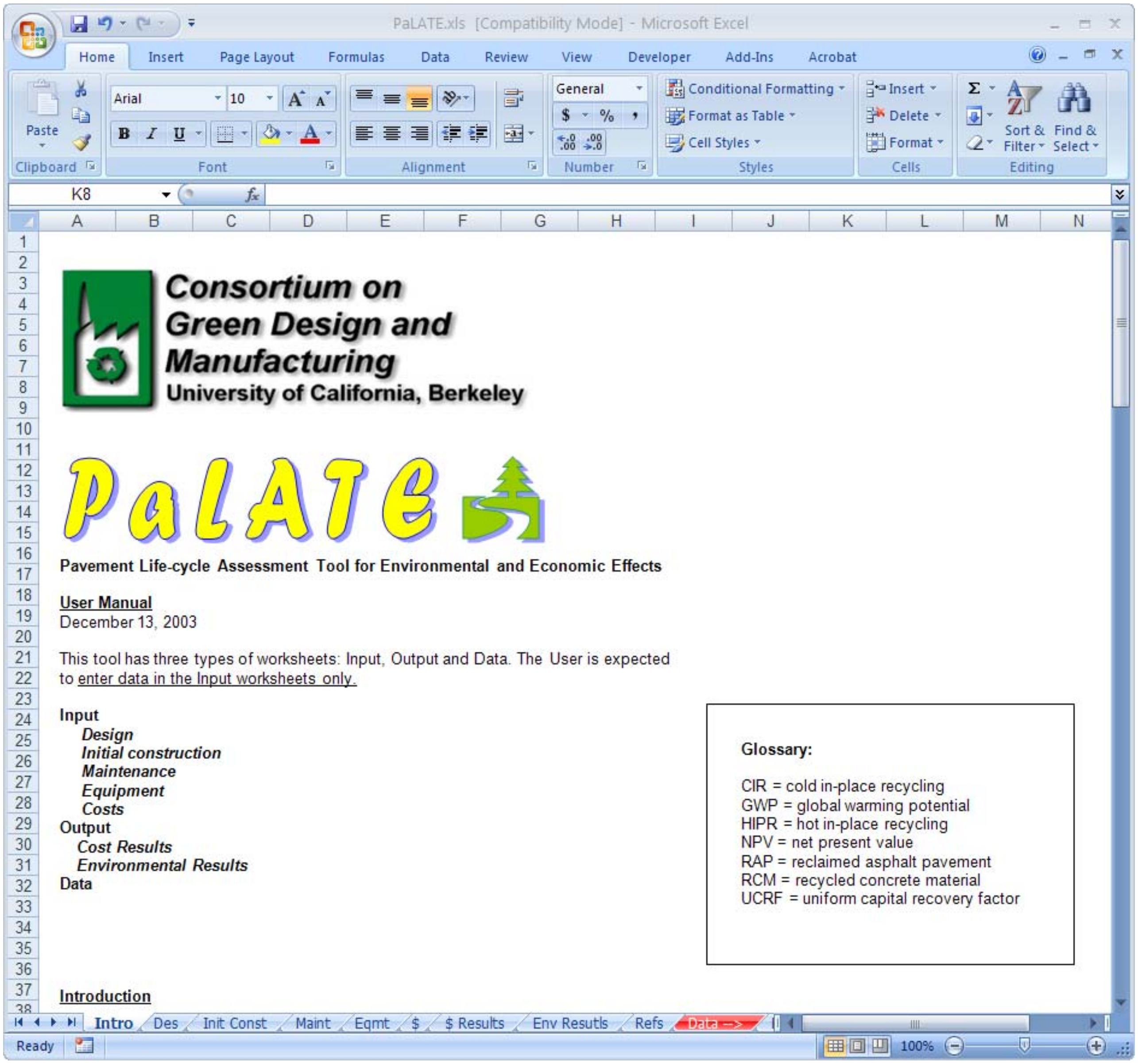
Documentation:
The sectors of the economy used in this model.
The environmental, energy, and other data used and their sources.
Frequently asked questions about EIO-LCA.

This sector list was contributed by Green Design Institute.

	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 11.1. 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.

PaLATE. 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 11.2). It is primarily built on the EIO-LCA method, but 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 in some cases, thus rendering PaLATE essentially useless. Recently, the University of Washington has rebuilt and simplified PaLATE (Figures 11.3 and 11.4) for interim use in their performance metric and have posted a working version on their website (www.greenroads.us). This version has not been validated by any outside party.



Consortium on Green Design and Manufacturing
University of California, Berkeley

PaLATE

Pavement Life-cycle Assessment Tool for Environmental and Economic Effects

User Manual
December 13, 2003

This tool has three types of worksheets: Input, Output and Data. The User is expected to enter data in the Input worksheets only.

Input
- ***Design***
- ***Initial construction***
- ***Maintenance***
- ***Equipment***
- ***Costs***

Output
- ***Cost Results***
- ***Environmental Results***

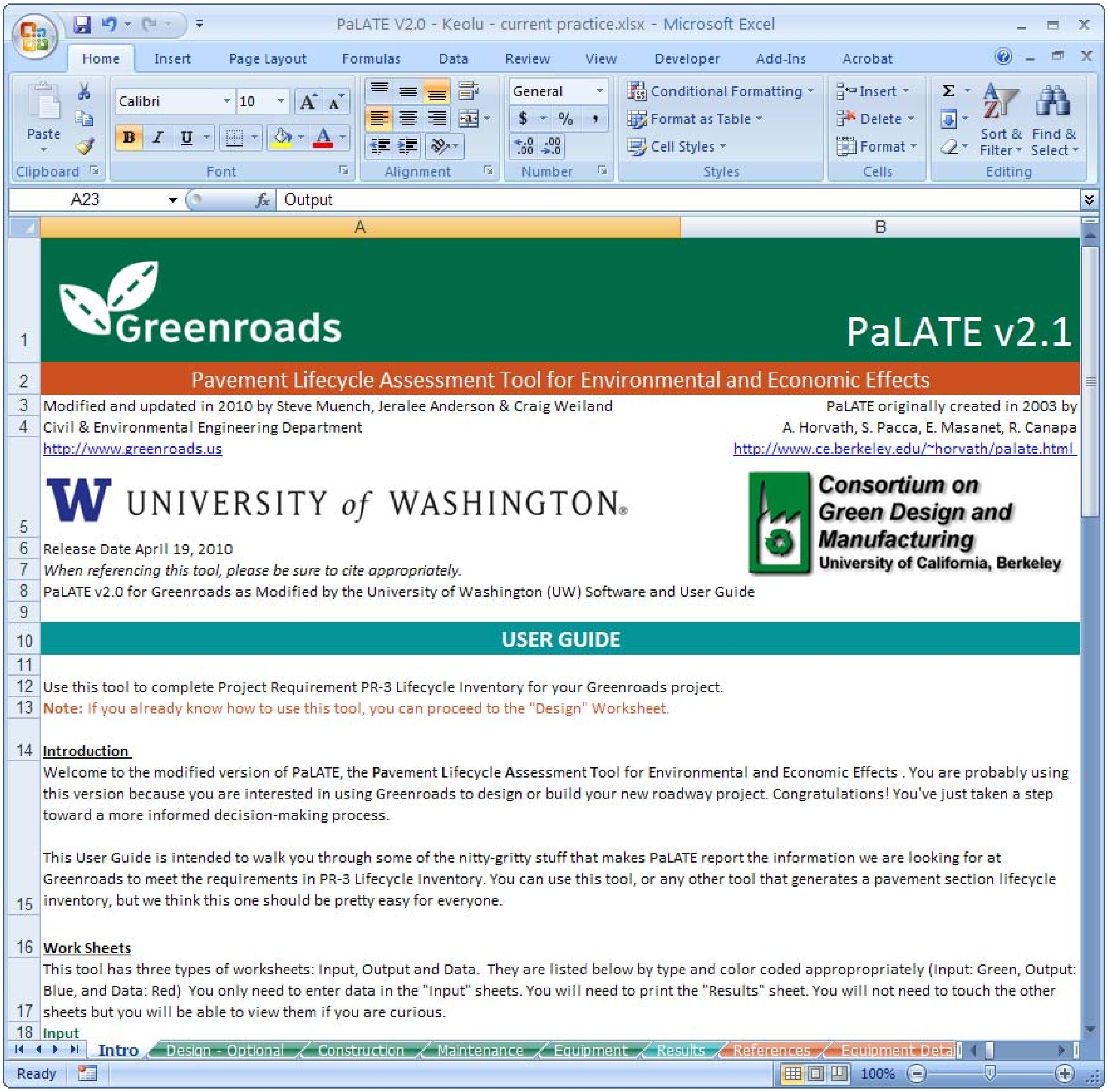
Data

Glossary:

CIR = cold in-place recycling
GWP = global warming potential
HIPR = hot in-place recycling
NPV = net present value
RAP = reclaimed asphalt pavement
RCM = recycled concrete material
UCRF = uniform capital recovery factor

Introduction

Figure 11.2. Introduction screen for PaLATE from the Consortium on Green Design and Manufacturing at the University of California, Berkeley.



Pavement Lifecycle Assessment Tool for Environmental and Economic Effects

Modified and updated in 2010 by Steve Muench, Jeralee Anderson & Craig Weiland
Civil & Environmental Engineering Department
http://www.greenroads.us

PaLATE originally created in 2003 by
A. Horvath, S. Pacca, E. Masanet, R. Canapa
http://www.ce.berkeley.edu/~horvath/palate.html

UNIVERSITY of WASHINGTON

Consortium on Green Design and Manufacturing
University of California, Berkeley

Release Date April 19, 2010
When referencing this tool, please be sure to cite appropriately.
PaLATE v2.0 for Greenroads as Modified by the University of Washington (UW) Software and User Guide

USER GUIDE

Use this tool to complete Project Requirement PR-3 Lifecycle Inventory for your Greenroads project.

Note: If you already know how to use this tool, you can proceed to the "Design" Worksheet.

Introduction

Welcome to the modified version of PaLATE, the **P**avement **L**ifecycle **A**ssessment **T**ool for **E**nvironmental and Economic Effects . You are probably using this version because you are interested in using Greenroads to design or build your new roadway project. Congratulations! You've just taken a step toward a more informed decision-making process.

This User Guide is intended to walk you through some of the nitty-gritty stuff that makes PaLATE report the information we are looking for at Greenroads to meet the requirements in PR-3 Lifecycle Inventory. You can use this tool, or any other tool that generates a pavement section lifecycle inventory, but we think this one should be pretty easy for everyone.

Work Sheets

This tool has three types of worksheets: Input, Output and Data. They are listed below by type and color coded appropropriately (Input: Green, Output: Blue, and Data: Red) You only need to enter data in the "Input" sheets. You will need to print the "Results" sheet. You will not need to touch the other sheets but you will be able to view them if you are curious.

Input

Figure 11.3. Introduction screen for PaLATE as modified by the University of Washington for use with Greenroads.

CHANGER. 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 the Traffic Facilities Laboratory (LAVOC) of the Swiss Federal Institute of Technology (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.).

(iii) Example LCA using PaLATE as modified by the University of Washington for Greenroads

A local collector road in Kailua, HI is scheduled for repaving. The work essentially involves removing 6 inches of HMA with a milling machine and replacing it with two layers of HMA: a 4-inch base course and a 2-inch 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-inch deep milling

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

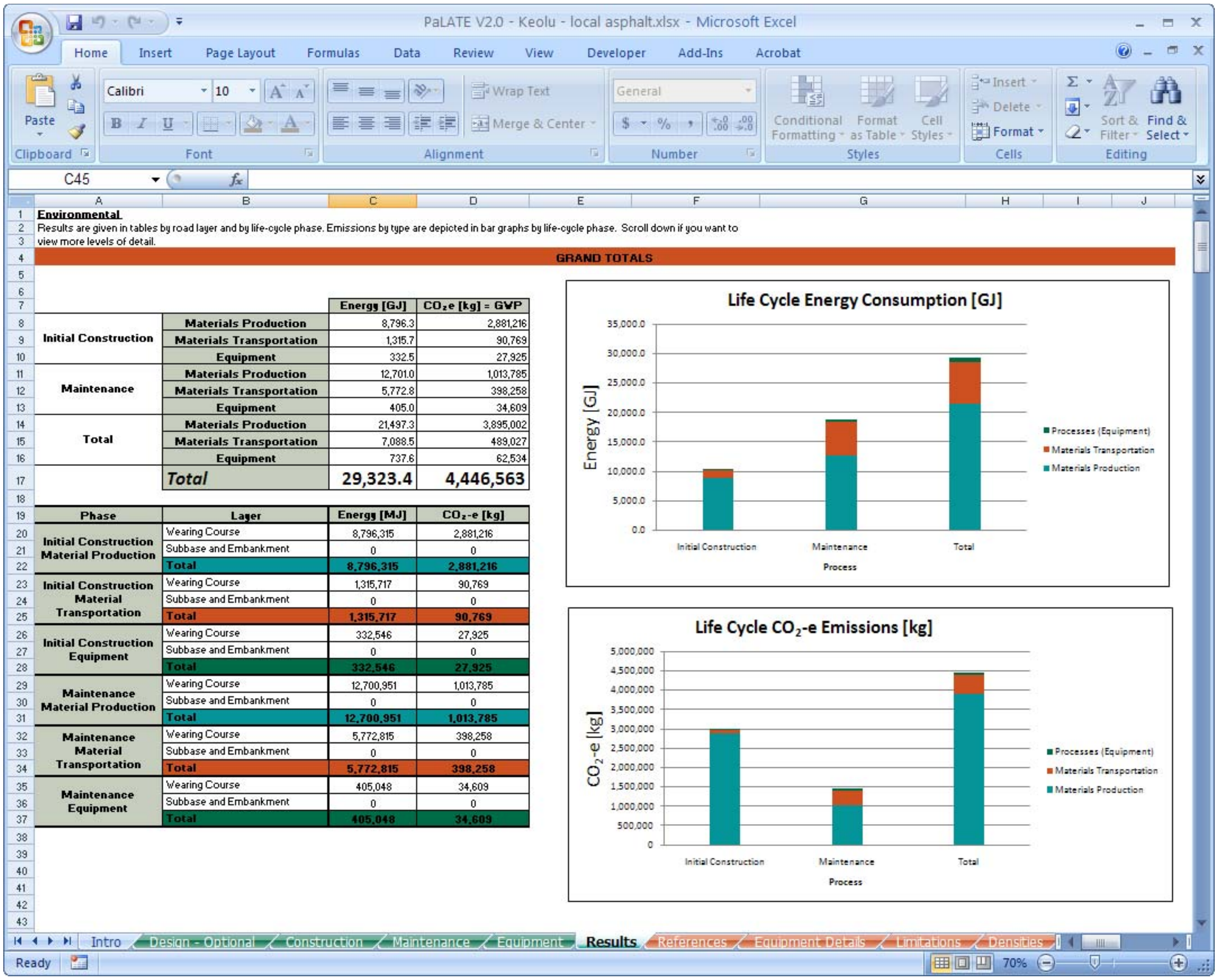
- Surface course: 7,913 tons of HMA
  - 5.5% asphalt by total weight of mix
  - No recycled material in the mix
- Tack coat: 0.15 gallons/yd^2 over 79,386 yd^2 = about 61 yd^3 of asphalt emulsion
- Milling: 79,386 yd^2 of 2-inch deep milling

Materials for this process come from the following locations:

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

Results from this analysis are (Figure 11.4):

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



Environmental

Results are given in tables by road layer and by life-cycle phase. Emissions by type are depicted in bar graphs by life-cycle phase. Scroll down if you want to view more levels of detail.

		Energy [GJ]	CO₂e [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]	CO₂-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 11.4. Screenshot of PaLATE output as modified by the University of Washington for use with Greenroads.

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

- 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).

Figure 11.5 through 11.7 shows the percentage change from the baseline practice in terms of energy consumption.

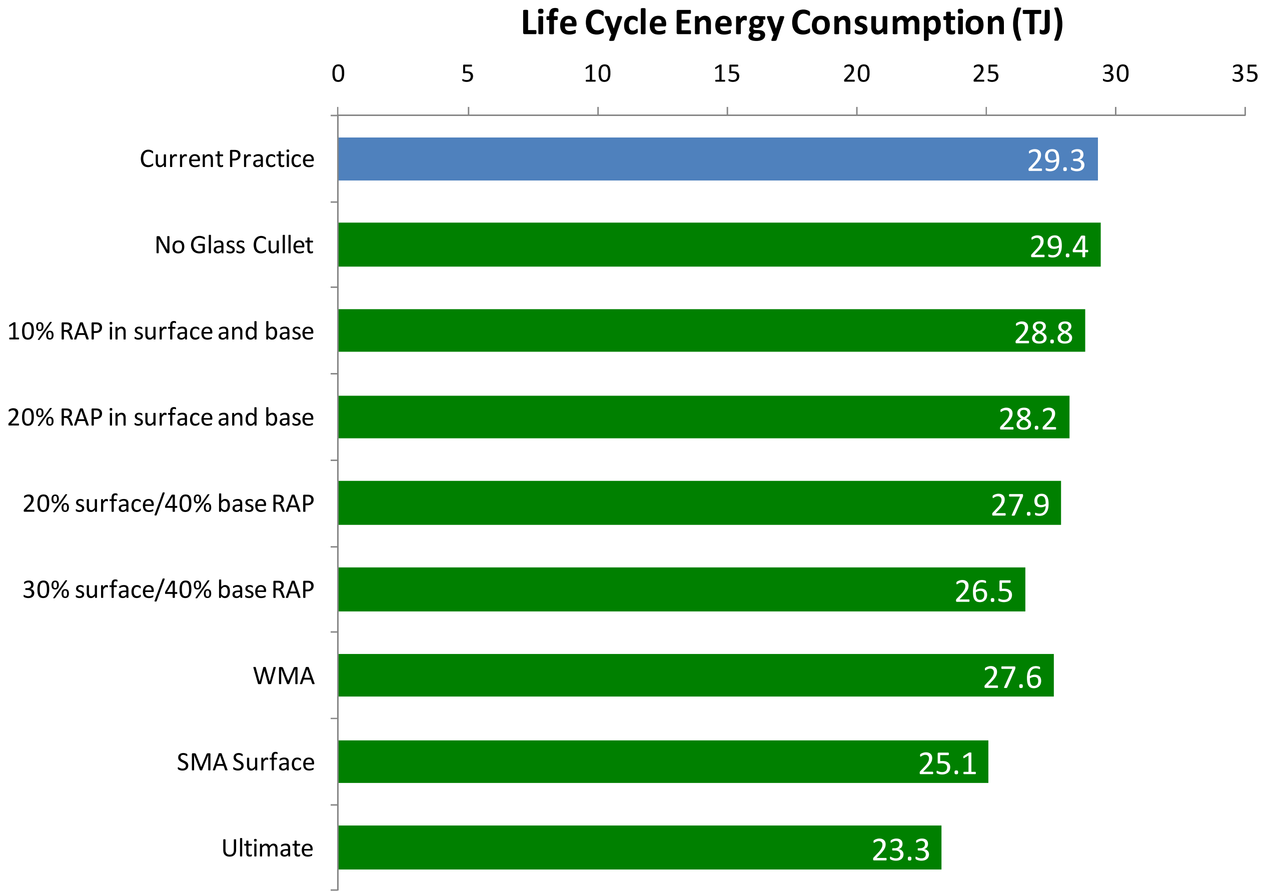


Figure 11.5. Life cycle energy consumption for the current practice and eight alternate scenarios for the example LCA.

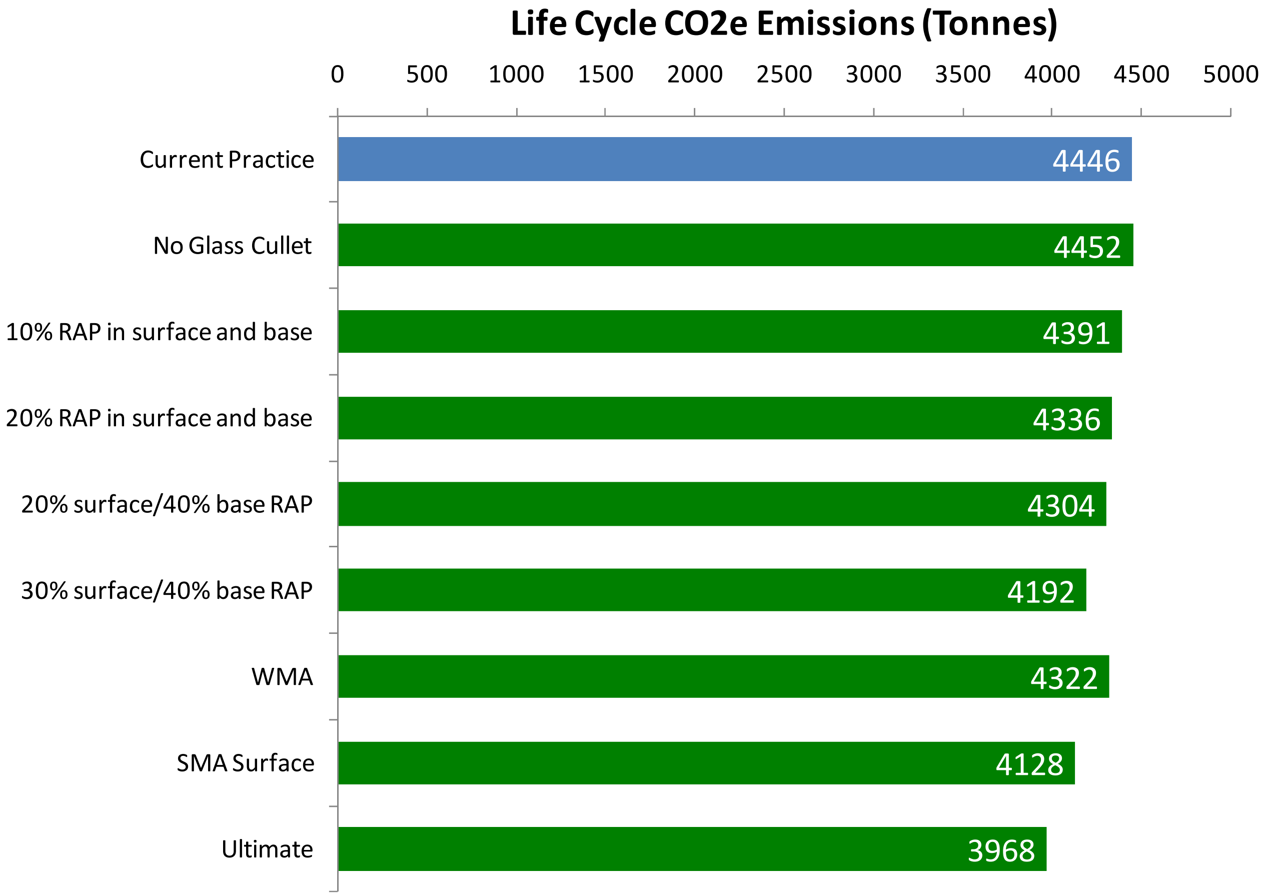


Figure 11.6. Life cycle CO_2 equivalent emissions for the current practice and eight alternate scenarios for the example LCA.

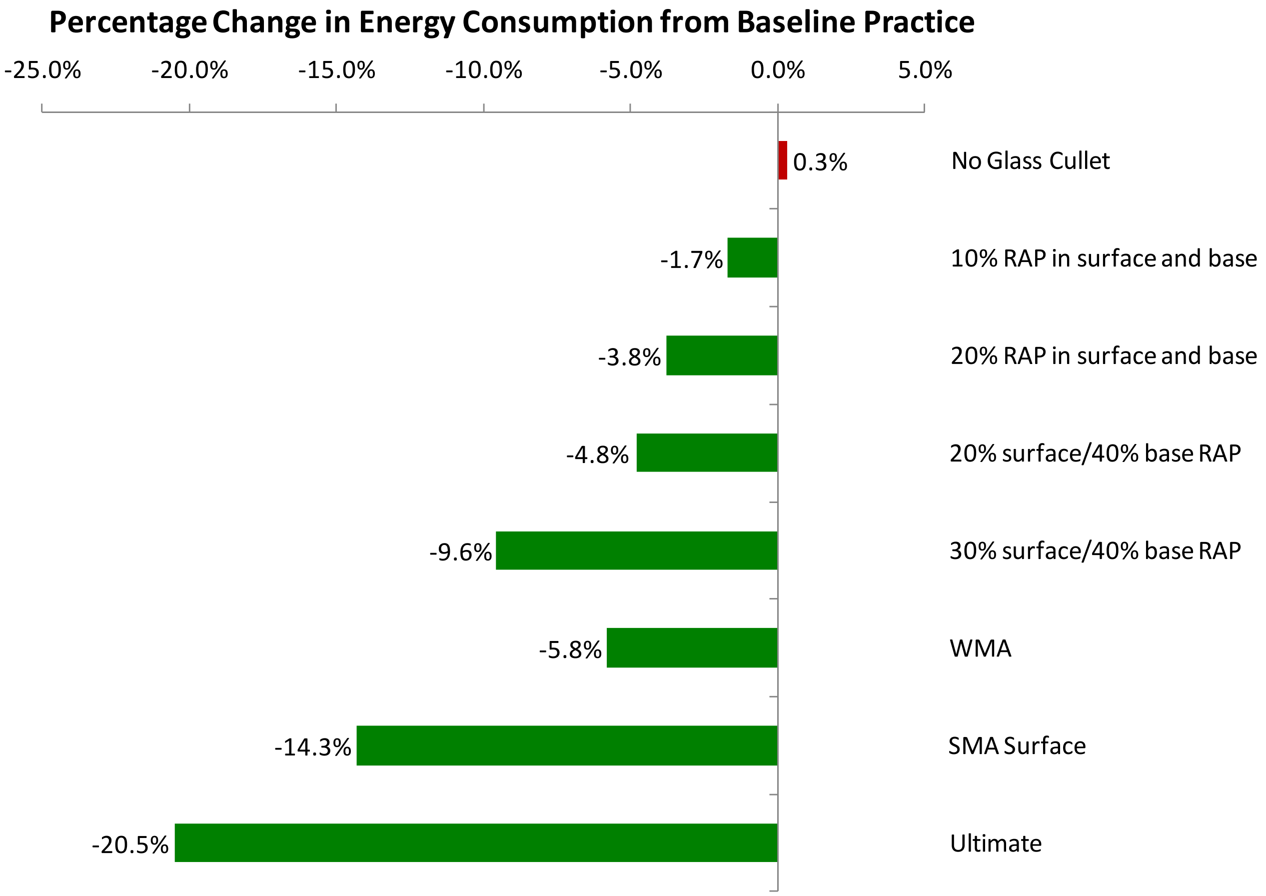


Figure 11.7. The percentage change from the baseline value of energy consumption for a number of alternate scenarios for the example LCA.

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

- 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.

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Section 12
Miscellaneous Material Properties

12.1 Purpose

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

12.2 Material Properties

Table 12.1 shows typical layer moduli for several material conditions. Table 12.2 shows information about rubblized PCC and Table 12.3 about crack and seat renewal.

Table 12.1 HMA Pavement Typical Moduli and Ranges of Moduli.

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

Table 12.2 PCC Pavement Rubblization Typical Moduli and Ranges of Moduli

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

Table 12.3 Crack and Seat and Break and Seat Renewal

Material	Value or Property
Typical modulus of crack and seated PCCP	200,000 psi
Modulus of crack and seated PCCP	Range: 200,000 to 800,000 psi
Modulus of break and seat PCCP	Range: 250,000 to 2,000,000 psi

12.3 References

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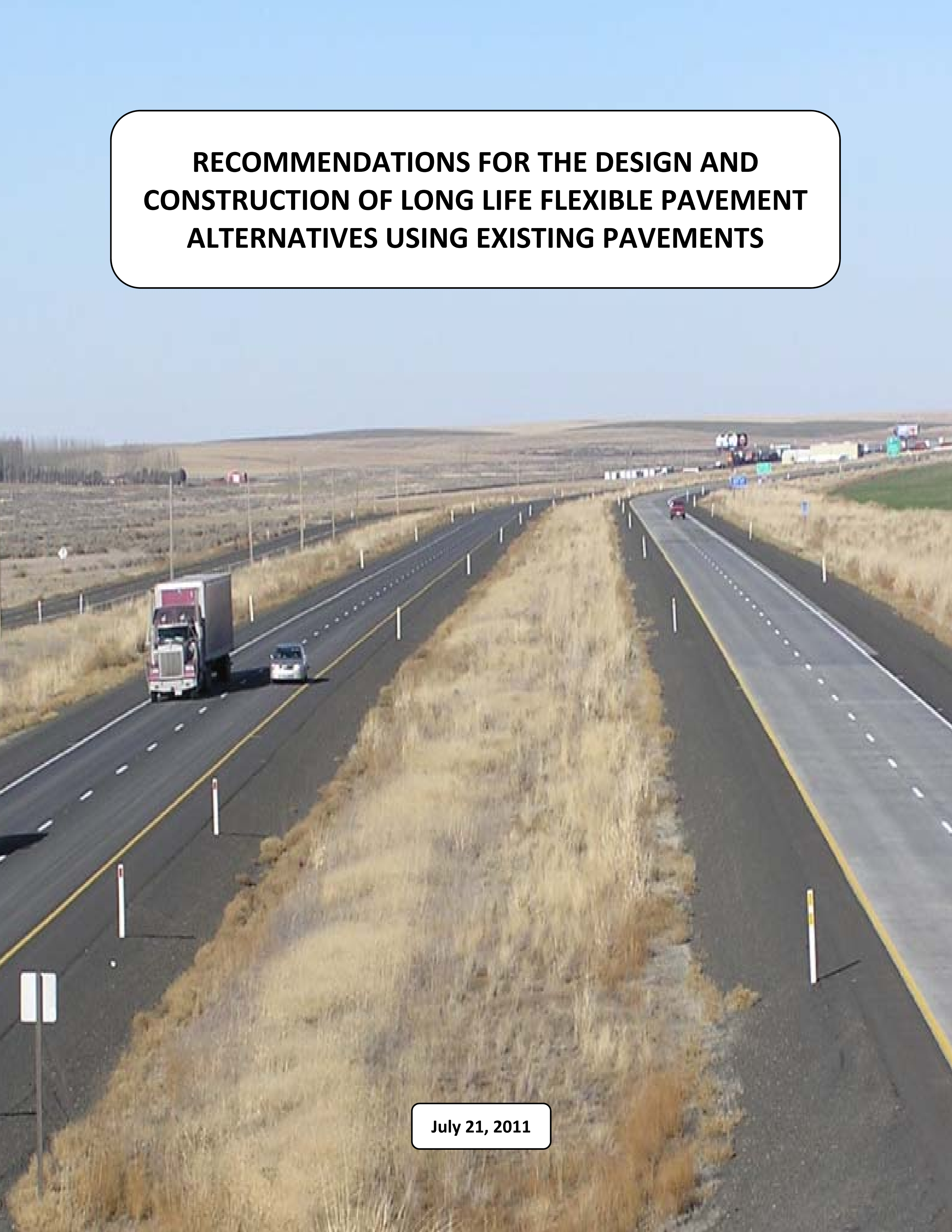
APPENDIX E-2

RECOMMENDATIONS FOR THE DESIGN AND CONSTRUCTION OF LONG LIFE FLEXIBLE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

RECOMMENDATIONS FOR THE DESIGN AND CONSTRUCTION OF LONG LIFE FLEXIBLE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS



July 21, 2011

Table of Contents

RECOMMENDATIONS FOR THE DESIGN AND CONSTRUCTION OF LONG LIFE FLEXIBLE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

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 DOTs and 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, environmental and material characteristics. While 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, e.g., 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 irrespective of the existing pavement type (i.e., distressed HMA or 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.

HMA Renewal Strategies

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

- 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 JPC pavements
    - HMA over saw, crack and seat JRC pavements
    - HMA over rubblized JPC pavements
    - HMA over existing 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/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 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 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 will be 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 1 shows a typical milling operation.

Figure 1. Typical milling operation of existing HMA layer. (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 prior to paving.

It is essential that bonding between the new wearing course and the existing pavement be assured 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 percent surface coverage (by ensuring double or triple coverage during spraying). Sufficient time should be allowed for the emulsion to break and dry before applying the next layer of HMA. Figure 2 shows examples of good and poor tack coat application. Milling the existing surface prior to an overlay significantly aids the bond between the two layers.

