SHRP 2 Reliability Project R23

Guide to Using Existing Pavement in Place and Achieving Long Life

Addendum 1

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

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

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Guide to Using Existing Pavement in Place and Achieving Long Life

Addendum 1

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Introduction

Phase 3 of this project was directed primarily to modify the guidelines to provide design options for 30 to 50 year design lives instead of the 50 year design options originally developed. One of the principal concerns noted by the state highway agencies working with the R23 team was that they designed their projects for 20 to 40 year design periods, so the guidelines based on 50 years would not be used. As such, modifying the guidelines for 30 to 50 year design periods would be more useful to most agencies.

In addition to modifying the guidelines to provide 30 to 50 year design options, Phase 3 included the following tasks:

- Develop training modules and incorporate into the interactive software
- Provide web hosting and support
- Provide outreach support, including preparation of material for meetings and webinars

As this work started, the contractor was asked to consider a possible Phase 4 activity that would add modular and composite pavement designs based on the products from R05 and R21 projects, respectively. As part of Phase 4, the project team proposed the conversion of the current R23 interactive program from a program based on Adobe Air and Flash to a more-web-compliant HTML5 platform. Since it was much more efficient to convert the existing program to the HTML5 platform as part of Phase 3 (rather than doing the modifications in Phase 4), the tasks in Phase 3 were changed to accommodate the program conversion in Phase 3.

To make the program conversions within the Phase 3 budget, the contractor proposed that the tasks for development of the training module and web hosting be delayed and added to Phase 4, if funding permitted.

Scope of Work

The revised Phase 3 activities were modified to include the following scope of work.

Task 12. Revise guidelines to provide guidance for 30 to 50 years of service life

The guidelines will be modified to consider approaches that were eliminated in Phase 1 and 2 because they would likely provide only 30 to 40 years of service. These approaches would show up in the guidance only when an agency input a 30 to 35 year design period. If an agency specified a 40 to 50 year design period, then those approaches would not be included in the recommended approaches. The guidance and specifications for long-life pavements would not change because they represent the best practices. To account for the 30 to 40 year design life window, the following actions were proposed:

- Revise decision tables to include bonded portland cement concrete (PCC) overlays and asphalt concrete (AC) overlays of continuously reinforced concrete pavement (CRCP) and add design thickness estimate tables to match added approaches
- Circulate revised decision tables to agencies and Industry for review comments
- 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
- Prepare addendum to final report to document revisions

Task 13. Update application to incorporate additional service life options (modified to account for new program and added logic for height restrictions and added lanes)

A new web-based program was developed in Task 14 based on the current program functions and logic. The new web-based program was revised to incorporate the added decision process from Task 12, quality control tested, and packaged for web-based delivery. This required the following actions:

- Design business logic and interface components to reflect Task 12
- Add new logic to provide more detailed recommendations for height restrictions and added lanes
- Develop a beta version of the application
- Internally test application functionality, identify bugs, and fix issues
- Develop static website template for hosting application
- Package documentation for application
- Package for end-user release and distribution via web-based application
- Provide hosting and support for web-based application during this task

Task 14. Convert existing application to web-based, standards-compliant format

The following actions were required for the program developers to modify the existing program into a standards-compliant HTML5/JavaScript platform and notation:

- Develop database, interface, and security elements for web-based application to provide users the ability to load, save, share, and compare various individual application results
- Convert Flash-based business logic and user interfaces into standards-compliant HTML5/JavaScript platform and notation
- Internally test application functionality, identify bugs, and fix issues
- Package application for testing and external review by NCE
- Address feedback from NCE review
- Update and package application for delivery via web-based platform
- Deploy to Pavia-hosted production server for unlimited access by end users during hosting and support period

Task 15. Application hosting and support

This task was dropped for Phase 3.

Task 16. Develop and provide workshops, training sessions, and presentations (unchanged)

To fully implement the SHRP 2 guidelines for long-life renewal of existing pavement structures will require a number of contacts with potential users across the country. This may consist of workshop presentations either at an agency location or via the web (possibly as a TRB webinar). It would also be helpful to make presentations either at conferences or at meetings, such as the annual TRB meeting, or more agency-involved meetings such as the Joint Technical Committee on Pavements of the American Association of State Highway and Transportation Officials (AASHTO), or regional meetings at which highway agencies would be present.

Summary of Work Accomplished

The following describes in more detail the work that was accomplished by task in Phase 3. Tasks 12 and 14 were conducted concurrently with the R23 team working on Task 12 and Pavia working on the program conversion in Task 14. In Task 13, the revised guidelines for 30 to 50 year design lives were added to the web-based program developed in Task 14.

Task 12 Modifications for 30 to 50 year design life

The primary effort for Task 12 was to revise the Phase 3 decision tables to include bonded PCC overlays and AC overlays of CRCP as well as add design thickness estimate tables to support the added approaches. In addition, the design thickness table for the unbonded PCC overlay table was expanded to account for a range in subgrade stiffness. The expanded design thickness tables were developed based primarily on DARWin-ME (AASHTO Pavement ME) runs with consideration of thickness limits noted in the final report for long-life pavements.

As the R23 team started to make test runs with the revised program, it became clear that the form of the decision tables from Task 2 did not expand well to meet the requirements for Phase 3. It was also anticipated that in Phase 4 the program logic was going to be expanded to include added elements from the SHRP 2 R05 and R21 projects, which would add more complexity to the existing decision tables. The form of the decision tables was modified to make them easier to understand and to better fit the programming logic used in the new program. The modified decision tables and design thickness tables are included in Appendix A of this report.

In addition the Rigid Pavement Best Practices document was revised to include the added treatments. The revised Rigid Pavement Best Practices is included in Appendix B of this report. Those revisions reflect the information contained in the Phase 1 report as well as additional information that has become available since that report was prepared. Specifically the R23 team revisited the most recent information available from the Texas Department of Transportation (TxDOT) on bonded PCC overlays of CRCP, as well as on test installations of bonded PCC overlays over hot-mix asphalt (HMA) pavements at MnRoad and other states.

Members of the R23 team met with personnel from the Minnesota DOT (MnDOT) and visited the MnRoad test track to review its performance data and view those test sections that were still in place. The results from that tour are contained at the end of Appendix B. The conclusions based on that information led the R23 team to conclude that it is still a little too early to include bonded PCC overlays over HMA as a viable approach in the guidelines when a minimum of 30 to 35 years of service is expected. Table 1 shows the service life for the bonded PCC overlays placed at the MnRoad test track.

Table 1. 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

Note: TWT = thin whitetopping; UTW = ultra-thin whitetopping.

Of the three unbonded PCC test sections that were built on the test track, only one was still in place in 2012. It has now been in service for 15 years and is in reasonable condition but has experienced enough faulting to require diamond grinding to restore the ride. The other two were in service only 13 years before they were taken out of service. It is unlikely that any of these test sections would provide 20 to 25 years of service. MnDOT is currently designing its bonded PCC overlays over HMA for a 20-year design period that fits the service life found at the test track. However, it is questionable at this time if that design option will provide 30 to 40 years of service and thus was not included in the guidelines for long-life pavement renewal.

Bonded CRCP overlays over CRCP appears to provide a longer service life particularly at the higher thicknesses. TxDOT built a number of 4-in. bonded overlays in the Houston area. Those bonded CRCP overlays have provided 20 to 25 years of service, but most are being replaced before 25 years. The photograph in Figure 1 was taken on Interstate 610 in the Houston area during Phase 1 of this project. This section of I-610 was being overlaid with a 12-in. unbonded PCC overlay. That particular section of pavement was about 24 years old.

Figure 1. Photo of 24-year-old bonded PCC overlay on I-610 in Houston.

The Houston area does have a number of 6-in.-thick bonded CRCP overlays that appear to be providing much better performance at 10 to 15 years of service. Consequently, bonded CRCP overlays of CRCP were added to the guidelines for structurally sound pavements starting at a minimum thickness of 6 inches. The details can be found in the revised Rigid Pavement Best Practices document (Appendix B) and in the trip reports from Phase 1 of this project.

To support these changes, the R23 decision tables and design tables were also modified to include bonded PCC overlays of CRCP pavement, and the unbonded PCC overlays were expanded to include subgrade support and a range of HMA thickness that can be used for elevation adjustments when height restrictions apply. The table values were developed based on numerous DARWin-ME runs for bonded PCC overlays and unbonded PCC overlays by using the same design input used in Phase 2 of this study. As noted earlier, the resulting decision and design tables are appended to this report as Appendix A.

Chapter 11 in the Project Assessment Manual was also updated to include updated information on new web-based programs that are now available for life-cycle assessment. The revised Project Assessment Manual is provided in Appendix C of this report.

Task 14. Convert existing application to web-based, standards-compliant format

The R23 scoping tool application was originally designed for self-contained delivery via CD (or other disconnected media) to minimize the upfront cost, but ultimately this limited its broader function and use. To increase use and adoption and provide additional functionality, the project team converted the existing Flash-based tool to a web-based application during Phase 3.

In order to provide access on the broadest number of devices and browsers, the team evaluated several web platforms and selected a delivery platform that consisted of the elements seen in Figure 2.

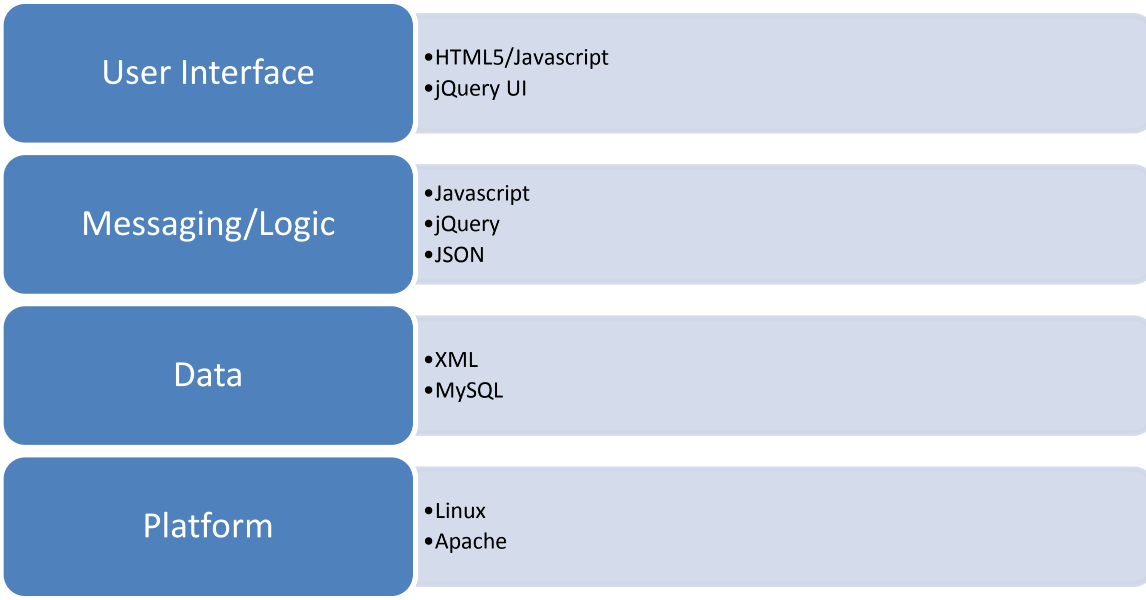

Figure 2. Phase 3 web-based platform elements.

Application Design

Following selection of the delivery platform, the project team designed the system architecture and data elements to achieve the project objectives on the new platform. This involved designing a new data structure that can be passed back and forth across the web efficiently and be associated with user accounts. To do so, the team evaluated using either structured data tables within a MySQL database or utilizing a JSON data structure that would be stored in the database to store the user data. Following discussions with the team and upon learning that input and output requirements may evolve over time, the team determined that a JSON structure would provide the most flexibility and least maintenance to implement as enhancements came over time.

Following development of the data structures to store the inputs and outputs in the database, the team began developing interface designs for the updated application to take advantage of the web-based delivery mechanism. The interface itself was designed to allow for responsive movement within a web browser to ensure optimal viewing experience for end users regardless of their device. To limit scope, the team determined the most likely screen resolutions to design for and then applied those to the interface elements within the application. Following feedback from the research team on interface designs, a final design was selected and built out for each screen as seen in Figure 3.

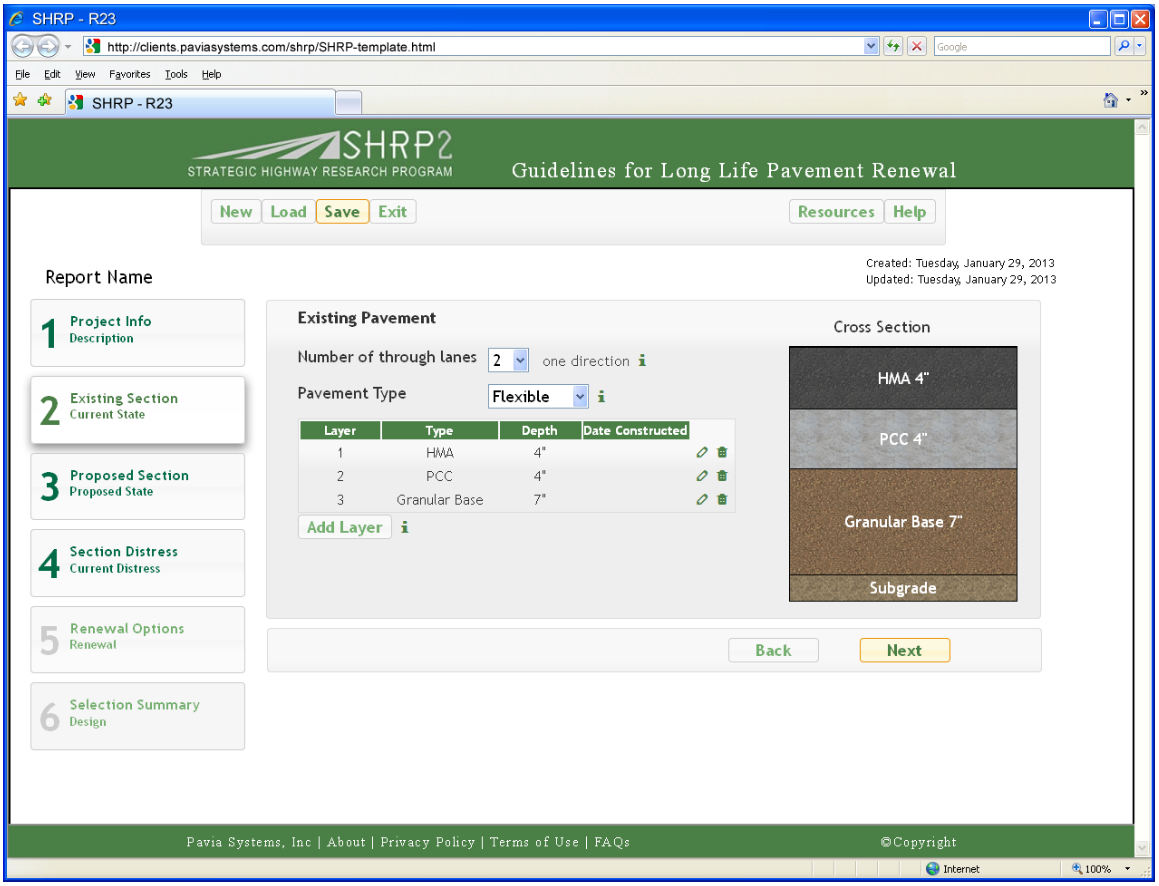

Figure 3. Screenshot of the design of the R23 scoping tool interface.

Application Development

In addition to the design of the interface the project team had to convert the input validation logic for the interface from the existing application to perform on the new platform. This included ensuring that proper input values, ranges, and characters were used by end users. To provide feedback to the user on any incorrect inputs, an error messaging system was added to the interface to provide responsive immediate feedback to the user.

Once the data and interface elements were complete, the team began converting the business logic and functions within the existing application from ActionScript (a native Adobe Flash language) to JavaScript. To ensure that functions performed similarly in both environments the team conducted unit testing for each function whereby inputs and outputs on the old and new platform were compared. When discrepancies arose, further debugging and modification were made on the new JavaScript code base.

New features were then developed by the project team to tie in user accounts and security infrastructure to provide storage and retrieval functions to the user. In addition, the project team developed a new web-based report and printing function. Following conversion of the code, development of the linkages between the database, user interface, and business logic elements was performed.

Application Testing and Deployment
At this stage, the application was ready for initial testing by the development team and comparison against the soon-to-be-deprecated Flash application. The team conducted numerous trials using consistent inputs between the applications and performed debugging and code modifications as necessary on the new platform to ensure consistency.

Following testing, the development team published the application to a private, secure web location so the project team could begin to review the application and provide feedback. The development team then tracked all feedback and, once the review period was concluded, acted on the feedback. Prior to packaging the application for release, the team performed a broad series of interface tests on various web browsers and platforms to ensure consistent delivery and interaction with the application whether using browsers such as Firefox, Internet Explorer, or Chrome. There were numerous style and interface tweaks that were made during this step to ensure consistency.

Finally, once final testing was complete, the team published to a live site that was accessible by SHRP for review. Following a review period, the project team made modifications to the application and underlying logic as required to address the feedback.

Task 13. Update application to incorporate additional service life options
The updated decision logic from Task 12 was incorporated into the interactive web-based program developed in Task 14. The program and new decision and design tables were submitted to the same agencies that reviewed and commented on the products from Phase 2:

- Michigan Department of Transportation (Michael Eacker)
- Minnesota Department of Transportation (Shongtao Dai)
- Missouri Department of Transportation (John Donahue, William Stone)
- Texas Department of Transportation (Magdy Mikhail)
- Virginia Department of Transportation (Trenton Clark, Alex Teklu)
- Washington State Department of Transportation (Jeff Uhlmeyer)

The R23 team also conducted extensive testing of the interactive program and all errors or omissions were addressed in the interactive program. The interactive program rePave was made available to all in mid-July 2013 on a new website established for this product (http://www.pavementrenewal.org/).

The full logic used in the new program is very close to that in Phase 2 with the addition of the bonded PCC overlays and more consistent action description terms and warnings.

Figures 4 through 13 show some typical screen shots from the new interactive program rePave.

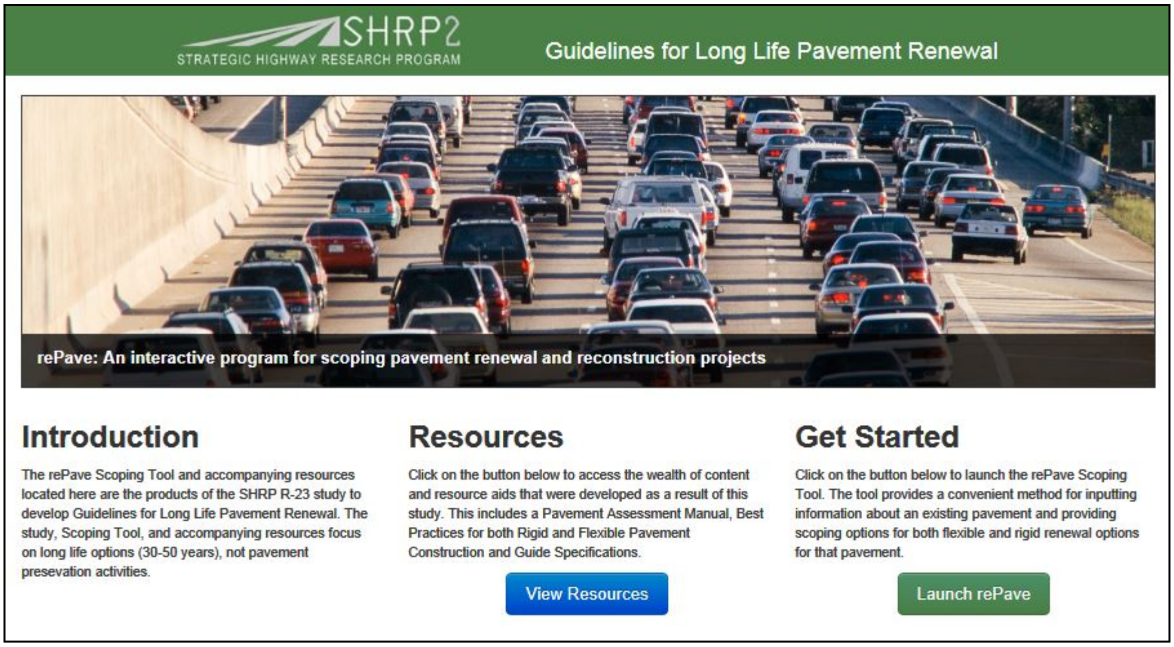

Figure 4. Opening page for rePave—Guidelines for Long-Life Pavement Renewal.

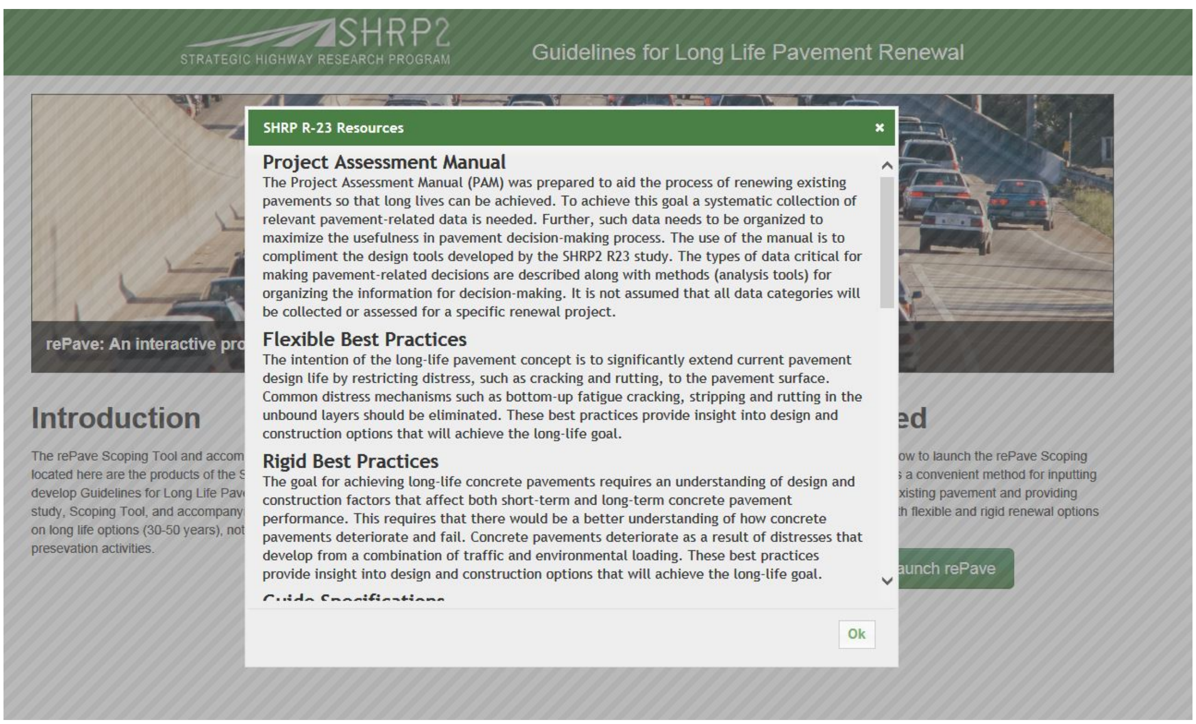

Figure 5. Resources tab for downloading resource documents.

In Step 1, the interactive program saves specific projects for the user if desired. To save a file the user's name, e-mail address, and password are required. The program will send a file name to the user to access the file.

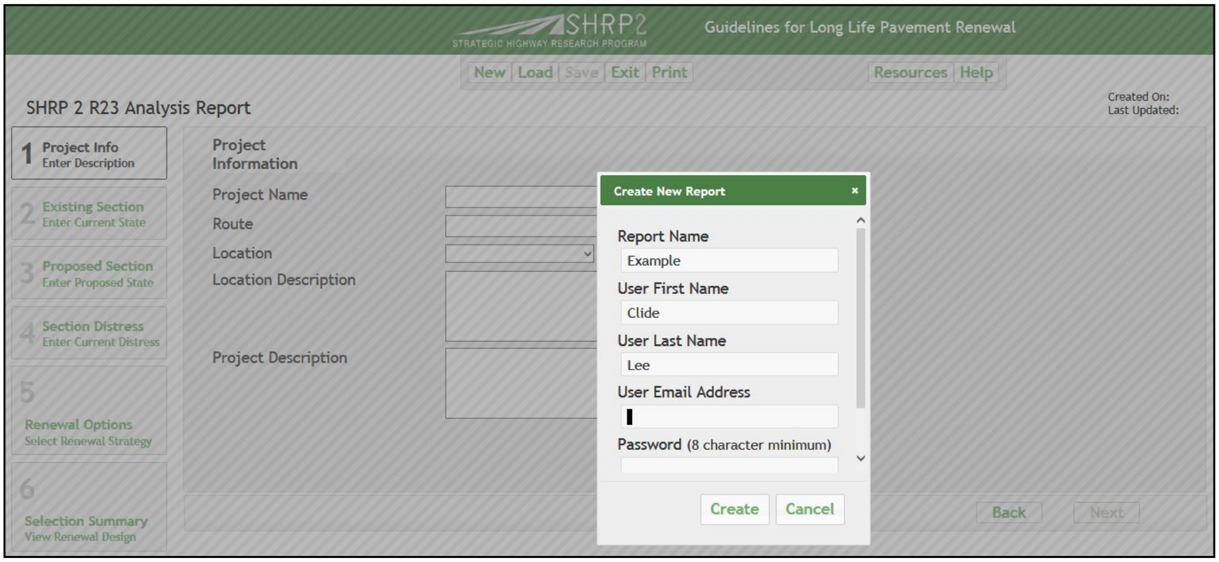

Figure 6. Report Information for web-based project file.

In Step 2, the program will ask the user to list the pavement layers in the existing pavement. The following example shows all the pavement layers when known.

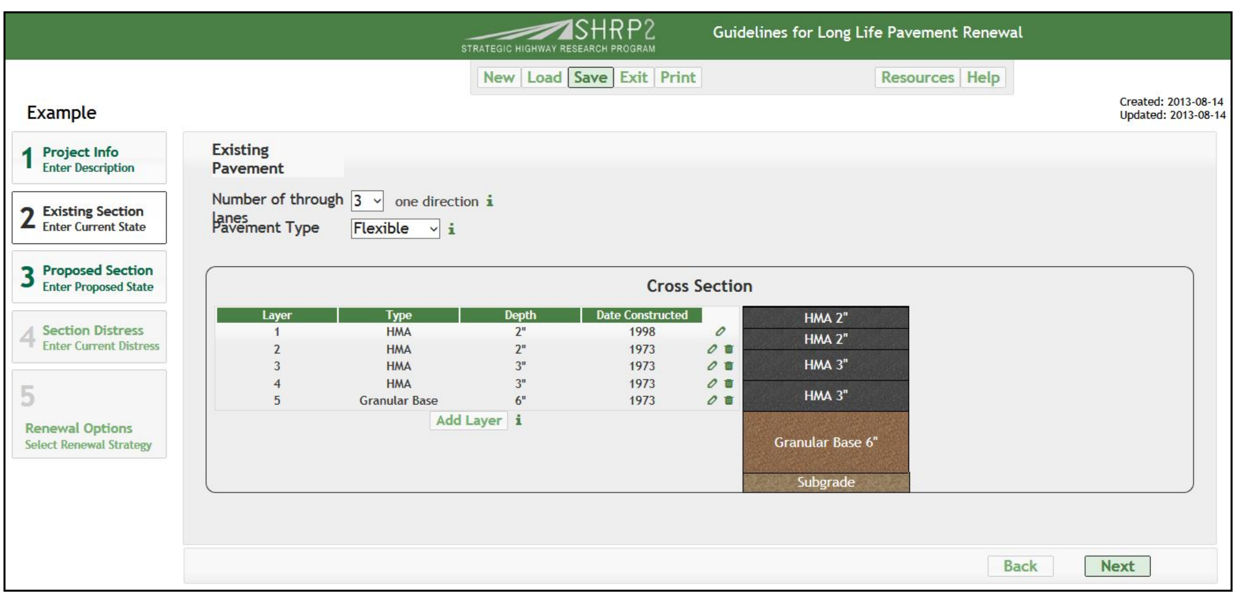

Figure 7. Existing pavement layer information.