(a) Good tack coat (b) Poor tack coat (left portion of photo)

Figure 2. Illustration of good and poor tack coat. (WSDOT, 2010)

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 percent relative to the density of the main mat. Further, 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.

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/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 included in Table 1. The table includes a brief explanation why the issue is of special interest along with examples from the R23 guide specification recommendations. 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.
- The foundation (subgrade) support is good (e.g., the backcalculated 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 (Hall et al, 2001): (1) use of an excessive amount of recycling agent, (2) application of a surface seal prematurely, (3) recycling only to the depth of an asphalt layer, resulting in de-lamination 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 3 shows pictures of FDR construction with different stabilizing agents.

Partial-depth reclamation by cold in-place recycling (CIPR) is limited to correcting only those distresses which 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.

Table 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 prior to HMA overlay to reduce potential for reflection cracks in the new HMA layer. Milling is considered superior to crack sealing prior to 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 feet in length. An alternate grade referencing attachment may be used if approved by the Engineer prior to use. **[Refer to Elements for AASHTO Specification 409 for more details]**[1]
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/SY. • 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. **[Refer to Elements for AASHTO Specification 404 for more details]** [1]
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 inches of a longitudinal joint, including the pavement on the traveled way side of the shoulder joint, shall not be less than 2.0 percent below the specified density when unconfined. **[Refer to Elements for AASHTO Specification 401 for more details]** [1]

[1] Contained in Appendix E-4

(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

Figure 3. FDR construction with different stabilizing agents. (Bang et al, 2010)

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 re-screened. 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 minutes of curing and drying, the material is compacted with a large rubber-tired roller, followed by a vibratory steel drum roller. Curing of about two weeks during favorable weather conditions (preferably at temperatures at or in excess of 60°F) is needed before the new HMA overlay is applied (FHWA, 1997). The addition of quick lime has been used to significantly reduce the cure time. Figure 4 shows typical CIPR train operations.

(a) CIPR Train with engineered asphalt emulsion

(b) CIPR Train with addition of lime slurry or cement in slurry

Figure 4. Typical CIPR train operation. (Hot-Mix Magazine, 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 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 percent at a minimum of 97 percent of laboratory maximum density (FHWA, 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. The table includes a brief explanation why the issue is of special interest along with examples from the study guide specification recommendations. One major practice is featured which is the gradation of the pulverized material.

Table 2. Best practices and specifications for HMA over reclaimed HMA pavement.

Best Practice	Why this practice?	Typical Specification Requirement
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 to 100% passing the 1.5 in. sieve. • Reject subgrade materials that can contaminate the pulverized asphalt pavement. **[Refer to Elements for AASHTO Specification 411[1] and the AASHTO Guide Specification 411]**

[1] Contained in Appendix E-4

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 5 shows a photo of an exposed concrete pavement after removal of the HMA overlay.

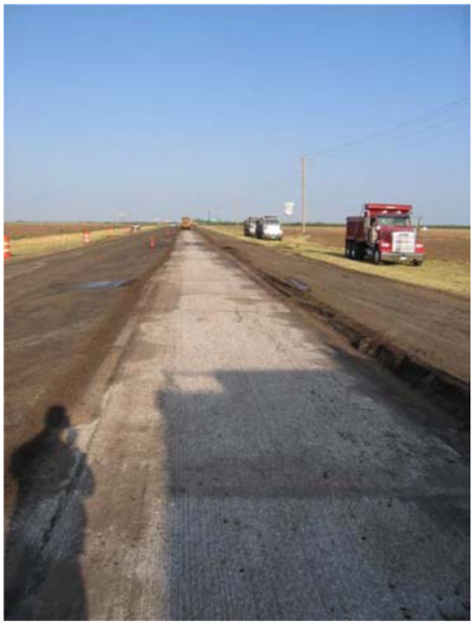


Figure 5. Existing concrete pavement exposed after removal of HMA layer.
(Sebesta and Scullion, 2007)

HMA over Crack and Seat 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 are 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 6).

The HMA overlay over crack and seat concrete 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/subbase).
- The existing drainage system is in good working condition.

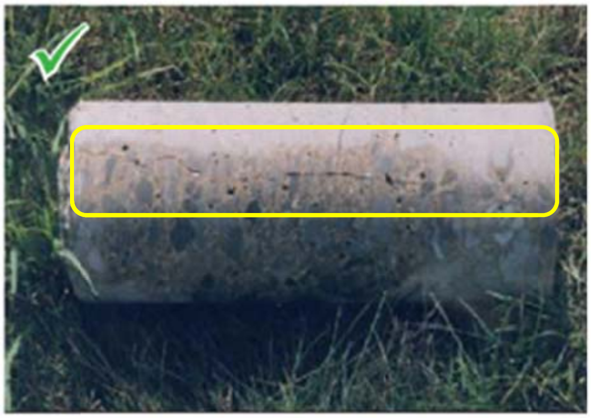
However, the following limitations and additional cautions are warranted:

- The performance of HMA overlays on crack and seat concrete pavements has been variable in the US; 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 strict quality control process and HMA overlay thickness in excess of 7 in. Thinner overlays like those commonly used in the US 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 job site 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.


(a) Example showing excessive longitudinal cracking


(b) Example showing good transverse cracking


(c) Non-compliant core: Over-cracked


(d) Compliant core: Fine, full-depth vertical crack

(e) Compliant crack illustrated by core hole (f) Compliant crack illustrated by reassembled core

Figure 6. Examples of poor and good practices of crack and seat.
(Jordan et al. 2008 (a)-(d); WSDOT 2011)

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 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 ESALs). Starting in 2003 with the Interstate 710 rehabilitation of existing 8 in. thick PCC slabs near Long Beach, CA (Monismith et al, 2009a and 2009b), the crack and seat process is followed by HMA overlays totaling 9 in. thick. The design ESAL levels for these sections of I-710 have ranged between 200 to 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 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 the recent California data does 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 which precede paving HMA over crack and seated PCC pavement was selected and included in Table 3. The table includes a brief explanation why the issue is of special interest along with examples from the study guide specification recommendations.

HMA over Saw Crack and Seat JRC Pavements

Criteria for Long Life Potential

It has been established that the crack and seat technique of fracturing reinforced concrete pavements (JRCP) has not been successful because of the inability to break the bond between the reinforcing steel and concrete nor to 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.

Table 3. Best practices and specifications for HMA over crack and seated PCC pavement.

Best Practice	**Why this practice?**	**Typical Specification Requirement**
Cracking Operations	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 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 Caltrans, suggests that a larger cracking pattern can work well for 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). **[Refer to Elements for AASHTO Specification 567[1] and AASHTO Guide Specification 567 for more details]**

[1] Contained in Appendix E-4

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 7. The same precautions as noted for crack and seat construction apply. The depth of the cut can be determined from coring and/or ground-penetrating radar (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 8). The spacing of saw-cuts should be similar to the cracking pattern used in the crack and seat procedures. The UK 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 7. Sawing of concrete slabs. (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 Department of Transport (UK, 2010) also requires cores to verify that the steel reinforcing has been cut and the slab is fully cracked. In addition, they require FWD deflection testing and backcalculation to verify a minimum modulus (termed effective stiffness modulus) of the PCC layer following cutting, cracking and seating.

Following cutting and cracking, the Department of Transport (UK, 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).

(a) Non-compliant core: Steel reinforcement not severed

(b) Compliant core: Fine, full-depth, vertical crack

(c) Spalling of saw-cut because of no strike plate use

(d) Strike plate in use

Figure 8. Examples of poor and good practices of sawcut, crack and seat. (Jordan et al. 2008)

HMA over Saw, Crack, 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 4. The table includes a brief explanation why sawing the existing reinforcing steel is of special interest along with examples from the study guide specification recommendations.

Table 4. Best practices and specifications for HMA over saw, crack and seat jointed reinforced PCC.

Best Practice	Why this practice?	Typical Specification Requirement
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: Prior to sawing, the following work must be complete: a. If required, construct pavement drainage systems at least two 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 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. **[Refer to R23 Guide Specifications for Saw, Crack and Seat Elements[1] for more details]**

[1] Contained in Appendix E-4

HMA over Rubblized Concrete Pavements

Criteria for Long Life Potential

In principle, rubblization effectively eliminates the problem of reflection cracking, since the technique is supposed to completely disintegrate the existing concrete slab. However, it also reduces the strength of the existing concrete pavement substantially since 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; although, crack and seat processing is generally preferred to rubblization since 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/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/subbase and/or the existing subgrade and produce an unstable base layer.
- Moisture problems, soft spots, and voids underneath the slab should be addressed prior to rubblization for enhanced performance.

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

Construction Operations

Rubblizing involves breaking the existing concrete pavement into pieces, and thereby destroying any slab action, and overlaying with HMA. The sizes of the broken pieces usually range from 2 to 6 in. (APA, 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 CRCP presents challenges and requires extra care in QC/QA 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 NAPA indicated that strength of the rubblized layer is 1.5 to three 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 between 100 to 135 ksi.
- For slabs 8 to 14 in. thick: E_{rub} ranges between 135 to 235 ksi.
- For slabs > 14 in. thick: E_{rub} ranges between 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 percent. Further, 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 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.

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 maintaining a strong base and/or subgrade soil. Poor performance can occur when the underlying soils are saturated. Installation of edge drains prior to rubblization has proven to be successful for this type of condition. If the existing concrete pavement is deteriorated due to 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 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 counter-weight situated on top of the beam. The principle on which the resonant breaker operates is that a low amplitude (about 0.5-inch) 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-degrees 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/subbase material and is approximately 1.0 to 1.5 lane-miles/day.

(a) Resonant breaker machine (b) Close-up of the sonic shoe

Figure 9. Resonant frequency pavement breaker. (Baladi et al, 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 shut down. Bituminous patches or un-milled overlays can also be problematic, as the shoe penetrates the asphalt causing a large loss in the energy delivered to the concrete. Lastly, the type of base/subbase material, the roadbed/subgrade soil and the condition of the concrete pavement being rubblized all affect the quality of the rubblized product. For example, if the base/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 subgrade soil.

The Process: It is recommended to begin rubblization 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. Reduce any particle greater than 6 inches in largest dimension remaining on the pavement surface to an acceptable size or remove and fill the area with granular base. Cut off any projecting reinforcing steel below the rubblized surface and dispose of it. 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, and
- Follow with at least two more passes with the vibratory roller.

The rolling pattern may be changed as directed.

The **multi-head breaker** operation includes multiple drop hammers arranged in two rows on a self-propelled unit and a vibratory grid roller (Figure 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 degrees 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/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 multi-head breaker to reduce the size of the broken concrete. The rate of production of the multi-head breaker depends on the type of base/subbase material and is about 0.75 to 1 lane-mile/10 hour shift. Several variables affect the rubblization process including: speed, height, weight and frequency of the drop hammers. The multi-head 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.

The Process: It is recommended to rubblize the entire lane width in one pass. Provide a screen to protect vehicles from flying particles. Reduce any particle greater than 6 in. in largest dimension remaining on the pavement surface to an acceptable size or remove and fill the area with granular base. Cut off any projecting reinforcing steel below the rubblized surface and dispose of it. 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, and
- At least two passes from a medium weight pneumatic roller

The rolling pattern may be changed as directed.

(a) Multi-head breaker

(b) Grid roller

Figure 10. Multi-head pavement breaker. (Baladi et al., 2000)

Figure 11 shows examples of good and poor rubblization outcomes.

(a) Rubblized Layer from MHB (b) Rubblized Layer from RMI

(c) Partial debonding of temperature steel (d) Partial destruction of the joint integrity

Figure 11. Examples of rubblized concrete pavements.
(Sebesta and Scullion, 2007 and Baladi et al., 2000)

Rubblized Concrete Size Requirements

Construction related problems with non-uniform particle size distribution throughout the PCC slab thickness will lead to underperforming pavements. Also, pavement sections that have been “over rubblized” (i.e., with rubblized pieces less than 2 inches in size) have a higher probability of cracking prematurely. Table 5 summarizes size requirements by various state highway agencies in the US. In addition, recent rubblization particle size information was summarized for the Wisconsin DOT (WisDOT, 2010). The results available in Table 5 and those from WisDOT differ somewhat; thus, the information shown must be used with significant judgment.

Table 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 ≤ 8 in
Arkansas	d < 6 in 100% @ d ≤ 8 in 51% @ 1 in < d < 3 in	d < 6 in 100% @ d ≤ 8 in 51% @ 1 in < d < 3 in	d < 6 in 100% @ d ≤ 8 in 51% @ 1 in < d < 3 in
Illinois	See next columns	75% @ d ≤ 3 in 100% @ d ≤ 9 in	75% @ d ≤ 9 in 100% @ d ≤ 12 in
Ohio	N/A	100 % @ d < 6 in 100% @ 1 in < d < 2 in	100 % @ d < 6 in 51% @ 1 in < d < 2 in
Pennsylvania	d < 6 in 100% @ d ≤ 8 in 51% @ d ≤ 4 in	d < 6 in 100% @ d ≤ 8 in 51% @ d ≤ 4 in	d < 6 in 100% @ d ≤ 8 in 51% @ d ≤ 4 in
Indiana	d < 6 in 51% @ 1 in < d < 2 in	d < 6 in 100% @ 1 in < d < 2 in	d < 6 in 51% @ 1 in < d < 2 in
Texas	60% @ d ≤ 3 in 100% @ d ≤ 6 in	60% @ d ≤ 3 in 100% @ d ≤ 6 in	75% @ d ≤ 9 in 100% @ d ≤ 12 in
FAA	75% @ d ≤ 3 in d ≤1.25 D	75% @ d ≤ 3 in d ≤1.25 D	75% @ d ≤ 12 in 100% @ d ≤ 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 project to be analyzed for its suitability for rubblization. Performing the following steps enables making this determination (Sebesta and Scullion, 2007):

- Evaluate the DCP data using a modified version of the IDOT rubblization selection chart (shown in Figure 12) as follows:
  - Plot the concrete thickness versus the CBR of the base. These data are used to gauge whether the concrete will rubblize, since 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 inches 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).

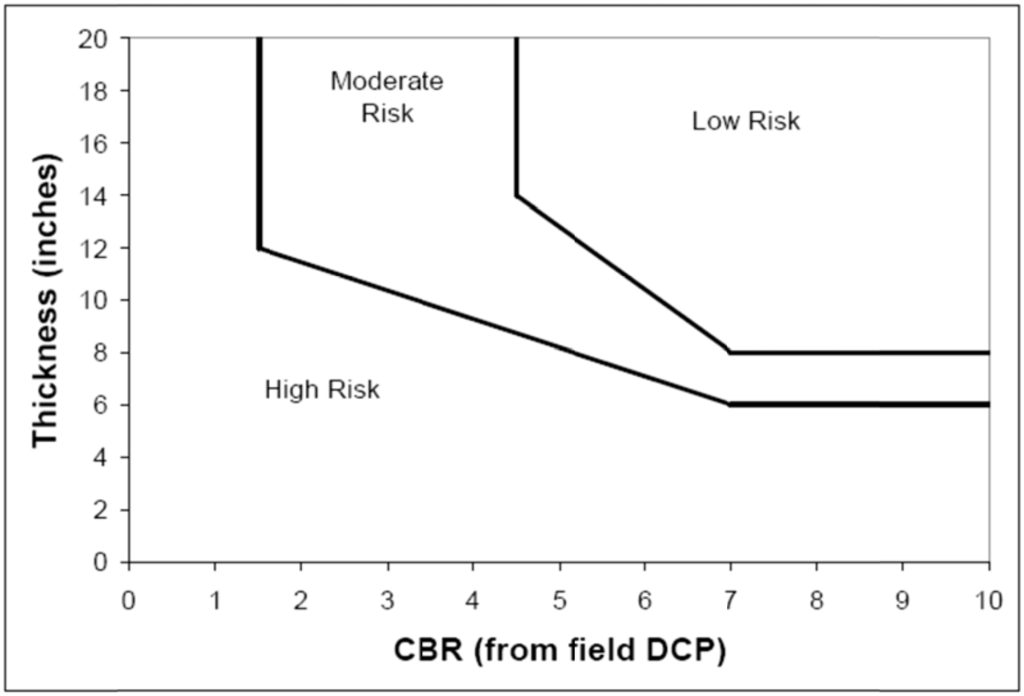


Figure 12. Modified IDOT rubblization selection chart as proposed by TTI-TxDOT. (Sebesta and Scullion, 2007)

- If all the data points fall in the zones that indicate rubblization is feasible, the project should be suitable for rubblization.
- If all the data points fall in the High Risk zone of the chart, rehabilitation options other than rubblization (crack and seat for JPCP, sawcut 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 on older concrete pavements where there is no or insufficient base support. Perform additional analysis as follows:
  - Determine the average CBR of the first 12 inches 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 inches 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 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 are included in Table 6. The table includes a brief explanation why the issue is of special interest along with examples from the study guide specification recommendations. Four major practices are featured: (1) work needed prior to rubblization, (2) the rubblization process and associated compaction, (3) verification of rubblization, and (4) traffic control.

HMA over 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 CRCP is in good condition and a good bond between the HMA overlay and the CRCP 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 CRCP is good (i.e., the deflection is low and there are no major defects such as spalling, punchouts, depressions and 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/subbase).
- The existing drainage system is in good working condition or a drainage system can be put in place.

Table 6. Best practices and specifications for HMA over rubblized jointed plain PCC pavement.

Best Practice	Why this practice?	Typical Specification Requirement
Work prior to 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. **[Refer to Rubblization Guide Specification for details]**[1]
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 percent of broken pieces less than 3 inches in size. At the surface of the rubblized layer, all pieces shall be less than 6 inches. Below the reinforcing steel or in the lower half of the pavement, the maximum particle size shall be 9 inches. **[Refer to Rubblization Guide Specification for details]** [1]
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 ft. 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. **[Refer to Rubblization Guide Specification for details]** [1]
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 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. **[Refer to Rubblization Guide Specification for details]** [1]

[1] Contained in Appendix E-4

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 US 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/Repair and Overlay Depths

For HMA over CRCP pavements, the following surface preparations and/or repairs are recommended by 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 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/seal the cracks prior to overlay (see Figure 14).
- 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 mm and 1.5 mm) (see Figure 16), 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 15), settlement, faulted cracks, and severe spalls, all distressed areas should be repaired with concrete, before overlaying with HMA.

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

(a) closely spaced tight transverse cracks

(b) tight bifurcated cracks

Figure 13. Examples of minor cracks in CRCP. (Jordan et al. 2008)

(a) Spalled cracks

(b) Intersected crack pattern

Figure 14. Examples of major crack defects in CRCP. (Jordan et al. 2008)

(a) Punchout strip

(b) Severe punchout block

Figure 15. Examples of “punchouts” in CRCP. (Jordan et al. 2008)

HMA over Existing CRC Pavement and Specifications

A significant issue associated with paving HMA over existing CRCP was selected and included in Table 7 full depth patching. The table includes a brief explanation why the issue is of special interest along with examples from the study guide specification recommendations.

Table 7. Best practices and specifications for HMA over existing continuously reinforced PCC.

Best Practice	Why this practice?	Typical Specification Requirement
Full depth patching process	The described steps are a systematic process for making any needed patches in the CRCP prior to 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 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. **[Refer to Elements for AASHTO Specification 558 for more details]**[1]

[1] Contained in Appendix E-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/bridge structure abutment or before passing under a structure where there is not sufficient clearance to meet standards. The length of this transition section depended upon 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 when designing 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 below in Figures 16 and 17.

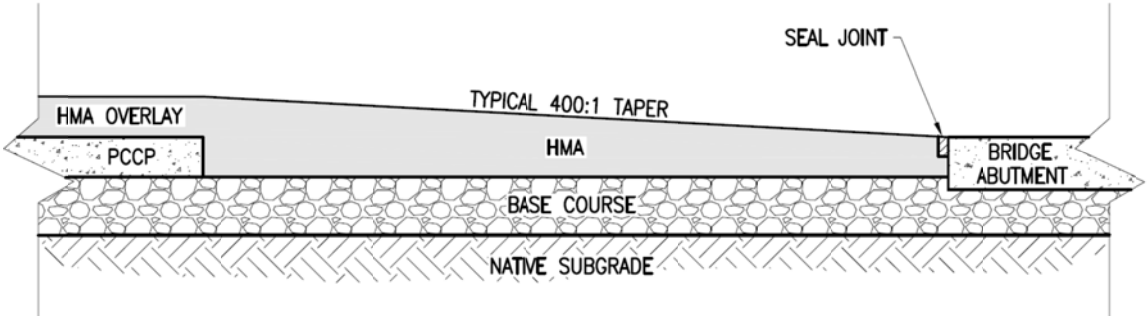


Figure 16. Diagram of transition to bridge approach

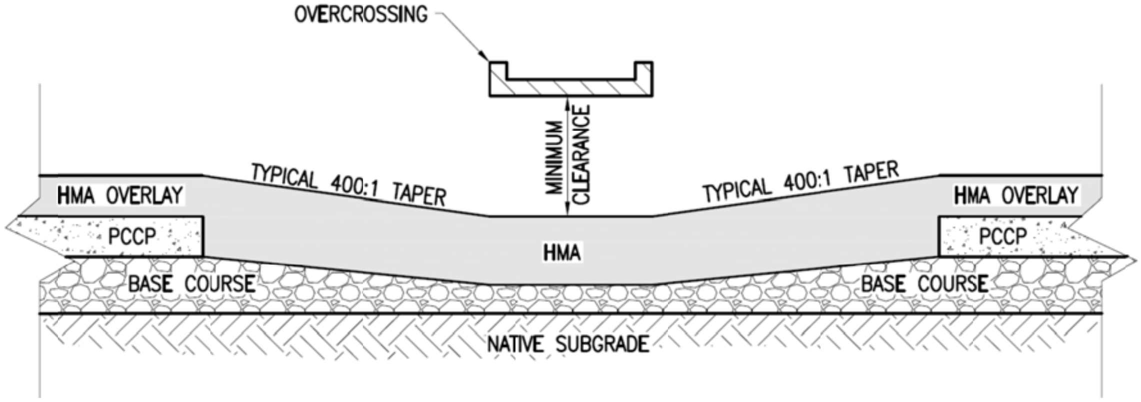


Figure 17. Diagram of transition beneath structure.

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 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 old PCC pavement. Specifically, the SHRP 2 guidance for "Geotechnical Solutions for Transportation Infrastructure" and their 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 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.

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 prior to 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.

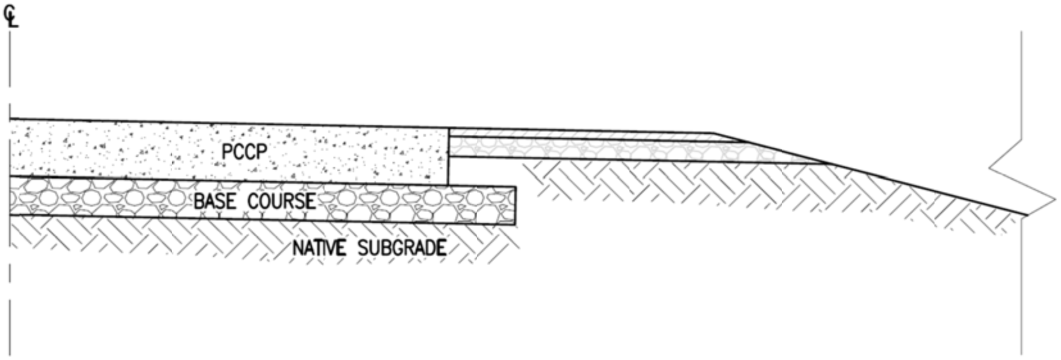


Figure 18. Showing existing PCC pavement.

Figure 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.

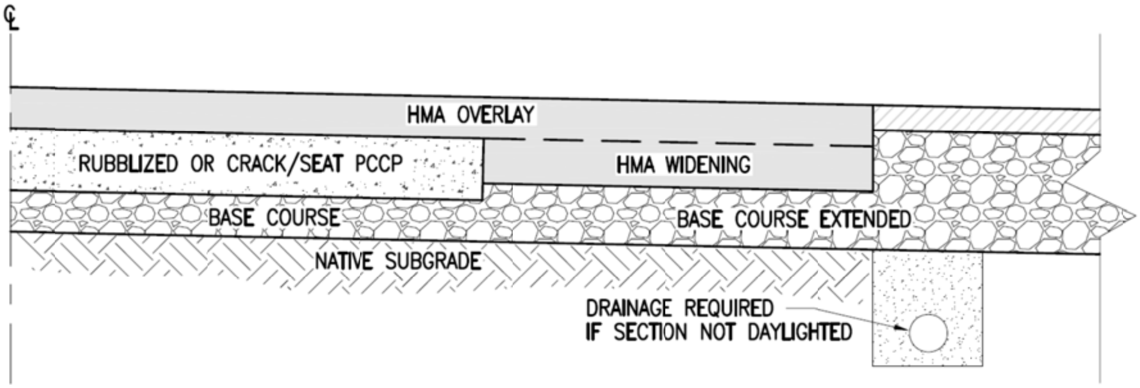


Figure 19. Illustration of widening the shoulder with daylighting or drainage installed.

Widening Next to Cracked and Seated or Saw 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 upon 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 (Hoierner, 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 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 thresholds (endurance limits) where the distresses begin to occur. Thus, an asphalt pavement could be designed for an "indefinite" structural life by designing 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 20 (Newcomb et al, 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, 1999). 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 NCAT suggests even higher fatigue endurance limits could be possible (Willis et al., 2009).

Pavement Design Software

In principle, adopting the limiting strain criteria for design allows for using any layered elastic analysis computer program, since the main output needed is the strain values at specific depths. However, a program that was developed specifically for the purpose of design long life HMA pavements is the PerRoad software (Timm, 2008). The program uses the basic M-E 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:

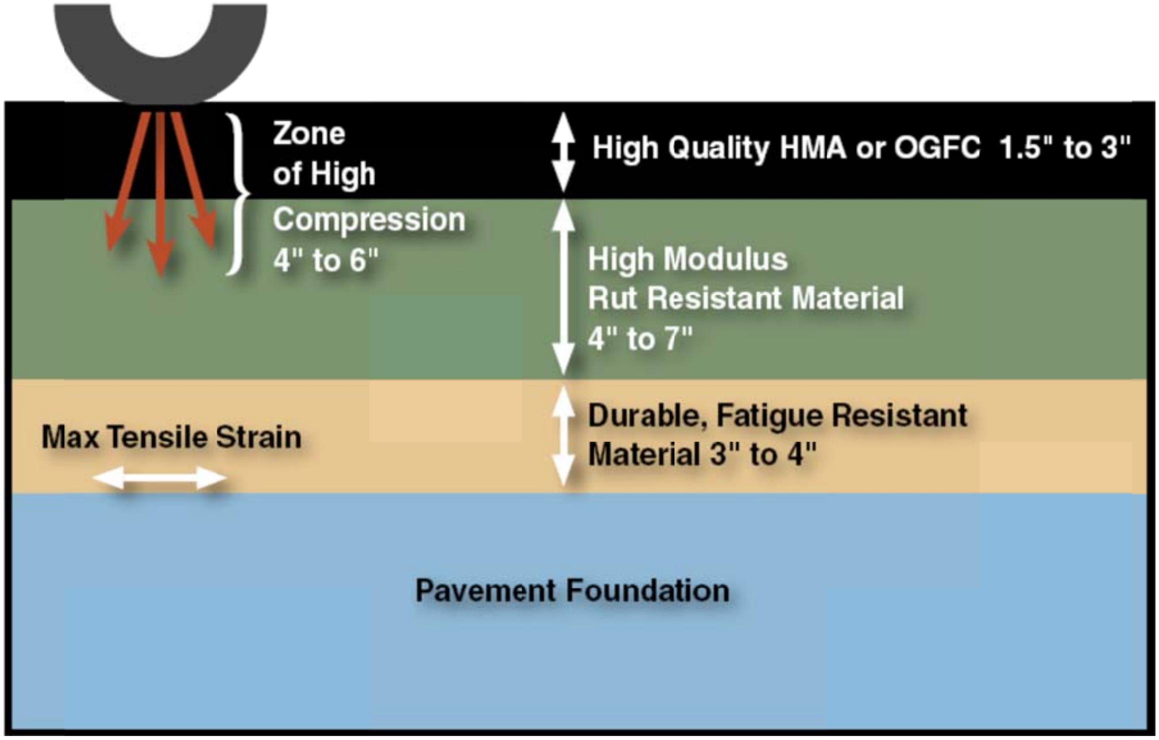


Figure 20. Long life HMA pavement design concept. (Newcomb et al., 2010)

- 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
- 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 (D = 1.0 is considered failure). On 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 percent 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 (AASHTO, 2008) can be used for long life pavement design, by using 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 one inch (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 one inch open graded wearing course) on a cracked and seated concrete pavement (Monismith and Long, 1999b, Monismith et al, 2009a and 2009b). The HMA overlay does not have a more fatigue resistant bottom layer (often referred to a "rich bottom" layer), since 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 one inch 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 8 shows the total HMA thickness along with HMA mix type recommended for the surface course.

Table 8. Michigan design catalog for long life HMA pavements over rubblized concrete. (after APA, 2002; Von Quintus and Tam, 2001)

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

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 jointed reinforced concrete pavement (JRCP) (Renteria and Hunt, 2006; 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 CRCP 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 done to distressed areas before overlaying (see CRCP section above):
  - A thin overlay (about 2 in. or less) can be used when:
    - 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.
    - 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:
    - The existing CRC pavement has large crack widths (between 0.5 mm and 1.5 mm).
    - The existing CRC pavement has surface spalling and scaling.
  - A thick overlay (greater than 4 in.) should be used when:
    - The existing CRC pavement has localized deformation and settlement due to poor subgrade condition.
    - The existing CRC pavement has structural defects such as “punchouts”, settlement, faulted cracks, and severe spalls.
    - The existing CRC pavement needs strengthening to accommodate higher traffic loading levels.

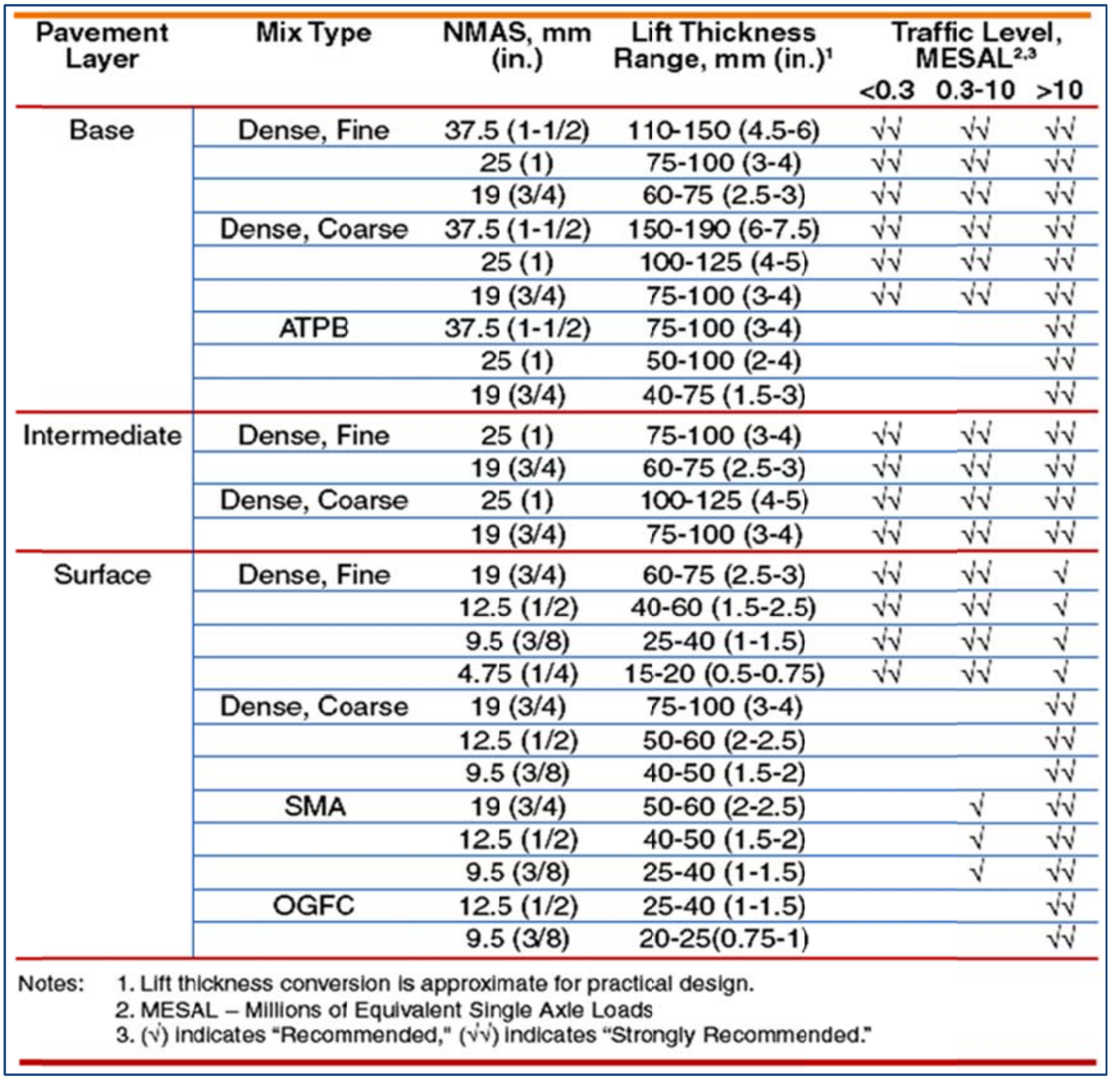
Broadly, for HMA overlays over processed PCC, thicknesses will typically be in the range of 8 to 10 in. for long life pavements. Many agencies will find this level of thickness costly; however, the issue is whether to spend more initially, minimizing future costs, or to enter into an endless cycle of 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/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 is 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 since 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 21.



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 21. Mix type selection guide for long life HMA pavements.
(Newcomb and Hansen, 2006)

For heavily trafficked roads, the need for rutting resistance, durability, impermeability, and wear resistance would dictate the use of SMA (EAPA (2007), Michael et al (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 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) as compared to a range of hot mix types. A study from the European Asphalt Pavement Association (EAPA, 2007), found SMA mixtures to have an average life of 20 years, while traditional hot mixes averaged 15 years. Similar performance trends were noted by those Agencies who regularly use SMA in their paving program. Methods for SMA mix design are given in NCHRP Report No. 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 mm 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 et al., 2001). The two most common HMA rut tests are the Hamburg Wheel Track Test (AASHTO T 324) and the Asphalt Pavement Analyzer (AASHTO TP 63). Later in this document (“HMA Stripping—Causes, Assessment, Solutions”), the Hamburg test is discussed in additional detail (note Figure 26 within that section).

In western and southern regions of the United States, open-graded friction courses (OGFC) are used to improve wet-weather friction. Some northern states such as Massachusetts, New Jersey, and Wyoming use OGFC 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 percent 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 OGFC 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 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 (LTPP, 2010). Normally, 95 percent or 99 percent reliability should be used, depending upon availability and cost.

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

- 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 percent 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). Further, 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 and 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 nominal maximum aggregate size (NMAS) can have a significant effect on the permeability of coarse graded Superpave designed mixes. Further, 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 four inches 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; and 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 insuring 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 for the surface to resist rutting. However, the low temperature requirement could probably be relaxed one grade, since 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 (LTPP, 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 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 non-linearity 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 US 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, which 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 insure 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 percent in the pavement structure (EAPA, 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, 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 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 23. The presence of moisture can also accelerate the formation of potholes or promote delamination between pavement layers (Figure 24) (Santucci (2002); Santucci (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 capillary rise of subsurface water. Moisture can also be present in the asphalt mix as a result of inadequately dried aggregate.

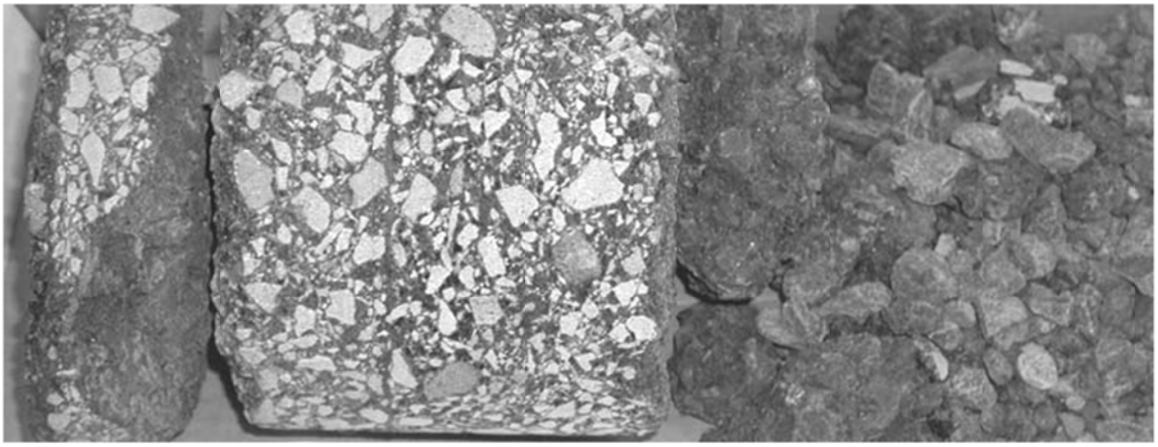


Figure 22. Moisture-induced stripping. (Photo courtesy Rita Leahy)

Rutting

Fatigue cracking

Figure 23. Moisture-weakened asphalt pavement induces premature failure. (Photo courtesy Rita Leahy)

Pothole

Delamination

Figure 24. Moisture exacerbates local pavement distress.

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

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), while 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 anti-strip 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 prior to paving (Santucci, 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 percent 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 collecting water on the surface of the underlying pavement unless adequate drainage is provided prior to 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 et al, 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 anti-strip 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 percent 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 percent or less showed little or no moisture damage. Sections with air void contents greater than 7 percent 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, since 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: (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 anti-strip agents improved the moisture resistance of asphalt mixes. Hydrated lime and liquid anti-strip agents were also effective in improving moisture resistance during a conditioning period of up to one year. The effectiveness of the liquid anti-strip agents remained constant over the one year period while, 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 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, 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 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.

Table 9. Moisture sensitivity tests.

Category	Test	Output
Component, Compatibility, and Loose Mixes	Sand Equivalent (AASHTO T 176)	Relative amount of clay material in the fine aggregate
	Plasticity Index (ASTM D 1073)	Plastic nature of fine aggregate or soil
	Methylene Blue (AASHTO TP 57)	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 TP 62 AASHTO TP 34	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 TP 63)	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

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/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, Santucci, and Ashenbrener (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 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 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.

More recently, agencies have found greater success with the Hamburg Wheel Tracking Test (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.

Table 10. Post-SHRP agency use of moisture sensitivity tests.
(Hicks et al, 2003; Solaimanian et al, 2003)

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

Table 11. Success rates of moisture sensitivity test methods.
(Kiggundu and Roberts, 1988)

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% to 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

The results from the HWTT define four phases of mix behavior: post compaction consolidation, creep slope, stripping slope, and stripping inflection point (Figure 25). The post compaction 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 one year (Aschenbrener, 1995; Aschenbrener et al, 1995).

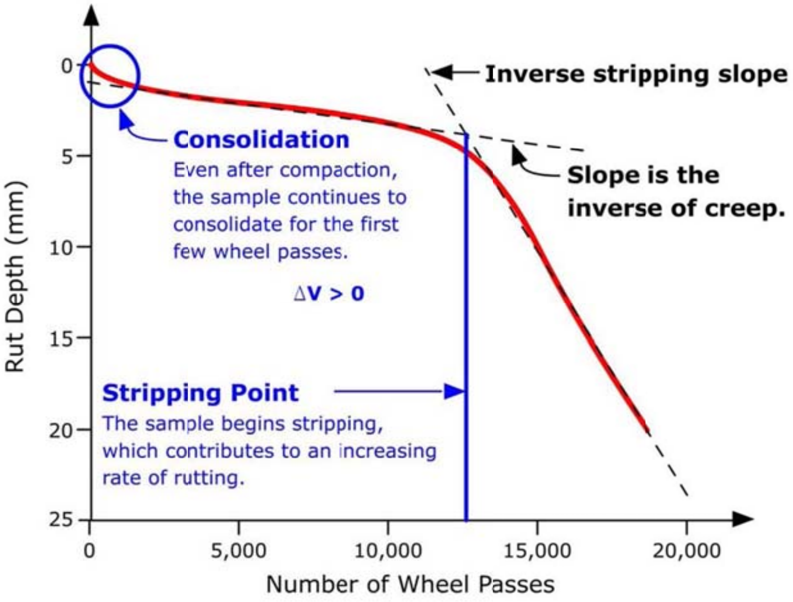


Figure 25. Typical Hamburg wheel-tracking data.
(From but not original to Pavement Interactive, 2011)

Texas DOT's evaluation of the HWTT yielded similarly positive results, i.e., 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 anti-stripping 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 anti-strip additives or lime. The use of organosilane compounds has also shown promise in reducing moisture damage in asphalt pavements (Santucci (2002); Santucci (2010)).

Most liquid anti-strips 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 anti-strip is typically added at a rate of 0.25 to 1.00 percent by weight of asphalt. Liquid anti-strip additives are designed to act as coupling agents that promote better adhesion at the asphalt-aggregate interface. It is important to pre-test any liquid anti-strip 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; Santucci, 2010; Epps-Martin et al, 2011; TRB, 2003).

Lime treatment is widely used throughout the US 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 percent by weight of dry aggregate or 20 to 40 percent 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 due to 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 percent 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 percent 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 percent with a mean value of about 7 percent. 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 percent reduction in air void content can increase mix fatigue life by 4 to 10 percent and increase rut resistance by 34 percent.

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 anti-strip additives or lime. Liquid anti-strips 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 US, 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 granular base layer, as suggested by the Illinois Department of Transportation chart (Figure 26). A similar foundation design practice is used in the UK, as shown in Table 12. The CBR of the subgrade dictates the thickness of the overlying granular layers. For a subgrade CBR of less than 15, a minimum six-inch thickness of subbase (equivalent to high quality base in the US) 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 5800 psi on top of the subgrade and 9500 psi at the top of the subbase under an FWD load of 9000 lb (Newcomb et al, 2010). Insufficient existing granular base/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.

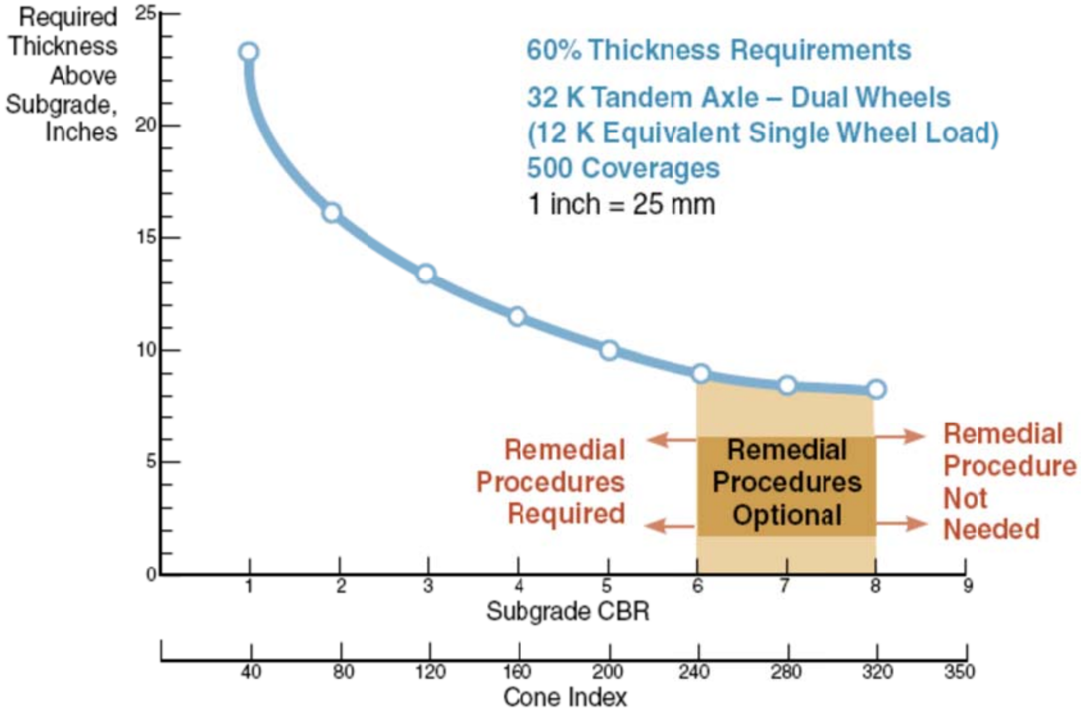


Figure 26. Illinois granular thickness requirement for foundation.
(IDOT, 1982)

Table 12.Transport Research Laboratory foundation requirements.
(Nunn et al., 1997)

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 UK, while a subbase is called capping.

When the existing pavement is concrete, FWD data should be collected at 0.2 mile 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 backcalculation 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 insure 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 foot 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 prior to rubblization, but after the contractor mills off all HMA.
- FWD: Collect FWD data on the project at 0.2 mile 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 backcalculation 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 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 inches 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 inches of subgrade.

Top-Down Cracking

It is critical that coring of the existing flexible pavement identify top-down cracking if it occurs in the existing pavement. The reasons for this are at least three: (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). Further, this type of cracking has been reported for a variety of climate and traffic conditions, which are illustrated by Figures 27 to 30. Figure 27 shows top-down cracking in cores taken in Panama with significantly different core thicknesses. Figure 28 shows views of top-down cracking which occurred on both an Interstate highway and local streets in Washington State. Figure 29 shows longitudinal top-down cracking on a US Interstate highway and transverse and longitudinal top-down cracking in Panama (near Colon). Figure 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 et al, 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. Further, 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 (2001) 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, 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 (2001) and Uhlmeyer (200) 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).

Figure 27. Top down cracking in cores from Panama.
(core thicknesses ranged from 6 to 12 in. thick)

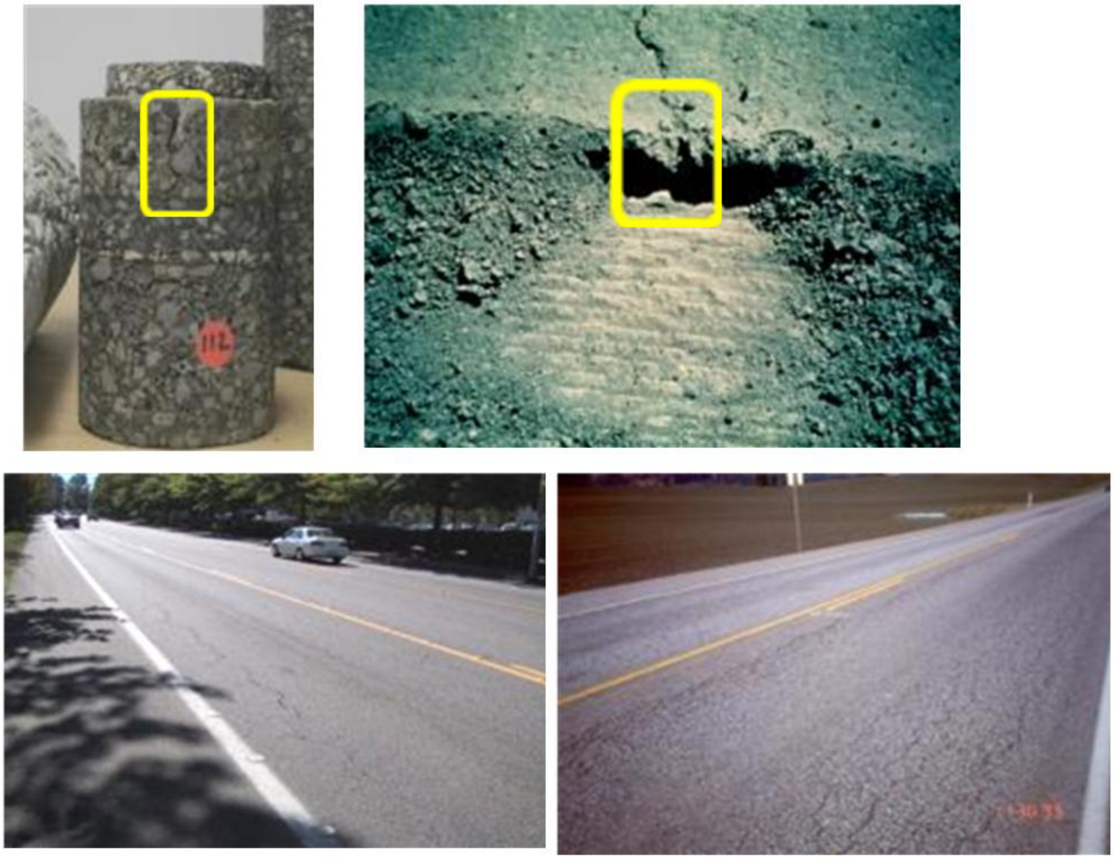


Figure 28. Illustrations of top-down cracking in Washington State.
(the upper photos are from Interstate 90; the bottom photos local streets in western Washington)

Figure 29. Illustration of longitudinal top-down cracking following crack sealing for a US Interstate Highway (left) and longitudinal and transverse top-down cracking in Panama (right).

Figure 30. Two views of longitudinal top-down cracking in Michigan.
(the photo on the right is HMA placed over rubblized PCC pavement)

Construction of a long life pavement should not be much different than 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 as 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 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 APEC (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 which allow the material to be placed and compacted at temperatures anywhere from 35 to 100^{o}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 percent and 97 percent of maximum theoretical density (i.e., 3 to 8 percent air voids) for surface courses will generally provide good performance (based on mixes with gradations passing through or above the Superpave-defined restricted zone). Further, 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 three months following construction. This is somewhat counter to prior views that held most of the post-construction densification occurs within two years. Further, 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 percent of TMD. The lower specification density requirement ranged between 91 and 92 percent.

Given the evidence available, it is suggested that an average density value for dense-graded mixes ≥ 93 percent 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, 2006). Segregation may be measured with infrared temperature techniques and laser texture methods such as the Rosan procedure (Stroup-Gardiner and Brown, 2000). Figure 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.

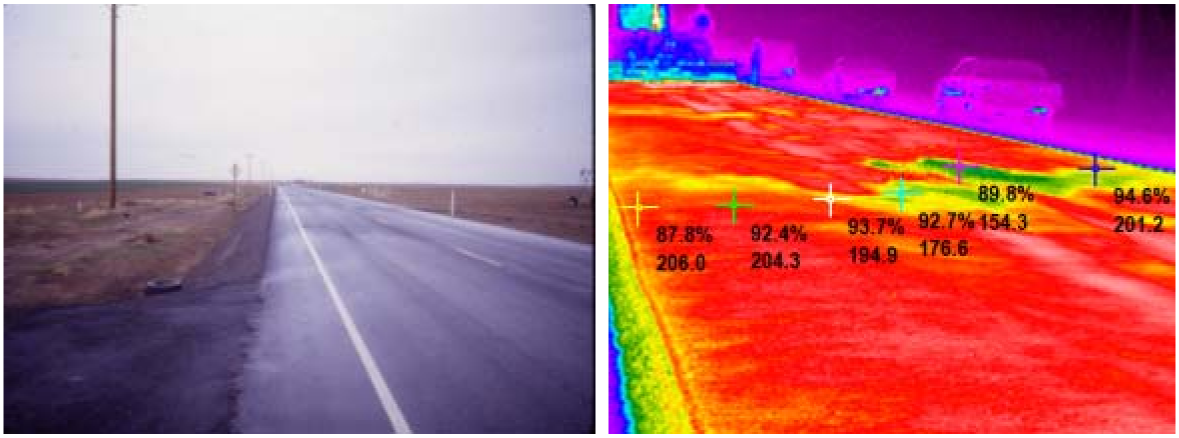


Figure 31. HMA segregation caused by temperature differentials.

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 prior to placement can help in avoiding thermal segregation. Also, the selection 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 insure 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 percent 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 percent 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 group in systematic or cyclic areas as shown in Figure 26. Temperature variations of 50 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 percent (Willoughby et al, 2001).

A number of HMA specification modifications have been crafted largely by State DOTs to address non-uniform 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 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 non-uniform conditions.

A relatively new solution is to measure whether temperature variation is a major factor on a paving project by 100 percent 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. Locations 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 32.

Figure 32. Pave-IR Thermal Imaging System. (Photos: Study Team)

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 (NAPA, 2002). The use of echelon paving or full-width paving has the effect of essentially eliminating the longitudinal joint, since the two paving lanes are placed at the same time. This should be considered as the best solution, although it may not always be possible to implement due to 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, since 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 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 in order 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 et al., 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 13 that are relevant for HMA quality control. The table includes a brief explanation why the issue is of special interest along with examples from the study guide specification recommendations. These specification elements are sorted by (1) HMA density, (2) HMA segregation, (3) longitudinal joints, and (4) interlayer bonding.

Table 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 (mix, layer thickness, weather, etc.) and is crucial in constructing long-lasting HMA layers. Air void levels greater than 7 to 8% result in accelerated fatigue and increased permeability.	• The average target % 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. **[Refer to Elements for AASHTO Specification 401 for more details]**[1]
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. **[Refer to Elements for AASHTO Specification 401 for more details]**[1]
Longitudinal 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 that 2% higher voids in the joint than 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 inches of a longitudinal joint, including the pavement on the traveled way side of the shoulder joint, shall not be less than 2.0 percent below the specified density when unconfined. **[Refer to Elements for AASHTO Specification 401 for more details]**[1]
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/SY. • Consider the use of a hot tack (paving grade asphalt cement). **[Refer to Elements for AASHTO Specification 404 for more details]**[1]

[1] Contained in Appendix E-4

Summary

A summary of the flexible pavement best practices is provided in Table 14. They are grouped by:

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

Table 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 vs. 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. Crack and seated PCCP is preferred over rubblization, if possible. 2. A wide range of crack spacings have been suggested for crack 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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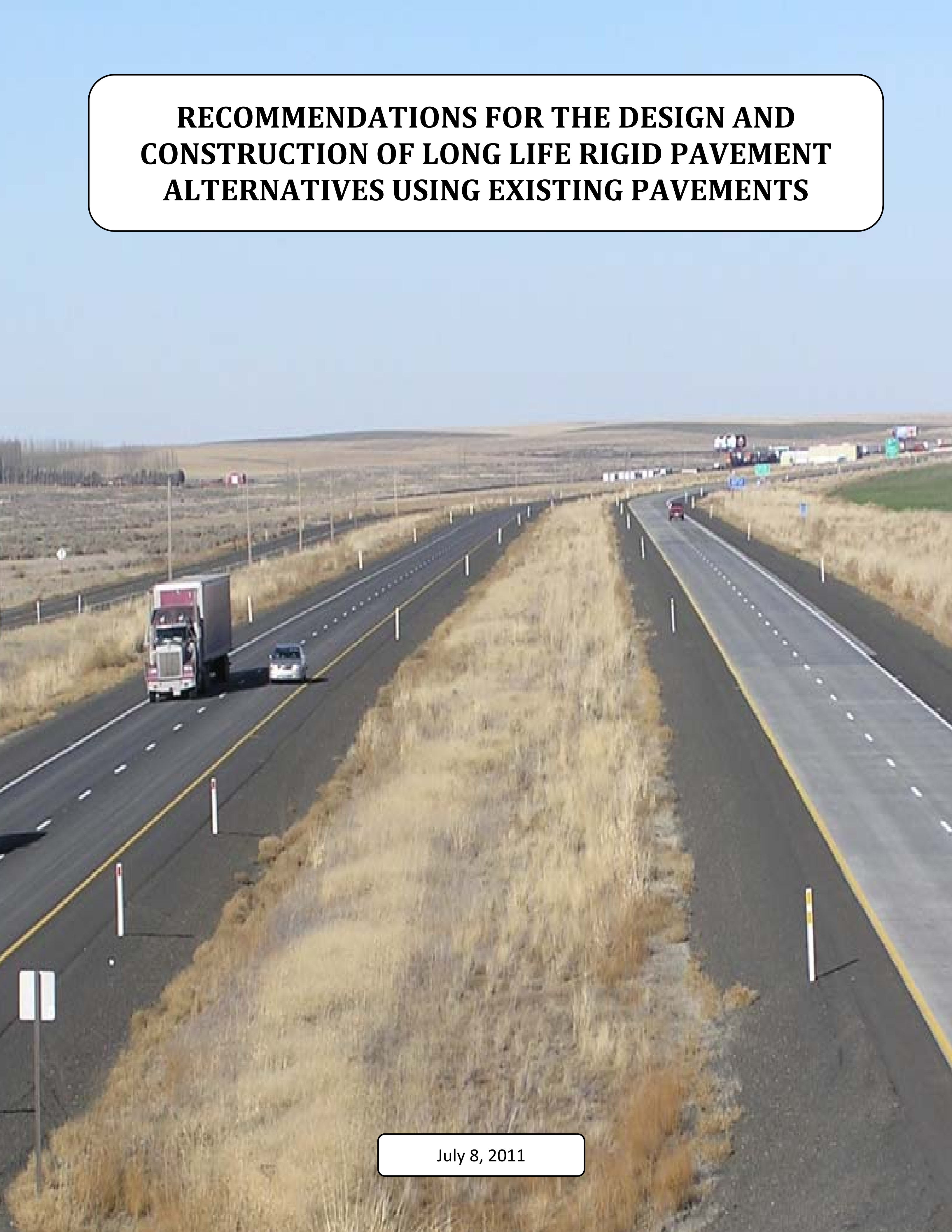
APPENDIX E-3

RECOMMENDATIONS FOR THE DESIGN AND CONSTRUCTION OF LONG LIFE RIGID PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

RECOMMENDATIONS FOR THE DESIGN AND CONSTRUCTION OF LONG LIFE RIGID PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS



July 8, 2011

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RECOMMENDATIONS FOR THE DESIGN AND CONSTRUCTION OF LONG LIFE RIGID PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

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 in excess of 50 years old that remain in service; 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 is:

- 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.
- Reduce life cycle costs and user costs.

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 1.

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 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).

JPCP constructed on HMA base

CRCP Constructed on HMA Base

Figure 1. Completed and under construction JPCP and CRCP. (Photos: Joe Mahoney)

Table 1. Threshold values for concrete pavement distresses. (Tayabji and Lim, 2007)

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

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

An illustration of the different types of concrete overlays is shown in Figure 2.

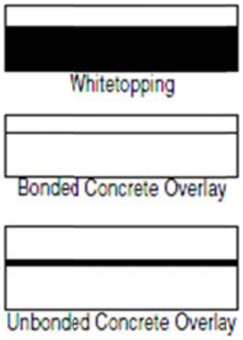


Figure 2. Types of concrete overlays—earlier descriptions.
(Rasmussen and Rozycki, 2004)

The Texas DOT 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) (TxDOT, 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 Manual (PDM) (TxDOT, 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. 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.

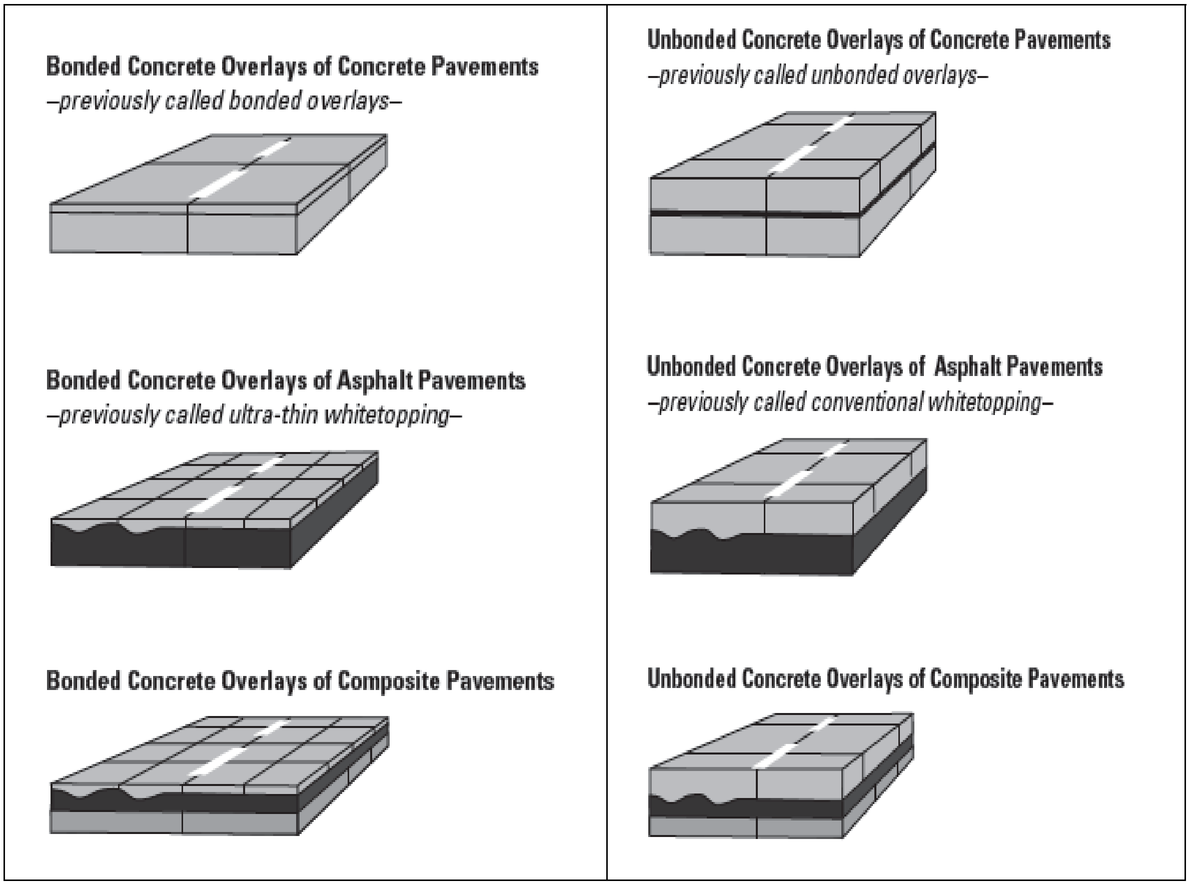


Figure 3. Types of concrete overlays—more recent descriptions. (Harrington, 2008)

Rigid Renewal Strategies

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

- Unbonded concrete overlays of concrete pavements
- 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 and Neville (1975) both noted that for flexural stress ratios less than 0.55 (applied flexural stress/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, 1995) produced a fatigue-based highway design method for the 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 last 20 years via the Long Term Pavement Performance (LTPP) program. These results, as relevant to long life rigid 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 GPS-9 and 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 I of this study was 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 either aggregate interlock or steel dowels. A summary of the sections and major findings from that assessment include:

- Of the unbonded overlays reviewed, the thicknesses were:
  - ~ 6 in. thick: 22%
  - ~ 8 in. thick: 22%
  - ~ 9 in. thick: 11%
  - ~ 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. It also contributed towards 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 I 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 between 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/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%
  - 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 ESALs/year. The JPCP overlays ranged in thickness from 3.0 in. to 5.4 in. (see below) with an average of 4.3 in. These overlays were placed on existing JPCP which had a 20 ft. spacing between transverse joints. Prior to placing the bonded overlays 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 five years following construction are shown in Table 1.

Table 1. Overlay thickness and performance over five years.

Target Overlay Thickness (in.)	Overlay Thickness (in.) based on cores	No. Transverse Cracks Prior to Overlay (JCPC constructed in 1955, 10 in. slabs)	No. of Transverse Cracks 5 years After Const.
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

Sources: (1) Smith and Tayabji, 1998, and (2) 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 in their 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 in. 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 is desirable.

Texas DOT CRCP Overlays

During the conduct of the R-23 study, a field trip to review concrete overlays was made with the Texas DOT. 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. bonded overlays and views expressed by TxDOT personnel, it appears that bonded CRCP overlays within that thickness range can be expected to perform about 25 years. One unbonded 12 in. 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. CRCP over 1 in. of HMA over 6 in. 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 (low coefficient of thermal expansion material).

Washington State DOT Bonded Concrete Overlays

Bonded JPCP concrete overlays constructed in 2003 over existing HMA were reviewed (Figure 4). Three thicknesses of concrete overlays were used: 3, 4, and 5 in. each placed on I-90 east of Spokane, WA 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.

Construction of bonded PCC overlays in July 2003 which were placed directly on rotomilled HMA.