Basic design information is then added in Step 3.

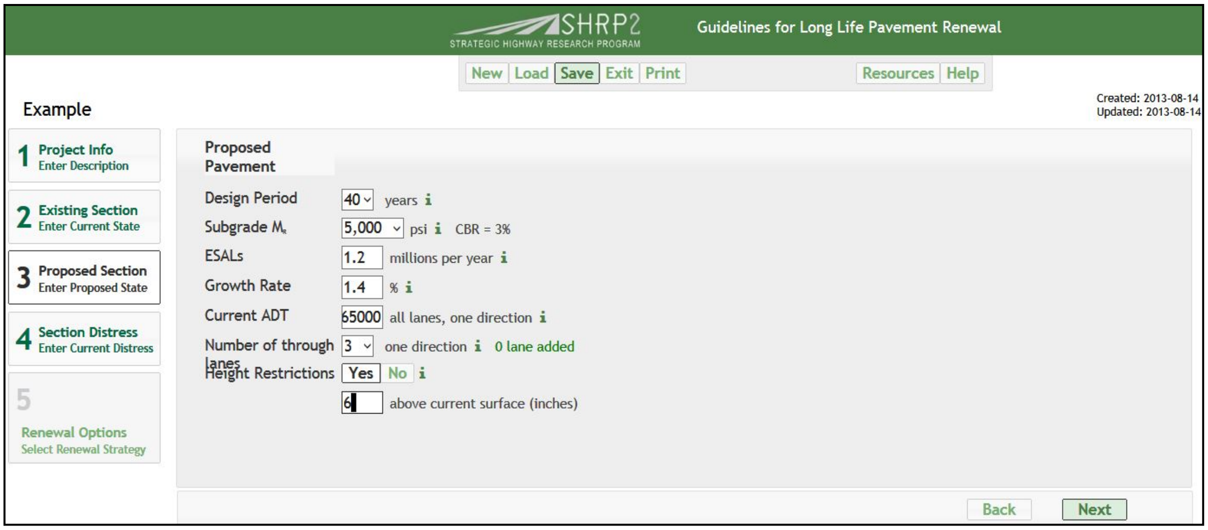

Figure 8. Design information for pavement renewal design.

In Step 4, pavement distresses are input for a specific section of highway. If there is much variability in pavement section and condition along the roadway, then individual designs will need to be run for each change in roadway section and significant change in pavement condition.

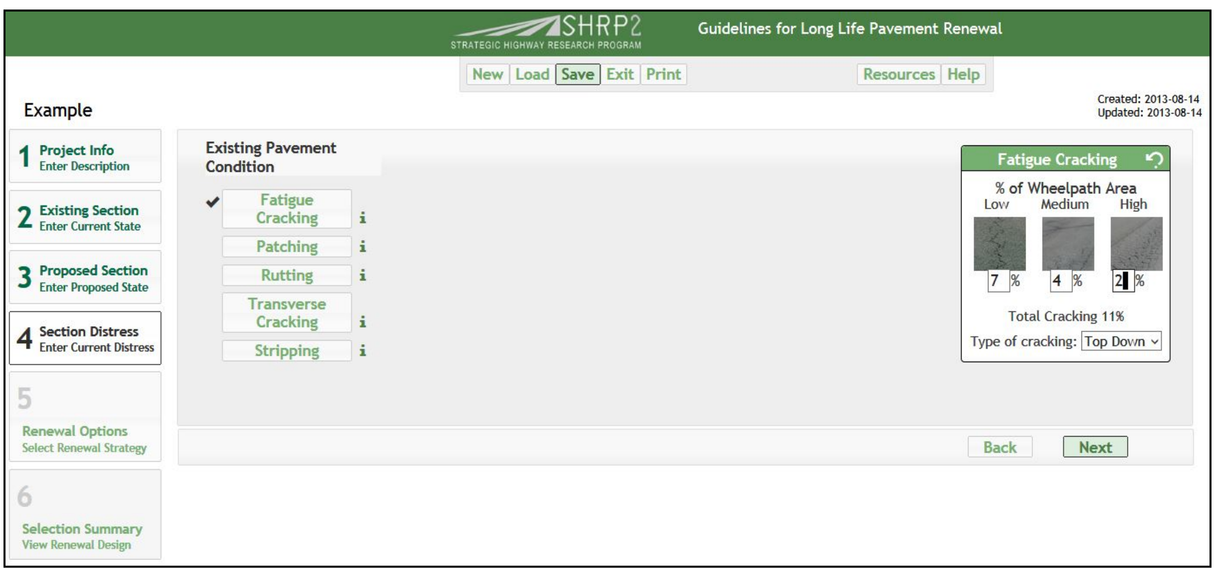

Figure 9. The amount of different pavement distress is input.

Note that for the different pavement types there are different critical pavement distress categories and quantity of distress that need to be entered into the program. When a category of distress is checked, then a screen will appear on the right of the monitor with the category and distress severity shown when appropriate. In other cases only the category will be shown. For flexible pavements there can be stripping evident in specific HMA layers so those layers should also be identified. Figure 10 shows how that is identified in the program.

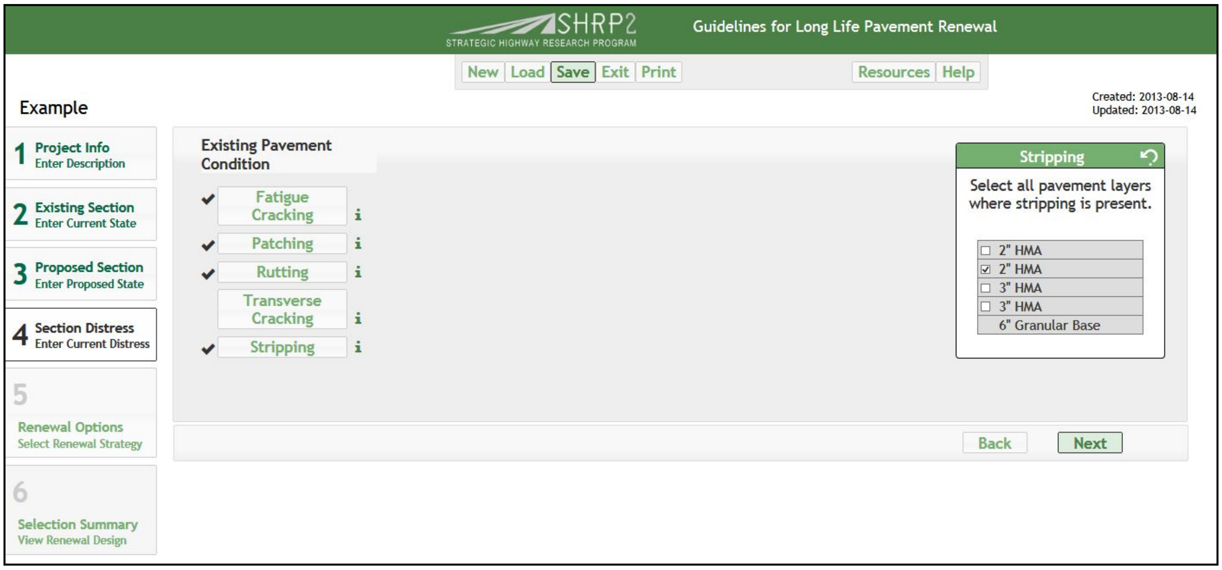

Figure 10. Distress identification where stripping can be shown in a specific layer.

The program logic never covers a problem, so all stripped HMA layers are always removed to ensure long-life performance. An example where stripping is removed will be shown later in Figure 12.

Once the data are accepted, then a list of renewal approaches will be listed based on the user's choice of rigid or flexible approaches. If desired, all approaches can also be shown.

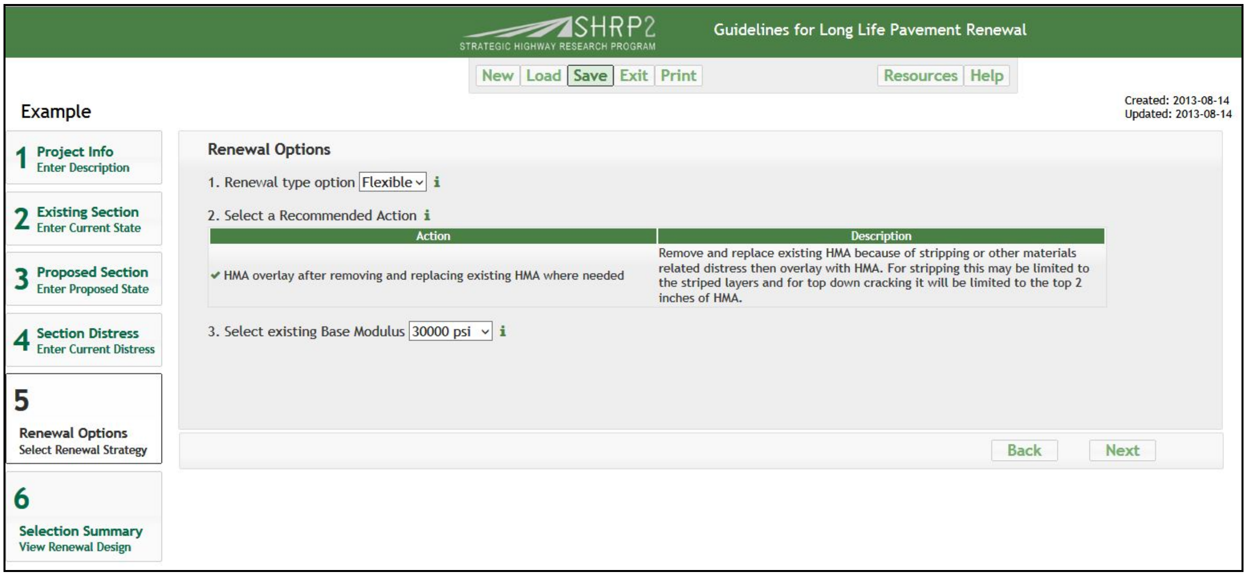

Figure 11. Renewal options for the pavement type and information provided.

In Step 5 the treatment or treatments that could be considered for either rigid or flexible options are shown, and the choice is selected by the user.

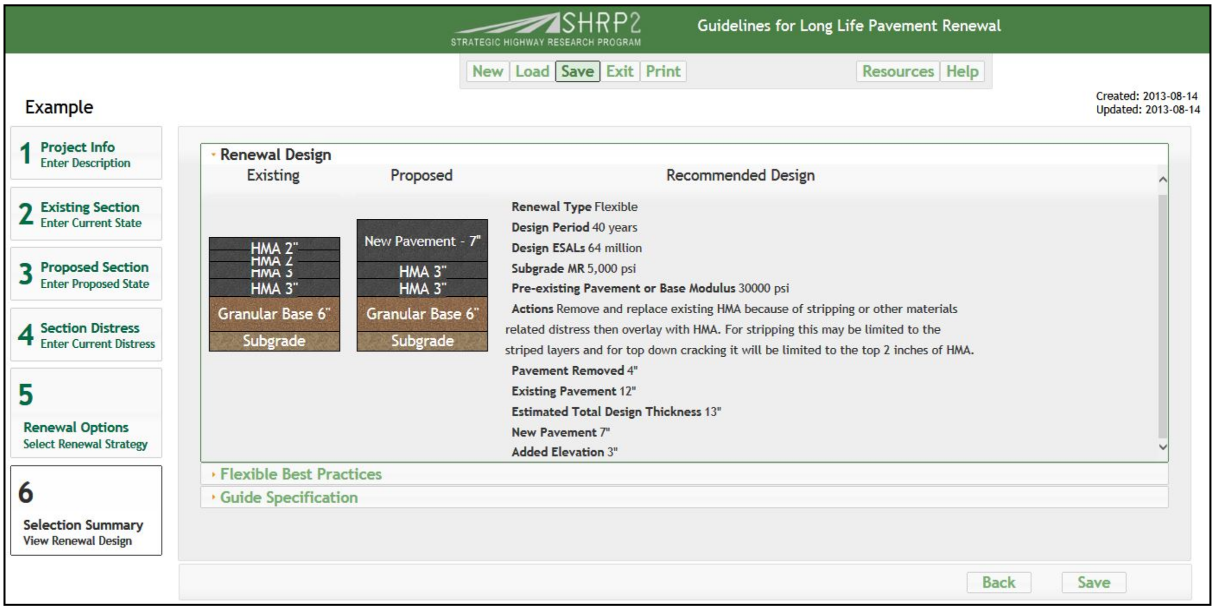

Figure 12. Renewal design is shown for flexible option.

In Step 6 the design summary is shown. Note in this particular case in which stripping is shown in the second layer then both the current wearing course and the second layer are removed before placing the required overlay.

The user may then go back in the program and choose a different pavement type. In Figure 13, the rigid pavement option was also selected, and the design summary is shown for a rigid design.

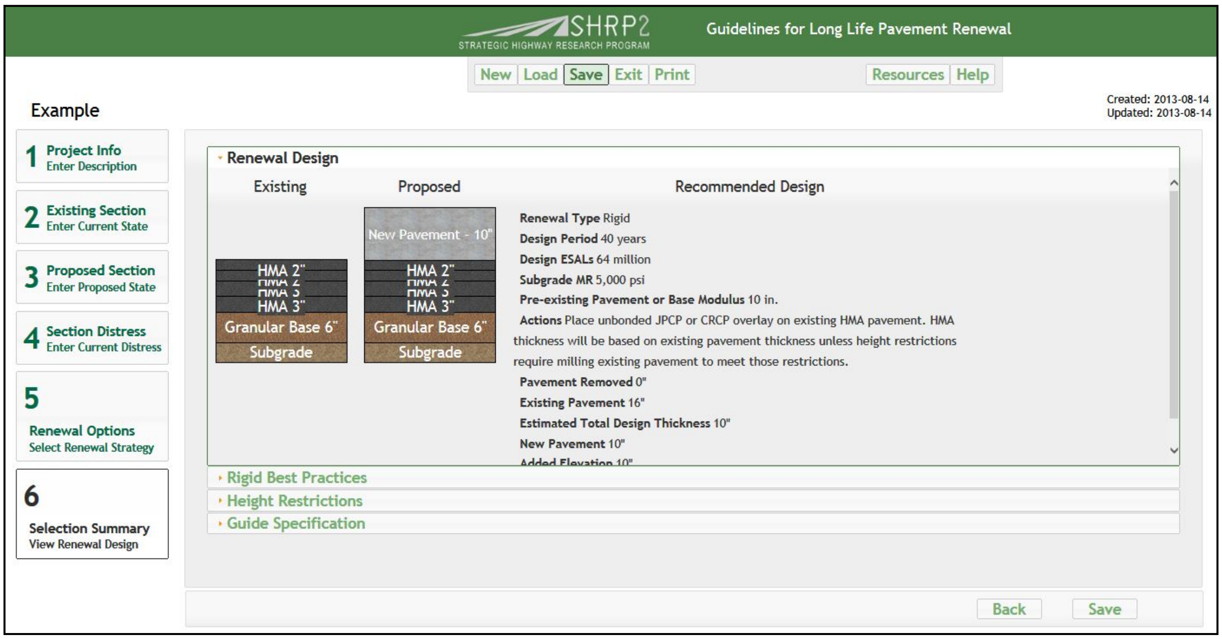

Figure 13. Example design when a rigid pavement option is selected.

In addition to the renewal design summary, the screen also provides direct links to the best practices and guide specifications, as can be seen on the folder tabs below the design summary. The project information can be saved at this point, and the print command allows one to print the summary page on paper or to a separate file as a PDF document.

The interactive program also contains a feedback form for users who have experienced a problem or want to make comments on the program.

Task 16. Develop and provide workshops, training sessions, and presentations

Task 16 provided funding for preparing and making presentations at various venues. A number of presentations were prepared and presented as part of this task. The more notable were the following:

- Technical Exchange, Beijing, China, June 16–25, 2011
- TRB Annual Meeting Workshop Presentation, January 22, 2012

- International Conference on Long-Life Concrete Pavements, September 18–21, 2012, Seattle Washington
- SHRP 2 Oversight Committee Poster Presentation, Washington, D.C., November 27–29, 2012

Also included were a number of presentations at TRB committee meetings at the 2012 and 2013 annual meetings.

Conclusions

The Phase 3 tasks were originally set up to modify the Pavement Renewal Guidelines so that they could be used by a wider range of agencies who would design pavements for 30 to 40 years of service but not for 50 years of service. The guidelines were expanded to provide design lives of 30 to 50 years. As the R23 team started to work on Task 12, SHRP 2 staff approached NCE about including the products from R05 (modular pavements) and R21 (composite pavements) in the guidelines. To support this expansion of the guidelines to a full web-based application, it was logical to move the program over from a flash-based program meant for CD delivery to an HTML5 platform for web-based delivery. Phase 3 was modified to include this conversion as a new Task 14 so that the revised guidelines would be available on a platform designed for web-based delivery. With that work completed in Phase 3, the expansion to include modular and composite pavements will be much cleaner in Phase 4. There is also the added benefit of having the Pavement Renewal Guidelines available for release and implementation in a fully web-compliant platform at the conclusion of Phase 3. The Pavement Renewal Guidelines are now available to all users at http://www.pavementrenewal.org/.

Reference

Burnham, T. 2008. Thin and Ultra-Thin Concrete Overlays. MnRoad Lessons Learned. Office of Materials, Minnesota Department of Transportation, February 8.

Appendix A

Scoping Methodology

A new version of the scoping methodology was developed. See the SHRP 2 publication *Guide to Using Existing Pavement in Place and Achieving Long Life: Addendum 2* at http://www.trb.org/Main/Blurbs/171517.aspx.

APPENDIX B

Revised Rigid Pavement Best Practices

Introduction

Long-life pavements as considered in this document are pavement sections designed and built to last 30 to 50 years or longer without requiring major structural rehabilitation or reconstruction. Periodic surface renewal activities are expected over the 30- to 50-year duration. The study primarily focused on the longer service lives, but feedback, largely from state departments of transportation (DOTs), recommended a lower threshold of 30 years. Long-lasting concrete pavements are readily achievable, as evidenced by the number of pavements in excess of 30 to 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. The longer service lives are desirable in providing lower life cycle costs as well as reduced user and environmental impacts. A more detailed working definition as suggested by Tayabji and Lim (2007) of long-life concrete pavement includes the following:

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

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

Photos of completed and under-construction jointed plain concrete pavements (JPCPs) and continuously reinforced concrete pavements (CRCPs) are shown in Figure B.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 B.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). CRCP pavements have moved toward thicker slabs as well—which were commonly about 8 in. thick during the 1960s and have increased to 11 to 13 in. today.

JPCP constructed on HMA base

CRCP constructed on HMA base

Figure B.1. Completed and under-construction JPCP and CRCP. (Photos: J. Mahoney)

Table B.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
Materials-related distress (JPCP and CRCP)	None
Punchouts, number/mi (CRCP)	12–16

Note: IRI = international roughness index.

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 have been described by Rasmussen and Rozycki (2004). Even though the industry has changed how concrete overlays are described, these original terms are still widely used and are described below:

- Unbonded concrete overlays. A portland cement concrete (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 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 B.2.

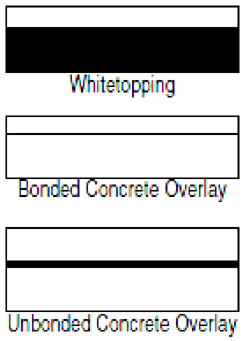

Figure B.2. Types of concrete overlays—earlier descriptions. (Rasmussen and Rozycki, 2004)

More recent concrete overlay terminology has been described by Harrington (2008). The new definitions provide a simplified description of concrete overlays as shown in Figure B.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.

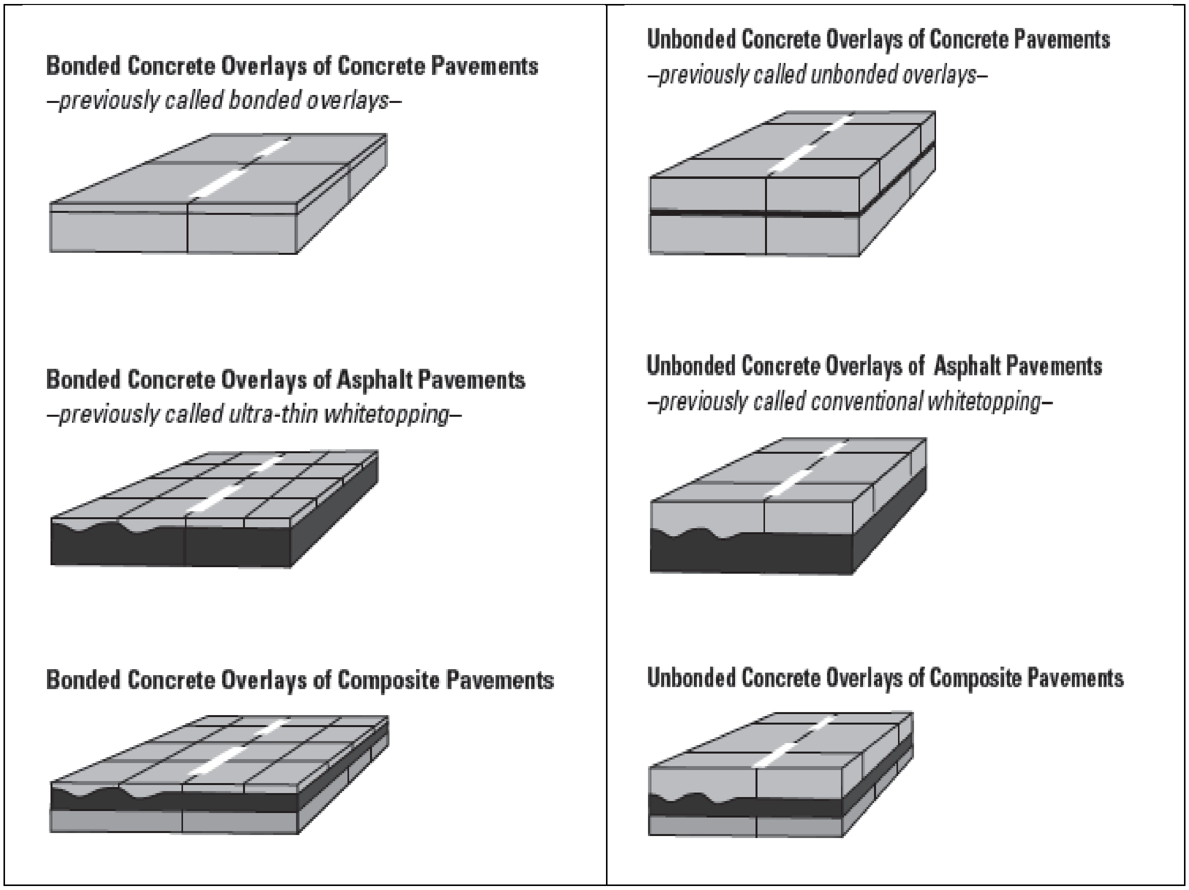

Figure B.3. Types of concrete overlays—more recent descriptions. (Harrington, 2008)

Rigid Renewal Strategies

The renewal strategies examined for long life (≥30 years) using existing pavements as described in this best practices document are as follows:

- Bonded concrete overlays of existing HMA or CRCP pavements
- Unbonded concrete overlays of existing HMA or concrete pavements

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 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 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 30 or more years. This conclusion is based on several sets of information that includes, but is not limited to, (1) existing pavement design criteria, (2) state DOT criteria and field projects, (3) Long-Term Pavement Performance (LTPP) program results, (4) state field visits, and (5) the National Concrete Pavement Technology Center (Harrington, 2008).

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 LTPP program. These results, as relevant to long-life rigid renewal best practices, are summarized in the Supplemental Documentation at the end of this appendix.

The pavement performance information presented in these best practices is largely based on field experiments and projects. Thus, a wide range of traffic conditions are not available; however, the thickness design information available in the study developed "app" does reflect the use of formal design processes and a wide range of traffic conditions.

From the information summarized, the performance of concrete overlays over existing HMA or concrete is a function of slab thickness and design details such as joints and remaining HMA thickness, condition of the existing concrete, aggregate type, reinforcing, etc. Given Interstate types of traffic [~1 million equivalent single-axle loads (ESALs) per year], Table B.2 shows typical pavement lives that can be expected for various slab thicknesses along with bonding condition and joint details over existing HMA. The expected lives shown are tentative and reflect an extrapolation from the field data reviewed.

Based on Texas DOT (TxDOT) experience, CRCP overlays over existing CRCP can achieve a 20-year life for a range of thicknesses (those reviewed ranged from a minimum of 2 in. up to 6.5 in.). TxDOT has accumulated substantial experience on both design and construction practices for this type of overlay. The thinnest CRCP overlays appear to address functional issues with the existing pavement. The most commonly applied CRCP overlay found in the TxDOT literature is typically 4 in. thick; however, more recent designs in the Houston, Texas, area have been in the range of 6 to 8 in. thick (R23 Houston Trip Report).

Only unbonded concrete overlays ≥8 in. thick meet the threshold for long life as defined in this study. This assumes that thicker bonded overlays (≥7 in. thick) are rarely applied.

Table B.2. Bonded and Unbonded JPCP Concrete Overlays over Existing HMA with 1 Million ESALs per Year with Sufficient Existing HMA Thickness

Slab Thickness (in.)	Bonded or Unbonded	Joints	Dowels?	Expected Life (years)
3	Bonded	5 ft by 6 ft	No	5
4	Bonded	5 ft by 6 ft	No	5 to 10
5	Bonded	5 ft by 6 ft	No	10 to 15
6	Bonded	6 ft by 6 ft	No	15 to 20
7	Bonded	6 ft by 6 ft	Optional	20 to 25
8	Unbonded	12 ft by 12 ft	Yes	25 to 30
9	Unbonded	15 ft by 12 ft	Yes	30 to 35

Note: Additional information about this table is contained in the Supplemental Documentation at the end of this appendix.

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 30 to 50 years.

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

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

- Foundation support [uniformity, volumetric stability (including stabilizing treatments)]
- Drainage design (moisture collection/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)
- Accuracy of design inputs
- Construction parameters (including paving operations, surface texture, initial smoothness, etc.)
- QA/QC (certification, prequalification, inspection, etc.)

All of the potential failure mechanisms (including those associated with structural or functional deterioration) must be addressed to ensure that the pavement system achieves the desired level of performance over 30 to 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 30 to 50 or more years without preventing the pavement from failure caused by other distresses in less than 30 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 B.4. 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).

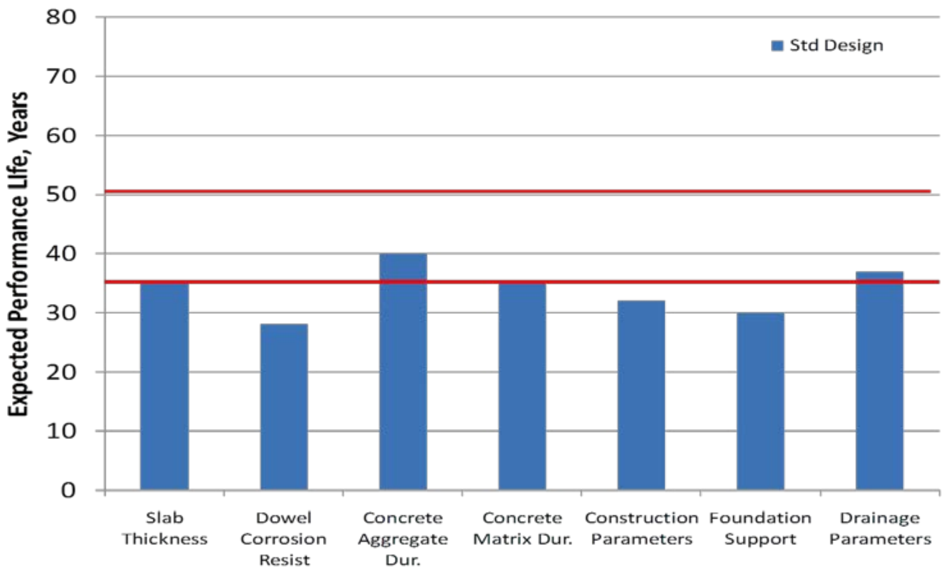

Figure B.4. Illustration of pavement designed and built for 35-year service life.

The chart in Figure B.5 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.). Although 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 B.6 shows that consideration of all of the potential improvement areas is necessary to ensure a performance life of at least 50 years.

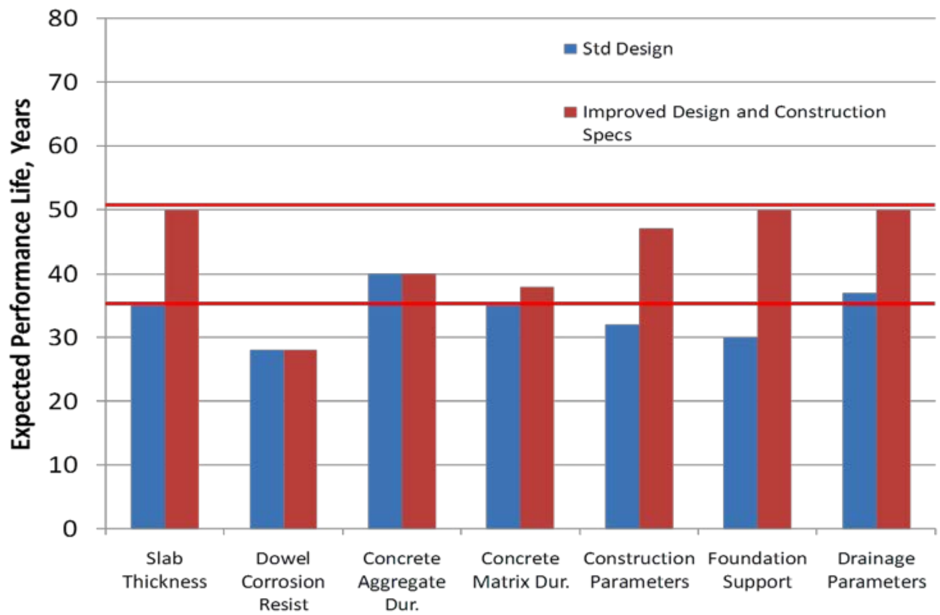

Figure B.5. Illustration of improved design and construction specifications.

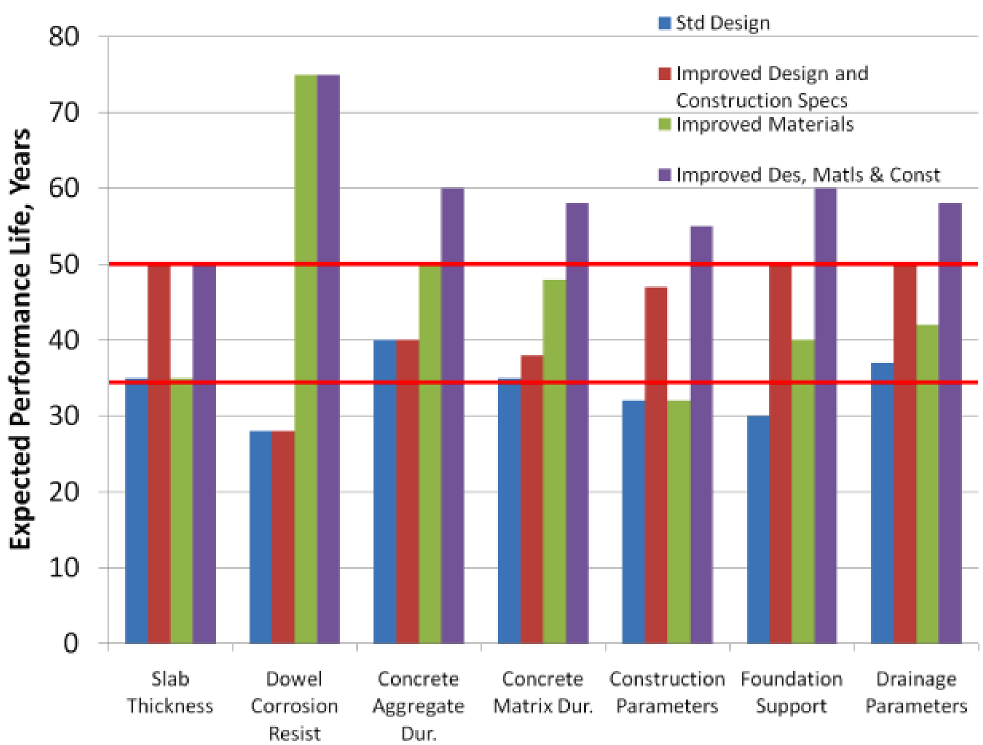

Figure B.6. Illustration that all areas of improvement need to be considered for long life.

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 an SCM. Although not a common practice, some agencies allow use of ternary concrete mixtures that incorporate portland cement and two SCMs.

Supplementary Cementitious Materials

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

Fly ash and slag are covered under the U.S. Environmental Protection Agency’s (EPA’s) Comprehensive Procurement Guidelines (CPG) (EPA, 2011). The CPGs are federal law that requires 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.

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 B.7.

Typical dosages for Class F fly ash are generally between 15% and 25% by mass of cementitious materials. Sources must be evaluated for typical usage rates. As the amount of fly ash increases, some air-entraining and water-reducing admixtures are not as effective and require higher dosage rates because of 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%.

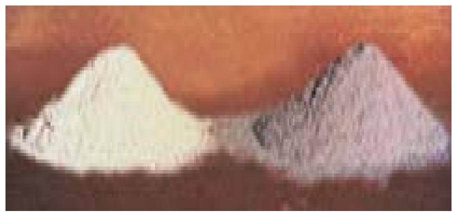

Figure B.7. Class F fly ash. (Photo: FHWA)

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 B.8) results when the molten slag is quenched with water. The rapid cooling forms glassy silicates and aluminosilicates of calcium. Once ground to a suitable particle size, the end result is GGBFS. This is commonly referred to as "slag cement."

GGBFS must meet the requirements of ASTM 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 United States.

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% and 50% 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 by considering the importance of early strengths for the panel fabrication process. However, if the slag will be used to control expansions caused by ASR, the minimum slag content used is that needed to control ASR.

Figure B.8. Preprocessed blast furnace slag. (Photos: J. 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% 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 C 33. Some of the key aggregate requirements are discussed below. Tables B.3 and B.4 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 B.9 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, which potentially can lead 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 by using coarse and fine aggregates. Today, agencies are moving toward the use of a combined gradation that may require use of more than two aggregate sizes. A combined gradation is based on an 8-to-18 specification. The percentage retained on all specified standard sieves should be between 8% and 18%, except for the coarsest sieve, and 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 fewer cementitious materials, exhibits less drying shrinkage, and may be more economical (Richardson, 2005).

Figure B.9. Aggregate processing, which includes stockpiles, conveyors, and screening. (Photos: J. Mahoney)

Table B.3. 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 ASR, expansive pressures, reflection cracking from underlying layer, traffic	• Coefficient of thermal expansion • Coarse aggregate-mortar bond • Shrinkage	•Coefficient of thermal expansion •Gradation •Size •Mineralogy •Shape, angularity,

		loading, loss of support		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 B.3. 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 B.4. 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 reevaluated 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 water to cement (w/c) ratio and lead to lower strength and an increased potential of plastic and drying shrinkage (Folliard and Smith, 2003).

Los Angeles Abrasion Test
The Los Angeles Abrasion Test provides a relative assessment of the hardness of the aggregate. Harder aggregates maintain skid resistance longer and provide an indicator of aggregate quality.

Durability (D-Cracking)
Durability cracking (D-cracking) is a concern for coarse aggregate particles that typically are (1) sedimentary in origin, (2) have a high porosity, (3) have a small pore size (about ~0.1 μm), and (4) become critically (>91%) saturated and subjected to freezing and thawing. Cracking of the concrete is caused by the dilation or expansion of susceptible aggregate particles and will develop wherever the conditions of critical saturation and freezing conditions exist. 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 B.10. 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 B.10. Photos illustrating D-cracking. (FHWA, National Highway Institute)

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 and 5 μm generally experience the most freezing and thawing problems because of higher potential for saturation. Aggregates of sedimentary origin, such as limestones, dolomites, and cherts, are most susceptible to D-cracking (Van Dam et al., 2002).

Alkali-Aggregate Reactivity (AAR)

Two types of AAR reaction are recognized, and each is a function of the reactive mineral: silicon dioxide or silica (SiO_2) minerals are associated with ASR and calcium magnesium carbonate ($CaMg(CO_3)_2$, or dolomite) minerals with alkali-carbonate 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 B.11.

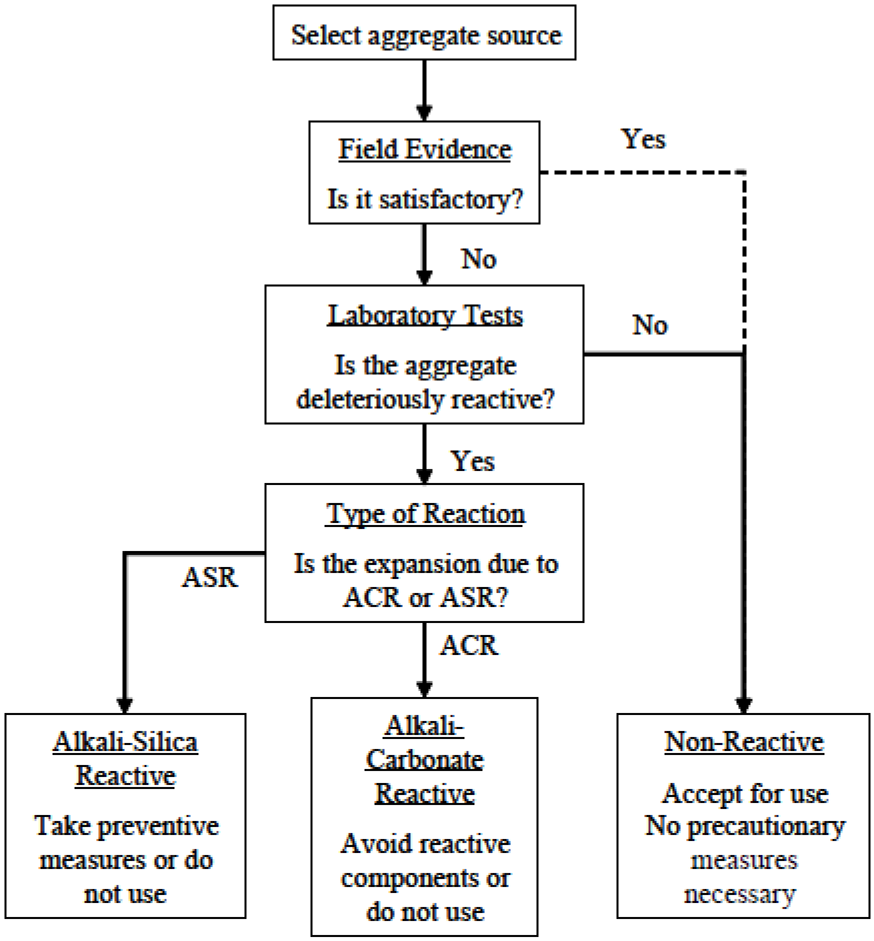

Figure B.11. Evaluation stages for alkali-aggregate reaction determination. (Source: Thomas et al., 2008)

ASR is of more concern because the aggregates associated with it are common in pavement construction. ASR is a deleterious chemical reaction between reactive silica constituents in aggregates and alkali hydroxides in the hardened cement paste. This constituent of concrete has a pore structure, and the associated pore water is an alkaline solution. This alkaline condition plus reactive silica provided by the aggregate produces a gel. The gel, unfortunately, has an affinity for water, which in turn grows and produces expansive stresses. These stresses generate polygonal cracking 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 B.12 and B.13. 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% is essential for ASR to occur.

Although the problem is widely known, and successful mitigation methods are available, ASR continues to be a concern for concrete pavement. Aggregates susceptible to ASR are either those composed of poorly crystalline or metastable silica materials, which usually react relatively quickly and result in cracking within 5 to 10 years, or those involving certain varieties of quartz, which are slower to react in field applications. ASR research is ongoing and the provisions associated with ASR-related testing are based on best current practices. Guidelines related to ASR will continue to be updated or replaced as more research becomes available.

The American Association of State Highway and Transportation Officials (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" (AASHTO, 2010). Additionally, reports from the Portland Cement Association (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 B.12. Illustration of ASR on a traffic barrier. (FHWA)

Figure B.13. 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 the w/c ratio, and cement pastes generally have higher CTEs than do concrete aggregates (as shown in Table B.5). Therefore, the concrete aggregate, which typically comprises 70% or more of the volume of concrete, tends to control the CTE of the hardened concrete: more aggregate and lower CTE aggregate results in concrete with lower CTE values. It should be noted that critical internal stresses may develop in the PCC if the thermal expansion characteristics of the matrix and the aggregates are substantially different, and large temperature changes take place.

Field sections in Texas clearly demonstrated the superior qualities of their limestone versus siliceous aggregates as used in bonded concrete overlays (Kim et al., 2012).

Table B.5. 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 United States is to specify use of epoxy coated dowel bars. However, the effectiveness of the current standard epoxy coating materials and processes beyond 15 to 25 years in service is considered suspect. Figure B.14 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 the Washington State DOT (WSDOT) on fully reconstructed JPCP construction following about 15 years of service. Several recent projects (Minnesota, Illinois, Iowa, Ohio, and Washington) have been constructed using stainless steel clad dowel bars (Figure B.15) 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 DOT (MnDOT) and Wisconsin DOT 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 to a governing state specification.

Figure B.14. Corroded epoxy coated dowel bars in a retrofitted dowel bar project (original bars 1.5” by 18”). (Photos: WSDOT)

Figure B.15. Stainless dowel bar. (Photo: J. 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; Harrington, 2008). The success of the strategy depends on the stability (structural integrity) and the uniformity of the underlying structure. Since the concrete overlay is “separated” from the underlying pavement, the preoverlay repairs are usually held to a minimum. Figure B.16 is a sketch of an unbonded overlay over concrete.

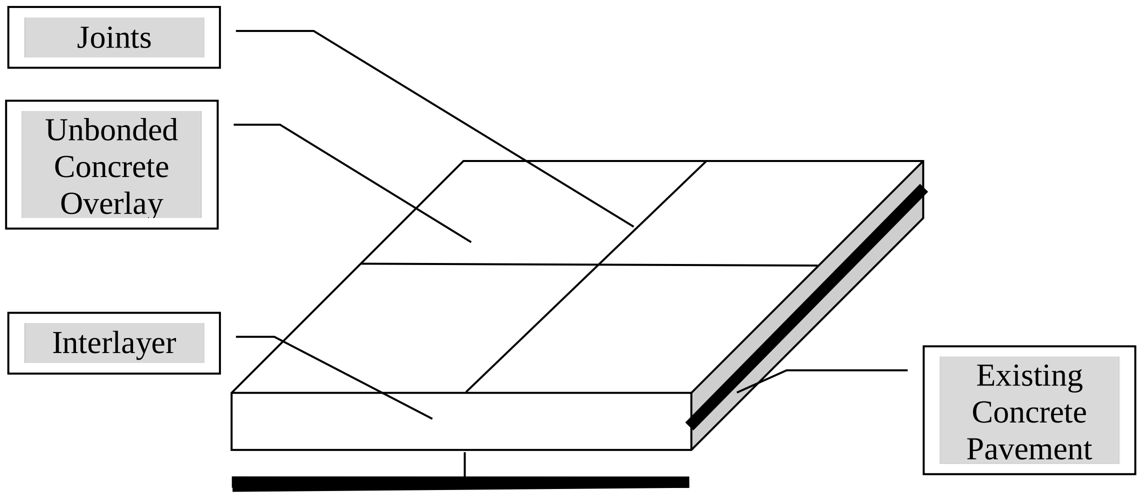

Figure B.16. Unbonded concrete overlay of concrete pavement. (Illustration: J. Mahoney)

Figure B.17 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 B.17. Unbonded 9-in. JPCP concrete overlay placed over concrete in Washington State (overlay 35 years old). (Photo: N. Jackson)

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, backcalculation of 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 B.6.

Table B.6. 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 with 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 ≥8 in. for Interstate applications with lives of about 30 years and 9 in. for about 50 years. Figure B.18 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 (≥50 years) 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 *Mechanistic–Empirical Pavement Design Guide* (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 do mechanistic-based procedures.
- MEPDG (or Pavement 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 international roughness index (IRI). For CRCP, the predicted distress types are punchouts and IRI. The production version of the MEPDG (Pavement ME) from AASHTO was released during 2011.

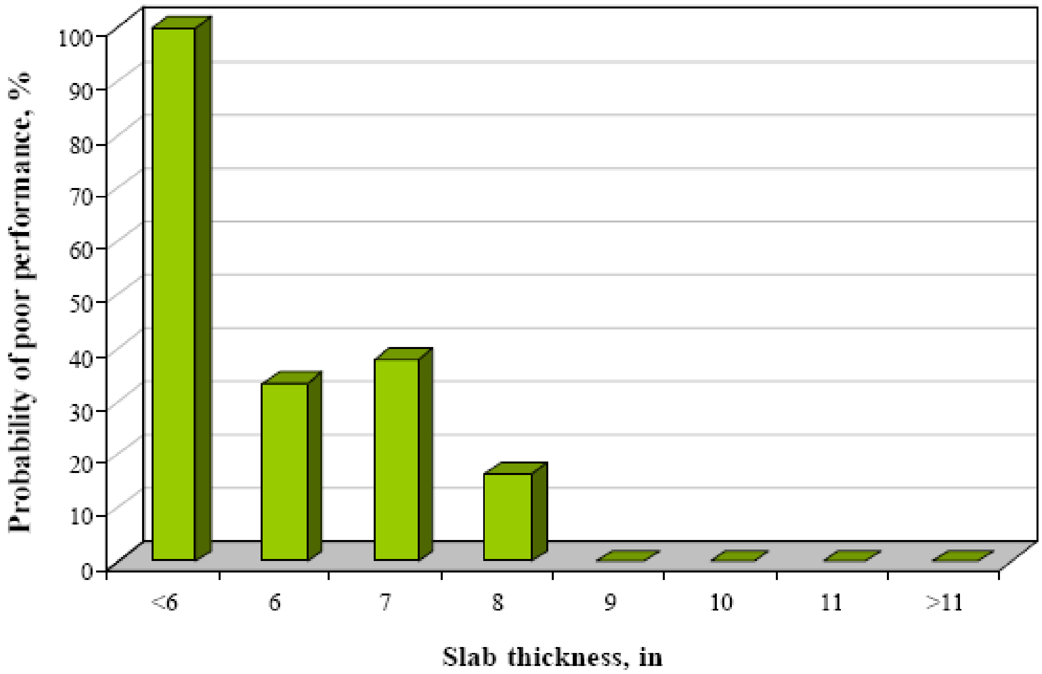

Figure B.18. Slab thickness versus probability of poor performance for unbonded JPCP overlays. (Smith et al., 2002)

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, nonstainless corrosion-resistant steel (ASTM A 1035), 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 ft is typically used for thick (> 9 in.) long-lived concrete pavements. Figure B.19 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).

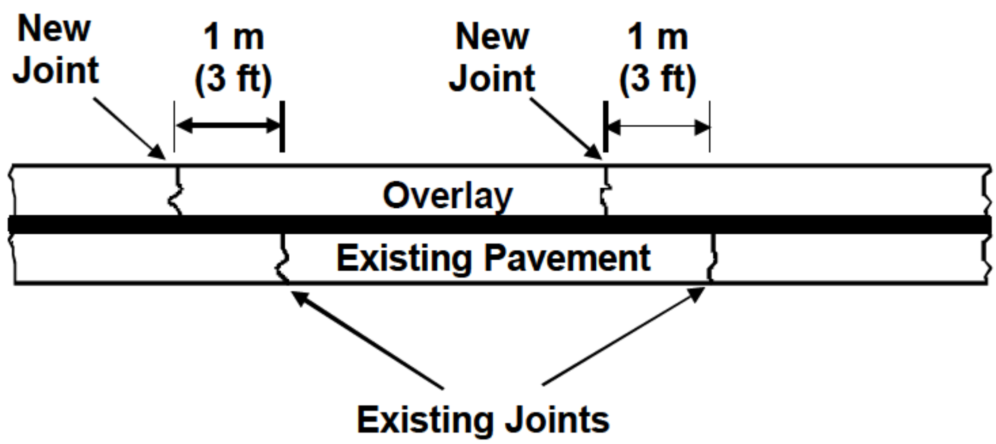

Figure B.19. Joint mismatching details. (Smith et al., 2002)

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 in.) separator layers should be avoided (Smith et al. 2002; Harrington, 2008).

Interlayers should be between 1 and 2 in. thick (Smith et al., 2002; Harrington, 2008). Thin interlayers (e.g., 1 in.) have been used successfully when the existing pavement has little faulting or other surface distress. Thicker separator layers have been used when faulting and distress levels are high. The use of dense-graded and permeable HMA interlayers is common. Other materials used in unbonded overlay interlayers (either alone or in conjunction with HMA material) include polyethylene sheeting, liquid asphalts, geotextile fabrics, chip seals, slurry seals, and wax-based curing compounds. Not all of these materials and material combinations may be suitable for long-life pavements.

In Germany, a nonwoven fabric material is placed between the stabilized subbase and concrete slab to prevent bonding between layers and to provide a medium for subsurface drainage. This technology has been adapted for use in the United States for unbonded concrete overlay interlayers and was showcased on a 2008 unbonded concrete overlay project in Missouri (Tayabji et al., 2009). Figure B.20 illustrates the placement of the fabric on the existing pavement surface. It is noted that no long-term performance data are currently available for the application of this technology in concrete overlays.

Figure B.20. Placement of nonwoven fabric as an interlayer. (Source: Tayabji et al., 2009)

Table B.7 summarizes the types of interlayers currently used in the construction of unbonded concrete overlays for concrete pavements. This information is based on extended meetings with pavement engineering and management professionals from the Illinois Tollway Authority and the Michigan, Minnesota, and Missouri DOTs.

Table B.7. Example State of Practice for 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%). Although other types of separator layers are also used, bituminous materials make up 91% of all separator layer types.

Performance Considerations

The performance of unbonded concrete overlays from the LTPP General Pavement Studies (GPS-9) sections is presented in this section. The pavement performance criteria selected for the summary includes 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 B.21 shows typical transverse cracks both for airfield and highway pavements. Figure B.22 shows the magnitude of average number of transverse cracks per a 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 B.21. Illustrations of transverse cracking on an airport apron and an Interstate Highway. (Photos: Joe Mahoney)

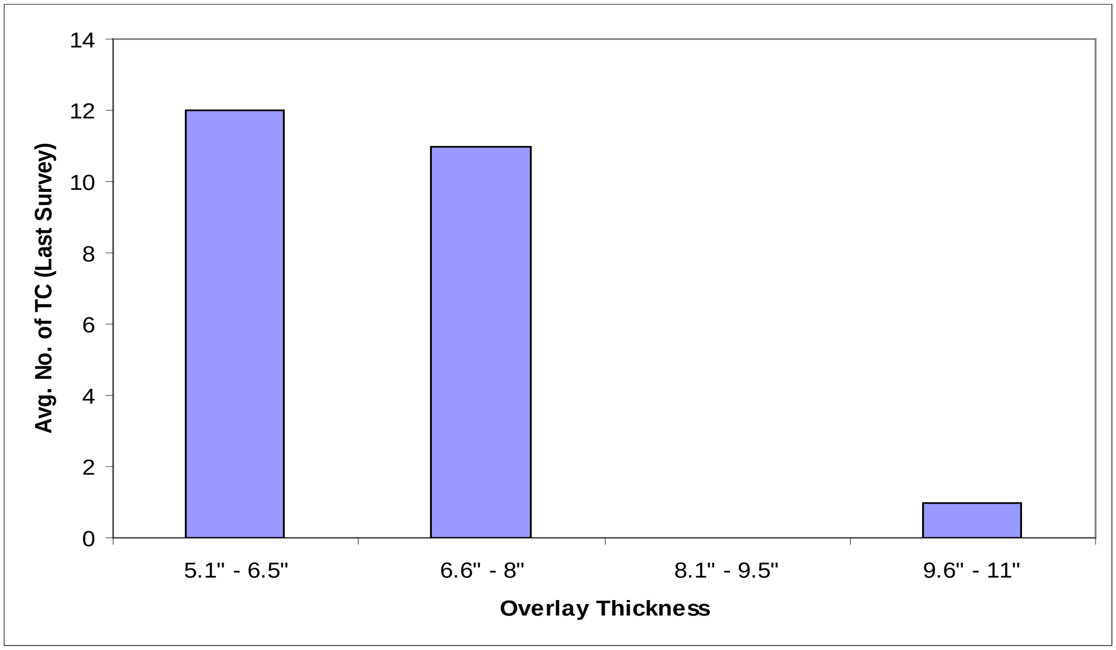

Figure B.22. JPCP overlay thickness versus average number of transverse cracks (TC).

International Roughness Index

Figure B.23 illustrates the progression of IRI and PSI for the various GPS-9 sections and the impact of overlay thickness on ride quality.

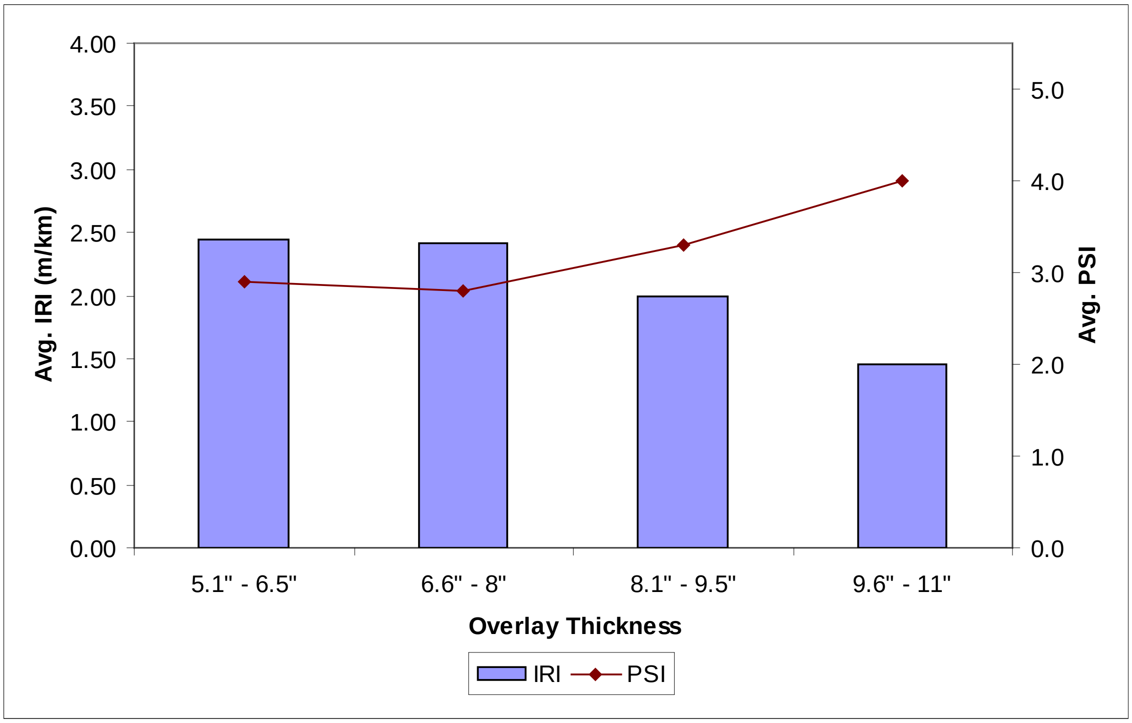

Figure B.23. Overlay thickness versus average IRI and average PSI (pavement age ranges from 6 to 20 years).

Joint and Crack Faulting

Figure B.24 illustrates transverse contraction joint faulting (faulting above 0.25 in. is significant), although the data from GPS-9 projects do not show the degree of severity that is illustrated in Figure B.25. 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 due more to the use of dowels rather than to 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.