Figure 4. Construction of bonded PCC overlays in Washington State. (Photos: WSDOT)

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 5. The mix had a specified minimum flexural strength of 800 psi with a minimum cement content of 800 lb per yd^3. Polypropylene fibers were added at a rate of 3 lb per yd^3. A carpet drag finish was applied to the surface (Andersen et al, 2006). The underlying

HMA thicknesses were 9 in. for the 3 in. slab, 8 in. of the 4 in. slab, and 7 in. for the 5 in. slab. Following one year of service, cracking in the three bonded JPCP sections were:

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

At the time of removal in 2011 (Figure 6), the 3 in. section was severely distressed as shown in Figure 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.

3 in. Bonded PCC overlay of HMA following 8 years of service.

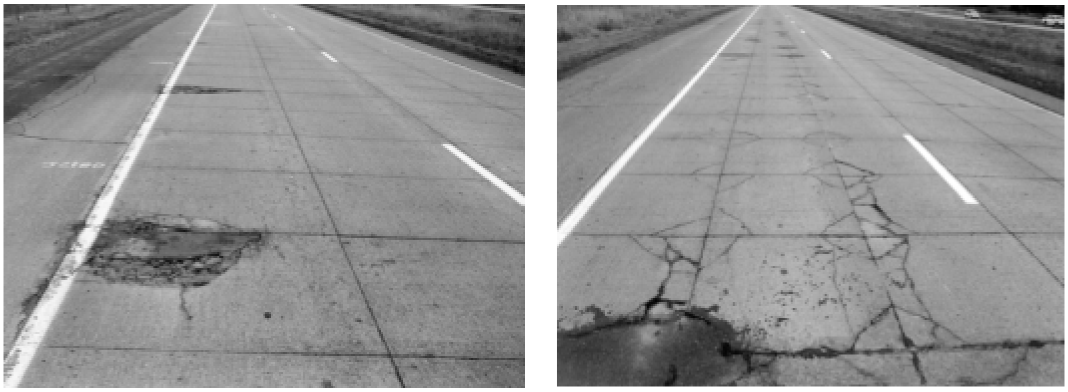
Figure 5. Condition of 3 in. bonded overlay in 2011. (Photos: WSDOT)

Removal of 3 in. PCC overlay prior to reconstruction of this portion of I-90

Figure 6. Bond between the PCC overlays were 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, which 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 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., but the amount of cracking for both configurations was extensive.



MnRoad Cell 95. Bonded concrete overlay 3 in. thick with a 5 ft by 6 ft joint spacing in November 2003.

MnRoad Cell 94. Bonded concrete overlay 3 in. thick with a 4 ft by 4 ft joint spacing in November 2003.

Figure 7. Condition of 3 in. bonded concrete overlays following 5 million ESALs and 6 years of service. (Photos: MnDOT)

Table 2 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 8 illustrates the performance of the 6 in. sections at MnRoad following 11 years of service.

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.

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.

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.

Figure 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).
(Photos taken in July 2008 by Tom Burnham, MnDOT)

Table 2. Initially constructed MnRoad bonded concrete overlay sections.
(after Burnham, 2008)

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

Recap on Concrete Overlays

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

- Bonded JPCP concrete overlays over HMA
- 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 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 the field data reviewed.

Table 3. 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 to 10
5	5 ft. by 6 ft	No	10 to 15
6	5 ft. by 6 ft	No	15 to 20

Note 1: All HMA thicknesses assume that the existing HMA materials are in good condition and exhibit no stripping.

A recent summary report from MnRoad (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 6x6-foot panels on higher volume roadways.”

Limited information on bonded CRCP overlays suggest 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 low coefficient of thermal expansion materials 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:

- Bonded Overlays: Use 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."
- Unbonded Overlays: Use "…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 State). These practices were updated based on recent information and summarized in Tables 4 and 5. Minnesota and Washington State were grouped together in Table 4 since their practices are for JPCP. Illinois and Texas are summarized in Table 5 to reflect their CRCP practices. While 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 materials and construction.

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.

Table 4. Examples of long-life JPCP standards for the Minnesota and Washington State. DOTs (Tayabji and Lim, 2007; MnDOT, 2005: WSDOT, 2010)

Item	**Minnesota DOT**	**Washington DOT**
Design Life	• 60 years	• 50 years
Typical Structure	• Slab thicknesses = 11.5 to 13.5” • 3 to 8” dense-graded granular base • Subbase 12 to 48” select granular (frost-resistant)	• Slab thickness = 12 to 13” (typical) • 4” HMA base • 4” crushed stone subbase
Joint Design	• Spacing = 15’ with dowels • All transverse joints are doweled	• Spacing = 15’ with dowels • Joints saw cut with single pass • Hot poured sealant
Dowel Bars	• Diameter = 1.5” (typical) • Length = 15” (typical) • Spacing = 12” • Bars must be corrosion-resistant	• Diameter = 1.5” • Length = 18” • Spacing = 12” • Bars must be corrosion-resistant Epoxy coatings not acceptable
Outside Lane and Shoulder		• 14’ lane with tied PCC or HMA • 12’ 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	• 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 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/CY 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	

Table 5. Examples of long-life CRCP standards for the Illinois and Texas DOTs. (Tayabji and Lim, 2007; TxDOT, 2011; TxDOT, 2009a; TxDOT, 2009b)

Item	Illinois DOT	Texas DOT
Design Life	• 30 to 40 years	• 30 years
Typical Structure	• Up to 14" CRCP slab • 4 to 6" HMA base • 12" aggregate subbase	• Up to 13" CRCP slab with one layer of reinforcing steel • 14 to 15" CRCP slab with two layers of reinforcing steel • Uses stabilized base either 6" CTB with 1" HMA bond breaker on top or 4" HMA • Recommends tied PCC shoulders
Tie Bars	• Use at centerline and lane-to-shoulder joints • Use 1" by 30" bars spaced at 24"	
CRCP Reinforcement	• Reinforcement ratio = 0.8% • Steel depth 4.5" for 14" slabs • All reinforcement in CRCP epoxy-coated	• Increased amount of longitudinal steel • Design details for staggering splices
Aggregate Requirements	• IDOT 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 to 90°F • Slipform pavers must be equipped with internal vibration and vibration monitoring • Curing compound must be applied within 10 minutes of concrete finishing and tining • Curing ≥ 7 days before opening to traffic	• Revised construction joint details

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:

- Foundation support (uniformity, volumetric stability [including stabilizing treatments])
- Drainage design (moisture collection/removal and design for minimal maintenance)
- Concrete mixture proportioning and components (selected to minimize shrinkage and potential for chemical attack, low CTE, provide adequate strength, etc.)
- 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, etc.)
- QA/QC (certification, pre-qualification, inspection, etc.)

All of 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 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).

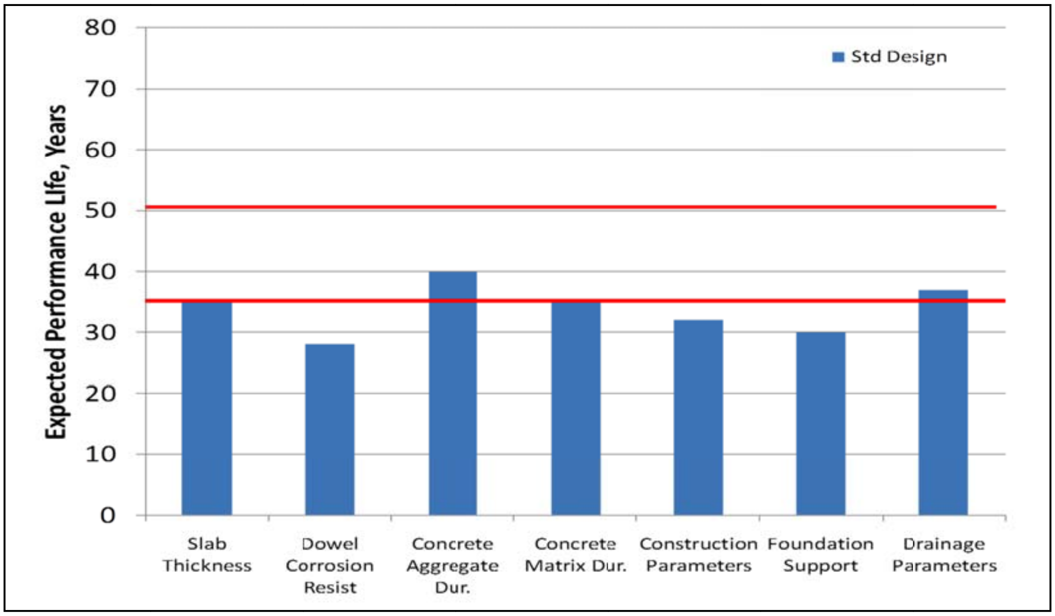


Figure 9. Illustration of Pavement Designed and Built for 35 Year Service Life

The chart Figure 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, etc.). While the development of 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 11 shows that the consideration of all of the potential improvement areas is necessary to ensure a performance life of at least 50 years.

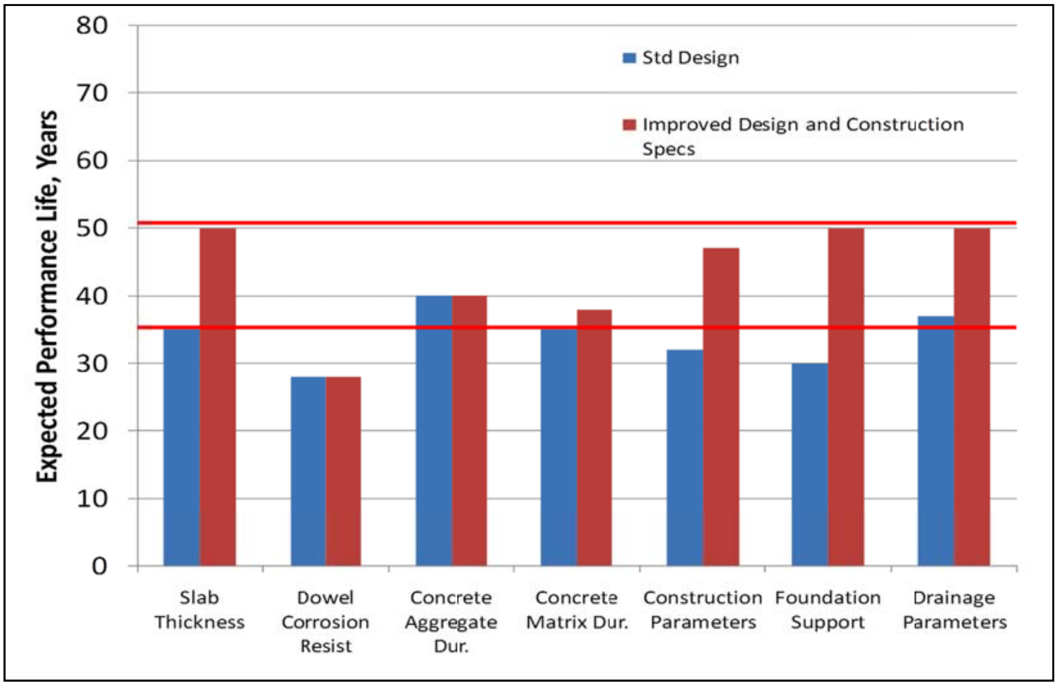


Figure 10. Illustration of Improved Design and Construction Specifications

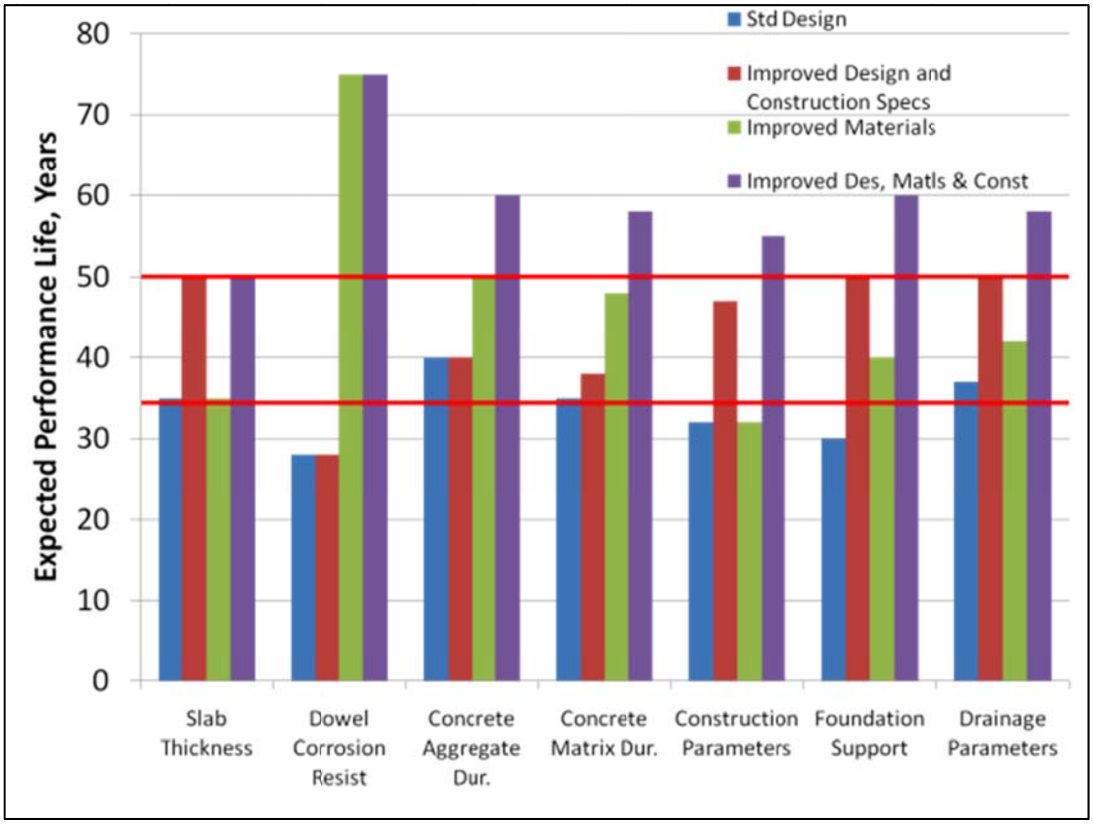


Figure 11. Illustration That All Areas of Improvement Need to be Considered for Long-Life

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.

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). Current practice for paving concrete is to incorporate portland cement and a 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) (EPA, 2011). The CPGs are Federal Law 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 C 618; however, care should be taken in applying ASTM C 618, as 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 12.

Typical dosages for Class F fly ash are generally between 15 percent and 25 percent 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 C 618 permits up to 6% loss on ignition (LOI), the state 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 percent.

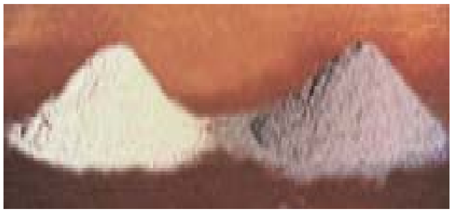


Figure 12. Class F fly ash. (Photo: FHWA)

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 (occasionally Type III for decreased cure times). Today, a wider range of cements are 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 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” (SCA, 2002).

GGBFS must meet the requirements of ASTM C 989. The following three grades are based on their activity index:

1. Grade 80. This is the least reactive and is typically not used for highway or airport projects.
2. Grade 100. This is moderately reactive.
3. Grade 120. This is the most reactive, with the increased activity achieved through finer grinding. Grade 120 can be difficult to obtain in some regions of the US.

It is common that blends of slag and portland cements are made (typically designated Type IS(X) where X = the % of GGBFS). Typical dosages of slag should be between 25 percent and 50 percent of cementitious materials. Concrete strength at early ages (up to 28 days) may be lower using slag-cement combinations, particularly at low temperatures or at high slag percentages. The desired slag content must be established considering the importance of early strengths for the panel fabrication process. However, if the slag will be used to control expansions due to ASR, the minimum slag content used is that needed to control ASR.

Figure 13. Preprocessed blast furnace slag. (Photo: Joe Mahoney)

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 percent of the PCC volume being composed of aggregates. Aggregate selection should maximize the volume of aggregate in the concrete mixture in order 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 6 and 7 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 14 shows typical aggregate processing prior to batching concrete for paving.

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 percent, except the coarsest sieve and 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).

Figure 14. Aggregate processing, which includes stockpiles, conveyors, and screening. (Photos: Joe Mahoney)

Table 6. Concrete pavement performance parameters affected by aggregate properties. (after Folliard and Smith, 2003)

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 affects 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
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

Table 6. Continued.

Performance Parameter	Manifestation	Mechanism(s)	PCC Properties	Aggregate Properties
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

Table 7. Standard aggregate, aggregate related and PCC test methods.
(Folliard and Smith (2003)

Property		Test Method
Basic Aggregate Property	Grading	AASHTO T 27
	Specific gravity	AASHTO T 84
	Absorption	AASHTO T 84
	Unit weight	AASHTO T 19
	Petrographic analysis	ASTM C 295
Durability	Soundness	AASHTO T 104
	F-T resistance	AASHTO T 161
	Internal pore structure	AASHTO T 85
	Degradation resistance	AASHTO T 96, ASTM C 535
Chemical reactivity	ASR	ASTM C 227, 295, 289
	ACR	ASTM C 295
Dimensional change	Drying shrinkage	ASTM C 157
Deleterious substances		AASHTO T 21
Frictional resistance		AASHTO T 242
Particle shape and texture		ASTM D 4791

Deleterious Substances

Deleterious substances are contaminants that are detrimental to the aggregate's use in concrete. ASTM C 33 lists the following as deleterious substances:

- Clay lumps and friable particles
- Chert (with saturated surface dry specific gravity < 2.40)
- Material finer than No. 200 sieve
- 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 C 88 test for soundness has a poor precision record. Aggregates that fail this test may be re-evaluated using ASTM C 666 or judged on the basis of local service history.

Flat and Elongated Particles

Flat and elongated particles impact 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, 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) small pore size (about ~ 0.1 μm), and (4) become critically (>91 percent) 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. Since 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 15. Also, since 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.

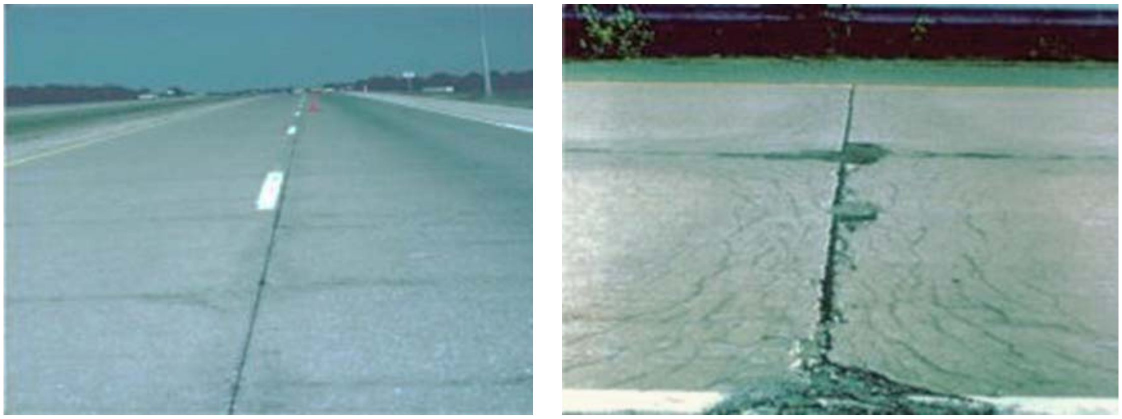


Figure 15. Photos illustrating D-cracking. (Sources: FHWA, NHI)

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 between 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 alkali-silica reaction (ASR) and calcium magnesium carbonate ($CaMg(CO_3)_2$ or dolomite) minerals with alkali-carbonate reaction (ACR) (Thomas et al, 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 16.

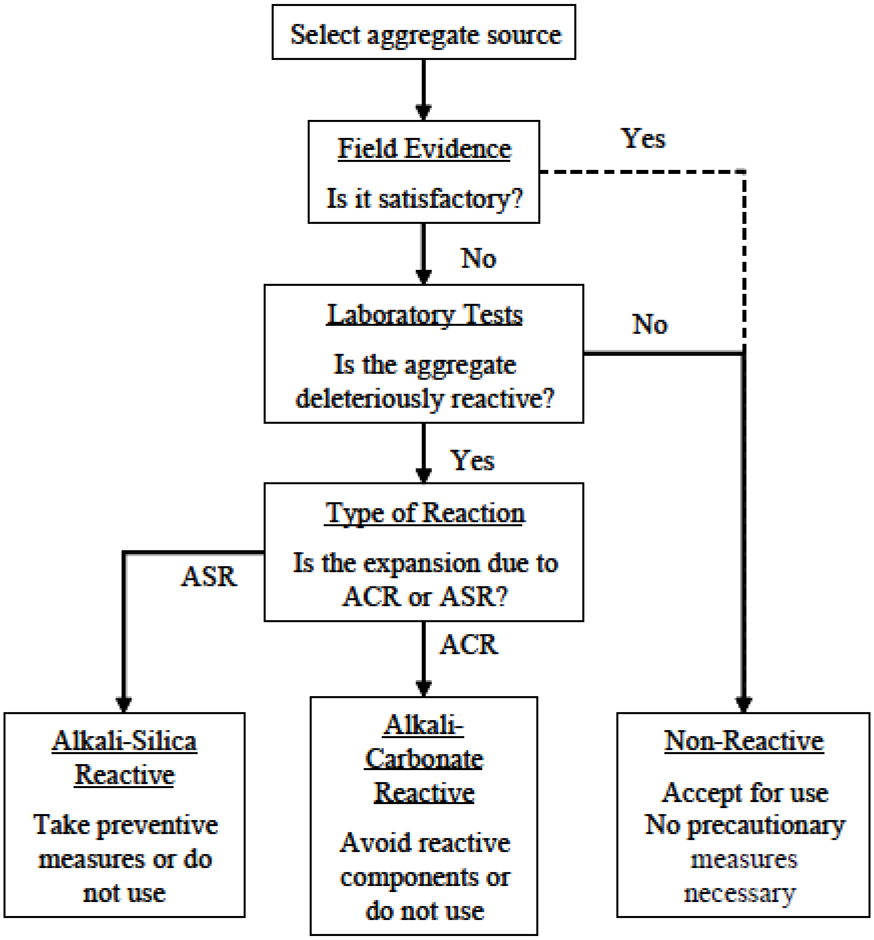


Figure 16. Evaluation Stages for Alkali-Aggregate Reaction Determination. (from Thomas et al, 2008)

Alkali-silica reaction (ASR) is of more concern since 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 either within the aggregate, 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 17 and 18. It is widely accepted 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 percent 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 on-going 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 "Determining the Reactivity of Concrete Aggregates and Selecting Measures for Preventing Deleterious Expansion in New Concrete Construction." Additionally, reports from the PCA (Farney and Kosmatka, 1997) and the FHWA (Thomas et al, 2008; Fournier et al, 2010) provide solid explanations on why ASR occurs, how it can be assessed, and mitigation measures that can be taken.

Figure 17. Illustration of ASR on a traffic barrier. (FHWA)

Figure 18. Illustration of ASR in concrete pavements.
(Source: D. Huft, South Dakota DOT)

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 upon 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 8). Therefore, the concrete aggregate, which typically comprises 70 percent 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 8. Typical CTE ranges for common PCC components. (ARA, 2004)

Material Type	**Typical Coefficient of Thermal Expansion ($10^{-6}/^{o}F$)**
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 w/c ratio 0.4 to 0.6	10.0-11.0
Concrete Cores from LTPP Sections	4.0 (lowest), 5.5 (mean), 7.2 (highest)

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 M 194 (ASTM C 494). 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 US 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 19 shows epoxy coated dowels with less than 15 years of service in Washington State. 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 (MN, IL, IA, OH, and WA) have been constructed using stainless steel clad dowel bars (Figure 20) and zinc-clad dowel bars with satisfactory performance (FHWA, 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 (WSDOT, 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 (MnDOT, 2005b; Wisconsin DOT, 2009). Additional guidance on dowel bar design can be found in a recent publication by the Concrete Pavement Technology Center (CP Tech Center, 2011).
- Tie bars 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 19. Corroded epoxy coated dowel bars in a retrofitted dowel bar project (original bars 1.5” by 18”). (Photos: WSDOT)

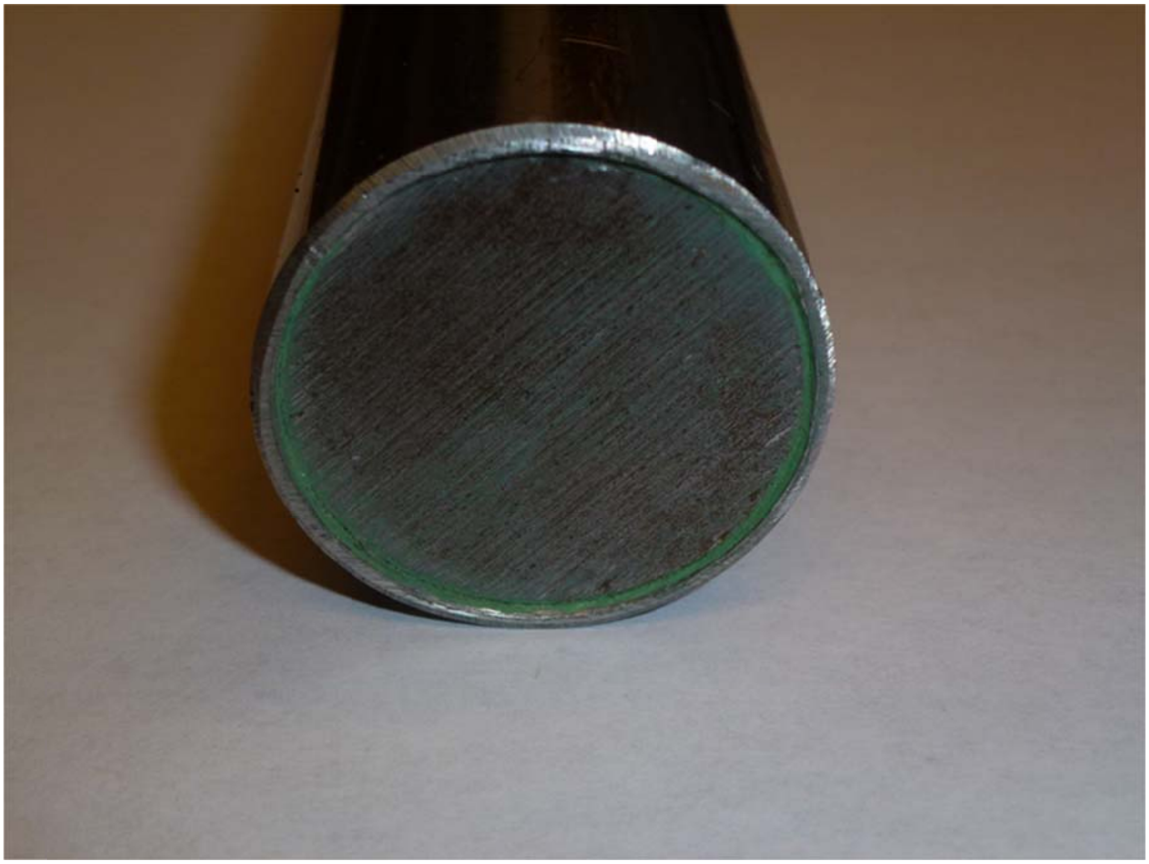


Figure 20. Stainless dowel bar. (Photo: Joe Mahoney)

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 et al (2002) and 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 pre-overlay repairs are usually held to a minimum. Figure 21 is a sketch of an unbonded overlay over concrete.

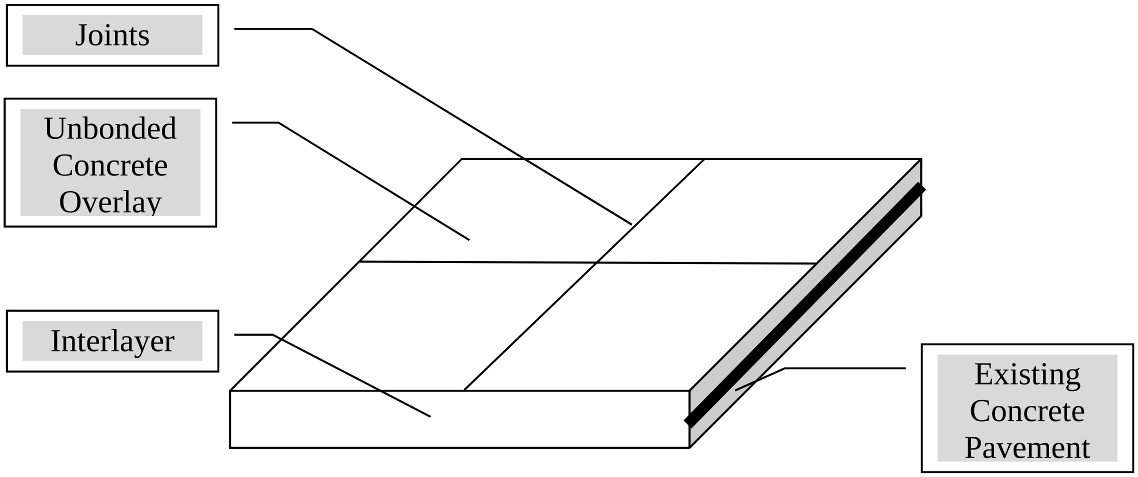


Figure 21. Unbonded concrete overlay of concrete pavement. (Illustration: Joe Mahoney)

Figure 22 illustrates an in-service unbonded undoweled concrete overlay. The photo shows a 35 year old JPCP overlay over an existing JPCP located on Interstate 90 in Washington State.

Figure 22. Unbonded 9 in. JPCP concrete overlay placed over concrete (Interstate 90 in Washington State [overlay 35 years old]). (Photo: WSDOT)

The following sections summarize some of the design and construction issues to consider for long life unbonded concrete overlays.

General Design Considerations

Smith et al (2002) and Harrington (2008) suggest 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, possible support characteristics under the slab corners and edges, backcalculate the modulus of subgrade reaction and modulus of the existing portland cement concrete pavement, and 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 9.

Table 9. Guidelines for preoverlay repairs. (Harrington, 2008)

Existing Pavement Condition	Possible Repairs
Faulting ≤ 10mm	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 w/punchouts	Full-depth repairs

- 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 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 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 MEPDG design methods. The key factors associated with these two methods are described below:

- 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.

- MEPDG (or Darwin-ME). The mechanistic-empirical 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.

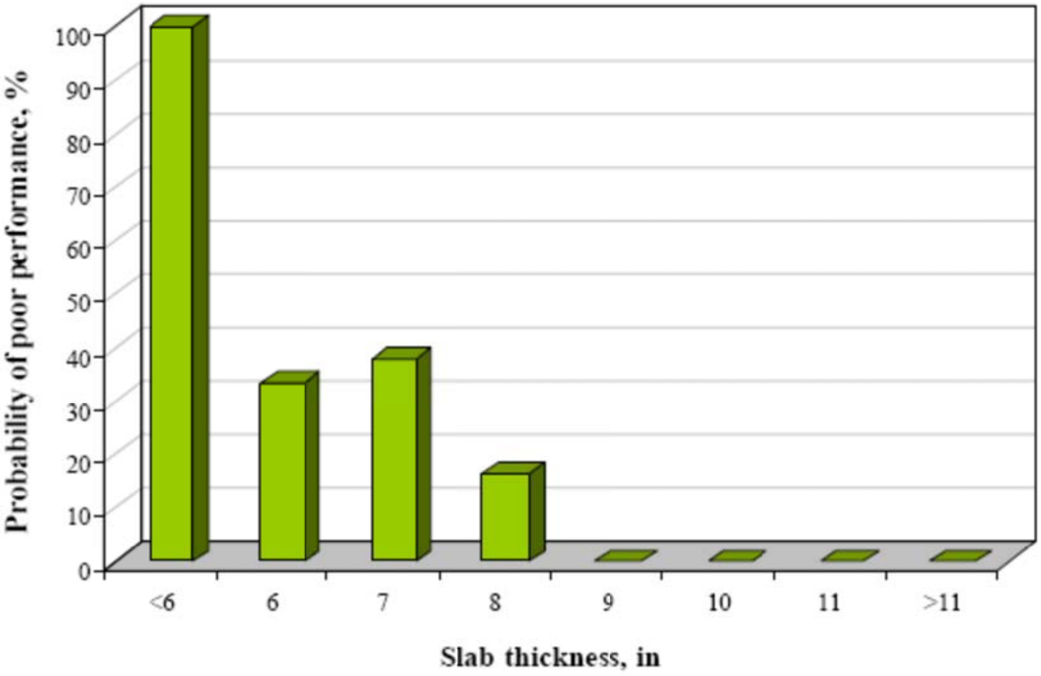


Figure 23. Slab thickness versus probability of poor performance for unbonded JPCP overlays. (Smith et al, 2002)

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 tie bars). 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, non-stainless 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 (CP Tech Center, 2011). Examples of three state DOT specifications and special provisions for the use of corrosion-resistant dowels were cited earlier.