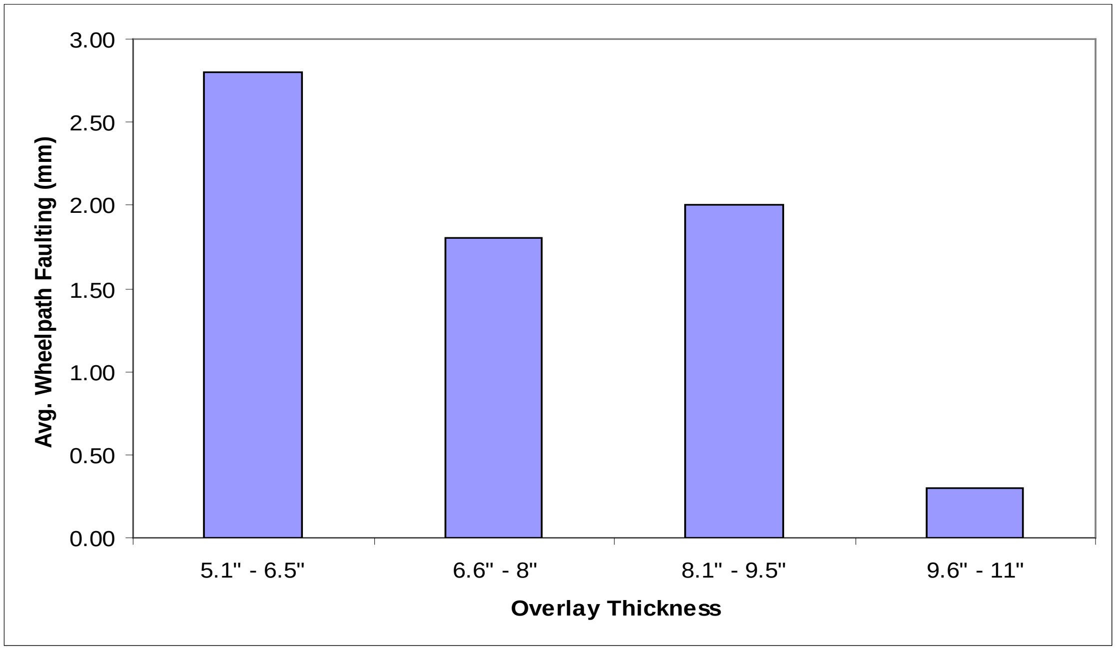

Figure B.24. Overlay thickness versus average wheelpath faulting.

Average fault ~0.25 to 0.5 in.

Average fault ~0.5 in.

Figure B.25. Illustration of contraction joint faulting of JPCP. (Photos: WSDOT)

Impact of Interlayer Design on Performance

Figures B.26 and B.27 illustrate the impact of the interlayer type and thickness on transverse cracking of the overlay. In general, thicker interlayers tend to inhibit transverse cracking.

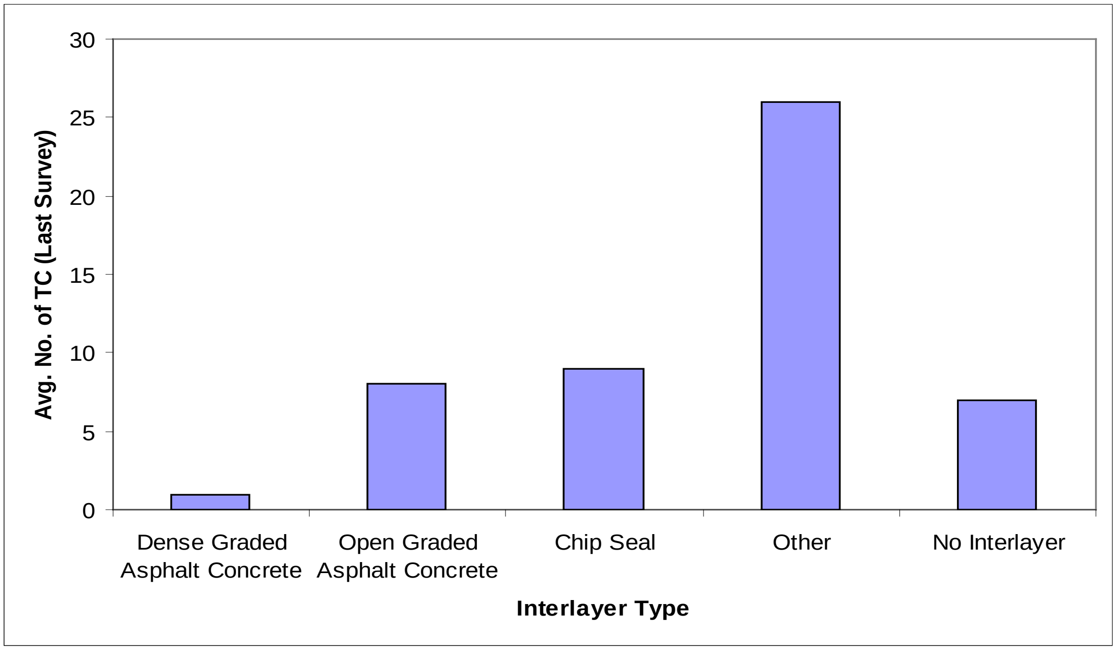

Figure B.26. JPCP interlayer type versus average number of transverse cracks.

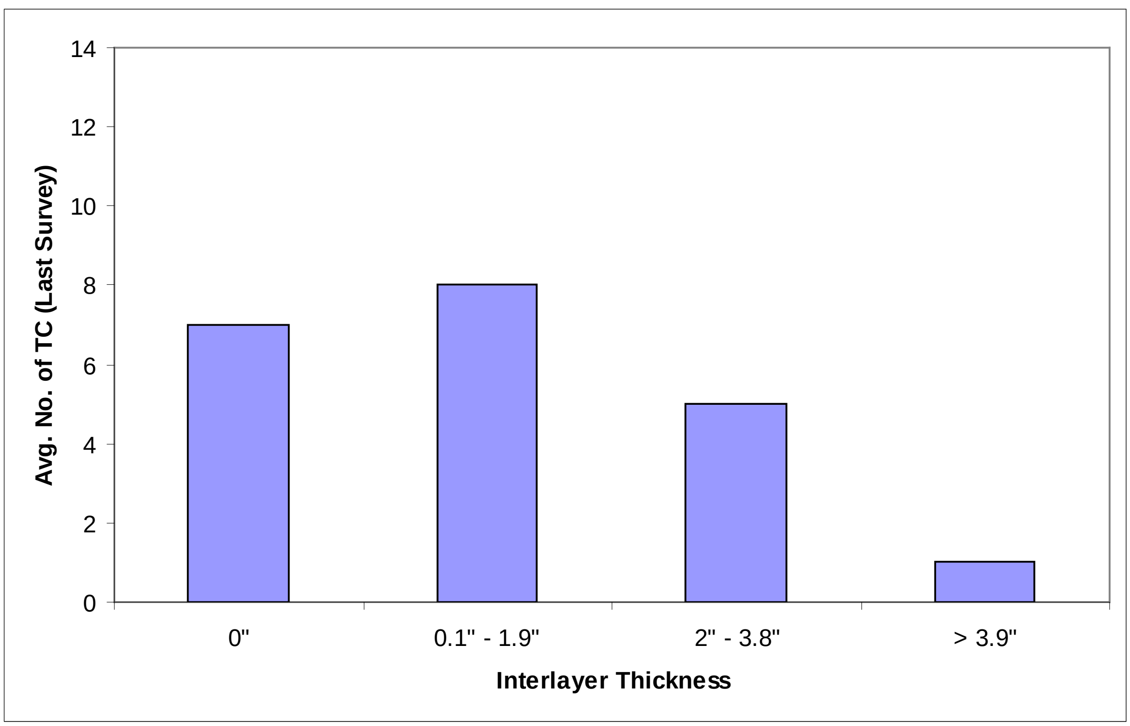

Figure B.27. JPCP interlayer thickness versus average number of transverse cracks.

Figure B.28 shows that thicker interlayers contribute to the integrity of the joint by controlling the amount of joint faulting (all other parameters being equal).

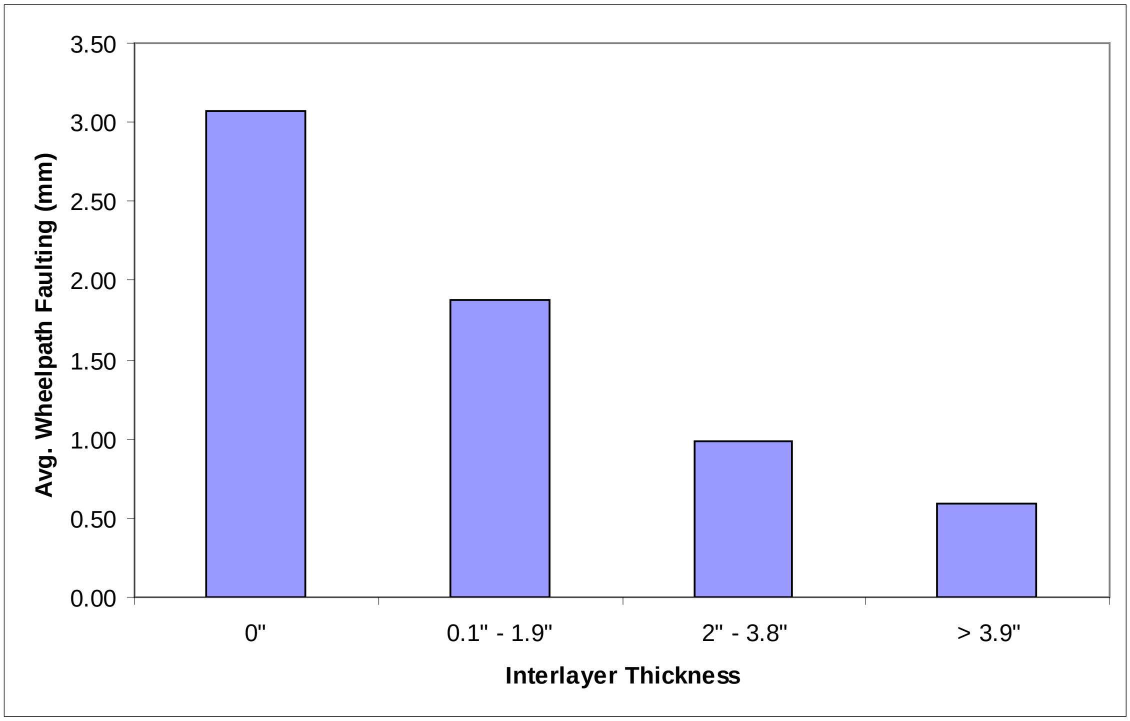

Figure B.28. JPCP interlayer thickness versus average wheelpath faulting.

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 (Harrington, 2008). There should be no standing water or moisture on the separator layer surface at the time of overlay placement. An alternative to this is to construct the PCC overlay at night. Whitewashing of the bituminous surface using lime slurry may also be performed 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 B.29, which illustrates the predicted tensile stress and strength in the concrete over the first 72 hours following placement.

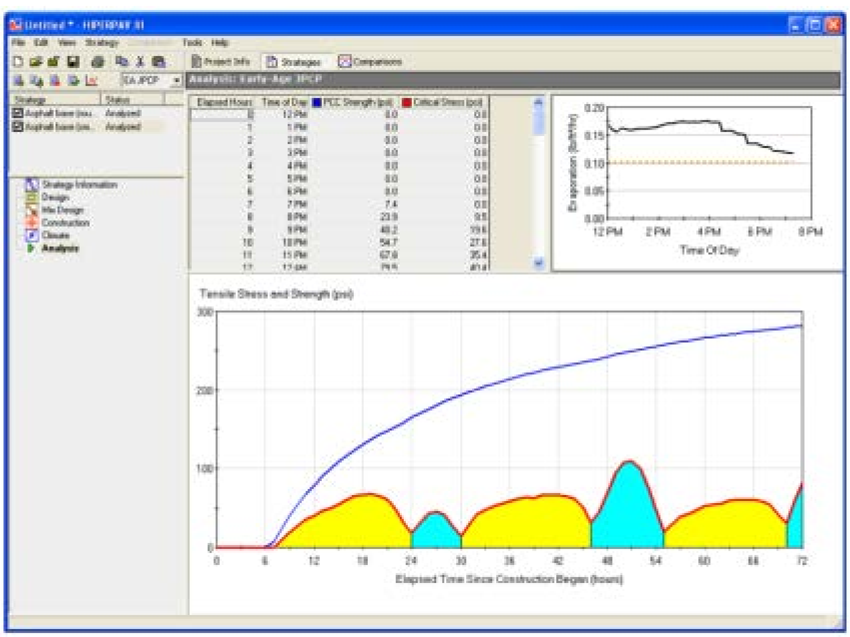

Figure B.29. Screen shot from HIPERPAV III software illustrating tensile stress and strength over first 72 hours. (HIPERPAV, 2010)

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 to reduce tire noise. 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 tends to improve texture durability and friction. For diamond grinding, polish-resistant hard and durable coarse aggregates are recommended. Narrow single-cut joints are recommended to minimize noise. Avoid faulted joints, protruding joint sealants, and spalled joints for quieter pavements (Rasmussen et al., 2008).

Dowel Placement
The use of dowel bars is critical for long-lasting JPCP. Numerous studies, including the AASHO Road Test, have shown 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 because of 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 B.30. These misalignments can cause various types of performance issues that range from slab spalling to cracking as shown in Table B.9. 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.

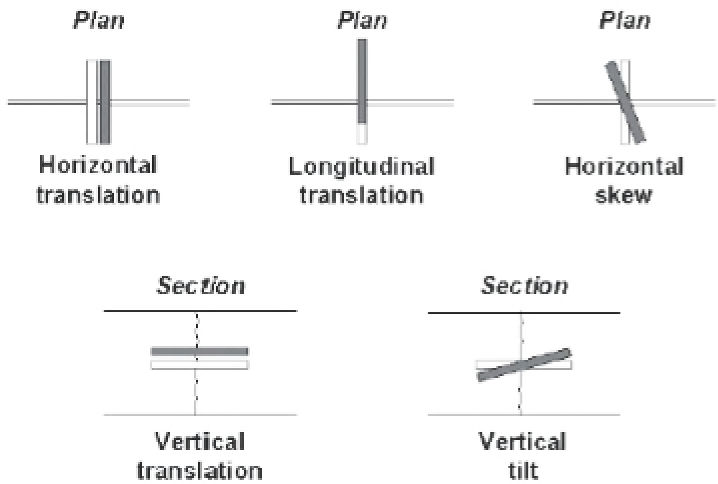

Figure B.30. Types of dowel bar misalignments. (Source: FHWA, 2007)

Table B.9. 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 caused by dowel misalignment is shown in Figure B.31. Additionally, an example of dowel "longitudinal translation" is also shown.

Failed contraction joint caused by dowel misalignment

Example of dowel longitudinal translation (joint is not the same as the one to the left)

Figure B.31. 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 postconstruction location of the dowel bars. There are multiple ways this can be done, but an instrument available from Magnetic Imaging Tools (MIT) is explored here. The device, MIT Scan-2, has been assessed and described in 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. An MIT Scan-2 device is shown in operation in Figure B.32.

Figure B.32. MIT Scan-2. (Source: 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 B.10 summarizes a selection of unbonded concrete overlays of concrete pavements constructed in the United States since 1993. The information presented in the table was compiled from National Concrete Overlay Explorer [a database provided by the American Concrete Pavement Association (ACPA, 2010)]. The website currently contains only a representative sampling of projects across the United States, and so the number of concrete overlay projects viewable online is expected to increase over time.

The common features for these unbonded concrete overlays in Table B.10 include the following:

- Slab thickness ranges from 9 to 12 in.
- Doweled joints spaced mostly at 15 ft
- HMA interlayers ranges 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 B.11.

A selection of significant practices and specifications associated with paving unbonded concrete overlays over existing concrete were selected and included in Table B.12. The table includes a brief explanation of 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 preoverlay 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 HMA concrete pavements are a viable long-lived renewal strategy. In general, this strategy is applied when the existing HMA pavements exhibit significant deterioration in the form of rutting, fatigue cracking, potholes, foundation issues, and pumping; however, the stability and the uniformity of the existing pavement are important for both renewal construction and long-life performance of the unbonded concrete overlay. Figure B.33 is a sketch of an unbonded overlay over preexisting flexible pavement.

The placement of the overlay can potentially do the following (Smith et al. 2002; Harrington, 2008):

- Restore and/or enhance structural capacity of the pavement structure
- Increase life equivalent to a full-depth pavement
- Restore and/or improve friction, noise, and rideability

Table B.10. A Selection of Unbonded Concrete Overlays Constructed in the United States 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	2002	• Slab thickness is 9”

Freeway, KY, existing pavement is JPCP and 28 years old		• 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 at 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

Note: JRCP = jointed reinforced concrete pavement.

Table B.11. Recommended Design Attributes for Long-Lived Unbonded Concrete Overlay (≥30 years)

Design Attribute	Recommended Range
Overlay slab thickness	Thickness ≥8 in. for ≥30 year life
Interlayer thickness (inches)	≥1 in.; 2 in. is likely optimal
Joint spacing	Maximum spacing of 15 ft. Shorter is preferred (12 ft)
Load transfer device	Mechanical load transfer device, corrosion resistant dowels to promote long life Dowel lengths of 18”
Dowel diameter	1.25 to 1.5 in. (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).

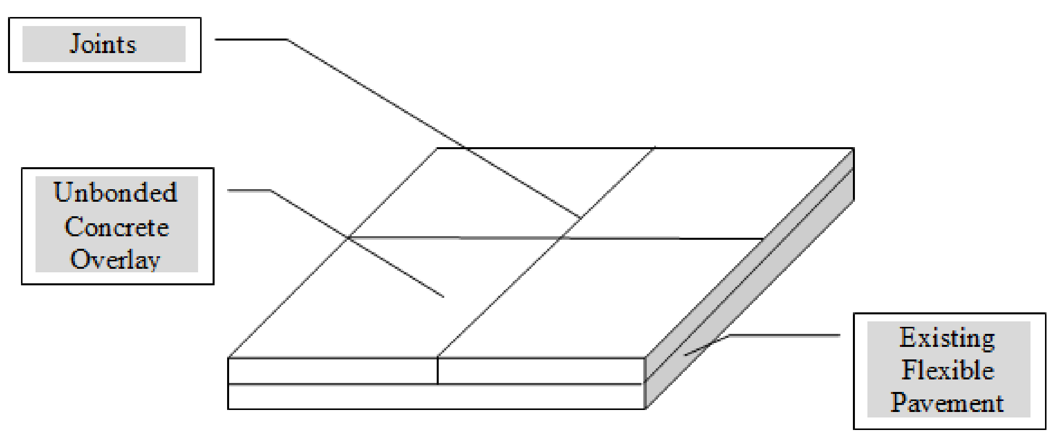

Figure B.33. Unbonded concrete overlay of flexible pavement. (Illustration: J. Mahoney)

Table B.12. Summary of Best Practices and Specifications for Unbonded Concrete Overlays over Existing Concrete

<table>
<tr><th>Best Practice</th><th>Why This Practice?</th><th>Typical Specification Requirements</th></tr>
<tr><td>Existing pavement and preoverlay repairs.</td><td>The preparation of the existing pavement is important for achieving long life from the unbonded concrete overlay.</td><td>
<table>
<tr><th>Existing Pavement Condition</th><th>Possible Repairs</th></tr>
<tr><td>Faulting ≤10 mm</td><td>No repairs needed</td></tr>
<tr><td>Faulting >10 mm</td><td>Use a thicker interlayer</td></tr>
<tr><td>Significant tenting, shattered slabs, pumping</td><td>Full depth repairs</td></tr>
<tr><td>Severe joint spalling</td><td>Clean the joints</td></tr>
<tr><td>CRCP with punchouts</td><td>Full depth repairs</td></tr>
</table>
[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 ≥8 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</td><td>Interlayer thickness and conditions prior to placing the</td><td>• The interlayer material shall be a minimum of 1-in.-thick new bituminous material.
• Surface temperature of HMA interlayer shall</td></tr>
</table>

existing pavement.	concrete overlay influence long-life performance and early temperature stress in the new slabs.	be <90°F prior to overlay placement. **[Refer to Elements for AASHTO Specification 563 for additional details]**[1]
Concrete overlay materials.		• 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]

[1] Contained in *Guide to Using Existing Pavement in Place and Achieving Long Life,* Chapter 4.

Preoverlay Repairs

The preoverlay requirements are minimal at best. Table B.13 summarizes the possible preoverlay repairs needed in preparation for the PCC unbonded concrete overlay of asphalt pavements (Harrington, 2008).

Table B.13. 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 an adequate load-carrying base for the concrete overlay (Smith et al., 2002; Harrington, 2008). The 1993 AASHTO design method does not consider the effects of bonding between the new overlay and the existing HMA pavement. The design method considers the composite k at the top of the HMA layer. Field studies have shown that there is some degree of bonding between the two layers. However, the longevity and the uniformity of this bond over the design life of the structure are 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 8 and 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 B.14 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 B.14 include

- Slab thicknesses ranging from 9 to 12 in.
- Doweled joints spaced mostly at 15 ft

Table B.14 Overview of Selected Unbonded Concrete Overlays of Flexible Pavements Constructed in the United States 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, Bakerville, MO. The existing pavement is 30 years old	2004	• Slab thickness is 12” • Doweled joints spaced at 15’ • 24% truck traffic
US 412, Bakerville, 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”

Added Lanes and Transitions for Adjacent Structures for Unbonded PCC Overlays over Existing Concrete and HMA Pavements

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

Bridge and Overcrossing Structure Approaches

In the transition where the unbonded PCC overlay connects to a bridge approach, or when the roadway section with an unbonded overlay passes under an existing structure, the new grade line and reduced vertical clearances usually require the construction of a new pavement section. The length of the new section depends 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 in the transverse direction.

Because the new roadway section will not be as thick as the renewal approach using the existing pavement, the difference in elevation is usually made up with HMA or a combination of HMA and untreated granular base material. 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 its recommendations for stabilization of the pavement working platform should be considered. Diagrams of possible transition profiles are shown in Figures B.34 and B.35.

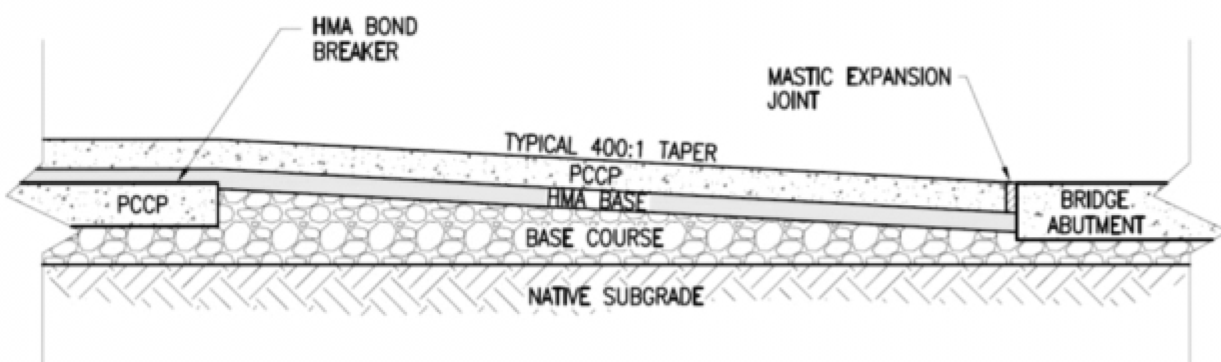

Figure B.34. Diagram of transition to bridge approach (unbonded PCC overlay of PCC pavement).

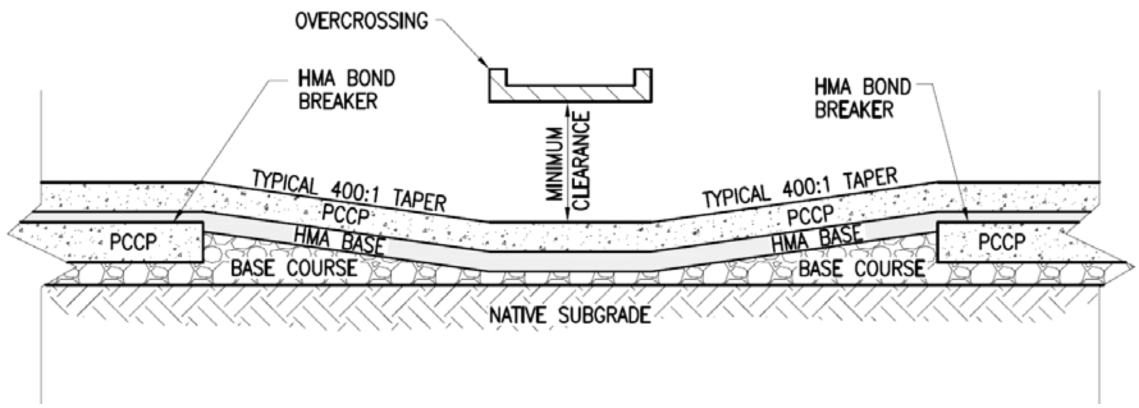

Figure B.35. Diagram of transition beneath structure.

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 its recommendations for stabilization of the pavement working platform should be considered.

Lane Widening

A number of agencies have reported that they have constructed a 14-ft-widened lane in the outside lane to provide improved edge support. One agency reported cracking along the edge of the old PCC pavement caused by nonuniform support at that location. The agency 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 an HMA bond breaker and the widened HMA pavement. Some agencies have widened the existing PCC pavement with PCC pavement and 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 B.36 and B.37. 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.

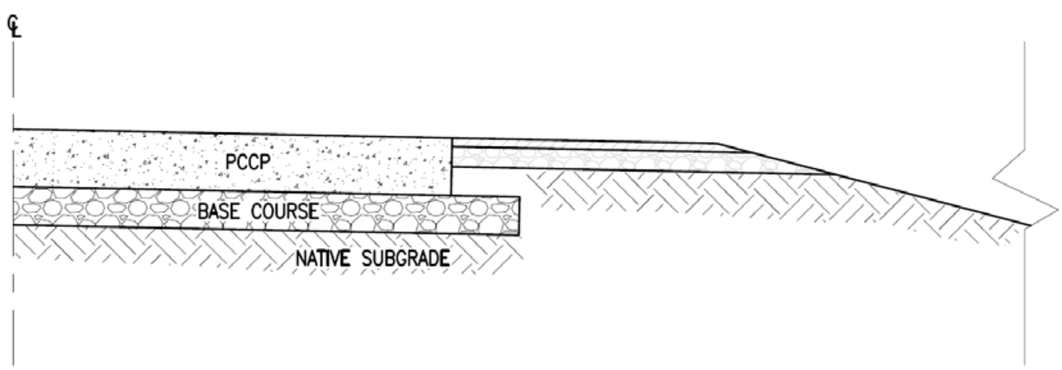

Figure B.36. Cross section showing existing PCC pavement without daylighted shoulders.

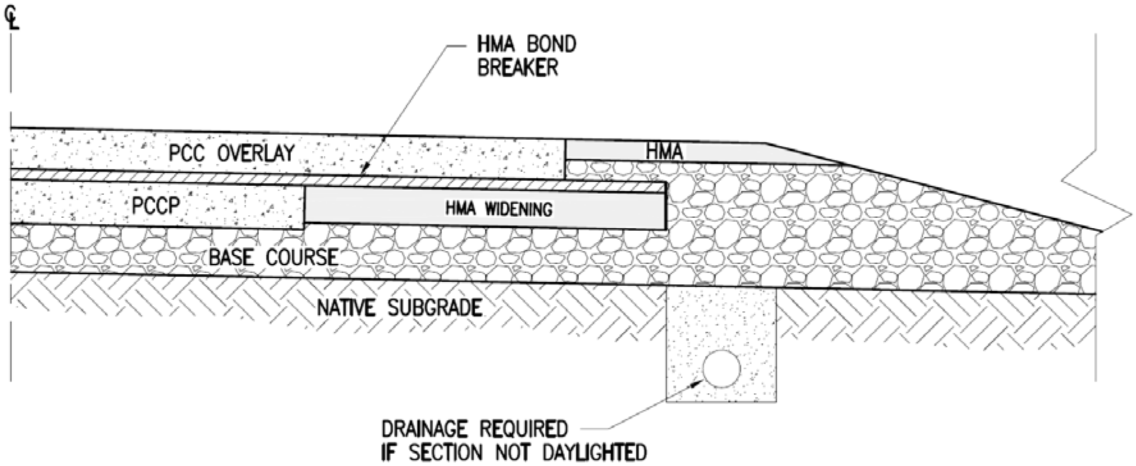

Figure B.37. Cross section showing widening of the shoulder with daylighting or drainage.

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 B.37 and B.38.

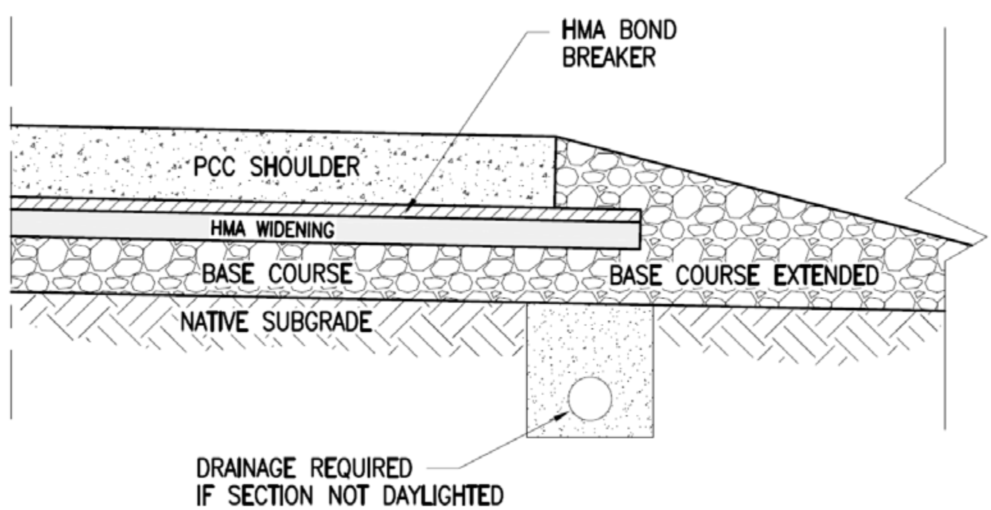

Figure B.38. Cross section detail with PCC shoulder.

Best Practices Summary

The definition of long-life renewal strategies is a design life of ≥30 years. To achieve this, unbonded concrete overlays of existing pavements are recommended. This recommendation is based on several sets of information that 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 30 to 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 B.11). Three major practices are featured: (1) existing pavement and preoverlay repairs, (2) overlay thickness and joint details, and (3) interlayer requirements.

The major findings are recapped in Table B.15.

Table B.15. Summary of Recommended Practices for Unbonded PCC Overlays

Factor or Consideration	Practice
Concrete Overlay Thickness	≥8 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 or appropriate for the slab thickness
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 maximum limits
Existing Pavement	Use criteria provided for preoverlay repairs.
Concrete Overlay Interlayer	Use an HMA interlayer 1 (minimum) to 2 in. thick.
Concrete Overlay Construction	Control mix and substrate temperatures during construction; tools such as HIPERPAV will help in planning and execution

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Supplemental Documentation: Concrete Overlays—Supporting Data and Practices

Given the initial definition of long-life renewal strategies and the constraints of life expectancy associated with timing and selection of pavement renewal strategies, the early findings of this study recommended that 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 was based on several sets of information that 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). When the definition of long-life pavement was reduced to include pavements lasting 30 or more years, bonded and thinner concrete overlays required reexamination.

It is and has been apparent that slab thickness is a major factor in long-life renewal options. Well-known design procedures for PCC systems have been available for several decades. For example, Packard (1973) used fatigue concepts for airport pavement design for the Portland Cement Association (PCA). Packard (1973) and Neville (1975) both noted that for flexural stress ratios less than 0.55 (applied flexural stress/modulus of rupture), the fatigue life of PCC is unlimited. Packard actually used a stress ratio of 0.50 to add a bit of conservatism to the PCA airfield design process. Additionally, Packard (1984) produced a fatigue-based highway design method for 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.

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 were used. The overlay thicknesses ranged from 5.8 to 10.5 in. Separator layers included dense-graded asphalt concrete, open-graded asphalt concrete, and chip seals. The average joint spacing was about 16 ft and load transfer mechanisms were either aggregate interlock or steel dowels. A summary of the sections and major findings from that assessment include the following:

- Of the unbonded overlays reviewed, the thicknesses were
  - ~6 in. thick: 22%
  - ~8 in. thick: 22%
  - ~9 in. thick: 11%
  - ~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. These also contributed toward the integrity of the joint by controlling the amount of joint faulting.
- Use of dowel bars in transverse joints had a positive impact on all pavement performance measures.

Bonded Concrete Overlays. From the SPS-7 experiment (Bonded Concrete Overlays on PCC Pavement), these sections were examined for Phase 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 as follows:
  - 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 that 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 B.16.

Table B.16. 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 Construction
3.0	4.4	1	21
3.0	3.0	0	11
3.0	3.6	9	43
3.0	3.0	0	15
5.0	4.8	6	102
5.0	4.9	3	101
5.0	5.2	2	94
5.0	5.4	4	130

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

- The cracking levels observed for these nominal 3- and 5-in.-thick bonded overlays suggest that these sections will not serve adequately for 30 to 50 years. The Missouri DOT (2002) notes in the "Missouri Guide for Pavement Rehabilitation": "(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, it appears that long-life concrete overlays for a 30 to 50 year life are only likely for thicker unbonded overlays. This is further supported by additional state experience, which follows. The remainder of this supplemental documentation will continue to explore largely the performance of bonded concrete overlays and evidence as to their performance particularly with respect to the potential for lives ≥30 years.

TxDOT Bonded Concrete Overlays

During the conduct of the R23 study, a field trip to review concrete overlays was made with the TxDOT. Most of TxDOT's bonded concrete overlays are located in the Houston area and are CRCP overlays over existing CRCP. Based on observed performance of 4 to 8 in. bonded overlays and views expressed by TxDOT personnel, it appears that bonded CRCP overlays within that thickness range can be expected to perform about 20+ 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). This performance was reconfirmed with TxDOT representatives during May 2012.

A recent study for TxDOT by Kim et al. (2012) provided updated information about a selection of bonded concrete overlays mostly in the Houston area. A summary of the information follows in Table B.17. This information provides an approximate estimate of performance for bonded concrete CRCP overlays over existing CRCP. It appears that the bonded concrete overlay thickness has a limited impact on performance—likely due to being placed on an existing CRCP. It is reasonable to conclude that with proper attention to good bonding and construction practices, a 20-year life can be expected for a range of CRCP overlay thickness (from a minimum of 2 in. up to 6.5 in. with most at 4 in.). It is expected that some distress will occur to these overlays during a 20-year period and be mostly related to delamination. Thin bonded overlays (2 in.) have been used to address functional issues in the existing pavement.

Kim et al. (2012) also reported on a 2010 CRCP bonded concrete overlay 7 in. thick placed on an existing 9-in. JPCP near Sherman, Texas. This is an interesting project to follow, but it is very early in its performance life.

Table B.17. Bonded CRCP Concrete Overlays in Texas over Existing CRCP with Moderate to Heavy Traffic Levels

Route	BCO Thickness	Age as of Most Recent Condition Survey	Existing Pavement and/or BCO Reinforcing	Comments
I-610 Houston	2-3 in.	27 years (original construction 1983)	8-in. CRCP over 6-in. CTB; multiple sections with no, or steel mat, or steel fiber reinforcement.	Delaminations detected after 7 years. Good condition as of 2010.
I-610 Houston	4 in.	24 years (original construction 1986)	8-in. CRCP over 6-in. CTB	Poor condition as of 2010. Early delams occurred within first 24 hours following construction. Mixed performance since there were several experimental sections. Removed and replaced in 2010.
I-610 Houston	4 in.	20 years (original construction 1990).	8-in. CRCP; wire mesh reinforcing.	Fair condition as of 2010—includes punchouts, spalling, and patching. Bonding agent PC grout and improved construction practices for original construction.
SH 146 Near Houston	3 in.	9 years (construction about 2001)	11-in. CRCP	Poor to good condition as of 2010. Localized areas of punchouts, minor spalls, and HMA patches.
Beltway 8 Houston	2 in.	14 years (construction 1996)	13-in. CRCP; steel fibers.	Fair to good condition as of 2010. Some patches, longitudinal cracks.
US 281 Wichita	4 in.	8 years (construction	8-in. CRCP; steel mat	Fair to good condition as of 2010. Some delams, spalling.

Falls		2002)	reinforcement.	Potential for punchouts.
I-10 El Paso	6.5 in.	14 years (construction 1996)	8-in. CRCP	Fair condition as of 2010. Original construction issues resulted in delams due to low w/c ratio and evaporation rates. As of 2010 some longitudinal cracking, PCC patches, and delams.

The Texas Pavement Design Guide (TxDOT, 2011) provides additional insight on bonded concrete overlays. The guide states that bonded concrete overlays placed over thin existing concrete pavement must behave as a monolithic layer. Further, TxDOT has constructed bonded concrete overlays ranging in thickness from 2 to 8 in. thick. Bonded concrete overlays have not performed well over existing JPCP. Conversely, bonded CRCP overlays over existing CRCP have performed successfully in several districts but have not been used widely throughout the state. A portion of Chapter 10 (TxDOT, 2011) follows:

> This chapter describes bonded concrete overlays (BCO) on continuously reinforced concrete pavement (CRCP), not on concrete pavement contraction design (CPCD). BCO is not a good option for the rehabilitation of CPCD.
>
> In the past, concrete pavements were designed and constructed with insufficient thicknesses for today's traffic demand. This insufficient thickness often resulted in pavement distresses such as punchouts for CRCP and mid-slab cracking or joint faulting in CPCD. If the Portland cement concrete (PCC) pavement is structurally sound (in other words, if the slab support is in good condition) except for the deficient thickness, BCO can provide cost-effective rehabilitation strategies to extend the pavement life. In bonded concrete overlays, new concrete layer is applied to the surface of the existing PCC pavement. This increases the total thickness of the concrete slab, thereby reducing the wheel load stresses and extending the pavement life. There are BCO projects in Texas that have provided an additional 20 yr. of service. At the same time, there are BCO projects that did not perform well. The difference between good and poorly performing BCOs is the bond strength between new and old concretes.
>
> The critical requirement for the success of BCO is a good bond between a new and old concrete layers. If a good bond is provided, the new slab consisting of old and new concrete layers will behave monolithically and increased slab thickness. The increased slab thickness will reduce the wheel load stress at the bottom of the slab substantially, prolonging the pavement life. On the other hand, if a sufficient bond is

not provided, the wheel load stress level in the new concrete layer will be high and the pavement performance will be compromised.

If the overlay is being placed only to remedy functional failures, normally a thinner overlay would suffice. However, 2 in. is the minimum practical constructible thickness for an overlay. For the steel design, when the overlay thickness is more than 40% of the existing CRCP, longitudinal steel should be provided for the overlay. If the steel is not provided:

- The longitudinal steel in the existing CRCP will be in much higher stress, diminishing its ability to restrain concrete volume changes
- The distance between the overlaid concrete surface and the existing steel will be increased and the ability of the existing steel to control the concrete volume changes at the surface will be diminished, resulting in more concrete volume changes and larger crack widths at the surface. The amount of steel needed should be sufficient to control the overlaid concrete volume changes. The guidelines to be developed in the current research study are expected to address steel design.

Steel should be placed at a depth that provides a minimum concrete cover of 3 in. If the overlaid thickness layer is not large enough, reinforcement steel bars can be placed directly over the surface of the existing pavement as shown in Figure 10-11 [see referenced TxDOT document], rather than at mid-depth of the overlay. For overlaid thickness that is not large enough, it may not be feasible to use a slip-form paving machine to place steel at the mid-depth of the overlay due to the use of vibrators. Placing steel directly on top of the surface of the existing pavement has advantages and disadvantages. Advantages include: saving construction time and costs, since it does not require chairs. Another advantage: the steel will restrain concrete volume changes at the interface most effectively, which will prevent or retard debonding. The only disadvantage is the reduction of the interface area between the new and old concrete. Taken together, for overlaid thickness up to about 5 in., placing steel on top of the existing concrete appears to be a better construction practice. A research study currently underway will address this issue. Guidelines will include recommendations.

WSDOT Bonded Concrete Overlays

Bonded JPCP concrete overlays constructed in 2003 over existing HMA were reviewed (Figure B.39). Three thicknesses of concrete overlays were used: 3, 4, and 5 in. each, placed on I-90 east of Spokane, Washington, which experiences about 1,000,000 ESALs/year. These sections were removed during 2011 because of pavement reconstruction; thus, they were in service for 8 years.

Construction of bonded PCC overlays in July 2003 that were placed directly on rotomilled HMA.

Figure B.39. 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 B.40. 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. for 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 as follows:

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

At the time of removal in 2011 (Figure B.41), the 3-in. section was severely distressed as shown in Figure B.40. 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 B.40. 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 B.41. Bond between the PCC overlays were assessed visually during removal in 2011. (Photo: WSDOT)

MnDOT Unbonded and MnROAD Bonded Concrete Overlays

During March 2012, the study team made an additional visit to the MnDOT. The purpose was to review the study guidelines and performance of their unbonded and bonded concrete overlays.

An example of the performance of one of their unbonded concrete overlays is shown in Figure B.42. Discussion with the MnDOT pavement team suggested that this type of overlay is expected to perform for 25 to 30 years. Given the specific section shown in Figure B.42, this section on I-35 at MP 156 (north of Minneapolis) was 25 years old at the time of the site visit. The transverse contraction joints were doweled, skewed, and placed 15 ft apart. It was placed over a preexisting JRCP. It is reasonable to expect this section to perform beyond a 30-year life given its excellent condition (no observed cracking or faulting).


Unbonded 8-in. overlay over pre-existing JRCP on I-35 in Minnesota. The transverse joints are spaced at 15 ft with dowels. The photos were taken during March 2012 and the section was 25 years old at that time.

Figure B.42. Condition of 8-in. unbonded concrete overlay on I-35 in Minnesota. (Photos: J. Mahoney)

The primary Minnesota experience with bonded concrete overlays is at the MnROAD facility. It constructed the 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 with the exception of Cell 96, which continues in service as of 2012. Figure B.43 shows the 3-in.-thick sections with two different joint layouts. The conclusion was that the 5 ft by 6 ft joint layout was superior to the 4 ft by 4 ft layout, but the amount of cracking for both configurations was extensive.

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 B.43. Condition of 3-in. bonded concrete overlays following 5 million ESALs and 6 years of service. (Photos: MnDOT)

Table B.18 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 B.44 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. Was 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 B.44. 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)

Figure B.45 shows Cell 96 at MnROAD, which is the only remaining 6-in.-thick section of the original bonded concrete overlays as of March 2012. The JPCP overlay is 6 in. thick over 7 in. of HMA. At the time the photos were taken, the section was 15 years old and had received about 1 million ESALs per year. Patching of joints and slab corners was observed and grinding had been done in 2011. MnROAD representatives noted that transverse joint faulting was the primary distress that triggered the grinding.

Figure B.45. Condition of the remaining 6-in. bonded concrete overlay—Cell 96 at MnROAD in March 2012 (Photos by J. Mahoney)

Table B.18. 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

Note: “Present” for Cell 96 is as of March 2012.

Recap on Concrete Overlays

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

- Bonded JPCP concrete overlays over HMA
- 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 B.19 shows an initial summary of typical pavement lives that can be expected for various slab thicknesses along with joint details. The expected lives shown are tentative and reflect a reasonable extrapolation from the field data reviewed.

Table B.19. 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: 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 6 x 6-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; Kim et al., 2012). 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.

Subsequent information gathered during 2012 allowed for the updating of Table B.19 and is shown as Table B.20 below. This shows that to reach a 30-year life an overlay thickness of about 8 in. is required over existing HMA. At this thickness, it would be classified as unbonded JPCP with dowels. A 9-in. overlay should achieve a 35-year life (based in part on MnDOT recommendations and other state DOT experience). As noted earlier CRCP bonded overlays should perform adequately at lesser thicknesses according to information from TxDOT; however, during a meeting with TxDOT pavement personnel during May 2012, its representatives stated that bonded concrete overlays over existing CRCP are not a standard practice in Texas. Currently, HMA overlays placed over existing CRCP are more common.

Table B.20. Bonded and Unbonded JPCP Concrete Overlays over Existing HMA with 1 Million ESALS per Year with Sufficient Existing HMA Thickness (an update of Table B.19 following meeting with MnDOT during March 2012)

Slab Thickness (in.)	Bonded or Unbonded	Joints	Dowels?	Expected Life (years)
3	Bonded	5 ft by 6 ft	No	5
4	Bonded	5 ft by 6 ft	No	5 to 10
5	Bonded	5 ft by 6 ft	No	10 to 15
6	Bonded	6 ft by 6 ft	No	15 to 20
7	Bonded	6 ft by 6 ft	Optional	20 to 25
8	Unbonded	12 ft by 12 ft	Yes	25 to 30
9	Unbonded	15 ft by 12 ft	Yes	30 to 35

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 B.21 and B.22. Minnesota and Washington State are grouped together in Table B.21 since their practices are for JPCP. Illinois and Texas are summarized in Table B.22 to reflect their CRCP practices. Although these practices were developed with new pavement construction in mind, they are also applicable to long-life concrete overlay systems.

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

Thickness Summary

Based largely on field sections in several states, unbonded JPCP overlays of ≥8 in. placed on existing HMA or concrete are expected to last about 30 years. Most experience from state DOTs suggests this type of overlay requires dowels at the transverse joints. Based on TxDOT experience, CRCP overlays over existing CRCP can achieve a 20-year life for a range of thicknesses (those reviewed ranged from a minimum of 2 in. up to 6.5 in.). TxDOT has accumulated substantial experience on both design and construction practices for this type of overlay.

Table B.21. 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 State 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 resistar	• 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 texturing

	• Longitudinal direction • Requires 1 mm average depth in sand patch test (ASTM E965)	
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 B.22. 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	• Illinois DOT applies tests to assess aggregate freeze-thaw and ASR susceptibilities	
PCC Mix		• Limits the coefficient of thermal expansion of concrete to ≤6 microstrains per °F
Construction Requirements	• Limits on concrete mix temperature = 50 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

References

References contained in this Supplemental Documentation are listed in the *Guide to Using Existing Pavement in Place and Achieving Long Life,* Chapter 3.

APPENDIX C

Revised Project Assessment Manual

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 need to be organized to maximize their 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 survey to insights on traffic impacts. The last section provides information on life-cycle assessments (environmental accounting). This type of assessment is receiving increasing usage and is likely to be widely applied in the future.

1.2 How to Use the Manual

The use of the manual is to complement the design tools developed by the SHRP 2 R23 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. Rather, the manual is designed as a reference document that provides information relevant to all renewal strategies considered in the SHRP 2 R23 project.

1.3 Assessment Data Categories

There are 10 categories of data contained in this manual. These are

- 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 C.1.1.

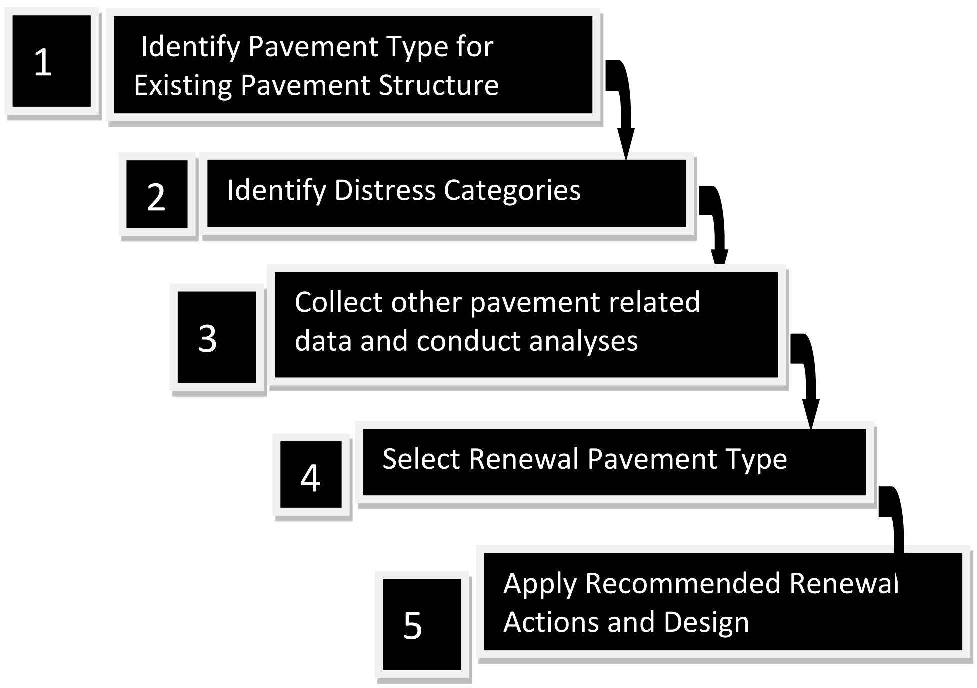


Figure C.1.1. Outline of assessment scheme.

The first three boxes (1 through 3) shown in Figure C.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). The types of input data include the distress types associated with the existing pavement structure, characterization of future traffic [in terms of equivalent single-axle loads (ESALs) and average daily traffic (ADT)], subgrade characterization (strength or stiffness), and more.

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 C.2.1. Almost all distress survey schemes use a subset of fracture, distortion, and/or disintegration.

Upon closer inspection of Table C.2.1 for flexible pavements, two of these subsets—fracture and disintegration—are responsible for most pavement rehabilitation and maintenance actions. More specifically, these can be categorized by fatigue, transverse cracking, and stripping/raveling. Tables C.2.2 through C.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 another appropriate location referencing system. For multilane highways, this information can be collected for the design lane or all lanes in one direction—as per project requirements.

Table C.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 the Long-Term Pavement Performance (LTPP) Distress Identification 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 hot-mix asphalt (HMA) layers in the pavement structure.

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

1. **Pavement Cracking**: Pavement cracking includes all major types of cracks that can occur in a slab. This can include corner breaks and longitudinal and transverse cracking as defined by Miller and Bellinger (2003). Corner break cracks intersect the adjacent transverse and longitudinal joints at approximately a 45° angle. Longitudinal and transverse cracking are parallel and transverse to the centerline, respectively.
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) ASR Cracking: Cracking of the portland cement concrete (PCC), which can be easily confused with D-cracking or shrinkage cracking.
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. **Punchouts**: 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. See Tables C.2.2 through C.2.9 and Figures C.2.1 through C.2.8.

Table C.2.2. Template for Flexible Pavement Distress—Fatigue Cracking

Location (milepost)	Depth		Distress		
			Fatigue Cracking		
	HMA (in.)	Base (in.)	Severity[1]	Extent[2]	Depth of Fatigue Cracks[3] (measured from the pavement surface)
			Low		
			Moderate		
			High		

1. 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).
2. Extent of fatigue cracking is based on % of wheelpath areas. Record extent for each level of severity.
3. 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 C.2.1. Illustrations of fatigue cracking severity levels.

Table C.2.3. Template for Flexible Pavement Distress—Transverse Cracking

Location (milepost)	Depth		Distress		
			Transverse Cracking		
	HMA (in.)	Base (in.)	Severity[1]	Extent[2]	Depth of Transverse Cracks (measured from the pavement surface)[3]
			Low		
			Moderate		
			High		

1. 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).
2. Extent of transverse cracking is based on the number of cracks per 100 ft. Record extent for each level of severity.
3. 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 C.2.2. Illustrations of transverse cracking severity levels.

Table C.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.

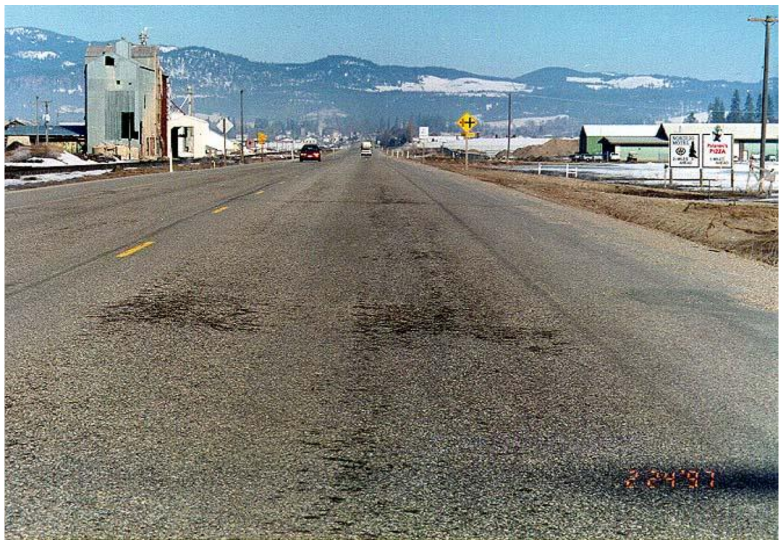


Figure C.2.3. Illustration of raveling. (Photo: WSDOT)

Using Table C.2.1 again, the most important JPCP distress types that initiate PCC pavement renewal actions are fracture (slab or pavement cracking), distortion (faulting—typically at transverse contraction joints), and disintegration, which includes materials-caused distresses of D-cracking and ASR cracking. These are shown in Tables C.2.5 through C.2.8. Tables C.2.9 and C.2.10 apply to CRCP and composite pavements.

Table C.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

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

Figure C.2.4. Illustrations of PCC slabs with multiple cracks. (Photos: Pavement Interactive and J. Mahoney)

Table C.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

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 C.2.5. Illustrations of various levels of joint faulting.

Table C.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		

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 from that used by LTPP.

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

Low severity
(Source: N. Jackson)

High severity
(Source: N. Jackson)

Figure C.2.6. Illustrations of D-cracking severity levels.

Table C.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	How Was ASR etected or Measured?

1. Three types of base underlying PCC: (1) granular base, (2) cement-treated base, or (3) asphalt-treated base.

Early stage of cracking (Source: N. Jackson)

Advanced stage of cracking (Source: N. Jackson)

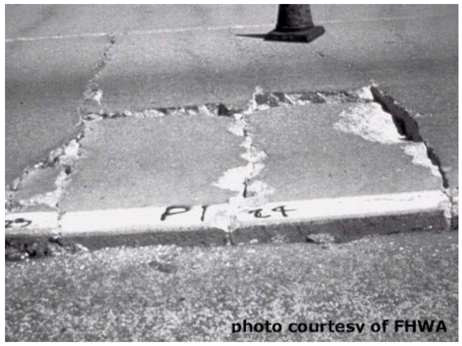
Figure C.2.7. Illustrations of ASR cracking severity levels.

Table C.2.9 applies to CRCP. A critical distress for CRCP is punchouts (which falls under "fracture" in Table C.2.1).

Table C.2.9. Template for Rigid Pavement Distress—CRCP—Punchouts

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

1. Three types of base underlying PCC: (1) granular base, (2) cement-treated base, or (3) asphalt-treated base.



Advanced stage for a punchout

Figure C.2.8. Illustration of a CRCP punchout. (Source: FHWA)

Table C.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	

1. Composite pavement definition assumes that 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 are specifically used in the renewal decision-making process is covered in a separate project report.

2.4 References

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

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.

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

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

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

Figure C.3.1 helps to define lateral locations of a typical highway lane (from AASHTO, 2001). Figure C.3.2 shows how rut depths are measured with automated equipment.