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 feet is typically used for thick (> 9 in.) long-lived concrete pavements. Figure 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]).

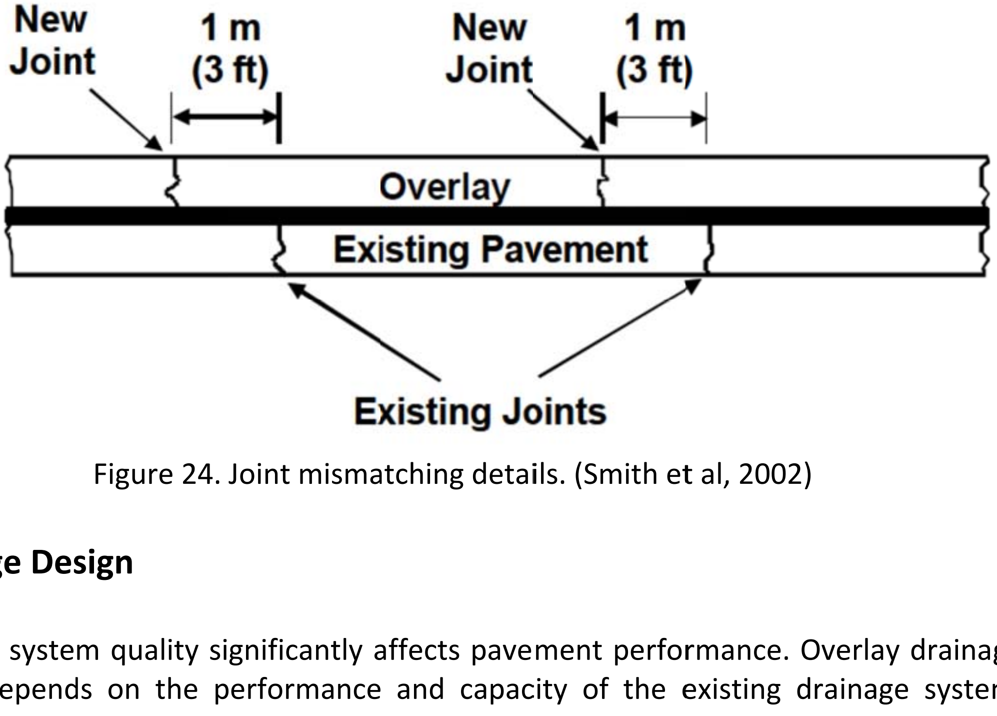


Figure 24. Joint mismatching details. (Smith et al, 2002)

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 inches) separator layers should be avoided (Smith et al (2002), Harrington (2008)).

Interlayers should be between 1 to 2 in. thick (Smith et al [2002], Harrington [2008]). Thin interlayers (e.g., 1 inch) 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 non-woven 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 US for unbonded concrete overlay interlayers, and was showcased on a 2008 unbonded concrete overlay project in Missouri (Tayabji et al, 2009). Figure 25 illustrates the placement of the fabric on the existing pavement surface. It is noted that no long-term performance data is currently available for the application of this technology in concrete overlays.

Figure 25. Placement of non-woven fabric as an interlayer. (From Tayabji et al [2009])

Table 10 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 Departments of Transportation.

Table 10. 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 as an interlayer.
Missouri	Typically use a 1 in. HMA or geotextile interlayer.

As reported by Smith et al (2002), the most commonly used separator layer is HMA (69 percent). Although other types of separator layers are also used, bituminous materials make up 91 percent of all separator layer types.

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), 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 26 shows typical transverse cracks both for airfield and highway pavements. Figure 27 shows the magnitude of 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.

Figure 26. Illustrations of transverse cracking on an airport apron and an Interstate Highway. (Photos: Joe Mahoney)

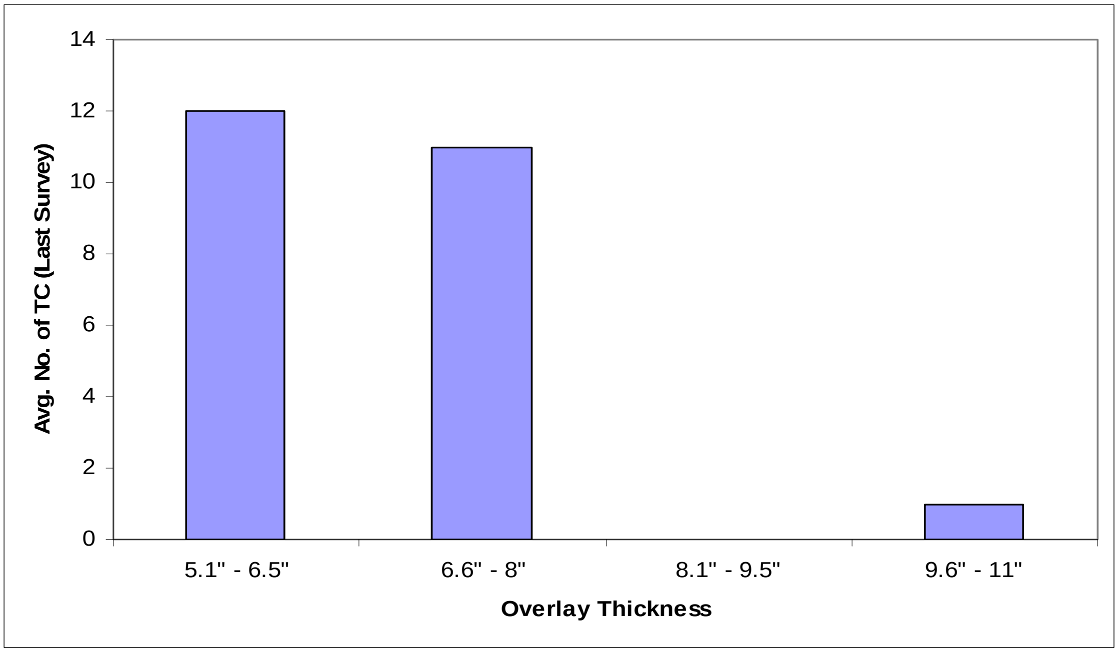


Figure 27. JPCP overlay thickness versus average number of transverse cracks.

International Roughness Index (IRI)

Figure 28 illustrates the progression of IRI and PSI for the various GPS 9 sections and the impact of overlay thickness on ride quality.

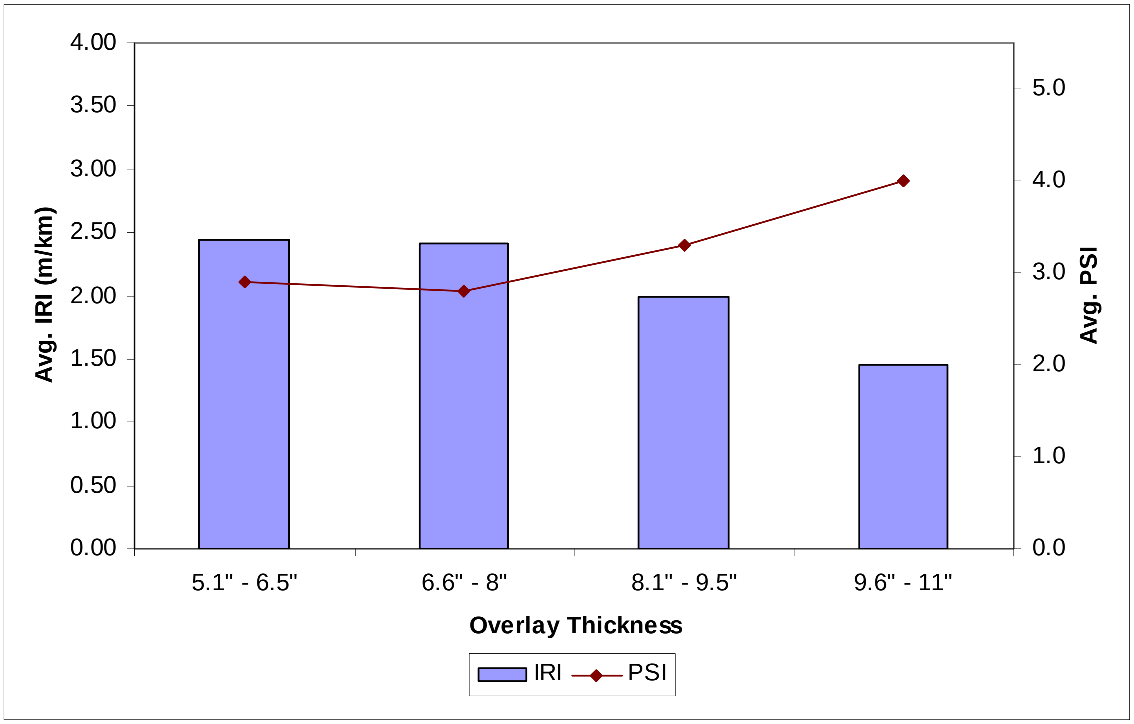


Figure 28. Overlay thickness versus average IRI and average PSI
(pavement age ranges from 6-20 years).

Joint and Crack Faulting

Figure 29 illustrates transverse contraction joint faulting (faulting above 0.25 in. is significant); although, the data from GPS-9 projects does not show the degree of severity that is illustrated in Figure 30. The overall magnitude of the faulting is below 0.25 in. and therefore does not appear to be an issue; however, slab thicknesses > 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 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.

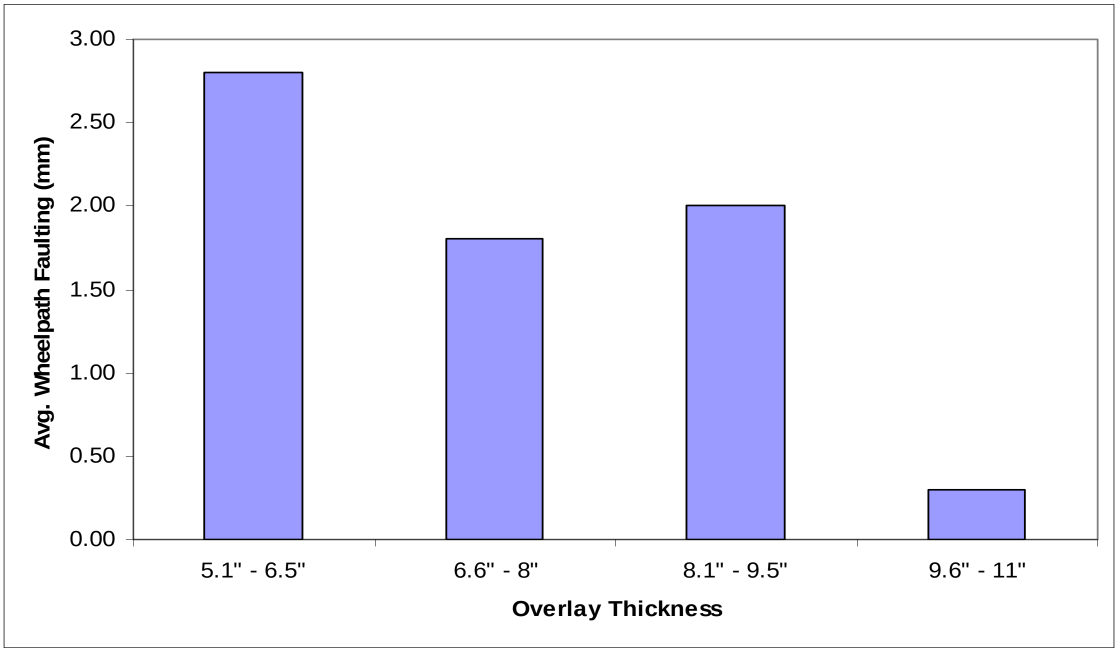


Figure 29. Overlay thickness versus average wheel path faulting.

Average Fault ~ 0.25 to 0.5 in.

Average Fault ~ 0.5 in.

Figure 30. Illustration of contraction joint faulting of JPCP. (Photos: WSDOT)

Impact of Interlayer Design on Performance

Figures 31 and 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.

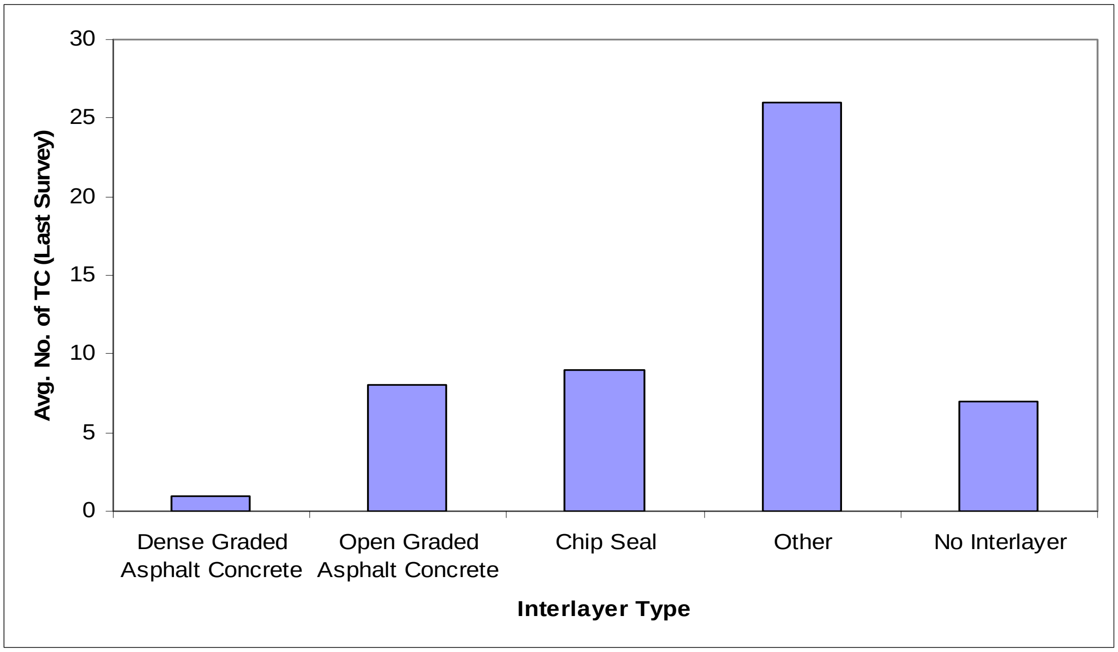


Figure 31. JPCP interlayer type versus average number of transverse cracks.

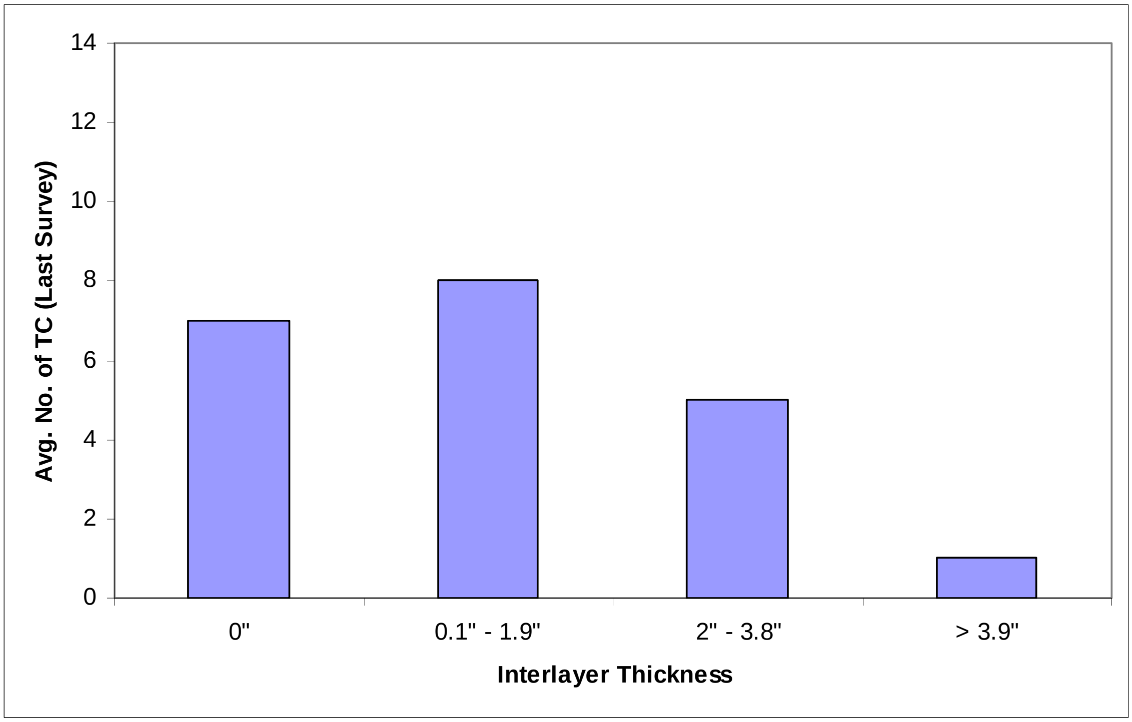


Figure 32. JPCP interlayer thickness versus average number of transverse cracks.

Figure 33 shows that thicker interlayers contribute to the integrity of the joint by controlling the amount of joint faulting (all other parameters being equal).

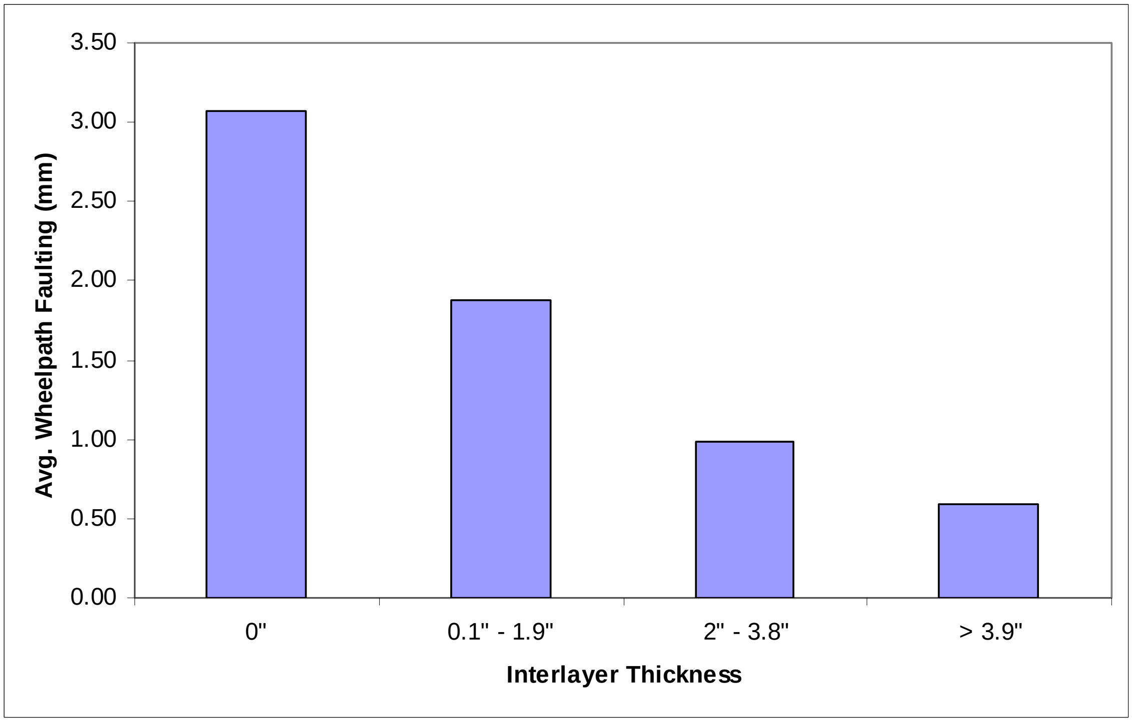


Figure 33. JPCP interlayer thickness versus average wheel path faulting.

Construction Considerations

Construction of the Separator Layer

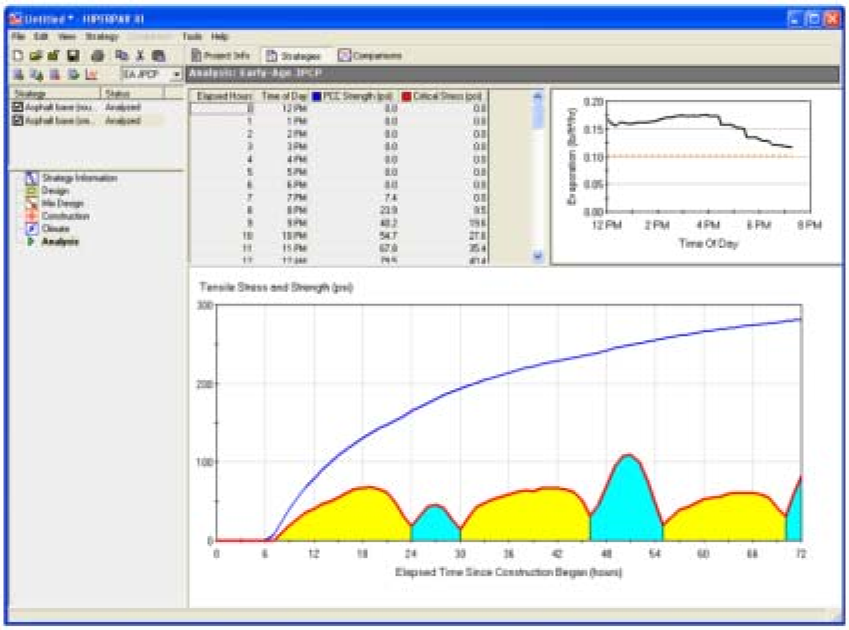
The placement of a separator layer is straightforward. The procedure depends on the interlayer material, but standard application procedures apply. The existing pavement surface needs to be swept clean of any loose materials. Either a mechanical sweeper or an air blower may be used (ACPA, 1990; McGhee, 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^{o}F to minimize the potential of early age shrinkage cracking (Harrison, 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 in order to cool the surface (ACPA, 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 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 34, which illustrates the predicted tensile stress and strength in the concrete over the first 72 hours following placement.



Elapsed Hours	Time of Day	🟦 PCC Strength (psi)	🟥 Critical Stress (psi)
0	12 PM	0.0	0.0
1	1 PM	0.0	0.0
2	2 PM	0.0	0.0
3	3 PM	0.0	0.0
4	4 PM	0.0	0.0
5	5 PM	0.0	0.0
6	6 PM	0.0	0.0
7	7 PM	7.4	0.0
8	8 PM	23.9	9.5
9	9 PM	40.2	19.6
10	10 PM	54.7	27.8
11	11 PM	67.8	35.4

Figure 34. Screen shot from HIPERPAV III software illustrating tensile stress and strength over first 72 hours. (HIPERPAV, 2010)

Surface Texture

For quieter pavements, the surface texture should be negative (i.e. grooves pointing downwards 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 (ACPA, 2006). It is believed that the use of siliceous sands tend to improves 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 dowelled 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 last 10 years can help do so.

There are five possibilities for misalignment as illustrated in Figure 35. These misalignments can cause various types of performance issues ranging from slab spalling to cracking as shown in Table 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.

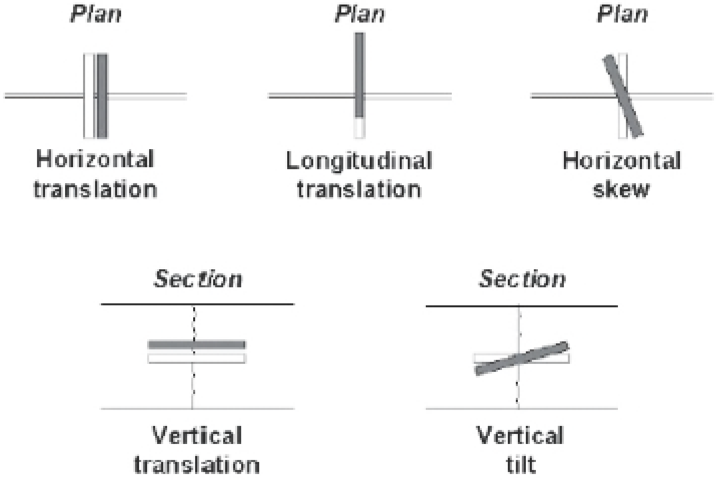


Figure 35. Types of dowel bar misalignments.
(from FHWA, 2007)

Table 12. Dowel misalignment and effects on pavement performance. (after FHWA, 2005)

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

An illustration of a failed contraction joint due to dowel misalignment is shown in Figure 36. Additionally, an example of dowel "longitudinal translation" is also shown.

Failed contraction joint due to dowel misalignment

Example of dowel longitudinal translation (joint is not the same as the one to the left)

Figure 36. Photos of dowel misalignment from an Interstate pavement.
(Photos courtesy of Kevin Littleton and Joe Mahoney)

A critical step for minimizing misalignment is to measure the post-construction 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; FHWA, 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 37.

Figure 37. MIT Scan-2.
(from Yu and Khazanovich, 2005)

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).

Various studies have been done to examine the issue of what are allowable dowel misalignments. A best practices document is available from the FHWA (FHWA, 2007).

Example Designs

Table 13 summarizes a selection of unbonded concrete overlays of concrete pavements constructed in the US since 1993. The information presented in the table was compiled from National Concrete Overlay Explorer (a database provided by the American Concrete Pavement Association (ACPA, 2010)). The website currently contains only a representative sampling of projects across the US, 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 11 include:

- Slab thickness ranges from 9 to 12 in.
- Doweled joints spaced mostly at 15 ft.
- HMA interlayers range in thickness from 1 to 3 in. with most dense-graded, but some open-graded mixes.
- Existing pavements were either jointed or CRCP.

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 14.

A selection of significant practices and specifications associated with paving unbonded concrete overlays over existing concrete were selected and included in Table 15. The table includes a brief explanation why the issue is of special interest, along with examples from the study guide specification recommendations. Three major practices are featured: (1) existing pavement and pre-overlay repairs, (2) overlay thickness and joint details, and (3) interlayer requirements.

Unbonded Concrete Overlay of Hot Mix Asphalt Concrete Pavements

Criteria for Long-Life Potential

Unbonded concrete overlays of hot mix asphalt concrete (HMA) 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 38 is a sketch of an unbonded overlay over preexisting flexible pavement.

The placement of the overlay can potentially (Smith et al (2002); Harrington (2008)):

- Restore and/or enhance structural capacity of the pavement structure
- Increase life equivalent to a full depth pavement
- Restore and/or improve friction, noise and rideability

Table 13. A Selection of unbonded concrete overlays constructed in the US since 1993. (Source information from ACPA, 2010)

Project Location and Details	Year of Overlay Construction	Design details of Overlay
I-77, Yadkin, South of Elkin, NC. The existing pavement is CRCP and 30 years old	2008	• Slab thickness is 11” • Doweled joints spaced at 15’ • Asphalt 1.5” interlayer
I-86, Olean, NY. The existing pavement is JRCP and 30 years old	2006	• Slab thickness is 9” • Doweled joints spaced at 15’ • Asphalt 3” interlayer • 30% truck traffic
I-35, Noble/Kay county, OK. The existing pavement is JRCP and 42 years old	2005	• Slab thickness is 11.5” • Doweled joints spaced at 15’ • Asphalt 2” interlayer • 25% truck traffic
I-40, El Reno, OK. The existing pavement is JPCP and 35 years old	2004	• Slab thickness is 11.5” • Doweled joints spaced at 15’ • Asphalt 2” interlayer
I-264, Louisville, KY. The existing pavement is JRCP and 36 years old	2004	• Slab thickness is 9” • Doweled joints spaced at 15’ • Drainable asphalt 1” interlayer
I-40, El Reno, OK (MP 119 and east), existing pavement is JPCP and 34 years old	2003	• Slab thickness is 10” • Doweled joints • Asphalt 2” interlayer
I-85 (SB), near Anderson,SC, existing pavement is JPCP and 38 years old	2002	• Slab thickness is 12” • Doweled joints • Asphalt 2” interlayer • 35% truck traffic • The NB lanes have been rubblized and overlaid. Performance comparison is recommended.
I-275 , Circle Freeway, KY, existing pavement is JPCP and 28 years old	2002	• Slab thickness is 9” • Doweled joints spaced at 15’ • Drainable asphalt 1” interlayer
I-65 , Jasper County, IN, existing pavement is JRCP and 25 years old	1993	• Slab thickness is 10.5” • Doweled joints spaced at 20’ • Asphalt 1.5” interlayer • 23% truck traffic
I-40, Jackson, TN, existing pavement is JPCP	1997	• Slab thickness is 9” • Doweled joints spaced at 15’ • Asphalt 1” interlayer
I-85, Granville, NC, existing pavement is CRCP and 25 years old	1998	• Slab thickness is 10” • Doweled joints spaced at 18’ • Permeable asphalt 2” interlayer • 25% truck traffic
I-265 @ I-71 , Jefferson County, KY, existing pavement is JRCP and was constructed in 1970	1999	• Slab thickness is 9” • Doweled joints spaced at 15’ • Drainable asphalt 1.3” interlayer
I-85 Newman, GA, existing pavement is JPCP and 38 years old	2009	• Slab thickness is 11” • CRCP overlay • Asphalt 3” interlayer

Table 14. Recommended design attributes for LLCP.

Design Attribute	Recommended Range
Slab thickness	Minimum thickness of 9”
Interlayer thickness (inches)	≥ 1 in. two inches is likely optimal
Joint spacing (feet)	Maximum spacing of 15 ft.
Load transfer device	Mechanical load transfer device, corrosion resistant dowels to promote long life Dowel lengths of 18”
Dowel diameter (inches)	1.5” (function of slab thickness)

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 backcalculate the resilient moduli of various pavement layers (although HMA layers less than 3 in. thick are difficult to backcalculate).

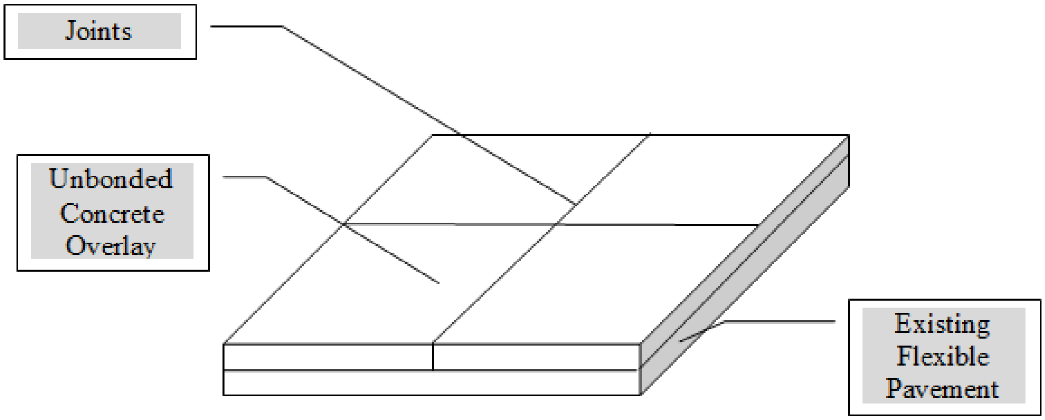


Figure 38. Unbonded concrete overlay of flexible pavement. (Illustration: Joe Mahoney)

Table 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 pre-overlay repairs.</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 ≤ 10mm</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 w/punchouts</td><td>Full-depth repairs</td></tr>
</table>
[Refer to Elements for AASHTO Specification 552, 557, 558 for additional details][1]</td></tr>
<tr><td>Overlay thickness and joint details.</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”: 1.50” diameter minimum
• Dowels should be corrosion resistant
[Refer to Elements for AASHTO Specification 563 for additional details][1]</td></tr>
<tr><td>Interlayer between overlay and existing pavement.</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.
• Surface temperature of HMA interlayer shall < 90°F prior to overlay placement.
[Refer to Elements for AASHTO Specification 563 for additional details] [1]</td></tr>
<tr><td>Concrete overlay materials.</td><td></td><td>• Supplementary cementitious materials may be used to replace a maximum of 40 to 50% of the portland cement.
[Refer to Elements for AASHTO Specification 563 for additional details][1]</td></tr>
</table>

[1] Contained in Appendix E-4

Preoverlay Repairs

The preoverlay requirements are minimal at best. Table 16 summarizes the possible preoverlay repairs needed in preparation for the PCC unbonded concrete overlay of asphalt pavements (Harrington, 2008).

Table 16. Suggested preoverlay repairs. (Harrington, 2008)

Existing Pavement Condition	Possible Repairs
Potholes	Fill with asphalt concrete
Shoving	Mill
Rutting ≥ 2”	Mill
Rutting < 2”	None or mill
Crack width ≥ 4”	Fill with asphalt

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. thick of competent material to ensure 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 between 4 to 11 in., however, to ensure long life performance the slab thicknesses of the overlay should range between 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 17 summarizes unbonded concrete overlays of concrete pavements constructed in the United States since 1995. The information presented in the table was compiled from National Concrete Overlay Explorer. 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 17 include:
- Slab thicknesses range from 9 to 12 in.
- Doweled joints spaced mostly at 15 ft.

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 SHAs surveyed in Phase 1 and those Agencies who 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 upon the elevation difference, but 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 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.

Table 17. Overview of selected unbonded concrete overlays of flexible pavements constructed in the US since 1995. (Source data from ACPA, 2010)

Project Location and Details	Year of Overlay Construction	Design details of Overlay
Cherry Street, North to H-17, IA	2004	• Slab thickness is 9” • Doweled joints spaced at 15’
Tiger Mountain, OK, existing pavement was 9 years old	2004	• Slab thickness is 10.5” • Doweled joints spaced at 15’ • 30% truck traffic
US 412, Bakervillie, MO. The existing is 30 years old	2004	• Slab thickness is 12” • Doweled joints spaced at 15’ • 24% truck traffic
US 412, Bakervillie, MO.	2003	• Slab thickness is 12” • Doweled joints spaced at 15’ • 24% truck traffic
I-55, Vaiden, MS	2001	• Slab thickness is 10” • Doweled joints spaced at 16’
E-33, IA	1998	• Slab thickness is 9” • Doweled joints spaced at 15’
P-33, IA	1998	• Slab thickness is 10” • Doweled joints spaced at 15’
I-10/1-12, LA	1995	• Slab thickness is 12”

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. Since 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 guidance for "Geotechnical Solutions for Transportation Infrastructure" and their recommendations for stabilization of the pavement working platform should be considered.Diagrams of possible transition profiles are shown in Figures 39 and 40.

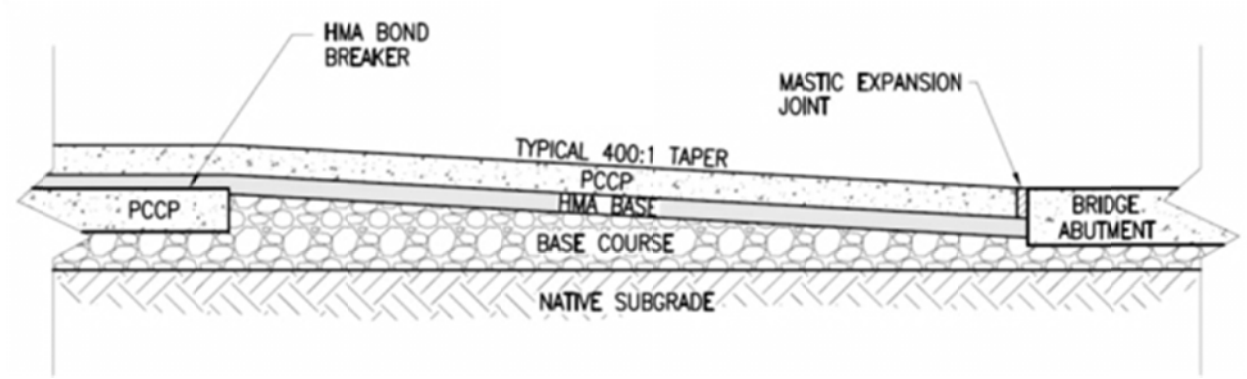


Figure 39. Diagram of transition to bridge approach
(unbonded PCC overlay of PCC pavement).

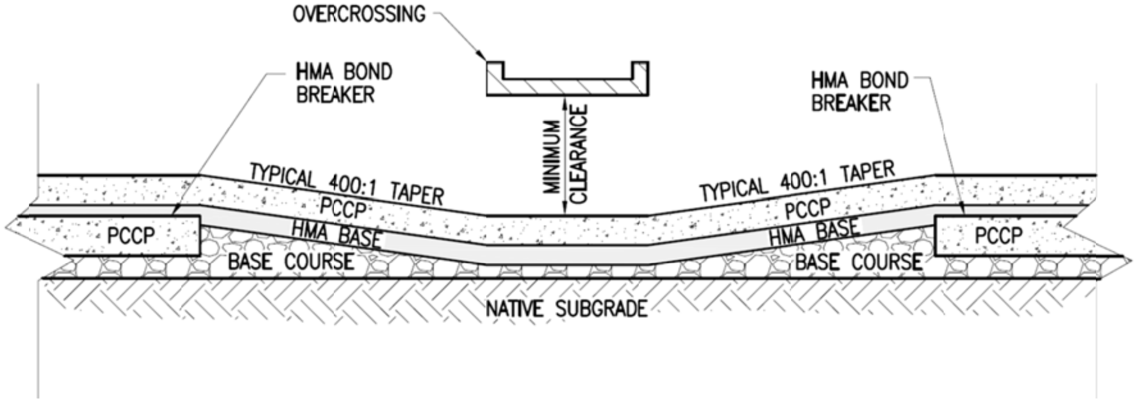


Figure 40. Diagram of transition beneath structure.

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 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 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 guidance for "Geotechnical Solutions for Transportation Infrastructure" and their 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 non-uniform 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 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 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 41 and 42. The minimum thickness of the HMA in the widening is usually controlled by the traffic loading during staging, but is usually a minimum of 6 in. thick, to minimize failure risk during staging and provide more uniform support for the PCC overlay.

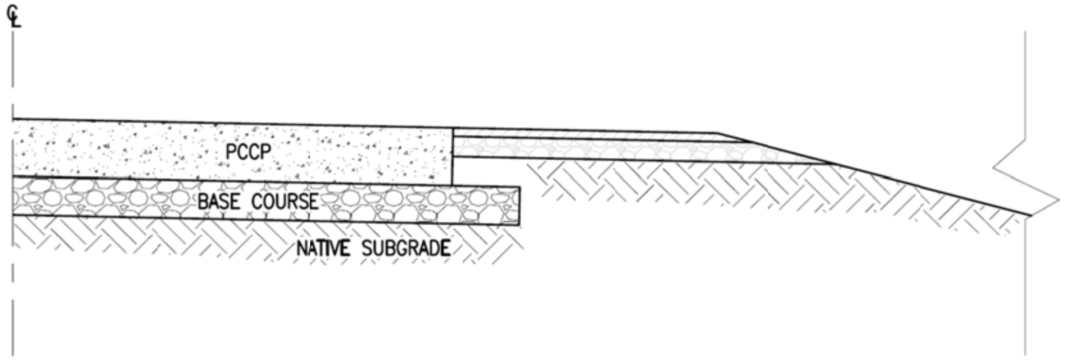


Figure 41. Cross section showing existing PCC pavement without daylighted shoulders.

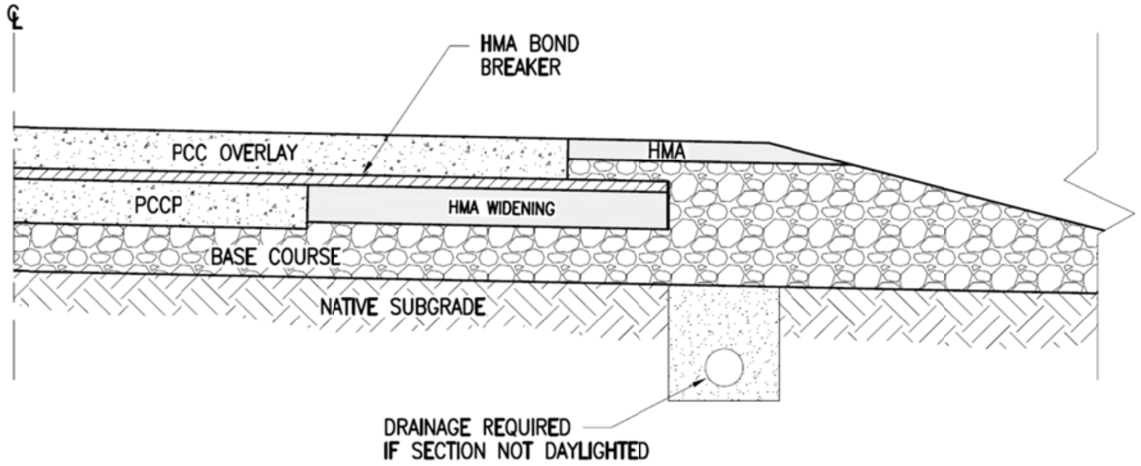


Figure 42. Cross section showing widening of the shoulder with daylighting or drainage.

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 42 and 43.

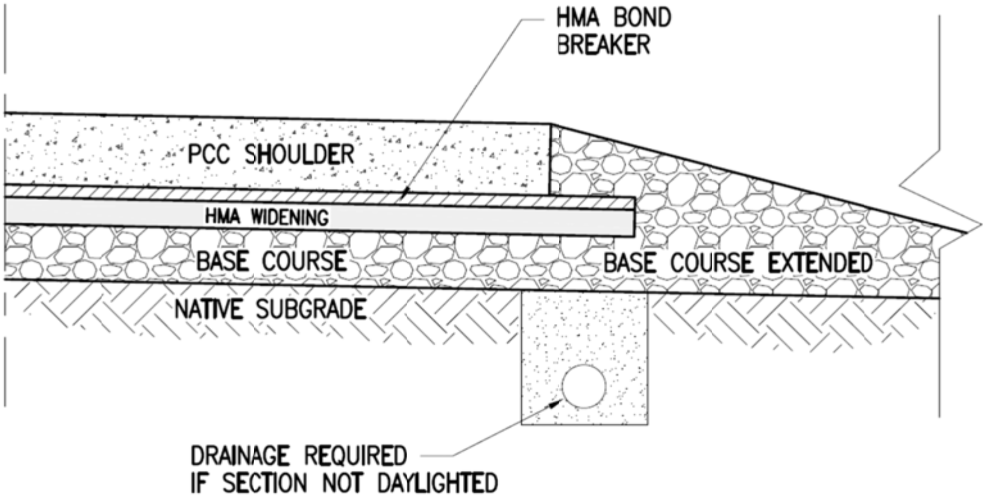


Figure 43. Cross section detail with PCC shoulder.

Best Practices Summary

The definition of long life renewal strategies is a design life ≥ 50 years. To achieve this, unbonded concrete overlays of existing pavements are recommended. This recommendation is based on several sets of information which includes but is 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.

Included is a summary of relevant best practices and related specification requirements (Table 14). Three major practices are featured: (1) existing pavement and pre-overlay repairs, (2) overlay thickness and joint details, and (3) interlayer requirements.

The major findings are recapped in Table 18.

Table 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 acceptable and may be superior to traditional portland cements; use state guidelines for max limits
Existing Pavement	Use criteria provided for pre-overlay repairs.
Concrete Overlay Interlayer	Use a HMA interlayer 1 (minimum) to 2 inches thick.
Concrete Overlay Construction	Control mix and substrate temperatures during construction; tools such as HIPERPAV will help planning and execution

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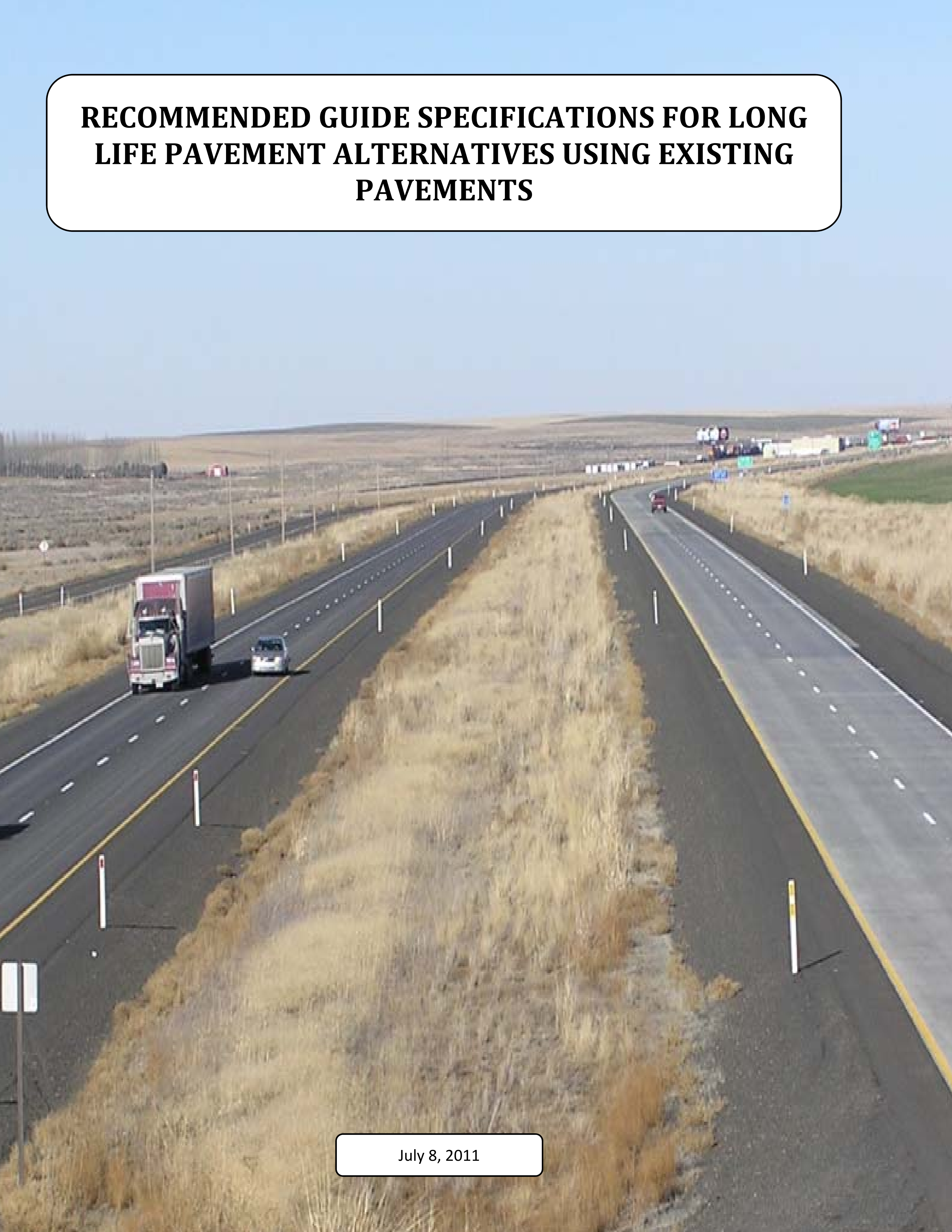
APPENDIX E-4

RECOMMENDED GUIDE SPECIFICATIONS FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

RECOMMENDED GUIDE SPECIFICATIONS FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS



July 8, 2011

Table of Contents

RECOMMENDED GUIDE SPECIFICATIONS FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

Introduction

The guide specifications developed by the SHRP2 R23 team are presented in this document. They are organized into three sections which are: (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 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, due to the fact that there are a wide variety of pavement-oriented specifications developed and maintained by AASHTO committees. Further, 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 and 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 felt necessary for this study. These are: Stone Matrix Asphalt (SMA), Open Graded Friction Course (OGFC), Rubblization of PCC, and Saw, Crack and Seat. Guide specifications were prepared and are contained in this document (see "Specifications not contained in the AASHTO Guide Specifications").

SPECIFICATIONS NOT CONTAINED IN THE AASHTO GUIDE SPECIFICATIONS

SHRP2 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, 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 table below:
<table>
<tr><th>Test Method and Description</th><th>% of 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 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:
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, though grains free of deleterious substances.
<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">Non-plastic</td></tr>
</table>
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
</td></tr>
</table>

<table>
<tr><th>Paragraph</th><th>Content</th></tr>
<tr><td>Materials (continued)</td><td>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:
<table>
<tr><th rowspan="2">Sieve Size</th><th colspan="2">0.5 in.</th><th colspan="2">0.375 in.</th></tr>
<tr><th>Lower Limit</th><th>Upper Limit</th><th>Lower Limit</th><th>Upper Limit</th></tr>
<tr><td>0.75 in.</td><td>100</td><td>100</td><td></td><td></td></tr>
<tr><td>0.5 in.</td><td>90</td><td>100</td><td>100</td><td>100</td></tr>
<tr><td>0.375 in.</td><td>26</td><td>78</td><td>90</td><td>100</td></tr>
<tr><td>No. 4</td><td>20</td><td>28</td><td>26</td><td>60</td></tr>
<tr><td>No. 8</td><td>16</td><td>24</td><td>20</td><td>28</td></tr>
<tr><td>No. 16</td><td>13</td><td>21</td><td>13</td><td>21</td></tr>
<tr><td>No. 30</td><td>12</td><td>18</td><td>12</td><td>18</td></tr>
<tr><td>No. 50</td><td>12</td><td>15</td><td>12</td><td>15</td></tr>
<tr><td>No. 200</td><td>8</td><td>10</td><td>8</td><td>10</td></tr>
<tr><td colspan="5">Typical asphalt content ranges between 6.0 and 7.5% by weight of total mix.</td></tr>
</table>
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 a minimum 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.</td></tr>
<tr><td>Construction</td><td>1. Hot Mix Plant Requirements: SMA shall not be stored at elevated temperatures for more than three 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 40ºF. Spreading operations shall be stopped when the air temperature is below 45 ºF 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 stone matrix asphalt mixture, the surface</td></tr>
</table>

Paragraph	Content
Construction (continued)	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 miles per hour, 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

ALDOT (2008), “Stone Matrix Asphalt (SMA) (Fiber Stabilized Asphalt Concrete), Section 423, Standard Specifications, Alabama Department of Transportation.

Brown, R. and Cooley, L. (1999), “Designing Stone Matrix Asphalt Mixtures for Rut-Resistant Pavements,” Report 425, Project 9-8, National Cooperative Highway Research Program, Transportation Research Board.

Prowell, B., Watson, D., Hurley, G., and Brown, R. (2010), “Evaluation of Stone Matrix Asphalt (SMA) for Airfield Pavements,” Paper, 2010 FAA Worldwide Airport Technology Transfer Conference, Atlantic City, NJ, April 2010.

SHRP2 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>

Aggregates: The aggregate shall be limited to 100% crushed, virgin aggregates.
<ol type="a">
The aggregate shall be combined into a total blend that will produce an acceptable job mix within the gradation limits shown below 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>

No RAP or RAS will be allowed.

Asphalt Binder: The liquid binder shall be a 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 % to 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.
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>

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.
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 40ºF. Spreading operations shall be stopped when the air temperature is below 45 ºF and falling.
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>

REFERENCES

ALDOT (2008), “Polymer Modified Open Graded Friction Course, Section 420, Standard Specifications, Alabama Department of Transportation.

SHRP2 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, 2000 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 two 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 percent of broken pieces less than 3 inches in size. At the surface of the rubblized layer, all pieces shall be less than 6 inches. Below the reinforcing steel or in the lower half of the pavement, the maximum particle size shall be 9 inches. 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 ft2 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.

Paragraph	Content
Construction (continued)	**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 non-conforming 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. **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 ft. 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 non-conforming locations, conducting required test pits, and equipment, labor, tools, and incidentals.

REFERENCES

Sebesta, S., Scullion, T., and Von Holdt, C. (2006), “Rubblization for Rehabilitation of Concrete Pavement in Texas: Preliminary Guidelines and Case Studies,” Report No. FHWA/TX-06/0-4687-1, Texas Transportation Institute, February 2006.

SHRP2 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 inches deep
Construction	**1. Preparatory Work:** Prior to sawing, the following work must be complete: a. If required, construct pavement drainage systems at least two 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 non-conforming locations, required coring, and equipment, labor, tools, and incidentals.

REFERENCES

Department for Transport United Kingdom (2009), "Manual of Contract Documents for Highway Works,” Volume 1, Series 0700, Road Pavement General.

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	1. Use AASHTO 313 unless other local criteria are more appropriate	AASHTO 313
	Draindown	1. ≤ 0.3%	Virginia 313
	Equipment	1. 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 Temps	1. Weather Limitations: Use AASHTO guidance unless other local criteria are more appropriate	AASHTO 313
		2. Plant discharge temperature range: 250 to 300°F.	Missouri 302
		3. Use an approved 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. NMAS: > 40 to 47% passing No. 8 sieve AASHTO M323 has a difference definition for coarse and fine-graded mixtures.	Mn/DOT 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 a Va = 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

AASHTO Paragraph	R23 Recommendations		Source
401.03 Construction (continued)	HMA Placement Temps	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 inches of a longitudinal joint, including the pavement on the traveled way side of the shoulder joint, shall not be less than 2.0 percent below the specified density when unconfined.	Missouri 403
	Lift Thickness	t/NMAS should conform to 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 % of TMD should range between 93 and 94% for dense graded mixes.	AASHTO 401 NCAT

AASHTO Paragraph	R23 Recommendations		Source
401.03 Construction (continued)	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: Base course mixtures: 3/8 to ¾-in. Leveling and top course mixtures: 1/8 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/SY. 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 • 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 between 0.35 to 0.45 • Cement content: range from to 560 to 598 lb/CY • Nominal Maximum Aggregate Size = 1.0 in. • Slump: 0 to 3 in. • Air content = 5.0 to 6.5%	AASHTO 501 Mn/DOT 2301 Missouri 501 Virginia 217
	Supplementary Cementitious Materials	Supplementary cementitious materials may be used to replace a maximum of 35 to 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: Membrane-forming compounds: The compound shall be applied under constant pressure at the rate of 100 to 150 square feet per gallon (or according to manufacturer's recommendation) by mechanical sprayers mounted on movable bridges. On textured surfaces, the rate shall be as close to 100 square feet as possible. • Protection in cold weather: The Contractor shall prevent protect the concrete from freezing during the first 72 hours immediately following concrete placement.	Virginia 316

AASHTO Paragraph	R23 Recommendations		Source
501.03 Construction (continued)	Curing (continued)	- 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.	
	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 3/4-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-inches, but not closer than 3-inches, 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 $^{1}/_{8}$ inch wide, on ¾ inch centers, and of sufficient length, thickness and resilience to form grooves approximately $^{3}/_{16}$ inch deep in the fresh concrete surface. Final texture shall be uniform in appearance with substantially all of the grooves having a depth between $^{1}/_{16}$ inch and $^{5}/_{16}$ inch. 3. Additional texturing methods: Methods that include astro-turf drag, diamond grinding and diamond grooving can be considered in accordance with local practice.”	AASHTO 501 Michigan 602 Washington 5-05 and Amendment dated 8-2-10
	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	Mn/DOT 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-pounds, and chipping hammers shall not weigh more than 15-pounds. All power driven hand tools used for the removal of pavement shall be operated at angles less than 45-degrees 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-inches. Repair areas shall be kept square or rectangular. Repair areas that are within 12.0-inches 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 1/2 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-feet 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 inch after initial grinding must be reground until faulting is less than 1⁄16 inch.	Michigan 603
	Texture	Grind to a parallel corduroy type texture consisting of grooves 1⁄16 to 1⁄8 inch wide, 1⁄16 inch deep and 1⁄16 to 1⁄8 inch 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 inch when checked with a 10-foot 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 inch, 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 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 between 0.35 to 0.45 • Cement content: range from to 560 to 598 lb/CY • Nominal Maximum Aggregate Size = 1.0 in. • Slump: 0 to 4 in. • Air content = 5.0 to 6.5%	AASHTO 501 and 563 Mn/DOT 2301 Missouri 501 Virginia 217
	Supplementary Cementitious Materials	Supplementary cementitious materials may be used to replace a maximum of 40 to 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 inches wide and one inch 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	

AASHTO Paragraph	R23 Recommendations		Source
563.03 Construction (continued)	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
	Curing	• Cure the concrete for at least 3 days immediately after the finishing operation. • Curing systems: Membrane-forming compounds: The compound shall be applied under constant pressure at the rate of 100 to 150 square feet per gallon by (or according to manufacturer's recommendation) mechanical sprayers mounted on movable bridges. On textured surfaces, the rate shall be as close to 100 square feet 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	• Min flexural strength opening ranges from 350 psi for thick slabs (≥ 9.5 in.) to 500 psi for thin slabs (6 in.). [Mostly Mn/DOT 2301} • The unbounded concrete overlay may be opened for light-weight traffic when the concrete has attained a minimum compressive strength of 2500 psi. The concrete pavement shall not be opened to all types of traffic until the concrete has attained a minimum compressive strength of 3000 psi. [Missouri 506.20]	Mn/DOT 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 to 1.8 ft^2 in area. Other state experience, such as Caltrans, suggests that a much larger cracking pattern can work well for 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 Dept 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 DOT	Not available.

AASHTO Specification Designation 313 “Materials”
Open Graded Bituminous Base (OGBB)

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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 percent 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 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 seconds.
Michigan DOT (Section 303)	Major construction related items	
	Equipment	Compact with 3 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 3 passes of 5 to 10 ton steel drum roller.
	Plant discharge temperature	250 to 300°F
	Maximum compacted layer thickness	≤ 4 in.
Texas DOT (Item 247)	Not available.	

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>Virginia DOT
(Section 313)</td><td>Major construction related items
<table>
<tr><td>Draidown</td><td>≤ 0.3%</td></tr>
<tr><td>Equipmnt</td><td>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 to 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.</td></tr>
<tr><td>Mix temperature</td><td>Mixtures shall be between 250 to 280°F</td></tr>
<tr><td>Surface Tolerance</td><td>The finished surface of the stabilized open-graded material shall be uniform and shall not vary at any point more than 0.5 inch above or below the grade shown on the plans.</td></tr>
<tr><td>Weather Limitations</td><td>Atmospheric temp > 40°F and the surface temp ≥ 35°F</td></tr>
<tr><td>Traffic Restrictions</td><td>The Contractor shall not use the open-graded course as a haul road or storage area. Construction trffic will not be permitted on the open-graded course except for equipment required to place the next layer.</td></tr>
<tr><td>Hydraulic Efficiecy</td><td>Stabilized open-graded material shall be designed to have an in-place coefficient of permeability of at least 1,000 feet per day when tested in accordance with VTM-84.</td></tr>
</table></td></tr>
<tr><td>Washington DOT</td><td>Not available.</td></tr>
</table>

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.
Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.
Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.
MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.
TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.
Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.
WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 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 rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>AASHTO (Section 315)</td><td>1. 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>2. 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.]
3. The minimum permittivity shall be 1.0 sec-1</td></tr>
<tr><td>Texas DOT</td><td>Not available.</td></tr>
<tr><td>Virginia DOT</td><td>4. Not available.</td></tr>
<tr><td>Washington DOT
(Section 2-12)</td><td>5. Geosynthetic roll identification, storage, and handling shall be in conformance to ASTM D 4873.
6. During periods of shipment and storage, the geosynthetic shall be stored off the ground.
7. 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.
8. Geosynthetics for separation shall conform to:
<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 sec-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 hours in xenon arc device.</td></tr>
</table>
</td></tr>
</table>

AASHTO Specification Designation 315 “Construction”
Separator Fabric for Bases

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="6">AASHTO
(Section 315)</td><td colspan="2">Major construction related items</td></tr>
<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 percent 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>
<tr><td>Michigan DOT</td><td colspan="2">Not available.</td></tr>
<tr><td>Minnesota DOT</td><td colspan="2">Not available.</td></tr>
<tr><td>Missouri DOT
(Section 1011)</td><td colspan="2">No major construction related items listed in Section 1011.</td></tr>
<tr><td>Texas DOT</td><td colspan="2">Not available.</td></tr>
<tr><td>Virginia DOT</td><td colspan="2">Not available.</td></tr>
</table>

Agency/Organization	Specification Section
	Construction
Washington DOT (Section 2-12)	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-inches 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-feet at all longitudinal and transverse joints, or the geotextile joints shall be sewn together. The initial lift thickness shall be 6-inches or more.

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 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 rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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 D 692 and AASHTO M 323. Provide aggregate of crushed stone, crushed slag, crushed gravel, or natural gravel.
b. **Fine Aggregate**. Meet AASHTO M 29 and AASHTO M 323. Provide aggregate of natural sand, manufactured sand, stone screenings, slag screenings, or a combination of these materials.
c. **Mineral Filler**. Meet AASHTO M 17.
d. **Lime for Asphalt Mixtures**. Meet AASHTO M 303.</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 to 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><td>NMAS (mm)</td><td>37.5</td><td>25.0</td><td>19.0</td><td>12.5</td><td>9.5</td></tr>
<tr><td>PCS (mm)</td><td>9.5</td><td>4.75</td><td>4.75</td><td>2.36</td><td>2.36</td></tr>
<tr><td>PCS Control Point % Passing</td><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>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>Minnesota DOT
(Section 2360)</td><td>Major materials related items
<table>
<tr><td>Design Air Void Content</td><td>
<table>
<tr><th>Locatin from surface</th><th>≤ 4-in.</th><th>>4-in.</th></tr>
<tr><td>DesigAir Voids (Va)</td><td>4.0%</td><td>3.0%</td></tr>
</table>
</td></tr>
<tr><td>PG Binder Selection with RAP</td><td>
<table>
<tr><th rowspan="2"></th><th rowspan="2">Specified PG</th><th colspan="2">PG to be used with RAP</th></tr>
<tr><th>≤20% RAP</th><th>> 20% RAP</th></tr>
<tr><td rowspan="2">Overlay</td><td>64-22</td><td>64-22</td><td>64-28</td></tr>
<tr><td>All others</td><td>No adjust.</td><td>No adjust.</td></tr>
<tr><td rowspan="6">New Const.</td><td>52-34</td><td>52-34</td><td>Not allow</td></tr>
<tr><td>58-28</td><td>58-28</td><td>58-28</td></tr>
<tr><td>58-34</td><td>58-34</td><td>Not low</td></tr>
<tr><td>64-28</td><td>64-28</td><td>64-28</td></tr>
<tr><td>64-34</td><td>64-34</td><td>Not allow</td></tr>
<tr><td>All others</td><td>No adjust.</td><td>Not allow</td></tr>
</table>
</td></tr>
<tr><td>VMA as a function of Fine and Coarse Gradations</td><td>
<table>
<tr><th>NMAS (in.)</th><th>Fine Mix % Pass No. 8</th><th>Min VMA</th><th>Coarse Mix % Pass No. 8</th><th>Min VMA</th></tr>
<tr><td>3/8</td><td>--</td><td>15.0</td><td>--</td><td>--</td></tr>
<tr><td>½</td><td>> 47</td><td>15.0</td><td>≤ 47</td><td>14.5</td></tr>
<tr><td>¾</td><td>></td><td>14.0</td><td>≤ 39</td><td>13.5</td></tr>
<tr><td>1</td><td>> 35</td><td>13.0</td><td>≤ 35</td><td>12.5</td></tr>
</table>
</td></tr>
</table>
</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>Missouri DOT
(Section 403)</td><td>Major materials related items
<table>
<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>12.5mm</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 percent virgin binder replacement provided testing according to AASHTO M 323 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 TP 58, *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 percent when compacted to 95 mm with 7 ± 0.5 percent air voids and tested in accordance with AASHTO T 283. SMA mixtures shall have a TSR greater than 80 percent when compacted to 95 mm with 6 ± 0.5 percent air voids and tested in accordance with AASHTO T 283.</td></tr>
</table>
</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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</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 @ 0.5" Rut Depth, Tested @ 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>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">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>9.0 mm</td><td>0 – 3</td><td>64-22</td></tr>
<tr><td></td><td>3 – 10</td><td>70-22</td></tr>
<tr><td></td><td>> 10</td><td>76-22</td></tr>
<tr><td>9.5 mm</td><td>0 – 3</td><td>64-22</td></tr>
<tr><td></td><td>3 – 10</td><td>70-22</td></tr>
<tr><td></td><td>> 10</td><td>76-22</td></tr>
<tr><td>12.5 mm</td><td>0 – 3</td><td>64-22</td></tr>
<tr><td></td><td>3 – 10</td><td>70-22</td></tr>
<tr><td></td><td>> 10</td><td>76-22</td></tr>
<tr><td>19,0</td><td><10</td><td>64-22</td></tr>
<tr><td></td><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 inches.</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>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>Washington 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-percent 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>½ in.</th><th>¾ 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 rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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 to 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>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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/SY)</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: 3/8 to ¾-in.
• Leveling and top course mixtures: 1/8 to ¼-in.</td></tr>
</table></td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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 (in.)</th></tr>
<tr><td>3/8-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
> -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 inches of a longitudinal joint, including the pavement on the traveled way side of the shoulder joint, shall not be less than 2.0 percent 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 percent of the theoretical maximum specific gravity for all mixtures except SMA. SMA mixtures shall have a minimum density of 94.0 percent of the theoretical maximum specific gravity.</td></tr>
</table>
</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>Texas DOT
(Items 340 and 341)</td><td>
<table>
<tr><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 Temp (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:
<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. Non-contact infrared thermometer
2. Thermal camera behind the paver
3. Paver mounted infrared bar (Pave-IR system)
The temperature measurements are applied to 150 ft. longitudinally measure portion of the mat behind the paver.</td></tr>
<tr><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>
</table>
</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>Virginia DOT
(Section 315)</td><td>
<table>
<tr><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 inches. 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-foot 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 1/4 inch.</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 to 12.5 mm</td><td>92.2 to 92.5%</td></tr>
<tr><td>19.0 mm</td><td>92.0 to 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 to 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>
</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="3">Washington DOT
(Section 5-04)</td><td>MTV</td><td>1. Direct transfer of HMA from the hauling equipment to the paving machine will not be allowed in the top 0.30-feet of the pavement section of hot mix asphalt (HMA) used in traffic lanes with a depth of 0.08-feet 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.</td></tr>
<tr><td>Cyclic Density</td><td>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-percent of the reference maximum density.
2. A $500 price adjustment will be assessed for any 500-foot section with two or more density readings below 90.0-percent of the reference maximum density.</td></tr>
<tr><td>Long. Joint Density</td><td>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-percent of the reference maximum density.
2. If one density reading, at either longitudinal joint, is below 90.0-percent of the reference maximum density, a $200 price adjustment will be assessed for that sublot.</td></tr>
<tr><td rowspan="2">NCAT
(Brown, et al, 2004)</td><td colspan="2">Recommendations included:
1. For fine-graded HMA: lift thickness/Nominal Maximum Aggregate Size (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 colspan="2">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 (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Brown, R., Hainin, R., Cooley, A., and Hurley G. (2004), “Relationship of Air Voids, Lift Thickness, and Permeability in Hot Mix Asphalt Pavements,” Report 531, National Cooperative Highway Research Program, Transportation Research Board.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 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 rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>AASHTO
(Section 404)</td><td>1. AASHTO references to Section 702 which lists:
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 micro-surfacing, 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 percent water provided that resulting material produces a uniform application of the tack.</td></tr>
<tr><td>Washington 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 1-part water to 1-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.