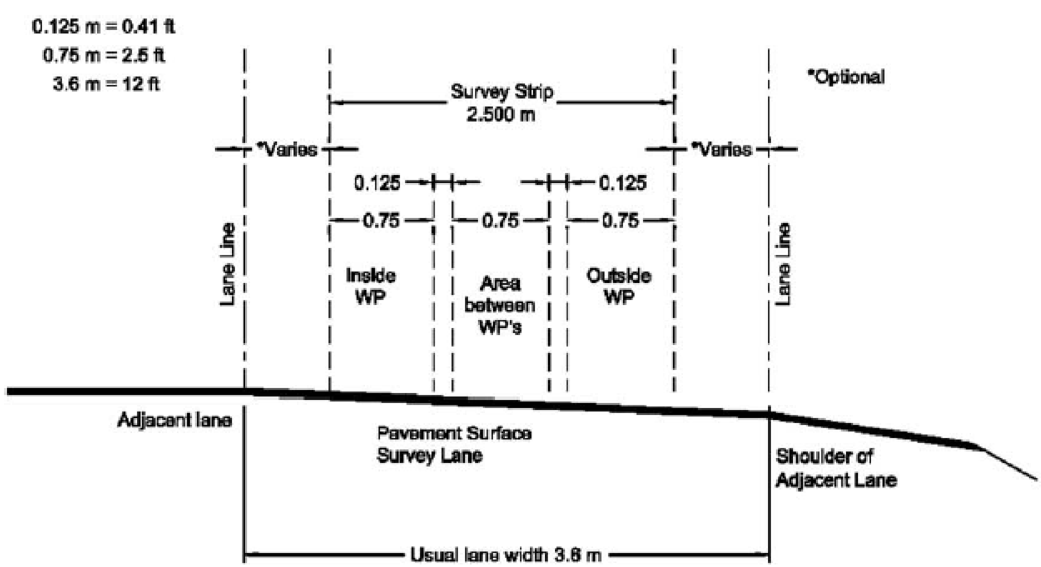


Figure C.3.1. Sketch Illustrating wheelpaths (WP) and between wheelpaths. (Source: McGhee, 2004, and AASHTO, 2001)

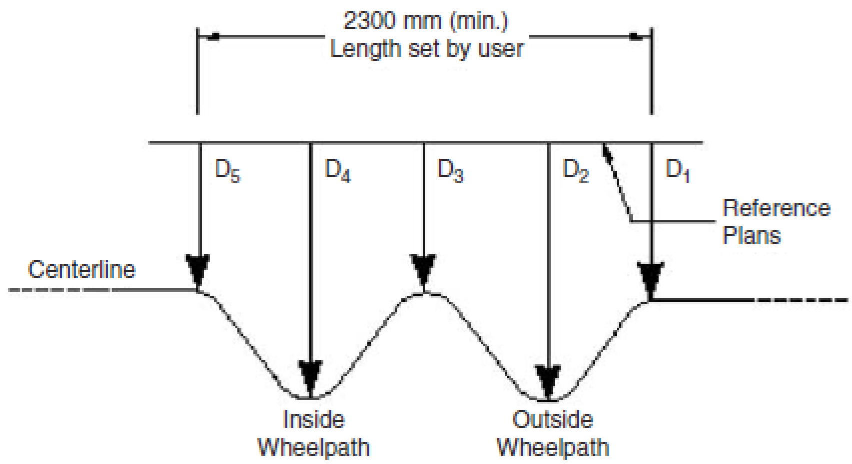


Figure C.3.2. Rut depth measurements. (Source: McGhee, 2004, and AASHTO, 2000)

(ii) International Roughness Index 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 the international roughness index (IRI) as the "pavement roughness index computed from a longitudinal profile measurement using a quarter-car simulation at a simulation speed of 80 kph (50 mph)." Further, **ASTM E1926** notes that "IRI is reported in either meters per kilometer (m/km) or inches per mile (in/mile)."

3.3 Analysis Tools

Some of the analysis tools available include allowable rut depths and recommended IRI levels, as shown in Tables C.3.1 through C.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 in. or greater. State DOTs such as the Washington State DOT (WSDOT) use a rehabilitation trigger level of 0.4 in. (10 mm). The Texas DOT (TxDOT; 1993) notes in its Hydraulic Design Manual that water depths of 0.2 in. or greater and Fwa (2006) notes that a rut depth of 0.5 in. or more can create the potential for hydroplaning. Thus, rut depths less than or equal to 0.5 in. appear to be a reasonable trigger-level range for rehabilitation decisions.

Table C.3.1. Typical Maximum Rut Depths

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

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

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

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

Ride Quality Terms	PSR	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

The IRI criteria used by the Federal Highway Administration (FHWA) have evolved as illustrated by review of Tables C.3.2 and C.3.3. In 1999, the most detailed breakdown, the criteria suggest that IRI values of less than 60 in./mile are quite good and values greater than 170 in./mile are poor. Interestingly, many newly paved HMA projects typically have IRI values close to the 60 in./mile value. Eventually, the FHWA simplified its criteria, as shown in Table C.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 in./mile acceptable as to ride quality (85% acceptable). The paper concluded that there was no evidence to change federal IRI guides (in essence, those shown in Table C.3.3).

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.

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.

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.

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

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. The device described is the Dynatest FWD. This device 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 these:

(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 the range of 20 to 60 ms with a rise time in range of 10 to 30 ms; (4) the loading plates standard sizes are 300 mm (12 in.) and 450 mm (18 in.); (5) the deflection transducers, which are used to measure the maximum vertical movement of the pavement, can be seismometers, velocity transducers, or accelerometers; (6) the load measurements must be accurate to at least ±2% or ±160 N (±36 lb), whichever is greater; (7) the deflection measurements must be accurate to at least ±2% or ±2 μm (±0.08 mils), whichever is greater (note that 0.08 mils = 0.00008 in. and 2 μm = 0.002 mm); and (8) a precision guide in ASTM D4694 notes that when a device is operated by a single operator in repetitive tests at the same location, the test results are questionable if the difference in the measured center deflection (D_0) between two consecutive tests at the same drop height (or force level) is greater than 5%. For example, if D_0 = 0.254 mm (10 mils), then the next load must result in a D_0 range less than 0.241 mm to 0.267 mm (9.5 to 10.5 mils).

(iii) Dynatest FWD

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

Locations for the seven velocity transducers vary. According to ASTM D4694, the number and spacing of the sensors is optional and depends on 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 the following distance (in inches) from the center of the load plate: 0, 8, 12, 24, 36, and 48.

The Strategic Highway Research Program (SHRP) sensor spacings with the 11.8-in. load plate uses the following distance (in inches) from the center of the load plate: 0, 8, 12, 18, 24, 36, and 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 itself on projects with large variations in surfacing layers. Deflections measured at the center of the test load combined with area values and E_{SG} computed from deflections measured at 24 in. from the center of the load plate are shown in the linear plot to provide a visual picture of the conditions found along the length of any project (as illustrated by data from a rural road in Figure C.4.1).

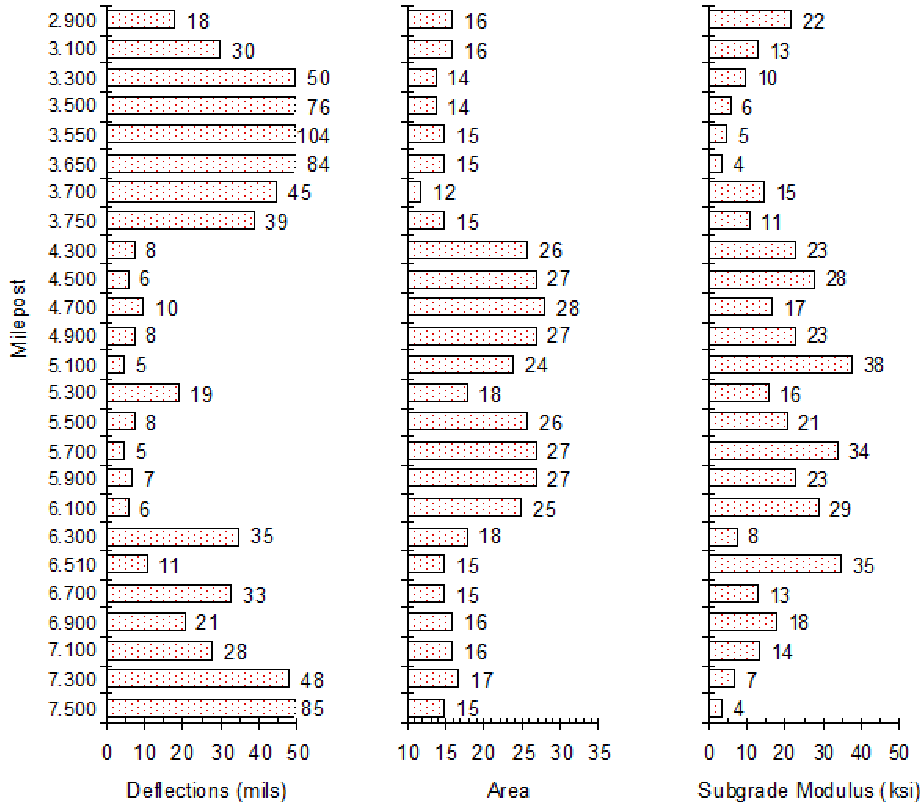


Figure C.4.1. Illustrations of FWD deflection data summarized by the three types of data.

The deflection data in Figure C.4.1 are “normalized” data in that the measured deflections are calculated for a 9,000 lb load. The modulus determination was based on the deflection of 24 in. from the center of the load plate.

Table C.4.1 provides general information about conclusions that can be drawn from the FWD parameters of area and D_0.

Table C.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 its temperature changes. D_0 ranges can be grouped into the following broad and approximate categories (Table C.4.2):

Table C.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 while 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 at center of test load,
D_1 = surface deflection at 1 ft,
D_2 = surface deflection at 2 ft, and
D_3 = surface deflection at 3 ft

The maximum value for the 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) = 36.0 in.

For all four deflection measurements to be equal (or nearly equal) would indicate 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). By 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 the 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 Table C.4.3.

Table C.4.3. Typical Area Values

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

(iii) Subgrade Modulus

An NCHRP study (Darter et al., 1991) which revised Part III of the American Association of State Highway and Transportation Officials (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 = *backcalculated* 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 (in.), and

r = distance from center of load plate to D_r (in.).

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 (in.),
D = total thickness of pavement layers (in.)
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 Table C.4.4 (after Chou et al., 1989).

Table C.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 Examples of Analyses of FWD Deflection Basins for Flexible Pavement

The following deflection basins shown in Table C.4.5 were obtained with a Dynatest FWD. The pavement temperature at the time of testing was 46°F (8°C). The deflection basins for the four FWD drops and normalized to 9,000 lb are shown in the table.

Table C.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 as follows:

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

(i) Requirements

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

(ii) Results

Maximum surface deflection

The maximum surface deflection = 0.01657 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)

$$M_R = P(1-\mu^2)/(\pi)(D_r)(r)$$
$$= 9000(1-0.45^2)/(\pi)(0.00418)\ (36)$$
$$\cong 15{,}200 \text{ psi}$$

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

The pavement subgrade modulus for an 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) Detailed Project Data Example

Table C.4.6 summarizes deflection data that were collected on a portion of an actual project. The project was about 5 miles in length, and FWD testing was performed every 250 ft, but only four of the FWD locations are shown (these locations were also coring sites). The average pavement temperature at the time the FWD data were collected was 46°F to 50°F. The timing of the survey was about 1.5 to 2 months after the spring thaw in this area.

As shown in Table C.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 and 5.3 in. with an average of 5.2 in., which constitutes a "medium" thickness of HMA (refer back to Table C.4.2).

The area values shown in the table suggest weak HMA, but not necessarily extreme weakness due to stripping. Table C.4.7 illustrates typical theoretical area values for various uncracked HMA thicknesses, which aids this type of comparison.

Table C.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 C.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 this range (above normal stiffness), or below this range (below normal stiffness). This comparison is shown in Table C.4.8.

Table C.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

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 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 by using measured deflection data. The concept of joint load transfer efficiency is illustrated in Figure C.4.2. Load transfer efficiency can be calculated using the following equation:

$LTE = (\Delta_U/\Delta_L)(100)$ = load transfer efficiency (%)

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 is illustrated in Figure C.4.3.

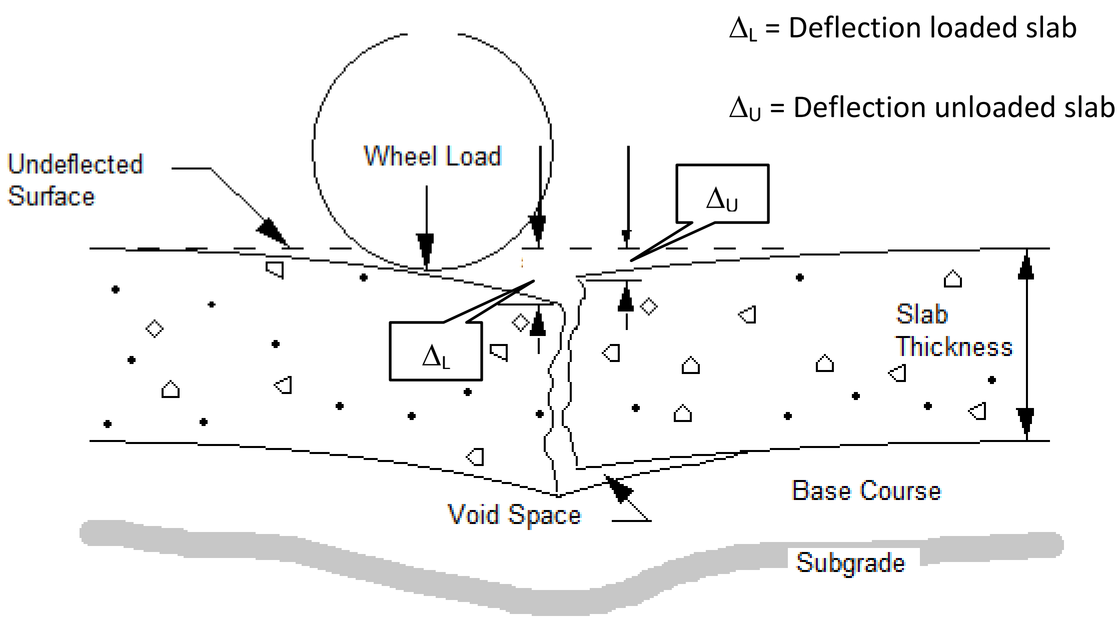


Figure C.4.2 Illustration of joint load transfer efficiency.

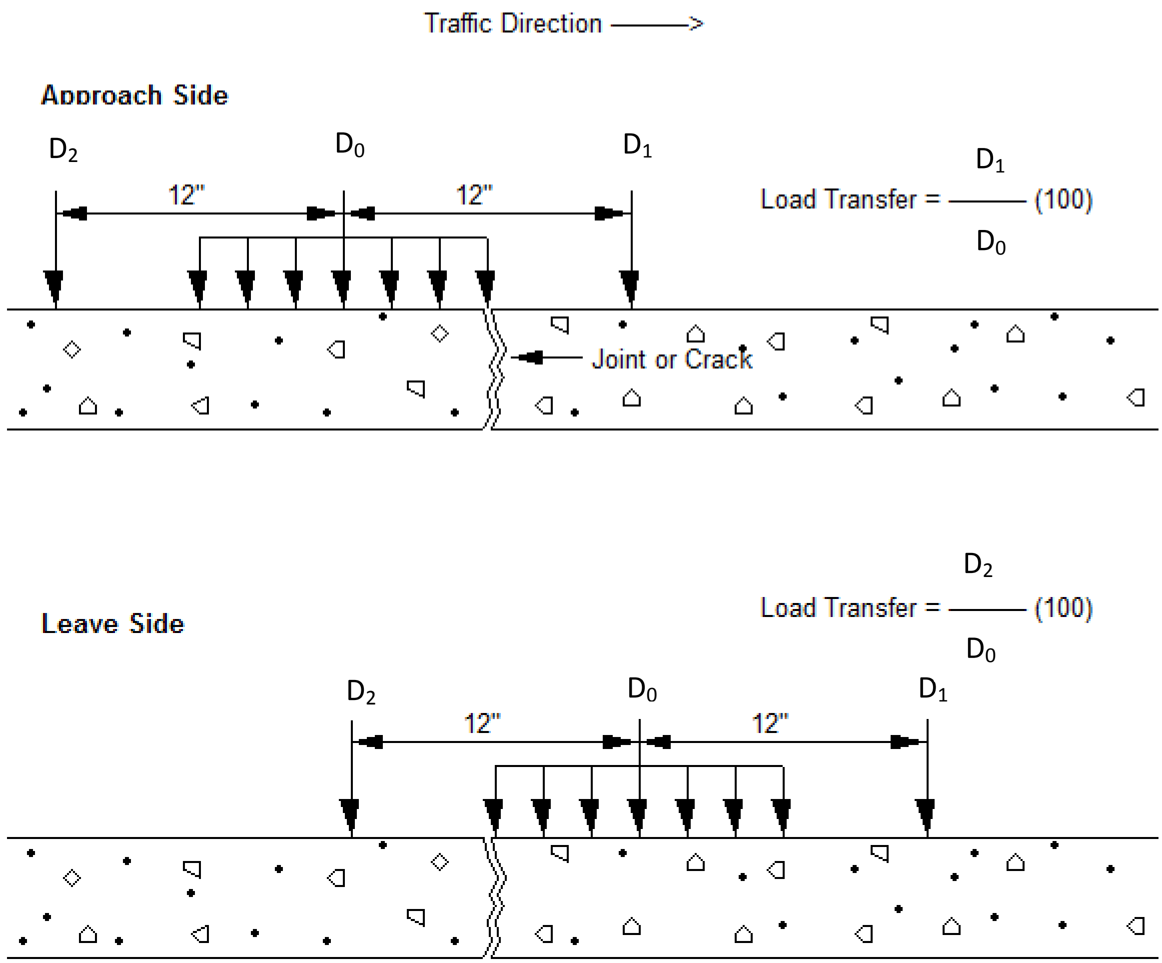


Figure C.4.3. Locations of FWD load plate and deflection sensors for determining load transfer efficiency.

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

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 United States. It is likely that within a specific state there is a preferred backcalculation software package to use.

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

(i) Number of Layers

Generally, use no more than three or four layers of unknown moduli in the backcalculation process (preferably, no more than three layers). If a three-layer system is being evaluated, and questionable results are being produced (weak or low stiffness base moduli, for example), it is sometimes advantageous to evaluate this pavement structure as a two-layer system. This modification would possibly indicate that the base material has been contaminated by the underlying subgrade and is weaker because of 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.

(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 of (or the potential for) 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 results in low base moduli. There are a number of reasons this can occur. One, a thin base is not a "significant" layer under a stiff, thick layer. Second, the base modulus may be relatively "low" because of 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. This is particularly true if a stiff layer is within a depth of about 20 to 30 ft 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 say 20, 30, or 50 ft if no stiff layer is otherwise indicated (i.e., use a semi-infinite depth for the subgrade). The depth to stiff layer should be verified whenever possible with other NDT data or borings.

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% 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 say 75 to 80°F).

Base and Subbase Moduli. Typical base and subbase moduli are shown in Table C.4.9.

Table C.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>>1,250</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 C.4.4.

(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.

4.4 References

AASHTO (1993), “AASHTO Guide for Design of Pavement Structures, 1993,” American Association of State Highway and Transportation Officials, Washington, D.C.

Chou, Y. J., Uzan, J., and Lytton, R. L. (1989), "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, 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.

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 the following:

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 ground penetrating radar unit is shown in Figure C.5.1. The radar antenna is attached to a fiberglass boom and suspended about 5 ft. from the vehicle and 14 in. above the pavement. This particular GPR unit can operate at highway speeds (70 mph); it transmits and receives 50 pulses per second and can effectively penetrate to a depth of around 20 to 24 in. All GPR systems include a distance measuring system, and many of the new systems also have synchronized/integrated video logging, so the operator can view both surface and subsurface conditions. GPS is also included in many new systems for identifying problem locations.

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

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

Figure C.5.1. Air-coupled GPR systems for highways.

(ii) Ground-Coupled GPR Systems

As shown in Figure C.5.2, a whole range of different operating frequencies is available for ground-coupled GPR systems. The selection of the best frequency for a particular application depends on the required depth of penetration. As the name implies, these antennas have to stay in close contact to the pavement under test.

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

Figure C.5.2. Ground-coupled systems, 1.5 GHz on left, lower frequency antennas with control unit on right.

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 electromagnetic (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 one trace for every 2 to 3 ft of travel. The captured return signal is often color coded and stacked side by side to provide a profile of subsurface conditions; this is analogous to an "X-ray" of the pavement structure. Examples of this 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 C.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.

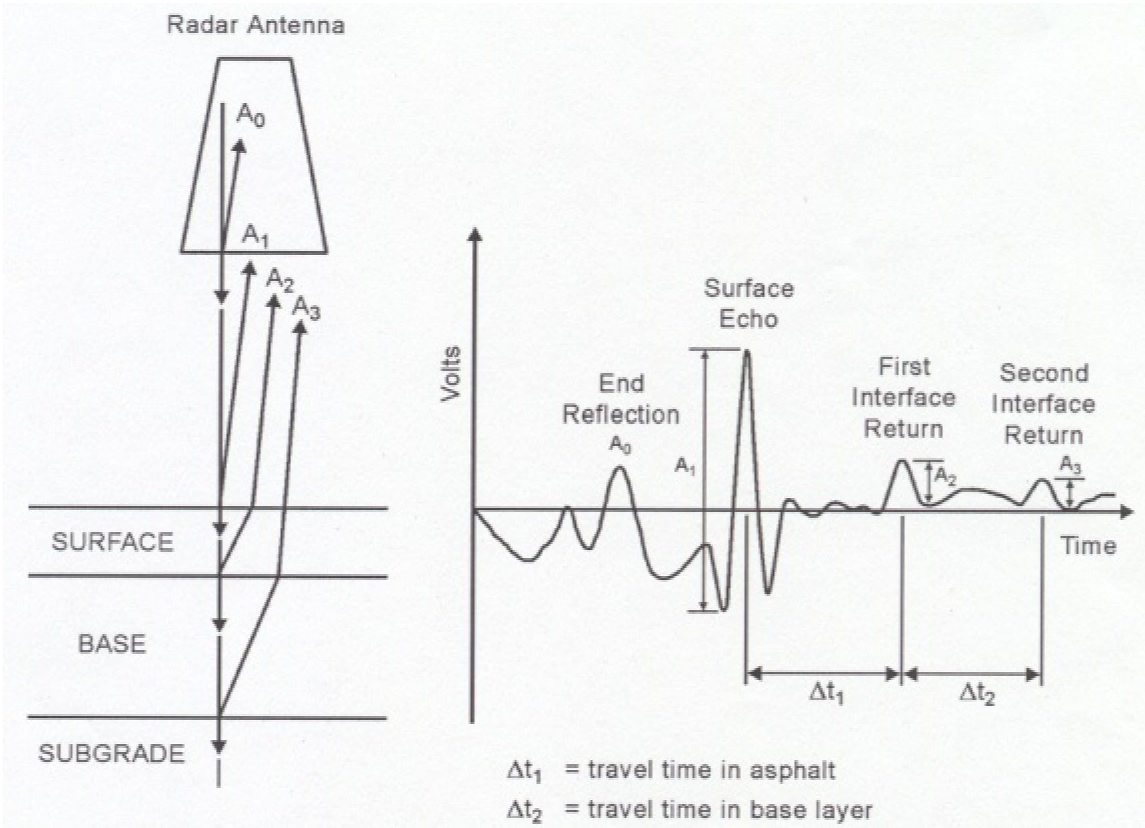


Figure C.5.3. Captured GPR reflections from a typical flexible pavement.

The reflection A_0 is known as the end reflection; it is an internally generated system noise that 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 that is most influenced by moisture content and density; it also governs the speed at which the GPR wave travels in the layer. An increase in moisture will cause an increase in layer dielectric. In contrast, an increase in air void content will cause a decrease in layer dielectric.

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

$$\in_a = \left[\frac{1 + A_1 / A_m}{1 - A_1 / A_m} \right]^2 \qquad \text{(Eq 1)}$$

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 C.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 as in Figure C.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 then stack them side by side so that a subsurface image of the pavement structure can be obtained. This approach is shown in Figure C.5.4.

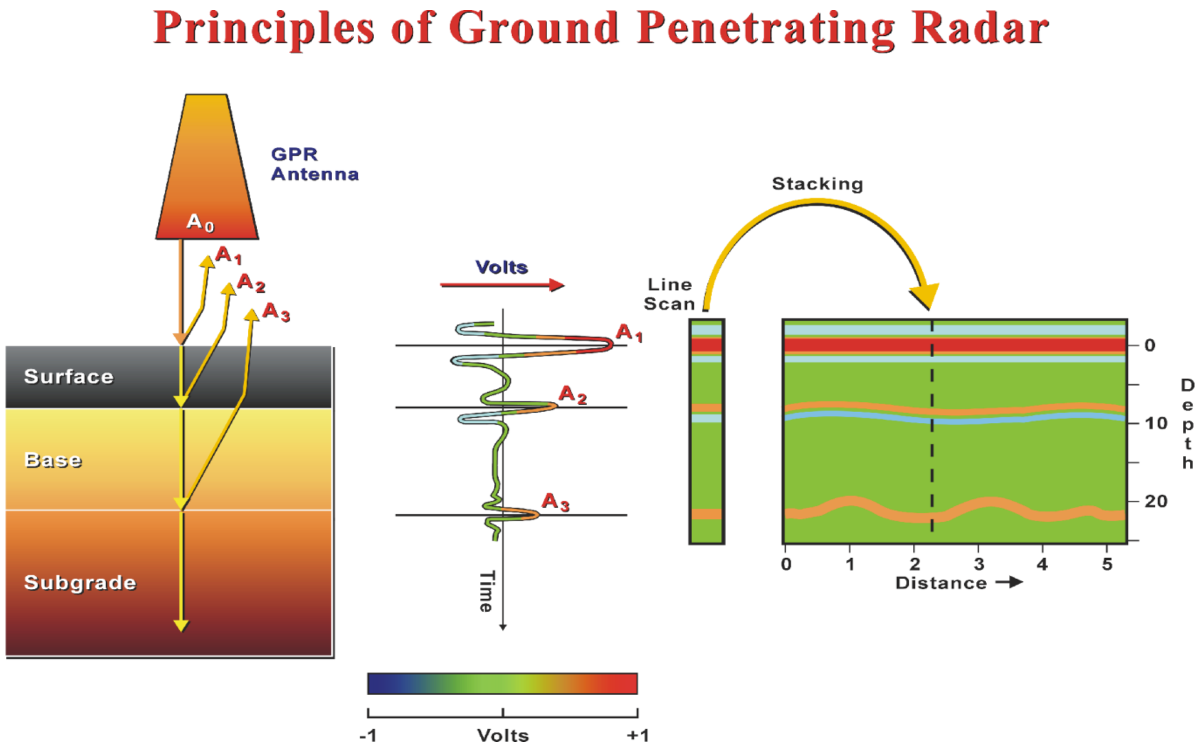


Figure C.5.4. Color coding and stacking individual GPR images.

The raw GPR image collection is displayed vertically in the middle of Figure C.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 by using the color scheme in the middle of Figure C.5.4. In the current scheme the high positive reflections are colored red and the negatives are colored blue. The green color is used when 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 in. deep of a thick flexible pavement is shown in Figure C.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.

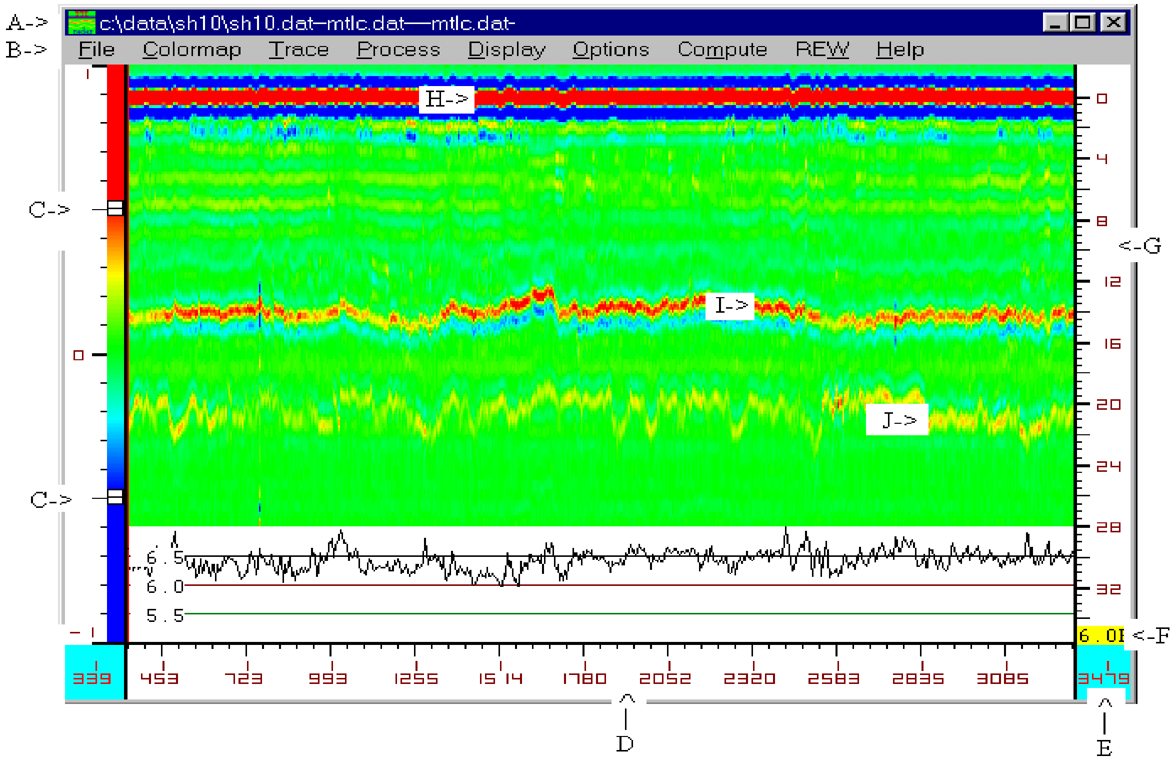


Figure C.5.5. Color-coded GPR traces.