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>Minnesota DOT
(Section 2357)</td><td>Major construction related items
<table>
<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</td></tr>
<tr><td>Application Rates</td><td>The bituminous material shall be applied at a uniform rate not to exceed:
(1) 0.05 gallon per square yard for cutback asphalt and undiluted asphalt emulsion (as supplied from the refinery).
(2) 0.20 gallon per square yard 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 to 160ºF, (2) RS-1: 70 to 140ºF, and (3) SS-2, CRS-1, CRS-2: 120 to 185ºF</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 percent by volume to improve the material application and distribution characteristics. However, the added water will be excluded from the pay quantities.</td></tr>
</table></td></tr>
<tr><td>Missouri DOT
(Section 407)</td><td>Major construction related items
<table>
<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 gallon per square yard and a maximum of 0.10 gallon per square yard.</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 percent 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></td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="5">Texas DOT (Item 340)</td><td colspan="2">Major construction related items</td></tr>
<tr><td>Preparation of Surface</td><td>Clean the surface before placing the tack coat.</td></tr>
<tr><td>Application Rates</td><td>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.</td></tr>
<tr><td>Tacked Surfaces</td><td>Apply a thin, uniform tack coat to all contact surfaces of curbs, structures, and all joints.</td></tr>
<tr><td>Adhesion Properties</td><td>The Engineer may use Tex-243-F to verify that the tack coat has adequate adhesive properties.</td></tr>
<tr><td rowspan="4">Virginia DOT
(Section 310)</td><td colspan="2">Major construction related items</td></tr>
<tr><td>Preparation of Surface</td><td>The existing surface shall be patched, cleaned, and rendered free from irregularities to the extent necessary to provide a reasonably smooth and uniform surface.</td></tr>
<tr><td>Tacked Surfaces</td><td>The edges of existing pavements that are to be adjacent to new pavement shall be cleaned to permit adhesion of asphalt.</td></tr>
<tr><td>Application Rates</td><td>Undiluted asphalt shall be applied at the rate of 0.05 to 0.10 gallons per square yard. Diluted asphalt shall be applied at the rate of 0.10 to 0.15 gallons per square yard.</td></tr>
<tr><td>Washington DOT
(Section 5-04)</td><td colspan="2">1. A tack coat of asphalt shall be applied to all paved surfaces on which any course of HMA is to be placed or abutted.
2. 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.
3. For Roadways open to traffic, the application of tack coat shall be limited to surfaces that will be paved during the same working shift.
4. The spreading equipment shall be equipped with a thermometer to indicate the temperature of the tack coat material.
5. 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.</td></tr>
</table>

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 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 DOT	No specification available.

AASHTO Specification Designation 409 “Construction”
Cold Mill Asphalt Pavement

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="4">AASHTO (Section 409)</td><td colspan="2">Major construction related items</td></tr>
<tr><td>Milling Equipment</td><td>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.</td></tr>
<tr><td>Milling Operations</td><td>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.</td></tr>
<tr><td>Surface Tests</td><td>Meet the specified surface tolerance, as verified using a 10-ft rolling straightedge operated parallel to centerline. Ensure no variation greater than [1/4 in.]</td></tr>
<tr><td rowspan="3">Michigan DOT
(Section 502)</td><td colspan="2">Major construction related items</td></tr>
<tr><td>Milling Equipment</td><td>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 feet in length. An alternate grade referencing attachment may be used if approved by the Engineer prior to use.</td></tr>
<tr><td>Milling Operations</td><td>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.</td></tr>
</table>

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 inch, 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 DOT	No specification available.	

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 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 DOT	Not available.

AASHTO Specification Designation 411 “Construction”
In-Place Cold Recycled Asphalt Pavement

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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] percent. 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>
</table>

Agency/Organization	Specification Section
	Construction
Michigan DOT	Not available
Minnesota DOT	Not available
Missouri DOT	No files available.
Texas DOT	No files available.
Virginia DOT	No files available.
Washington DOT	Not available.

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 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

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Materials</th></tr>
<tr><td rowspan="13">AASHTO
(Section 501)</td><td colspan="2">Major materials related items</td></tr>
<tr><td>Portland Cement</td><td>Conform to AASHTO M85</td></tr>
<tr><td>Fine Aggregate</td><td>Conform to AASHTO M6</td></tr>
<tr><td>Coarse Aggregate</td><td>Conform to AASHTO M80</td></tr>
<tr><td>Load Transfer Devices</td><td>Conform to AASHTO M31</td></tr>
<tr><td>Joint Filler</td><td>Conform to AASHTO M282 Poured Joint Sealants for Pavements</td></tr>
<tr><td>Reinforcing Steel</td><td>1. Conform to AASHTO M31 or M322.
2. Furnish deformed bars for concrete structures meeting the tensile properties for the grade specified.</td></tr>
<tr><td>Curing Materials</td><td>1. Burlap Cloth: AASHTO M182
2. Sheet Materials: AASHTO M171
3. Liquid Membrane Compounds: AASHTO M148</td></tr>
<tr><td>Air-Entraining Admixtures</td><td>Conform to AASHTO M154</td></tr>
<tr><td>Chemical Admixtures</td><td>Conform to AASHTO M194 as applied to (1) water-reducing, (2) set-retarding, and (3) set-accelerating.</td></tr>
<tr><td>Fly Ash</td><td>Conform to AASHTO M295</td></tr>
<tr><td>Ground Granulated Blast Furnace Slay (GGBFS)</td><td>Conform to AASHTO M302</td></tr>
<tr><td>Water</td><td>Conform to AASHTO M157
Potable-quality water requires no testing.</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>Michigan DOT
(Section 601)</td><td>Major materials related items
<table>
<tr><td>Cement</td><td>Section 901</td></tr>
<tr><td>GGBFS</td><td>Section 901</td></tr>
<tr><td>Fly Ash</td><td>Section 901</td></tr>
<tr><td>Coarse Aggregate</td><td>Section 902</td></tr>
<tr><td>Fine Aggregate</td><td>Section 902</td></tr>
<tr><td>Concrete Admixtures</td><td>Section 903</td></tr>
<tr><td>Water</td><td>Section 911</td></tr>
</table>
<table>
<tr><td>Certified Batch Plants</td><td>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 miles
of the project limits, the Engineer may waive NRMCA certification and/
or automation requirements</td></tr>
<tr><td>Additional Water at Placement Site</td><td>Do not add more water than the approved concrete mix design will allow based on maximum water content and maximum water/cementitious material ratio.</td></tr>
<tr><td>Concrete Placing Temp</td><td>Concrete must be between 45°F and 90°F at the time it is placed.</td></tr>
<tr><td>Air Content</td><td>At the time of placement, concrete must have 6.5 ± 1.5 percent entrained air. However, concrete furnished for slipform placement and having a slump of 1.5 inches or less, may have a minimum of 4.5 percent entrained air</td></tr>
</table>
</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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/CY with a minimum of portland cement = 400 lb/CY 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/CY</td></tr>
<tr><td>Water Cement Ratio</td><td>The target W/C ratio is 0.40 for large paving projects (>5,000 CY). Incentives and disincentives associated with lower or higher W/C ratios are shown below
<table>
<tr><th>Mean Value of W/C (termedQI)</th><th>Payment Incentive or Disincentive per CY ($/CY)</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>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>Missouri DOT
(Section 501, 507)</td><td>Major materials related items
<table>
<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 dmixtures</td><td>Section 1054</td></tr>
<tr><td>Water</td><td>Section 1070</td></tr>
</table>
<table>
<tr><td>Cement Requirement for Pavement Concrete</td><td>560 lb/CY</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 percent.</td></tr>
<tr><td>Supplementary Cementitious Materials</td><td>1. Supplementary cementitious materials may be used to replace a maximum of **40 percent of the Portland cement.**
2. Fly Ash: Class C or Class F fly ash may be used to replace a maximum of **25 percent 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 percent 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 non-destructive 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>
</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>Texas DOT
(Item 360)</td><td>Major materials related items
<table>
<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>
</table></td></tr>
<tr><td>Virginia DOT (Section 217)</td><td>Major materials related items
<table>
<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. Nominal Maximum Aggregate Size = 1.0 in.
4. Slump: 0 to 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">Equipmnt</th><th colspan="3">Cncrete Temperature</th></tr>
<tr><th>< 80°F</th><th>80 to 90°F</th><th>> 90°F</th></tr>
<tr><td>Agitator Type Haul Equipment
• Retarded
• Unretarded</td><td>2.5 hours
1.5 hours</td><td>2.0 hours
1.25 hours</td><td>1.5 hours
1.0 hour</td></tr>
<tr><td>Nonagitator Type Haul Equipment—all Concrete</td><td>1.0 hour</td><td>0.75 hour</td><td>0.5 hour</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 to 95°F apply.</td></tr>
</table></td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>Washington DOT
(Section 5-05)</td><td>Major materials related items
<table>
<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/CY</td></tr>
<tr><td>Water-Cementitious Ratio</td><td>≤ 0.44</td></tr>
</table>
</td></tr>
</table>

AASHTO Specification Designation 501 “Construction”
Portland Cement Concrete Pavements

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>AASHTO
(Section 501)</td><td>Major construction related items
<table>
<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 to 3 in.</td><td>T119</td></tr>
<tr><td>Cement Content
Without Air (min)
With Air (min)</td><td>
564 lb/CY
598 lb/CY</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 to 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>
</td></tr>
</table>

Agency/Organization	Specification Section		
	Construction		
AASHTO (Section 501) (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 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 to 24 hours 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 (±1/4 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 hours after placing concrete but before uncontrolled shrinkage cracking develops.
		Sealing	Similar to longitudinal joint construction.

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="4">AASHTO
(Section 501)
(continued)</td><td>Transverse Construction Joints</td><td>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.</td></tr>
<tr><td>Surface Tolerances</td><td>AASHTO provides for two profile measurement methods
1. 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.
2. Profilograph: Describes a California-type profilograph.</td></tr>
<tr><td>Curing</td><td>1. Cure the concrete for at least 3 days immediately after the finishing operation.
2. Protect the concrete for at least 10 days or until the concrete achieves a compressive strength of [2,200 psi].</td></tr>
<tr><td>Tolerance and Price Adjustments for Pavement Thickness</td><td>1. Determine pavement thickness according to AASHTO T148.
2. 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 to 0.20</td><td>100</td></tr>
<tr><td>0.21 to 0.30</td><td>80</td></tr>
<tr><td>0.31 to 0.40</td><td>72</td></tr>
<tr><td>0.41 to 0.50</td><td>68</td></tr>
<tr><td>0.51 to 0.75</td><td>57</td></tr>
<tr><td>0.76 to 1.00</td><td>50</td></tr>
<tr><td>> 1.00</td><td>Remove and Replace</td></tr>
</table></td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="5">Michigan DOT (Section 602)</td><td colspan="2">Major construction related items</td></tr>
<tr><td>Surface Texture</td><td>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 1⁄8 inch wide grooves spaced 1⁄2 inch on center and 1⁄8 to 1⁄4 inch deep.</td></tr>
<tr><td>Sealing Joints with Hot-Poured Sealants</td><td>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.</td></tr>
<tr><td>Profile</td><td>While the concrete is still plastic, test the slab surface for trueness to the required grade and cross section using a 10-foot straightedge. If high or low spots exceeding 1⁄8 inch in 10 feet (1⁄4 inch for concrete shoulders and inch for concrete base course and temporary concrete pavement) are found, suspend paving operations and correct the finishing procedures. Correct high spots in pavements that exceed these tolerances.</td></tr>
<tr><td>Weather and Temperature Limitations</td><td>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.</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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 pounds per cubic yard**. 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 **3000 psi** in 48 hours. 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 minutes, a standard header joint shall be constructed.</td></tr>
<tr><td>Joint Construction</td><td>Initial joint sawing shall be approximately **1/8 inch** 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>

Agency/Organization	Specification Section	
	Construction	
Minnesota DOT (Section 2301) (continued)	Surface Finish	Mn/DOT 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 hours. (4) Place the curing media on the pavement edges within 30 minutes after removal of the forms when side forms are used. (5) Extend the minimum curing period to 96 hours 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 foot** 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 Mn/DOT Concrete Engineering Unit.
	Thickness Requirements	Where the cores show a thickness deficiency exceeding **½ inch**, but less than **1 inch**, 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 inch** 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.

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="7">Missouri DOT
(Section 502)</td><td colspan="2">Major construction related items</td></tr>
<tr><td>Weather Limitations wrt Freezing Conditions</td><td>All concrete shall be effectively protected from freezing until a minimum compressive strength of 3500 psi has been attained.</td></tr>
<tr><td>Added Finishing Water</td><td>Moisture in any form shall not be applied to the surface of the concrete except for emergency conditions.</td></tr>
<tr><td>Required Texture Depth</td><td>The results of ASTM E 965 shall show a texture depth of any sublot, as defined in Sec 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.</td></tr>
<tr><td>Curing</td><td>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 minutes 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 one gallon for each 200 square feet.
2. Burlap</td></tr>
<tr><td>Straightedge</td><td>As soon as practical, the engineer will straightedge all segments of the paved surface not profilographed, including shoulders. Any variations exceeding 1/8 inch in 10 feet will be marked. Areas more than 1/8 inch high shall be removed</td></tr>
<tr><td>Air Entrainment during Paving Operations</td><td>Tests for entrained air content shall be performed on a random basis for each 500 cubic yards of material produced. The minimum air content in front of the paver shall be 5.0 percent 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.</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>Texas DOT
(Item 360)</td><td>
<table>
<tr><td>Concrete Placement</td><td>1. Do not allow the pavement edge to deviate from the established paving line by more than 1/2 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.</td></tr>
<tr><td>Temperature Restrictions</td><td>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.</td></tr>
<tr><td>Early Opening</td><td>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.</td></tr>
<tr><td>Tolerance and Price Adjustments for Pavement Thickness</td><td>
<table>
<tr><th>Deficiency in Thickness as Determined by Cores (in.)</th><th>Contract Price Allowed</th></tr>
<tr><td>Not Deficient</td><td>100</td></tr>
<tr><td>> 0 to 0.20</td><td>100</td></tr>
<tr><td>> 0.20 to 0.30</td><td>80</td></tr>
<tr><td>> 0.30 to 0.40</td><td>72</td></tr>
<tr><td>> 0.40 to 0.50</td><td>68</td></tr>
<tr><td>> 0.50 to 0.75</td><td>57</td></tr>
<tr><td>> 0.75</td><td>Zero pay or removal</td></tr>
</table>
</td></tr>
</table>
</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">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 **1/4 inch** between any two contact points when tested with a 10-foot 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 square feet per gallon by mechanical sprayers mounted on movable bridges. On textured surfaces, the rate shall be as close to 100 square feet 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 hours 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. These are:
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 to 0.20</td><td>100</td></tr>
<tr><td>0.21 to 0.30</td><td>80</td></tr>
<tr><td>0.31 to 0.40</td><td>72</td></tr>
<tr><td>0.41 to 0.50</td><td>68</td></tr>
<tr><td>0.51 to 0.75</td><td>57</td></tr>
<tr><td>0.76 to 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 pounds per square inch 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>

Agency/Organization	Specification Section	
	Construction	
Washington DOT (Section 5-05)	Subgrade	1. The Subgrade shall be prepared and compacted a minimum of 3-feet beyond each edge of the area which is to receive concrete pavement in order to accommodate the slip-form equipment. 2. Concrete shall not be placed on a frozen Subgrade nor 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-inch. 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. 6. 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 bondbreaking material such as polyethylene film, roofing paper, or other material as approved by the Engineer.

Agency/Organization	Specification Section	
	Construction	
Washington 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-inch outside diameter plain round steel bars 18-inches 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-percent of its design strength or 2500-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 2500-psi as determined from cylinders, made at the time of placement, cured under comparable conditions, and tested in accordance with AASHTO T 22.

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 DOT	Not available.

AASHTO Specification Designation 552 “Materials”
Subsealing and Stabilization

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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 DOT</td><td>Not available.</td></tr>
</table>

AASHTO Specification Designation 552 “Construction”
Subsealing and Stabilization

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="9">AASHTO (Section 552)</td><td colspan="2">All construction related items are:</td></tr>
<tr><td>Grout Plant</td><td>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 to 300 psi when pumping a grout slurry mixed to a 12-second flow cone time. Furnish an injection pump that can continuously pump at rates as low as 1.5 gal/min.</td></tr>
<tr><td>Vertical Movement Testing</td><td></td></tr>
<tr><td>Drilling and Subsealing</td><td></td></tr>
<tr><td>Radial Cracks</td><td></td></tr>
<tr><td>Hole Patching</td><td>Agency should specify drill hole fill material.</td></tr>
<tr><td>Weather Conditions</td><td></td></tr>
<tr><td>Unanticipated Conditions</td><td></td></tr>
<tr><td>Resealing Pavement Joints</td><td></td></tr>
<tr><td>Michigan DOT</td><td colspan="2">Not available.</td></tr>
<tr><td>Minnesota DOT</td><td colspan="2">Not available.</td></tr>
<tr><td>Missouri DOT</td><td colspan="2">Not available.</td></tr>
<tr><td>Texas DOT</td><td colspan="2">Not available.</td></tr>
<tr><td>Virginia DOT</td><td colspan="2">Not available.</td></tr>
<tr><td>Washington DOT</td><td colspan="2">Not available.</td></tr>
</table>

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 DOT (Section 5-01.3(5))	Partial Depth Spall Repair

AASHTO Specification Designation 557 "Materials"
Partial Depth Patching

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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 hours</td><td>Type P-MS</td></tr>
<tr><td>12 to 72 hours</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 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

Agency/Organization	Specification Section	
	Construction	
AASHTO (Section 557)	All construction related items are:	
	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 hours. 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 hours. 4. Epoxy Resin Patching Mortars: Use only Agency-approved materials. Prepare epoxy resin patching mortars according to the manufacturer’s recommendations.
	Preparation of Partial Depth Patch Area	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.
	Placing Patch Material	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.

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>Michigan DOT
(Section 603)</td><td>Relevant construction related items are:
<table>
<tr><td>Size of Patches</td><td>Make repairs 6 feet or longer. When the area to be repaired leaves a section of pavement less than 6 feet from an existing joint or less than 15 feet from the next area to be repaired, remove that section also. For repairs more than 15 feet 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 DOT
(Section 5-01.3(5))</td><td>1. If jackhammers are used for removing pavement, they shall not weigh more than 30-pounds, and chipping hammers shall not weigh more than 15-pounds. All power driven hand tools used for the removal of pavement shall be operated at angles less than 45-degrees 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-inches. Repair areas shall be kept square or rectangular. Repair areas that are within 12.0-inches of another repair area shall be combined.
3. A vertical saw cut shall be made to a minimum depth of 2.0-inches around the area to be patched. The Contractor shall remove material within the perimeter of the saw cut to a depth of 2.0-inches, or to sound concrete. The surface patch area shall be sand blasted 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-inch below the depth of repair. The compressible insert shall extend at least 3.0-inches 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 (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 DOT (Section 5-01.3(4))	Replace Portland Cement Concrete Panel

AASHTO Specification Designation 558 “Materials”
Full Depth Patching

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Materials</th></tr>
<tr><td>AASHTO (Section 558)</td><td>AASHTO references to Subsection 558.02 which lists:
<table>
<tr><td>Material</td><td>AASHTO Subsection</td></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><td>Time from Casting to Traffic Opening</td><td>Grade of Concrete</td></tr>
<tr><td>≤ 8 hours</td><td>Type P-MS</td></tr>
<tr><td>12 to 72 hours</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 hr 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 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

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>AASHTO (Section 558)</td><td>All construction related items are:
<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 hours.
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 hours.
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 Patch Area</td><td>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).</td></tr>
</table>
</td></tr>
<tr><td>Michigan DOT
(Section 603)</td><td>Refer to AASHTO 557 summary.</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>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="3">Texas DOT (Item 361)</td><td colspan="2">Construction related items are:</td></tr>
<tr><td>Repair Area</td><td>Make repair areas rectangular, at least 6 ft. long and at least 1/2 a full lane in width unless otherwise shown on the plans.</td></tr>
<tr><td>Repair Process Steps</td><td>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.</td></tr>
<tr><td>Virginia DOT</td><td colspan="2">Does not have a specific related specification</td></tr>
<tr><td>Washington DOT
(Section 5-01.3(4))</td><td colspan="2">1. Concrete slabs to be replaced as shown in the Plans shall be at least 6.0-feet long and full width of an existing pavement panel. The portion of the panel to remain in place shall have a minimum dimension of 6-feet 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-feet 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-inches to 18-inches 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.</td></tr>
</table>

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 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 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 rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="4">AASHTO (Section 560)</td><td colspan="2">Construction related items are:</td></tr>
<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 in. and 5/32 in. wide, with a distance between the grooves of 1/16 in. 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>
<tr><td rowspan="3">Michigan DOT
(Section 603)</td><td colspan="2">Relevant construction related items are</td></tr>
<tr><td>Faulted Pavement</td><td>Faulted areas at transverse cracks and joints in excess of 1⁄16 inch after initial grinding must be reground until faulting is less than 1⁄16 inch.</td></tr>
<tr><td>Texture</td><td>Grind to a parallel corduroy type texture consisting of grooves 1⁄16 to 1⁄8 inch wide, 1⁄16 inch deep and 1⁄16 to 1⁄8 inch 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 inch when checked with a 10-foot straightedge.</td></tr>
<tr><td>Minnesota DOT</td><td colspan="2">Does not have a specific related specification</td></tr>
<tr><td>Missouri DOT</td><td colspan="2">Does not have a specific related specification</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>Texas DOT (Item 361)</td><td>Relevant construction related items are:
<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>
<tr><td>Virginia DOT</td><td>Does not have a specific related specification</td></tr>
<tr><td>Washington DOT
(Section 5-01.3(9))</td><td>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-inches. Ninety-five-percent of the surface area of the pavement to be ground shall have a minimum of ⅛-inch 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-inches wide, and no deeper than 1⁄16-inch. The land area between the grooves shall be between 1⁄16 and ⅛-inches wide.</td></tr>
</table>

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 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 Mn/DOT Section 2232.
Missouri DOT	Not applicable.
Texas DOT	Not applicable.
Virginia DOT	Not applicable.
Washington DOT	Not applicable.

AASHTO Specification Designation 561 “Construction”
Milling Pavement

Agency/Organization	Specification Section
	Construction
AASHTO (Section 561)	Construction related items are: 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: 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 1/8 inch, 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 DOT	Does not have a specific related specification

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 tie bars. 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 DOT	No specific specification for PCC unbonded overlays.

AASHTO Specification Designation 563 "Materials"
Portland Cement Concrete Unbonded Overlays

Agency/Organization	Specification Section	
	Materials	
AASHTO (Section 563)	Major materials related items	
	Portland Cement	1. AASHTO Subsection 701.02. Meets AASHTO M85 2. Use only Type I or Type II cement
	Asphalt Cements	AASHTO Subsection 702.01(A). Meets AASHTO M320
	Asphalt Concrete	Place a uniform layer to a minimum depth of 1 in.
	Curing Materials	AASHTO Subsection 713.02. Includes three options: 1. Burlap cloth (AASHTO M182) 2. Sheet materials (AASHTO M171) 3. Liquid membrane forming compounds (AASHTO M148)
	Water	AASHTO Subsection 714.01(A). Meets AASHTO M157.
	Reinforcing Steel	Use deformed epoxy-coated bars
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 miles 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 Temp	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 percent entrained air. However, concrete furnished for slipform placement and having a slump of 1.5 inches or less, may have a minimum of 4.5 percent entrained air.

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="3">Specification Section</th></tr>
<tr><th colspan="3">Materials</th></tr>
<tr><td rowspan="5">Minnesota DOT
(Section 2301)</td><td colspan="3">Major materials related items</td></tr>
<tr><td>Minimum Cementitious Content</td><td colspan="2">530 lb/CY with a minimum of portland cement = 400 lb/CY when using fly ash or GGBFS.</td></tr>
<tr><td>Total Alkalis in Portland Cement</td><td colspan="2">0.60%</td></tr>
<tr><td>Total Alkalis in Cementitious Material</td><td colspan="2">≤ 5 lb/CY</td></tr>
<tr><td>Water Cement Ratio</td><td colspan="2">The target W/C ratio is 0.40 for large paving projects (>5,000 CY). 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 CY ($/CY)</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>
<tr><td rowspan="3">Missouri DOT
(Section 506.20)</td><td>Interlayer</td><td colspan="2">The interlayer material shall be a minimum of 1 in. thick new bituminous ….</td></tr>
<tr><td>Concrete</td><td colspan="2">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>
<tr><td colspan="3"></td></tr>
<tr><td>Texas DOT</td><td colspan="3">No specific specification for PCC unbonded overlays.</td></tr>
<tr><td>Virginia DOT</td><td colspan="3">No specific specification for PCC unbonded overlays.</td></tr>
<tr><td>Washington DOT</td><td colspan="3">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 to 3 in.	T119
Cement Content Without Air (min) With Air (min)	 564 lb/CY 598 lb/CY	
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 to 8%	T152, T196, or T199

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr>
<td>AASHTO
(Section 563)
(continued)</td>
<td>Placing and Finishing Concrete (continued)</td>
<td>
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.

3. Longitudinal Joints
<table>
<tr><td>Dimensions</td><td>Saw the first cut or insert the joint material to one third of the depth.</td></tr>
<tr><td>Tiebars</td><td>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.</td></tr>
<tr><td>Construction</td><td>Form or saw longitudinal joints in the plastic concrete. Saw the joints within 4 to 24 hours after placing the concrete and immediately after completing the transverse joints. Allow only the saw on the pavement during sawing operations.</td></tr>
<tr><td>Sealing</td><td>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.</td></tr>
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</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td>AASHTO
(Section 563)
(continued)</td><td>Placing and Finishing Concrete (continued)</td><td>4. Contraction Joints
<table>
<tr><td>Location and Dimensions</td><td>Form or saw joints as narrowly as possible, to at least one third of the pavement depth.</td></tr>
<tr><td>Load Transfer</td><td>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 [±1/4 in.]. Vibrate concrete around all dowel bars without misaligning them.</td></tr>
<tr><td>Construction</td><td>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 hours after placing concrete but before uncontrolled shrinkage cracking develops.</td></tr>
<tr><td>Sealing</td><td>Similar to longitudinal joint construction.</td></tr>
</table>

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.

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).</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="3">AASHTO
(Section 563)
(continued)</td><td>Placing and Finishing Concrete (continued)</td><td>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 to 0.20</td><td>100</td></tr>
<tr><td>0.21 to 0.30</td><td>80</td></tr>
<tr><td>0.31 to 0.40</td><td>72</td></tr>
<tr><td>0.41 to 0.50</td><td>68</td></tr>
<tr><td>0.51 to 0.75</td><td>57</td></tr>
<tr><td>0.76 to 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>

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="5">Michigan DOT
(Section 602)</td><td colspan="2">Major construction related items</td></tr>
<tr><td>Surface Texture</td><td>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 1⁄8 inch wide grooves spaced 1⁄2 inch on center and 1⁄8 to 1⁄4 inch deep.</td></tr>
<tr><td>Seling Joints with Hot-Poured Sealants</td><td>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.</td></tr>
<tr><td>Profile</td><td>While the concrete is still plastic, test the slab surface for trueness to the required grade and cross section using a 10-foot straightedge. If high or low spots exceeding 1⁄8 inch in 10 feet (1⁄4 inch for concrete shoulders and inch for concrete base course and temporary concrete pavement) are found, suspend paving operations and correct the finishing procedures. Correct high spots in pavements that exceed these tolerances.</td></tr>
<tr><td>Weather and Temperature Limitations</td><td>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.</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>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 pounds per cubic yard. 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 3000 psi in 48 hours. 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 Requirements 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 minutes, a standard header joint shall be constructed.</td></tr>
</table></td></tr>
</table>

Agency/Organization	Specification Section	
	Construction	
Minnesota DOT (Section 2301) (continued)	Joint Construction	Initial joint sawing shall be approximately **1/8 inch** 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.
	Surface Finish	Mn/DOT 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 hours. (4) Place the curing media on the pavement edges within 30 minutes after removal of the forms when side forms are used. (5) Extend the minimum curing period to 96 hours 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 foot** 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 Mn/DOT Concrete Engineering Unit.
	Thickness Requirements	Where the cores show a thickness deficiency exceeding **½ inch**, but less than **1 inch**, 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 inch** 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.

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="8">Missouri DOT
(Section 506.20)</td><td colspan="2">Major construction related items</td></tr>
<tr><td>Surface Preparation</td><td>All holes greater than 2 inches wide and one inch 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.</td></tr>
<tr><td>Bituminous Interlayer</td><td>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.</td></tr>
<tr><td>Dowel Bars</td><td>Dowel bars for eight-inch 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.</td></tr>
<tr><td>Tie Bars</td><td>Tie bars shall be installed between lanes in an eight-inch unbounded concrete overlay.</td></tr>
<tr><td>Concrete Temperature</td><td>The concrete temperature shall not exceed 95°F when delivered to the site.</td></tr>
<tr><td>Contraction Joints</td><td>Sawing of the contraction joints shall not cause excessive raveling. Standard joint spacing for a five-inch unbounded concrete overlay is 6 feet transversely and longitudinally. Standard joint spacing for an eight-inch 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 1/8-inch maximum. The joints shall not be sealed, unless open more than ¼ inch, but shall be cleaned of all deleterious material after sawing. Concrete panels with cracking outside of the sawed joints shall be considered unacceptable.</td></tr>
<tr><td>Opening Strength</td><td>The unbounded concrete overlay may be opened for light-weight traffic when the concrete has attained a minimum compressive strength of 2500 psi. The concrete pavement shall not be opened to all types of traffic until the concrete has attained a minimum compressive strength of 3000 psi. Compressive strength for opening to traffic shall be determined either by compressive strength tests in accordance with AASHTO T 22 or the maturity method.</td></tr>
<tr><td>Texas DOT</td><td colspan="2">No specific specification for PCC unbonded overlays.</td></tr>
<tr><td>Virginia DOT</td><td colspan="2">No specific specification for PCC unbonded overlays.</td></tr>
<tr><td>Washington DOT</td><td colspan="2">No specific specification for PCC unbonded overlays.</td></tr>
</table>

REFERENCES

AASHTO (2008), “Guide Specifications for Highway Construction,” American Association of State Highway and Transportation Officials.

Michigan DOT (2003), “Standard Specifications for Construction,” Michigan Department of Transportation.

Mn/DOT (2005), “Mn/DOT Standard Specifications for Construction,” Minnesota Department of Transportation.

MoDOT (2004), “Missouri Standard Specifications for Highway Construction,” Missouri Department of Transportation.

TxDOT (2004), “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges,” Texas Department of Transportation.

Virginia DOT (2007), “Road and Bridge Specifications,” Virginia Department of Transportation.

WSDOT (2010), “Standard Specifications for Road, Bridge, and Municipal Construction, M41-10, Washington State Department of Transportation.

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 (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 DOT	No specific specification.

AASHTO Specification Designation 567 “Materials”
Cracking and Seating

Agency/Organization	Specification Section
	Materials
AASHTO (Section 567)	No materials related specifications.
UK (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 DOT	No specific specification.

AASHTO Specification Designation 567 "Construction"
Cracking and Seating

Agency/Organization	Specification Section	
	Construction	
AASHTO (Section 567)	Construction related items are:	
	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 to 1.8 ft2 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

Agency/Organization	Specification Section	
	Construction	
AASHTO (Section 567) (continued)		verify the specified extent of breakage. Adjust the energy or striking pattern based on these check sections.
	Seating Operations	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.
	Maintenance	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.
UK Dept. for Transport Specifications (Section 716 and NG 716) Cracking and Seating of Existing Jointed Unreinforced Concrete Pavements and Hydraulically Bound Mixture Bases	Construction related items are:	
	Cracking and Seating Equipment	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 pre-set 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.
	Surface Preparation	Any existing asphalt overlay and surfacing shall be removed from the area to be treated for the full width of each lane.
	Test Section	3. The test section shall be no less than 250 m2 nor greater than 420 m2. 4. 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

Agency/Organization	Specification Section	
	Construction	
UK Dept. for Transport Specifications (Section 716 and NG 716) Cracking and Seating of Existing Jointed Unreinforced Concrete Pavements and Hydraulically Bound Mixture Bases (continued)	Test Section (continued)	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. 5. 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.’
	Cracking Operations	1. Proceed with pavement cracking at spaces determined by test section based on effective stiffness modulus computed from FWD tests (refer to UK specification 717). Generally a 0.75 m to 2 m spacing. 2. Surface cracking checked by applying water on all areas, allowing it to surface dry and then core every 300 m2 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.
	Seating	Minimum of six passes with a 20 tonne pneumatic tired

<table>
<tr><th rowspan="2">Agency/Organization</th><th colspan="2">Specification Section</th></tr>
<tr><th colspan="2">Construction</th></tr>
<tr><td rowspan="2"></td><td>Operations</td><td>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>
<tr><td>Michigan DOT</td><td colspan="2">No specific specification.</td></tr>
<tr><td>Minnesota DOT</td><td colspan="2">No specific specification.</td></tr>
<tr><td>Missouri DOT</td><td colspan="2">No specific specification.</td></tr>
<tr><td>Texas DOT</td><td colspan="2">No specific specification.</td></tr>
<tr><td>Virginia DOT</td><td colspan="2">No specific specification.</td></tr>
</table>

<table>
<tr><th rowspan="2">Agency/Organization</th><th>Specification Section</th></tr>
<tr><th>Construction</th></tr>
<tr><td>Washington DOT</td><td>Construction related items are:
<table>
<tr><td>Cracking and Seating Equipment</td><td>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.</td></tr>
<tr><td>Surface Preparation</td><td>1. 1. Prior to cracking, any existing HMA shall be removed from the PCCP to be cracked.</td></tr>
<tr><td>Test Section</td><td>1. 1. A test section will be used to assess early cracking operations (numerous details are associated with the test section).</td></tr>
<tr><td>Cracking Operations</td><td>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.</td></tr>
<tr><td>Seating Operations</td><td>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 5 passes over the cracked concrete. A pass shall be one movement of a roller in either direction.</td></tr>
<tr><td>Maintenance</td><td>1. Public traffic shall not be allowed on the cracked pavement until a minimum of 0.35 ft. of HMA has been placed.</td></tr>
</table></td></tr>
</table>

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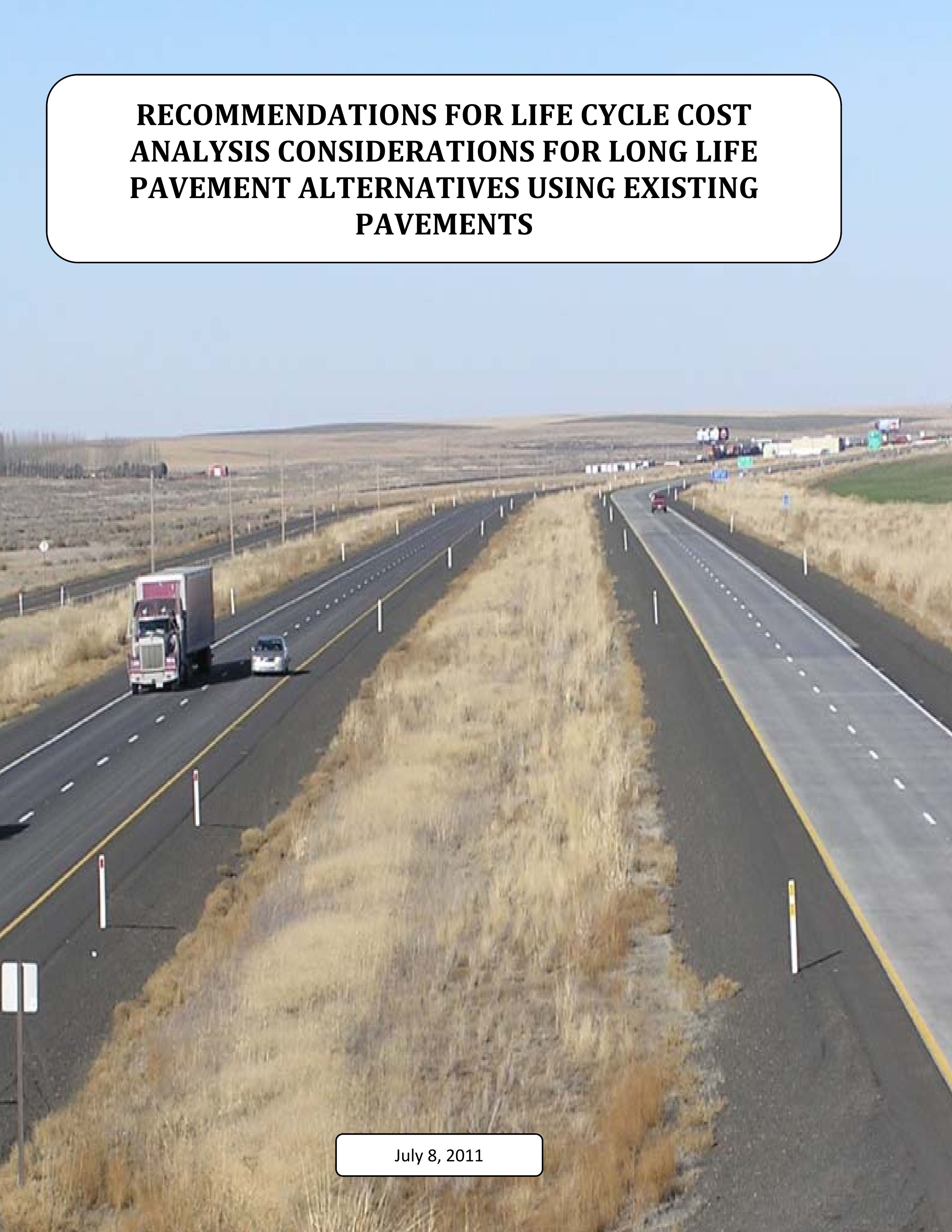
APPENDIX E-5

RECOMMENDATIONS FOR LIFE CYCLE COST ANALYSIS CONSIDERATIONS FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

RECOMMENDATIONS FOR LIFE CYCLE COST ANALYSIS CONSIDERATIONS FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS



July 8, 2011

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RECOMMENDATIONS FOR LIFE CYCLE COST ANALYSIS CONSIDERATIONS FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

Introduction

The SHRP2 R23 Guidelines provide a number of possible alternative designs using either rigid of 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 as 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): cost-benefit analysis, cost-effectiveness analysis, multi-criteria analysis, risk-benefit analysis, etc. At one extreme lies the purely multi-criteria 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 who 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.

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 into the following:

- 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 will be 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, which 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 1 shows a typical analysis period for a given pavement design alternative. Guidelines for the preservation of long life pavements is included in these guidelines based on the work performed in SHRP 2 Project R-26 "Preservation Approaches for High Traffic Volume Roadways." Preservation treatments and approaches recommended in those guidelines should be considered in the re-accruing maintenance or preservation costs associated with each design alternative. A simplified illustration of the activity and timing is illustrated in Figure 1.

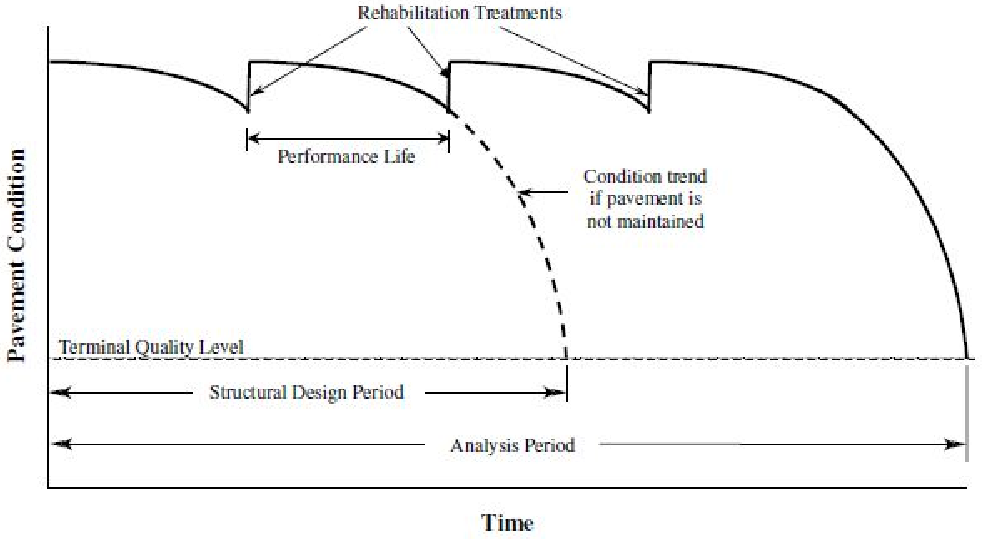


Figure 1. Example of Pavement Performance Life (WSDOT, 2010).

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. SHAs have historically planned to employ a variety of rehabilitation strategies to keep highway facilities in a functional condition. For example, Table 1 shows the Washington State Department of Transportation's (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 Volume 1 2009).

Table 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 Program (LTPP) 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 requirement 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/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 since 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 in order 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 little 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 net present value (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 utilized.

Salvage value, which at times is included as a negative cost, represents 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, and 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 percent) 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 (VOC), 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 as 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 Chapter 10 of the Project Assessment Manual in these Guidelines, can accomplish this, and is gaining use among SHAs in the US.

There is a range for the dollar value of time delay used by various Agencies. The following table is the Recommended Dollar Value used by WSDOT in 2010 dollars (WSDOT Pavement Guide Vol 1 2009).

Table 2. Recommended Dollar Values per Vehicle Hour of Delay.

Vehicle Class	Value per Vehicle Hour	
	Value	Range
Passenger Vehicles	$15.10	$13 to $17
Single Unit Trucks	$24.16	$22 to $26
Combination Trucks	$29.08	$27 to 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:

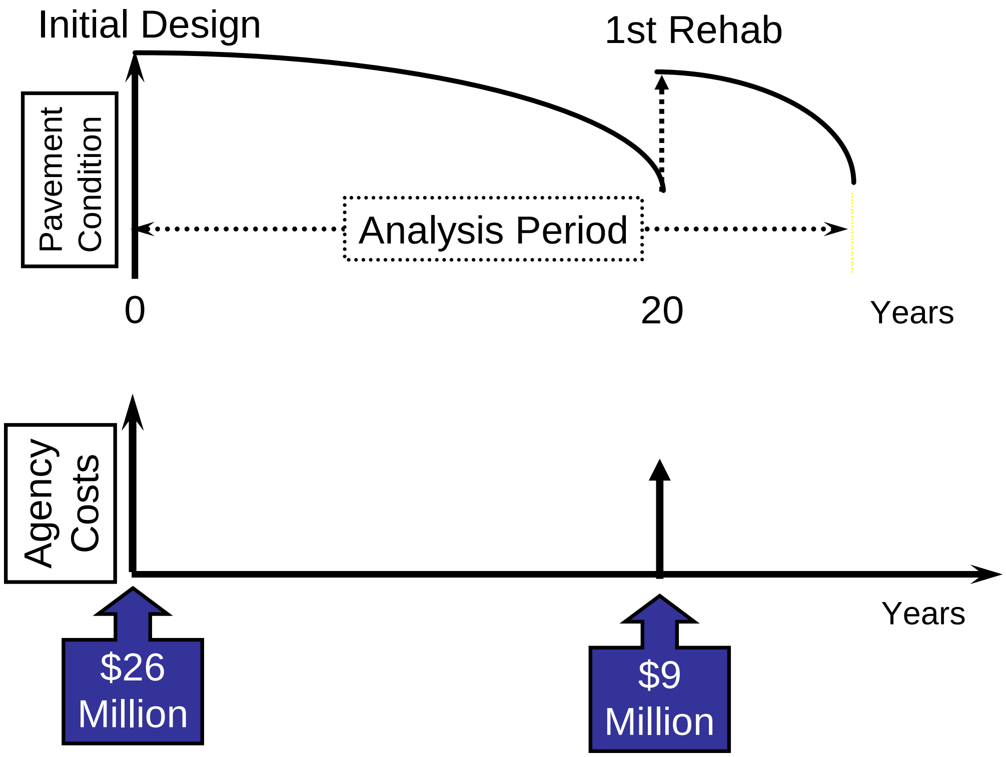
$$NPV = InitialCost + \sum_{k=1}^{N} FutureCost_k \times \left[\frac{1}{(1+i)^{n_k}} \right]$$

where:

i = Discount rate
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 percent, if the year of expenditure is 20 years, the NPV will become $30.1 million by Equation 1 as depicted by Figure 2. The NPV can be categorized in two ways: one being the agency NPV, and the other 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 + analyses have varied from a low of 2.7 percent (for 2009 and 2010) to a high of 7.9 percent (for 1982). On average, the real discount rate over a span of about 30 years is 4.3 percent.



Source: J. Walls and M.R. Smith (1998), FHWA-SA-98-0**61**

Figure 2. Net Present Value Computation Example.

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.

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 3 summarizes LCCA input variables and the general basis used to determine their values.

Table 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 the

FHWA) a SHA should consider integrating quantitative risk analysis concepts into the decision making process (Figure 3).

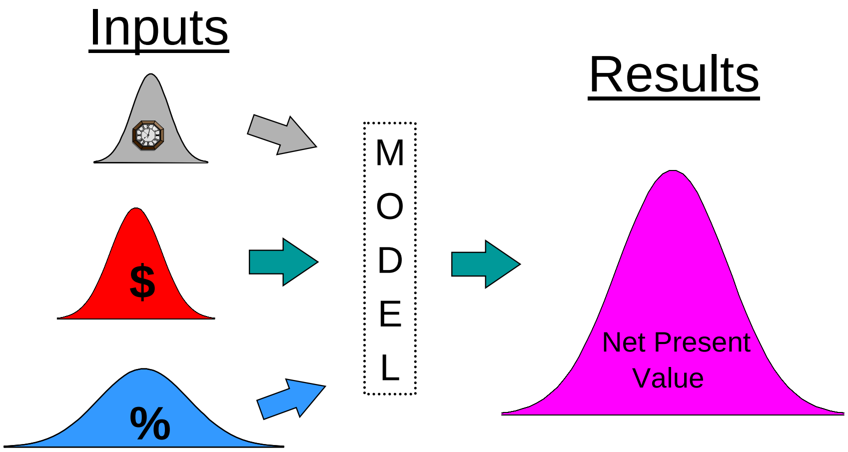


Figure 3. Risk analysis approach.

Source: J. Walls and M.R. Smith (1998), FHWA-SA-98-061

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.

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 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.
- Planning for additional lanes/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 1993 *AASHTO Guide for Design of Pavement Structures* (AASHTO, 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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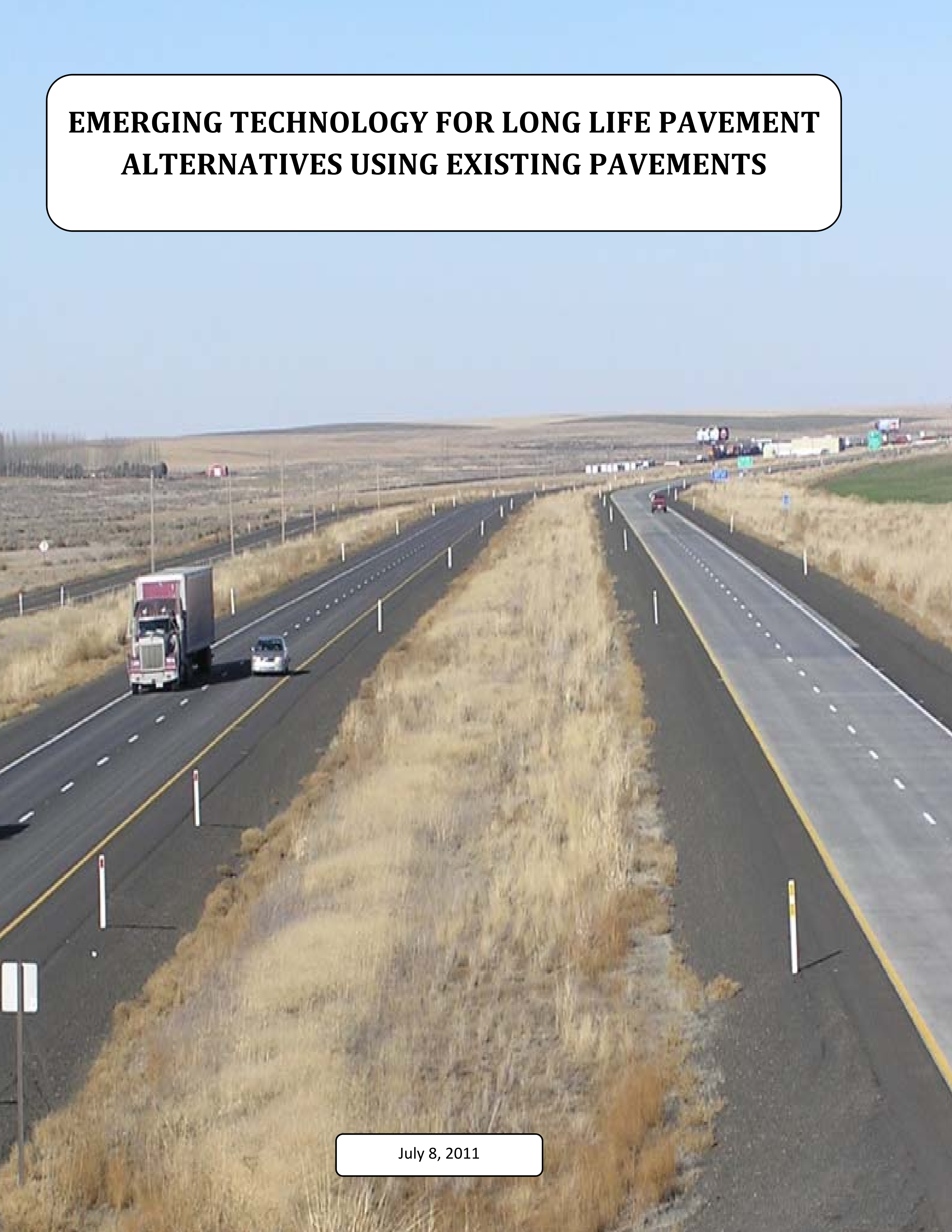
APPENDIX E-6

EMERGING TECHNOLOGY FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

EMERGING TECHNOLOGY FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS



July 8, 2011

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EMERGING TECHNOLOGY FOR LONG LIFE PAVEMENT ALTERNATIVES USING EXISTING PAVEMENTS

Introduction

There are 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:

- Rigid Pavements
    - Ultra Thin 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 become viable components for long-lasting pavement renewal.

Rigid Pavements

Ultra Thin 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 ultra thin fiber reinforced concrete overlays, which have been more widely evaluated in the US (such as the examples provided by Kuo et al, 1999). The UTCRCP approach has been extensively investigated in South Africa (Kannemeyer, et al, 2008). Figure 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 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 ESALs. Kannemeyer estimated that this type of overlay would last between 14 to 55 years—depending on average daily truck traffic (Kannemeyer 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 multi-lane 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.

Testing of thin CRCP near Heidelberg, South Africa.

HVS testing of 50 mm thin CRCP

Testing includes substantial instrumentation for in situ measurements.

HVS testing typically continues until a failure condition is reached.

Previously tested section illustrating reinforcing.

Close-up of reinforcing.

Figure 1. Thin CRCP Being Tested via the Heavy Vehicle Simulator in South Africa.

(Photos: Joe Mahoney)

A second UTCRCP project was constructed on the N1 highway northeast of Paarl (near Cape Town), South Africa. Photos captured from Google Maps are shown in Figure 2. This project currently serves 12,000 vehicles per day with 20 percent trucks (Civil Engineering, 2010) and serves as a climbing lane. The design loading is 40 million E80s over a 25 year span and has a 50 mm thickness. The mix makes use of polypropylene fibers, 5.6 mm diameter steel mesh with a spacing of 50 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 1500 psi.

Figure 2. UTCRCP on the N1 Highway near Cape Town.
(Photos: Wynand Steyn)

Potential for long-term performance: The South African experience with UTCRCP should be monitored since 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 US over the last 10 years—three well-documented projects include Texas (completed in 2001), California (completed in 2004) (Merritt et al, 2005), and Minnesota (completed in 2005) (Burnham, 2007). Subsequently, projects have been completed in Missouri (2005) and Iowa (2006) (FHWA, 2009). The purpose for 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 Iowa 4,300 ft^2. Earlier projects were documented by Merritt, et al (2000) which noted projects built in South Dakota, Japan, and Texas. 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 percent of conventional concrete pavement. An analysis done to compare 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 being: "… 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 (FHWA, 2009). The post-tensioned sections were 7 @ 250 ft., 1 @ 225 ft., and 1 at 325 ft. The panel installation rate was 25 panels per 6 hours. Figure 3 provides an aerial view of the project and Figure 4 photos taken during December 2010 to illustrate performance to date. The pavement was nine years old at the time the photographs were taken, and was exhibiting 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 one panel being delivered to the job site per truck cycle. The expansion joints were designed 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. (2 lanes plus shoulders) which used 10 ft. by 38 ft. panels ranging between 5.75 to 11.0 in. thick (the thinner sections are associated with the shoulders). The post-tensioned sections were 4 @ 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 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 a precast test section (Figure 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 post-tensioned, but rather were doweled at the transverse joints. The test section was ground about 5 months after construction with the IRI results summarized in Table 1. Load transfer efficiency measurements for the transverse joints were about 90 to 95 percent one year after construction.

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.

Figure 3. PPCP Section—Texas I-35 Frontage Road.
(Photos: Google Maps)

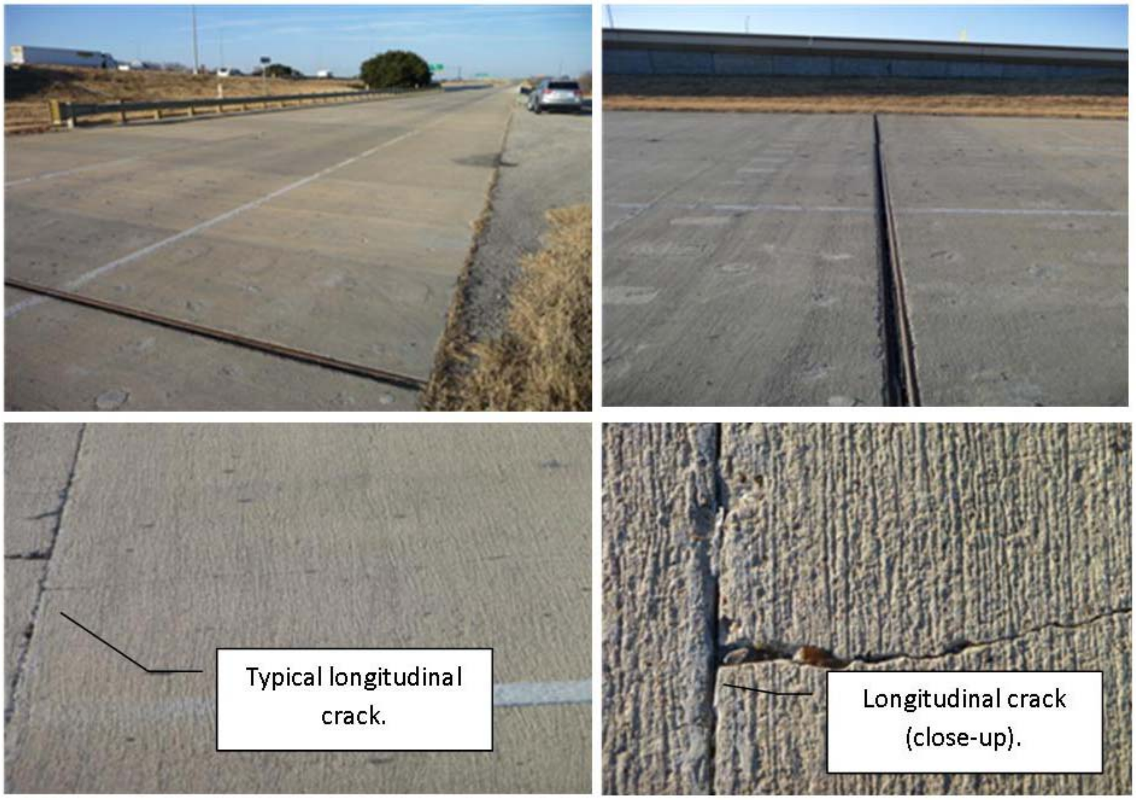

Figure 4. PPCP Section—Texas I-35 Frontage Road—Photos December 2010.
(Photos: Joe Mahoney)

Table 1. Summary of IRI Results—Precast Panels—Minnesota TH 62 (after Burnham, 2007).

Time and Activity	Average IRI (inches/mile) for both Wheelpaths
TH 62 Prior to Construction	150
New Precast Panels (Fall 2005)	140
After Grinding Panels (Fall 2005)	76
Six Months following Grinding (April 2006)	50

Figure 5. Precast Section—Minnesota TH 62.
(Photos from Burnham, 2007)

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 percent 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 1960’s.

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 USACOE testing, the cost of RMP ranged between that of traditional HMA and PCC.

More recent studies on RMP include a five year performance assessment by Battey and Whittington (2007) in Mississippi (see construction of RMP—Figure 6). Three systems were assessed for use in signalized intersections on US 72 in Corinth, Mississippi. These were:

1. RMP wearing course 2 inches thick.
2. Ultra-thin whitetopping 3 inches thick.
3. HMA overlay with PG 82-22 binder.

The comparison of these three options were 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 ultra-thin whitetopping, and RMP last. However, the assessment also showed that the RMP exhibited no rutting but was the most expensive. The ultra-thin 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 7 were taken in 2009 of a RMP that had been in service for 2 years at a truck weigh station (thus all traffic is trucks moving at a slow speed). As of 2009, no rutting or significant cracking had occurred.

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. As to whether RMP will out-perform 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.

Figure 6. Construction of RMP. Application of the grout to the open-graded HMA. (from Battey and Whittington, 2007)

Resin modified pavement at a truck weigh station on the N-3 near is near Johannesburg, South Africa.

A close-up of the resin modified cement which was placed on open-graded HMA.

Figure 7. Resin Modified Pavement. (Photos: Joe Mahoney)

Cost Comparisons

Cost comparisons for emerging technologies are a challenge on several levels due to use in experimental projects, limited production, exchange rates, etc. Further, 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 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 2). Table 3 provides performance lives and cost estimates for typical preservation treatments developed as part of the SHRP2 R26 study—which provide additional cost perspectives.

Table 2. Conventional and Emerging Technology Cost Estimates.

Traditional Paving Systems	**Typical Cost**	**Basis per Ton**	**Basis per yd^2**
Asphalt Concrete (HMA) @ 12” thick	\$64/ton	\$64/ton	\$42/yd^2
Portland Cement Concrete @ 12” thick	\$130/CY	\$64/ton	\$43/yd^2
HMA Overlay 2” thick	\$64/ton	\$64/ton	\$7/yd^2
Chip seal	--	--	\$2/yd^2
	Project Cost	Misc Info	Basis per yd^2
UTCRCP (N1 Freeway, South Africa—completed June 2010). Section contained ~ 16,000 m^2 of UTCRCP paving.	R590/m^2	~ \$85/m^2	~ \$70/yd^2
TH 62 Minnesota—Precast Panels 12’x12’x9.25” (completed June 2005). Cost/SY excludes traffic control, grinding, and striping. Cost/SY does include removal of pre-existing 8” JRCP. Contained 288 yd^2 of precast panels.	--	Test section was small ~ 288 yd^2	\$575/yd^2
Caltrans Precast Post-Tensioned Test Section—constructed 2004.	--	Test section was ~ 1,000 yd^2	\$224/yd^2

Note: Per SY basis based on equal thickness of HMA and PCC. Only the material costs were considered. Assumed densities are 145 lb/ft^3 for HMA and 150 lb/ft^3 for PCC.

Table 3. Expected Performance and Costs Associated with A Selection of Pavement Preservation Treatments (after Peshkin et al, 2010).

Pavement Type	**Expected Treatment Performance (years)**	**Estimated Unit Cost**
Existing HMA Surfaced Pavement		
Crack Filling	2 to 4	\$0.10 to 1.20/ft
Crack Sealing	3 to 8	\$0.75 to 1.50/ft
Slurry Seal	3 to 5	\$0.75 to 1.00/yd^2
Chip Seal—Single Course	3 to 7	\$1.50 to 4.00/yd^2
Thin HMA Overlay (Dense Graded; 0.875 to 1.5 in. thick)	5 to 12	\$3.00 to 6.00/yd^2
Profile Milling	2 to 5	\$0.35 to 0.75/yd^2
Ultra-Thin Whitetopping (2 to 4 in. thick)	Not Available	\$15.00 to 25.00/yd^2
Existing PCC Surfaced Pavement		
Joint Resealing	2 to 8	\$1.00 to 2.50/ft
Crack Sealing	4 to 7	\$0.75 to 2.00/ft
Diamond Grinding	8 to 15	\$1.75 to 5.50/yd^2
Partial-Depth Concrete Patching	5 to 15	\$75 to 150/yd^2 (based on patched area)
Full-Depth Concrete Patching	5 to 15	\$75 to 150/yd^2 (based on patched area)
Dowel Bar Retrofit	10 to 15	\$25.00 to 35.00/bar
Thin HMA Overlay (0.875 to 1.5 in. thick)	6 to 10	\$3.00 to 6.00/yd^2

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).

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