The labels on Figure C.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 (1 mile and 3,479 ft), G = depth scale in inches, with the zero (0) being the surface of the pavement, and F = default dielectric value used to convert the measure time scale into a depth scale. The important features of this figure are the lines marked H, I, and J. These lines are the reflections from the surface, top, and bottom of the base respectively. This pavement is homogeneous, and the layer interfaces are easy to detect.

When processing GPR data, the first step is to develop displays such as Figure C.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 grayscale for the pavement shown in Figure C.5.5 is shown in Figure C.5.6.

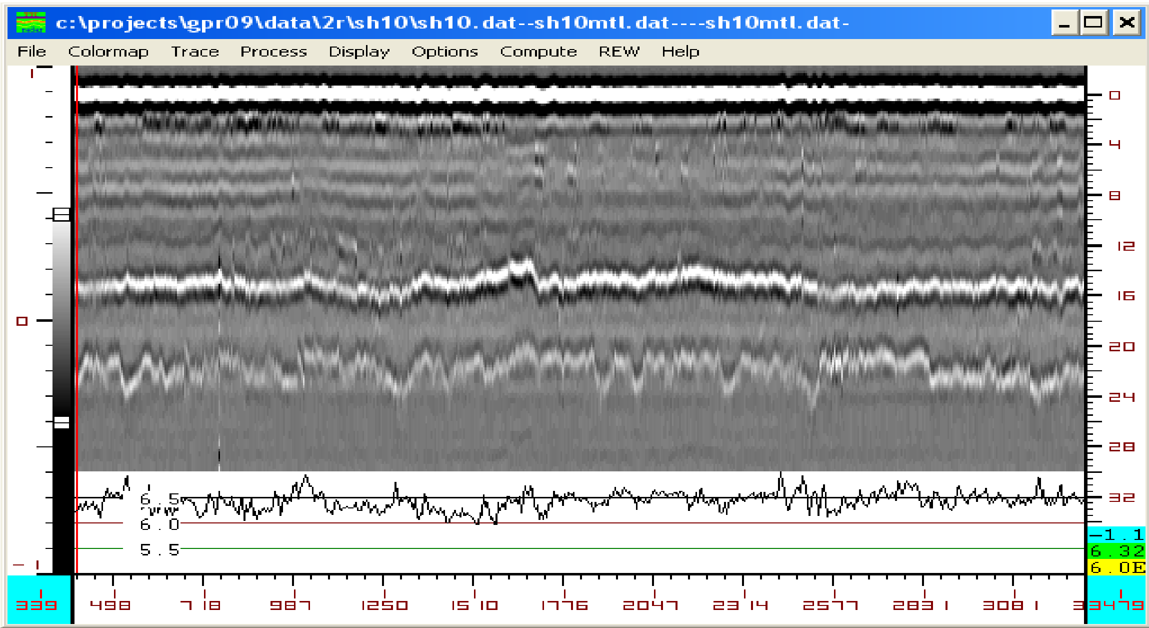


Figure C.5.6. Similar data to Figure C.5.5 presented as a grayscale.

All of the commercially available software packages produce both a color display of subsurface conditions, such as Figures C.5.5 and C.5.6, together with a table of computed layer thicknesses and dielectrics that is usually exported to Excel. A typical table is shown in Figure C.5.7, where E1 and Thick 1 are the top layer dielectric and thickness.

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 C.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; most DOTs often have poor 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 old 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, several major problems have occurred during construction, or poor pavement performance has occurred because of the failure to account for the variability of the existing pavement in the design phase. Lab designs are based on testing at localized sampling locations; sections that are either too thick or too thin have been documented to cause problems. GPR can help in this area.

It also must be recalled that processing FWD data as described in Chapter 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 up-front 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 designs, samples are taken from the existing pavement and taken back to 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 C.5.8 shows variations in asphalt layer thickness for an FDR candidate. At the sample location the structure was 5 in. of asphalt and 10 in. of granular base. Based on lab test results, the plan was to recycle to a depth of 10 in. blending 50% base with 50% existing base with 3% cement. However, from a review of Figure C.5.8, the 5 in. of HMA is common on this highway with several noticeable exceptions. The first 800 ft only has 3 in. of asphalt, and this is not thought to be a concern. However, for about 2,000 ft of this project the total HMA thickness is more than 12 in. From previous experience the 3% cement treatment does not work with 100%

RAP. In these locations it will be necessary to modify the construction plan. As a result, 5 in. of the existing HMA was milled and replaced with 5 in. of new flexible base. In that way the FDR process can continue, and in all locations the as-designed 50/50 blend can be treated with cement.

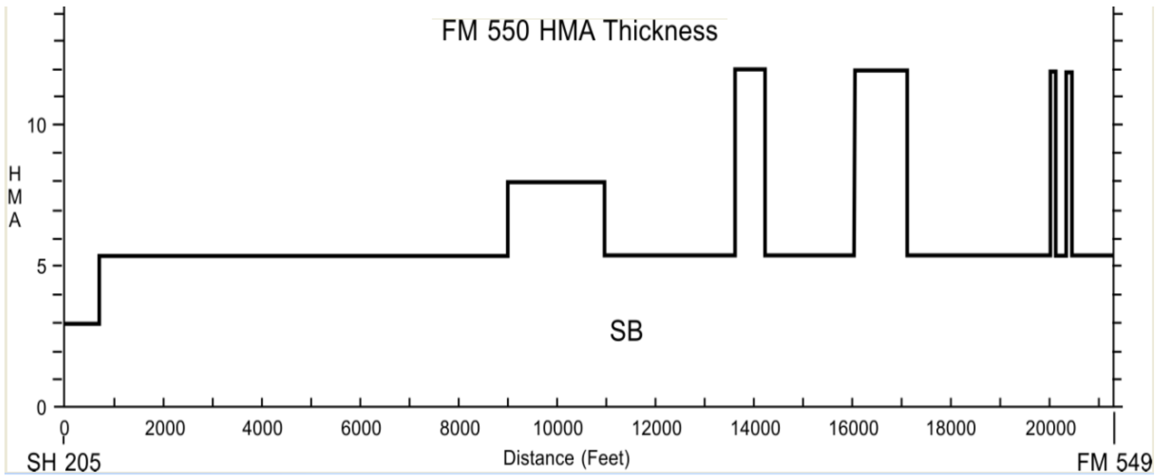


Figure C.5.8. Surface thickness variations from GPR profiling on FM 550.

(ii) Defect Detection Prior to Pavement Rehabilitation

In many cases the 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 exiting base layer is holding moisture. It must be recalled that GPR traces are collected frequently at 2 to 3 ft intervals, so very precise location of deflects is possible. The GPR color-coded profile shown in Figure C.5.5 is from a thick HMA section with no defects. This should be contrasted with the GPR profile shown in Figure C.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.

The presence of defects in either HMA or base layers can be easily detected by GPR, and their severity will then need to be confirmed by localized coring. This is valuable input to the pavement designer who has to make a decision as to whether these defects 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.

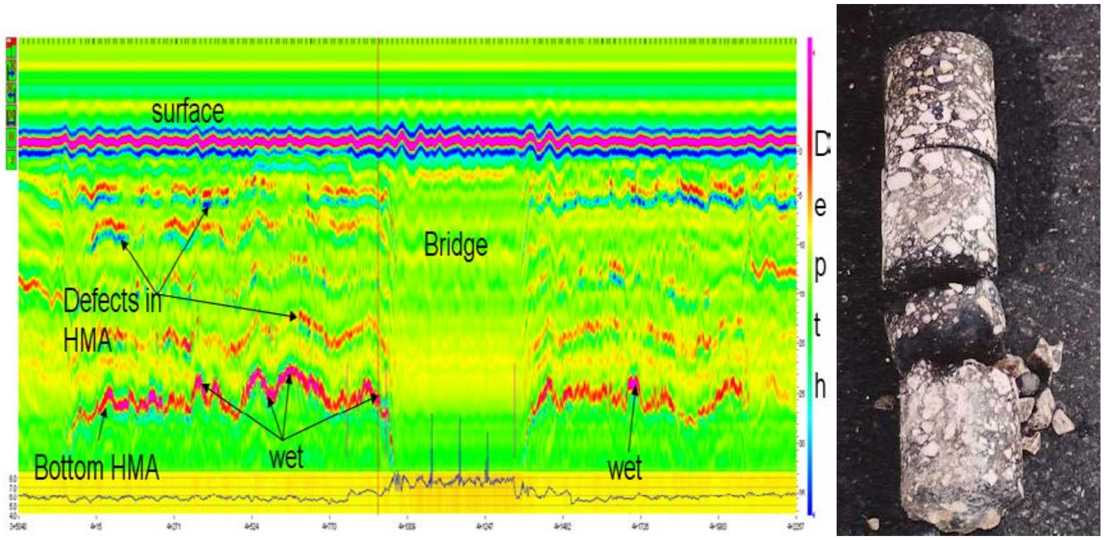


Figure C.5.9. Using GPR to identify defects in surface and base layers.

(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 in Figure C.5.10. This is a 1.8-mile section. The entire section had all received a thin overlay and so surface condition was very similar. However, the first part of the section was performing poorly. A GPR surface was undertaken, and from the display it is clear that this section has 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

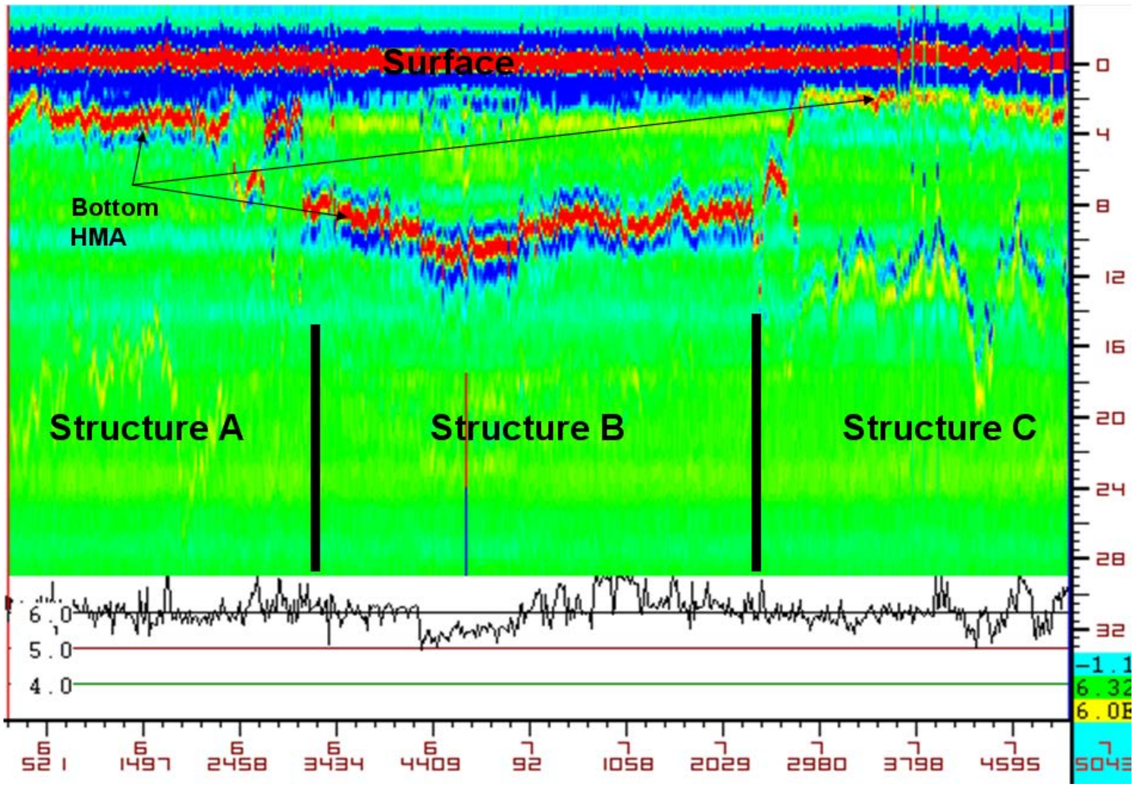


Figure C.5.10. Using GPR to map subsurface variability.

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 performed better than the air coupled. The high frequency ground-coupled systems such as the 1.5 GHz unit shown in Figure C.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 (2006) handbook on radar inspection of concrete has some very good examples of rebar detection. Figure C.5.11 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.

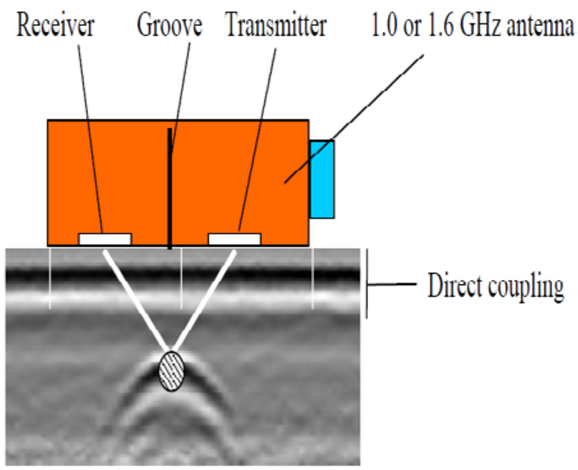


Figure C.5.11. Ground-coupled GPR signals from steel in concrete (GSSI, 2006).

By moving the GPR antenna slowly across the surface of the concrete, it is possible to map different layers of steel and the bottom of the concrete slab as shown in Figure C.5.12.

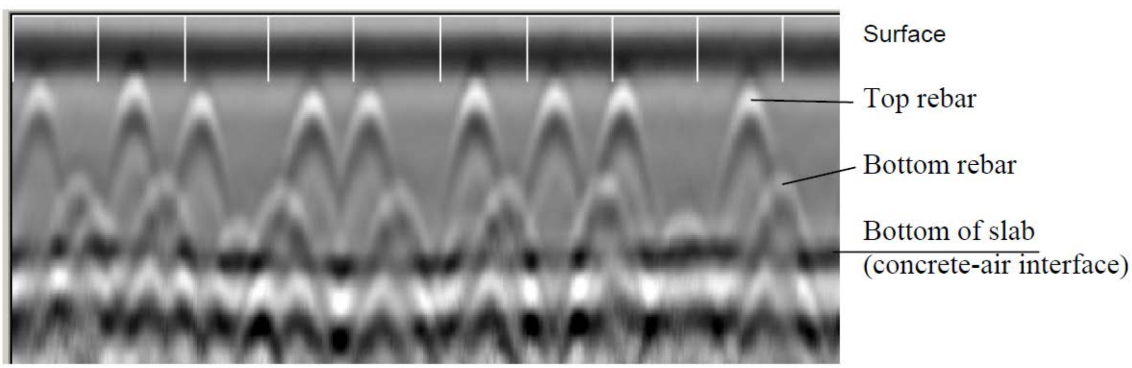


Figure C.5.12. Mapping multiple layers of steel in concrete (GSSI, 2006).

(ii) Void Detection

Detecting thin air voids with air-coupled GPR is often problematic, and furthermore, even very thin voids are very detrimental to slab performance. Controlled studies have found that air voids of less than 0.75 in. thick cannot be readily detected with air-coupled GPR. However, if the voids are larger or if they are moisture filled, then they can readily be detected. An example of a GPR color profile for an 8-in. PCC slab with water-filled voids is shown in Figure C.5.13. The strong reflections (red areas) are locations of trapped water.

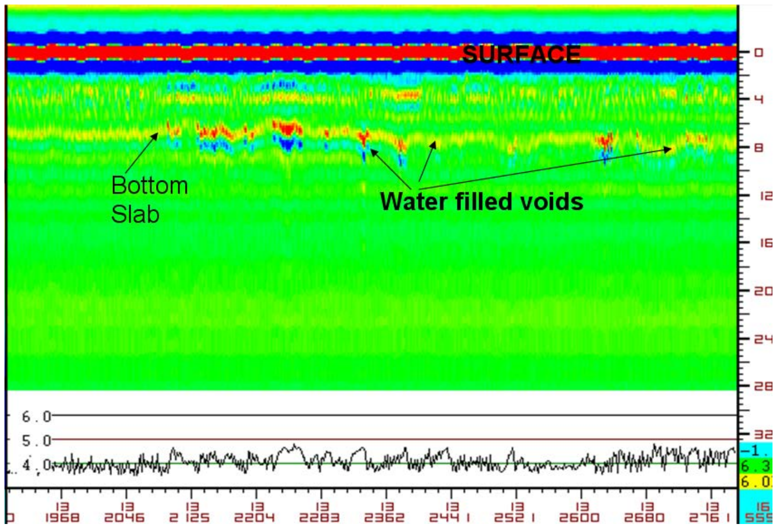


Figure C.5.13. Mapping subslab water-filled voids with GPR.

(iii) Deep Investigations of Subslab Conditions with GPR

The lower frequency ground-coupled GPR can be used to investigate deeply beneath concrete pavements to identify changes in support conditions and possibly to help explain the occurrence of surface distress. Figure C.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 toward the top of the figure. The steel is more closely spaced in the left of the figure. The anomaly on the left is a culvert. The bottom of the slab is indicated. There is a clear change in subgrade support at the top of the subgrade showing the transition from a cut to a fill area.

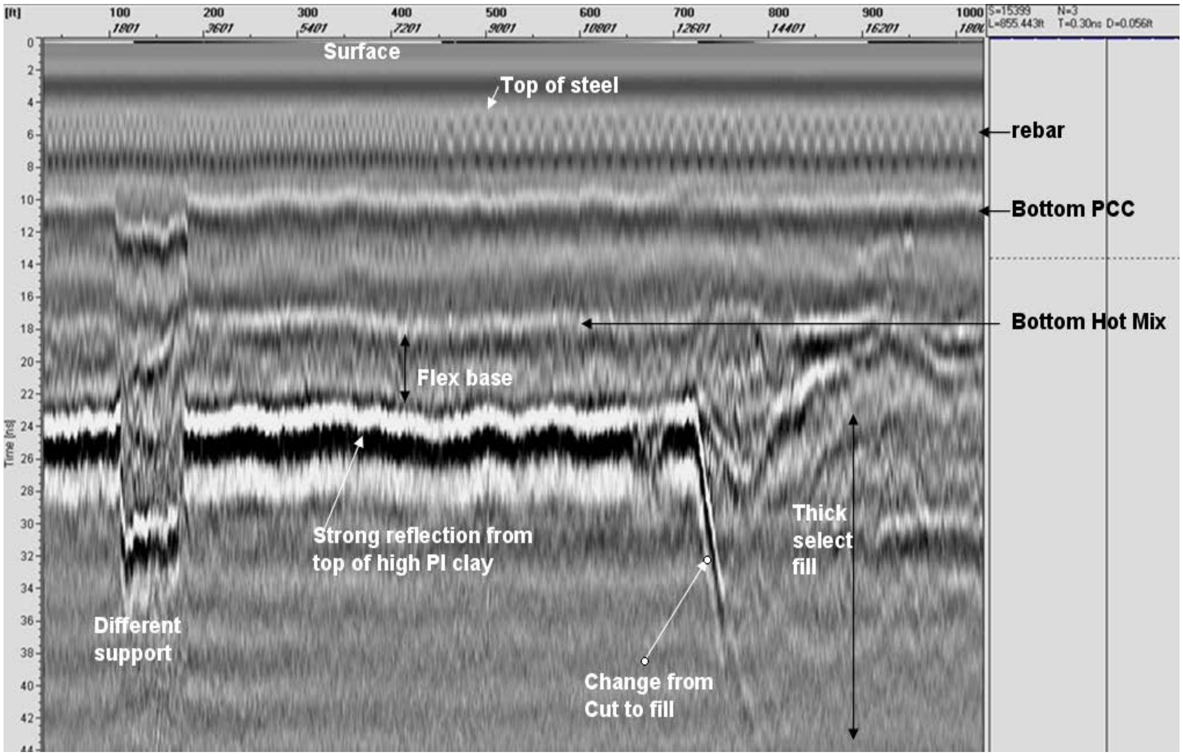


Figure C.5.14. Mapping concrete pavements structure with GPR.

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 painfully slow. There are several factors causing this, and these will be discussed in this section, but the main factors are the following;

1. The FCC ban on 1 GHz air-coupled systems in 2002 (these units can be purchased in any country worldwide except the United States). For the past decade most air-coupled GPR systems have been performed with systems built before 2002. Only recently have commercial systems become available such as GSSI's 2.2 GHz as shown in Figure C.5.1.
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 that 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 and consultants have implemented GPR technology in-house (for example, the Florida DOT, TxDOT, and others), 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 its equipment against the performance specs (which are available).
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 the following:

a. *GPR is only for layer thickness determination. My state has good as-built records so we do not need GPR.*
As stressed throughout this report, GPR is much more than 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 vehicle. Ground-coupled systems cost approximately $60,000. With the cost of pavement rehabilitation activities, these costs are minimal with the cost of rehabilitating sections of Interstate pavement. GPR costs substantially less than other nondestructive testing equipment such as FWDs.

c. *GPR is a black box that is impossible to understand.*
This is not true. The basics of GPR are very simple. The key here is that agency personnel should attend training schools to get 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 about 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 first training end user agency personnel. 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 Reference

GSSI (2006). "GSSI Handbook for Radar Inspection of Concrete," www.geophysical.com, August.

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 stems 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 cores not only reveal much about the existing pavement structure but also allow for use of the dynamic cone penetrometer. Knowing the HMA layer thickness to within one-fourth inch is essential in ensuring a more accurate prediction of layer moduli if a backcalculation procedure is used.

The number of cores obtained will depend on project specific conditions; however, a reasonable rule-of-thumb is to obtain a core at every 5th or 10th FWD test location. If the pavement thicknesses are found to vary substantially (not probable but this can be the case), then cores should be obtained at every FWD test location.

Typical core diameters are either 4 or 6 in.

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 C.6.1. 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 C.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

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

> 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. (ASTM D6951)

A sketch of a standard DCP is shown in Figure C.7.1.

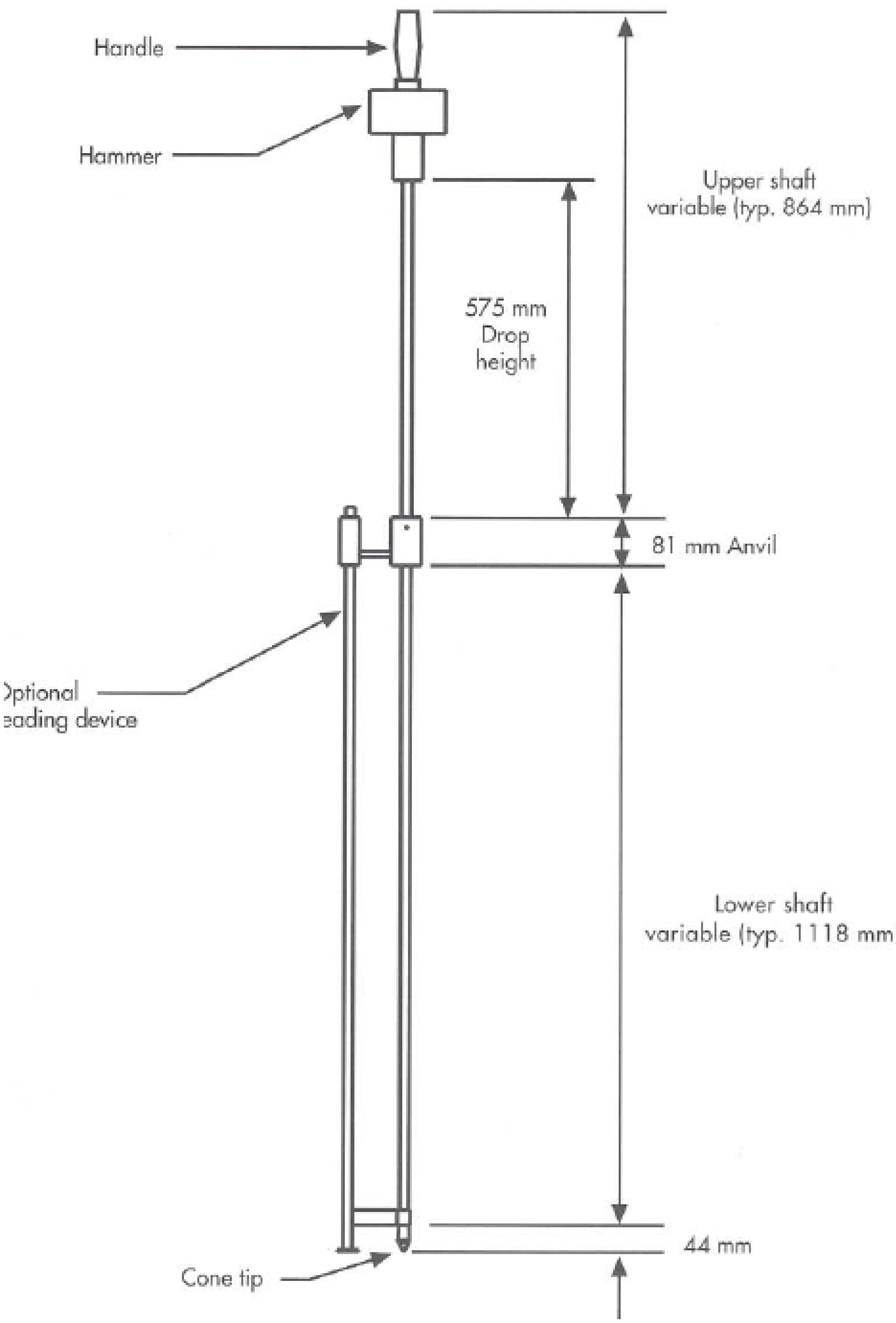


Figure C.7.1. Sketch of the MnDOT DCP (from MnDOT, 1993.)

7.3 Analysis Tools

DCP test results are typically expressed in terms of the 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 (MnDOT, 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 as follows:

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

where DPI = mm/blow

Table C.7.1 shows typical CBR and DPI ranges for three soil types [Minnesota DOT (MnDOT) 1993).

Table C.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

Note: 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 C.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 C.7.2. Minnesota DCP Results Following Placement of the Base Course

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

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

(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 C.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 MnDOT broadly agree in that subgrade DPI values greater than 25 mm/blow are of concern.

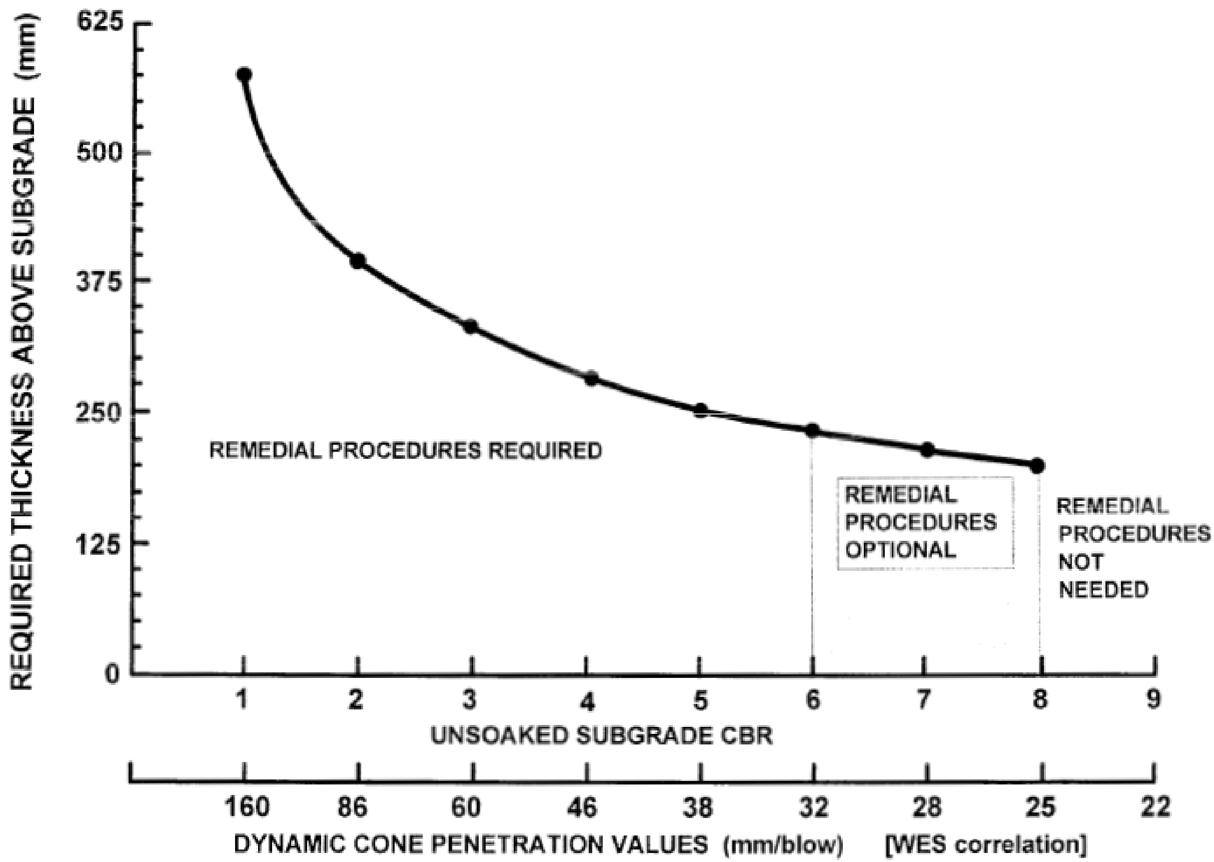


Figure C.7.2. DCP-based thickness design for granular backfill and subgrade modification for the Illinois DOT (figure from Burnham, 1997, but checked against Illinois DOT, 2005).

(iv) Use of DCP Data in Renewable Decisions

The Texas Transportation Institute developed guidelines for the TxDOT as to conditions suitable for rubblizing existing rigid pavements (Figure C.7.3). The high risk portion of the figure implies that the pavement is not a good candidate for rubblization since the supporting base and subgrade is excessively weak. Figure C.7.3 is similar to but modified from similar guidelines developed for Illinois (Figure C.7.4). Figure C.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 7.3(i), which was originally done by the U.S. Army Corps of Engineers.

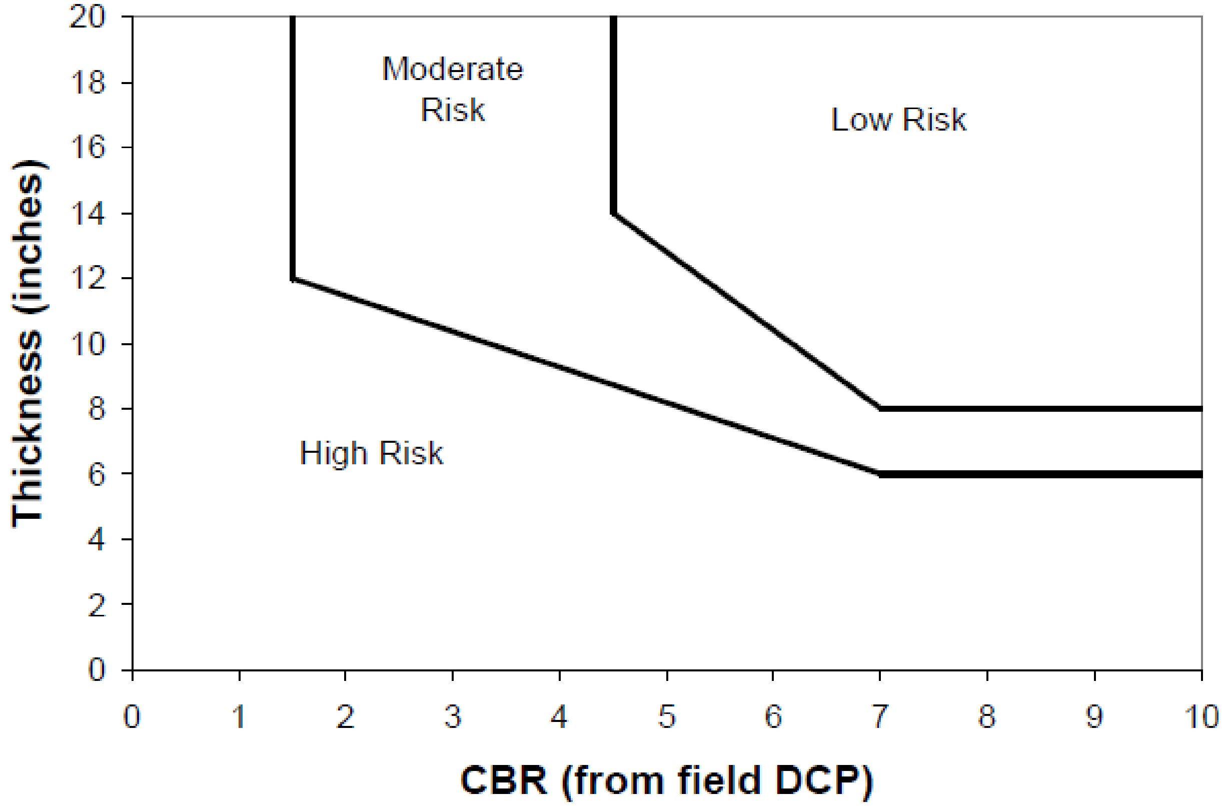


Figure C.7.3. Rubblization selection chart developed by the Texas Transportation Institute. (Source: Sebesta and Scullion, 2007)

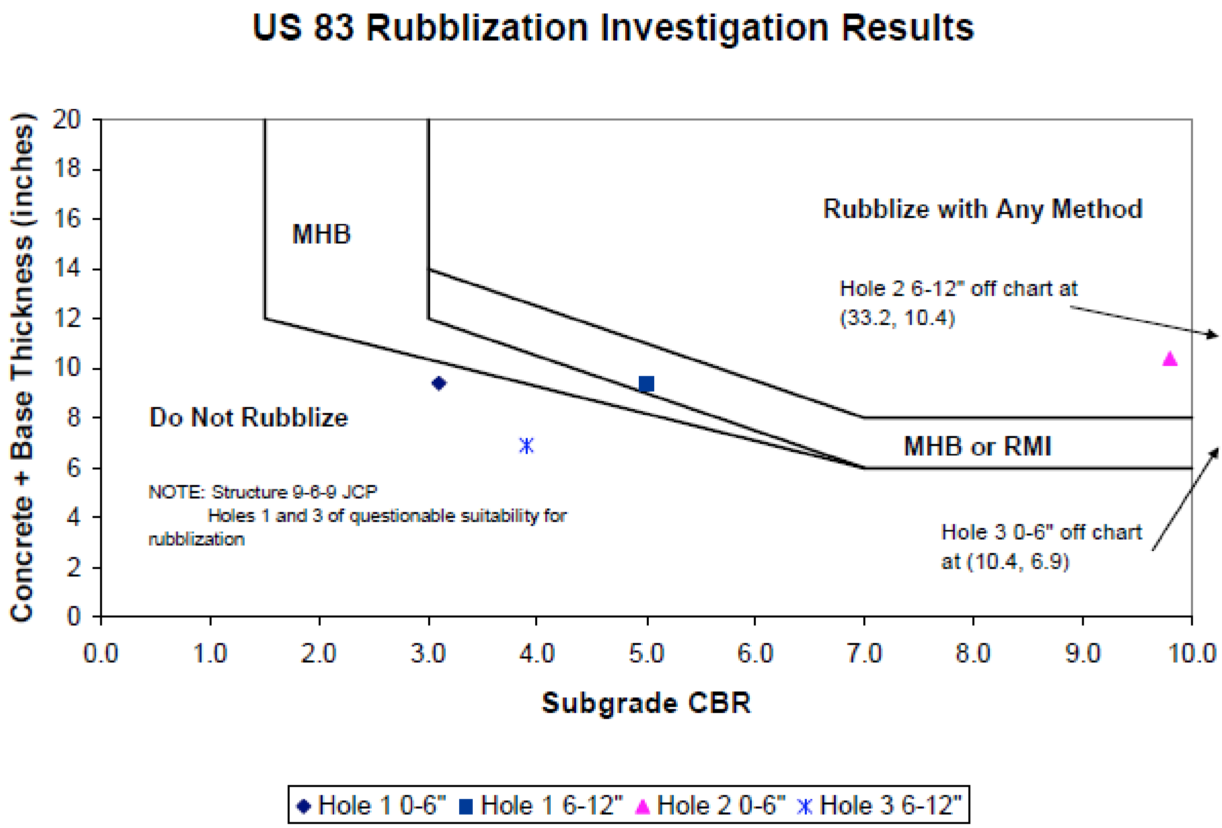


Figure C.7.4. Illinois rubblization selection chart with data from US 83 (Texas). (Source: Sebesta and Scullion, 2007; original Illinois DOT criteria from Heckel, 2002)

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.

Heckel, L. (2002), Rubblizing with Bituminous Concrete Overlay—10 Years' Experience in Illinois, Report IL-PRR-137, Illinois Department of Transportation, April. [The report contains an appendix entitled Guidelines for Rubblizing PCC Pavement and Designing a Bituminous Overlay. This appendix contains a similar version of Figure C.7.4.]

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.

MnDOT (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.

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.

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 stems 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 C.8.1.

Table C.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 careful 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 about 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 C.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 that can aid decisions about the potential frost susceptibility of a subgrade soil is to use the U.S. Army Corps of Engineers classification system for frost design (NCHRP Synthesis 26, 1974), as shown in Table C.8.2.

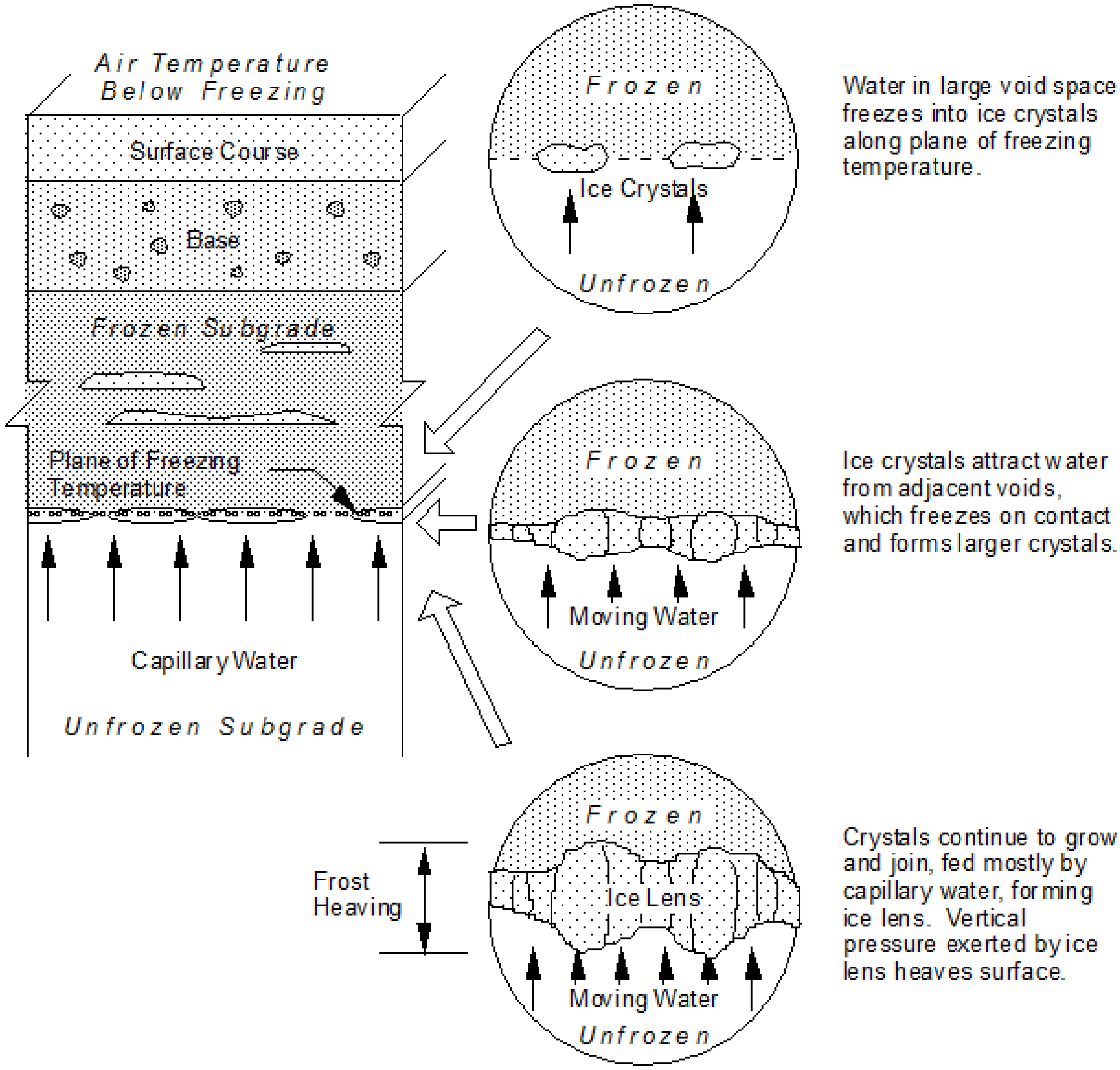


Figure C.8.1. Formation of ice lenses in a pavement structure.

Table C.8.2. U.S. Army Corps of Engineers Frost Design Soil Classification

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

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

(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

Groundwater issues, if apparent, may require the geotechnical engineer to sort them out.

8.4 References

Terzaghi, K., and Peck, R. (1967), "Soil Mechanics in Engineering Practice," John Wiley and Sons.

"NCHRP Synthesis of Highway Practice 26: Roadway Design in Seasonal Frost Areas," Transportation Research Board, National Research Council, Washington, D.C., 1974.

U.S. Army (1990), "Design of Aggregate Surfaced Roads and Airfields," Technical Manual TM 5-822-12, Department of the Army, September.

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).

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 C.9.1, which illustrates single and tandem axles with either single or dual tires.

States generally have regulations limiting allowable load per inch width of tire. 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.

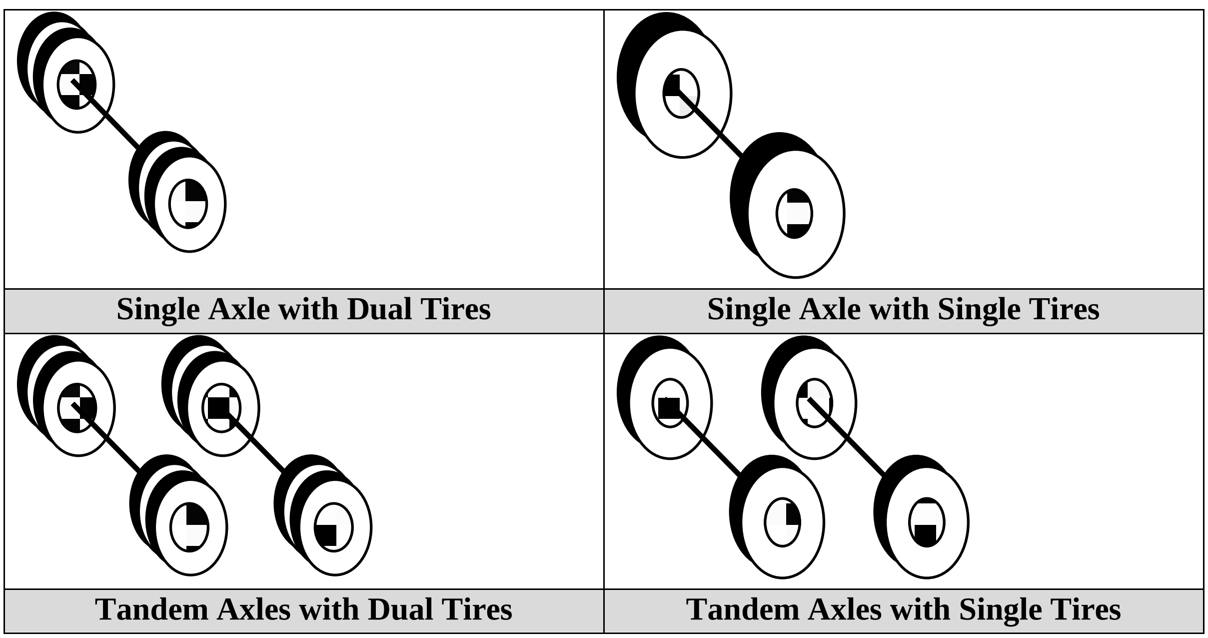


Figure C.9.1. Illustration of typical axle and tire configurations.

(ii) Typical Federal and State Axle Load Limits

Typical federal and state axle load limits are these:

- 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 U.S. trucks and buses is the FHWA Bridge Formula. The total gross weight in pounds imposed on the pavement by any group of two or more consecutive axles on a vehicle or combination of vehicles shall not exceed that weight calculated by use of Equation 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 as 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 that bridges are not "overstressed" by an almost infinite number of truck-axle configurations and weights.

$$W = 500(NL/(N-1) + 12N + 36) \qquad \text{Eq. 9.1}$$

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

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

Wheel load equivalency has been one of the most widely adopted results of the AASHO Road Test (1958 to 1960), i.e., 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 is shown in Table C.9.1.

Table C.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, load equivalency factors (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 C.9.2.

Table C.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 handlebars rather than wheels. This category includes motorcycles, motor scooters, mopeds, motor-powered bicycles, and three-wheel motorcycles. This vehicle type may be reported at the option of the state.
Class 2	*Passenger Cars*—All sedans, coupes, and station wagons manufactured primarily for the purpose of carrying passengers and including those passenger cars pulling recreational or other light trailers.
Class 3	*Other Two-Axle, Four-Tire Single Unit Vehicles*—All two-axle, four-tire, vehicles, other than passenger cars. Included in this classification are pickups, panels, vans, and other vehicles such as campers, motor homes, ambulances, hearses, and carryalls. Other two-axle, four-tire single unit vehicles pulling recreational or other light trailers are included in this classification.
Class 4	*Buses*—All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires or three or more axles. This category includes only traditional buses (including school buses) functioning as passenger-carrying vehicles. All two-axle, four-tire single unit vehicles. Modified buses should be considered to be a truck and be appropriately classified.
Class 5	*Two-Axle, Six-Tire, Single Unit Trucks*—All vehicles on a single frame including trucks, camping and recreational vehicles, 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 C.9.2 is to group all truck and bus traffic into three groups or units as shown in Table C.9.3.

Table C.9.3. Simplified Truck and Bus Groups

Simplified Vehicle Categories	Groupings of FHWA Vehicle Classes
Single units, which includes	(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, which includes	(i) 4 axle single trailer (FHWA Class 8) (ii) 5 axle single trailer (FHWA Class 9) (iii) 6+ axle single trailer (FHWA Class 10)
Multitrailers, which includes	(i) 5 axle multitrailer (FHWA Class 11) (ii) 6 axle multitrailer (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 C.9.4 shows typical ESALs per vehicle according to the groupings in Table C.9.3. The ESALs per vehicle were developed by a state DOT and appear to be typical for U.S. truck traffic. They may appear to be low, but the values are averages that include empty backhauls.

Table C.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 Multitrailers	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, 500 trucks with multitrailers, 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%).

The approximate annual ESALs were estimated for various levels of ADT and are shown in Table C.9.5. The primary (and major) assumptions were that the typical truck (average over FHWA Classes 4 through 13 based on actual weigh data) has an average ESAL = 1.0. Further, the range of truck percentages as a function of ADT is typically 5% to 10%. Given these assumptions, Table C.9.5 can be used as a guide.

Table C.9.5. Approximate Annual ESALs as a Function of ADT

Average Daily Traffic	Approximate Annual ESALs
10,000	250,000
25,000	750,000
50,000	1,500,000
100,000	2,500,000

The annual ESALs can vary significantly from the values in Table C.9.5. Examples include routes with large numbers of buses, which generally have higher ESALs per vehicle than an average truck (often by a factor of two to three or more). The percentage of trucks in the total traffic can vary significantly and particularly so in the vicinity of seaports. Further, rural Interstate routes typically have higher percentages of trucks than do urban Interstates along with differing mixes of trucks. Urban areas will have higher single unit truck percentages than do rural locations.

9.4 Reference

AASHTO (1993), “AASHTO Guide for Design of Pavement Structures, 1993,” American Association of State Highway and Transportation Officials, Washington, D.C.

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 involved in 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), ADT can be used and hourly traffic volumes can be developed by multiplying ADT by typical hourly distribution factors for the type of roadway being analyzed.

3. *Estimate the fraction of traffic that will cancel trips and the fraction of traffic that will use detour routes during the construction*. At best, these will be rough estimates unless more 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 and 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 and result in low overall productivity. In some scenarios, it may not be possible to make any meaningful progress in a short nighttime closure. Even if partial night closures cannot be used for mainline paving, they are often useful for prepaving work (e.g., PCC panel sawcutting, restriping lanes, milling HMA).
    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 with each roadway direction still open although with reduced capacity in at least one direction. These closures are often the first considered for lightly trafficked roadways when 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 during 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 short 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 important, 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 sometimes lighter than weekday traffic.

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

5. *Model traffic using the tool of choice (see Analysis Tools Section 10.3).* 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 by 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 C.10.1). This table is in 1996 dollars and should be adjusted to current dollars by using the consumer price index (CPI). A simple calculator is available from the U.S. Bureau of Labor Statistics at http://www.bls.gov/data/inflation_calculator.htm. Multiplying these values by total delay for each class of vehicle gives an estimate of total work zone user delay cost.

Table C.10.1. Recommended Values of Time (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 et al., 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 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 in which 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, if 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, the presence of ramps, fraction of heavy vehicles, lane width, lateral clearance, work zone grade, and more. The Highway Capacity Manual (HCM) procedures are very rough but the HCM (TRB 2000) does suggest that 1,600 passenger cars per lane per hour be used as a baseline for short-term work zones. Typically this number is adjusted downward based on other factors and can be as low as about half the original value.
- The more a work zone can be physically and visually separated from traffic (e.g., semipermanent barriers such as 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 by 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 are the following:

- *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. Although 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 some can estimate over much longer time periods. Some tools can only estimate delay on an hourly basis while some can estimate delays 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 the 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," (NAPA, 1996) 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. This is a Microsoft Access–based software tool that can be used to analyze highway pavement rehabilitation strategies that include 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 DOTs 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 by using CA4PRS with most inputs being held constant at typical values. The purpose of these graphs is to give only a rough estimate of typical productivity. CA4PRS should be used to produce more accurate numbers based on actual site-specific parameters for use in any project planning. In general, most inputs were fixed except for the trucking rates (i.e., removal of demolition from the site and delivery of paving material to the site). Thus, the 95% confidence intervals seen are mostly dependent on these delivery rates. In all cases, a 10-mile stretch of two lanes was analyzed (20 lane-miles total). As with all the data input values, this length of highway and total lane-miles has some influence on productivity. Table C. 10.2 through C.10.7 show input parameters used in CA4PRS to generate Figures C.10.1 through C.10.9. Estimates are given for the following:

- *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 job site at any one time), and concurrent operations (both major operations—demolition and paving—are occurring on the job site 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 in. thick with the exception of the top two lifts, which are either 2 or 1.5 in. thick. A lift is paved for each lane across before the next lift is paved on any lane.
- *Mill and fill with HMA*. Remove a predetermined thickness from the existing pavement with an HMA milling machine, then replace the same thickness with new HMA. The roadway lanes being paved are fully shut down; only one paver with a 12-ft-wide screed is used, and HMA is paved in lifts. Lifts are generally 3 in. thick with the exception of the top two lifts, which are either 2 or 1.5 in. thick. A lift is paved for each lane across before the next lift is paved on any lane.
- *Crack, seat, and overlay*. Crack and seat the existing PCC pavement, then overlay with HMA. The roadway being paved is fully shut down; only one paver with a 12-ft-wide screed is used, and HMA is paved in lifts. Lifts are generally 3 in. thick with the exception of the top two lifts,

which are either 2 or 1.5 in. thick. A lift is paved for each lane across before the next lift is paved on any lane.

- *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 C.10.2. CA4PRS Input Values for Remove and Replace with PCC

Input	**Value**	**Distribution/Comments**
Activity Constraints		
Mobilization	1.0 hr	None - Deterministic
Demobilization	2.0 hrs	None - Deterministic
Base Paving	none	N/A
Demolition-to-PCC Paving Lag Times for Sequential Method	1.0 hrs	Triangular (min = 0.5 hrs, max = 1.5 hrs)
Demolition-to-PCC Paving Lag Times for Concurrent Method	2.0 hrs	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 hrs	
Working Method	See graphs	

Table C.10.3. CA4PRS Input Values for Remove and Replace with HMA

Input	Value	Distribution/Comments
Activity Constraints		
Mobilization	1.0 hrs	None - Deterministic
Demobilization	2.0 hrs	None - Deterministic
Base Paving	none	N/A
Demo-to-HMA Paving Lag	1.0 hrs	Triangular (min = 0.5 hrs, max = 1.5 hrs)
Half Closure Traffic Switch	0.5 hrs	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)
Nonpaving 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 in. each, all other lifts are 3 in. each. Paver moves at 0.6 mph for top two lifts and 0.5 mph for all other lifts.
Change in Roadway Elevation	No Change	
Shoulder Overlay	Prepaving	Shoulder overlays are not accounted for.
Curing Time	12 hrs	
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 C.10.4. CA4PRS Input Values for Mill and Fill with HMA

Input	**Value**	**Distribution/Comments**
Activity Constraints		
Mobilization	1.0 hrs	None - Deterministic
Demobilization	2.0 hrs	None - Deterministic
Mill-to-HMA Paving Lag	1.0 hrs	Triangular (min = 0.5 hrs, max = 1.5 hrs)
Half Closure Traffic Switch	0.5 hr	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 in. Paver speeds are 0.5 to 0.6 mph.
Change in Roadway Elevation	No Change	
Shoulder Overlay	Prepaving	Shoulder overlays are not accounted for.
Curing Time	12 hrs	
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 C.10.5. CA4PRS Input Values for Crack, Seat, and Overlay

Input	Value	Distribution/Comments
Activity Constraints		
Mobilization	3.0 hrs	None - Deterministic
Demobilization	2.0 hrs	None - Deterministic
Half Closure Traffic Switch	0.5 hr	Triangular (min = 0.25 hrs, max = 0.75 hrs)
Resource Profile		
Paver	None	N/A (no base material)
Nonpaving 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 in. each, all other lifts are 3 in. each. Paver moves at 0.6 mph for top two lifts and 0.5 mph for all other lifts.
Change in Roadway Elevation	No Change	
Shoulder Overlay	Prepaving	Shoulder overlays are not accounted for.
Curing Time	12 hrs	
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 C.10.6. CA4PRS Input Values for Unbonded PCC Overlay

Input	Value	Distribution/Comments
Activity Constraints		
Mobilization	3.0 hrs	None - Deterministic (longer time accounts for surface preparation)
Demobilization	2.0 hrs	None - Deterministic
Base Paving	None	N/A
Demo-to-PCC Paving Lag Times for Sequential Method	0 hrs	No demolition occurs
Demo-to-PCC Paving Lag Times for Concurrent Method	0 hrs	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 hrs	
Working Method	See graphs	

Table C.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 Temperature	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 Temperature	60°F
Mix Specifications	
Mix Type	Dense Graded
PG Grade	64-22
Delivery Temperature	300°F
Stop Temperature	140°F
Lift Thicknesses	
3 in. of HMA total	2 lifts of 1.5 in. each
6 in. of HMA total	3 lifts of 2 in. each
9 in. of HMA total	3 lifts of 2 in., 1 lift of 3 in.
12 in. of HMA total	2 lifts of 1.5 in. 3 lifts of 3 in.

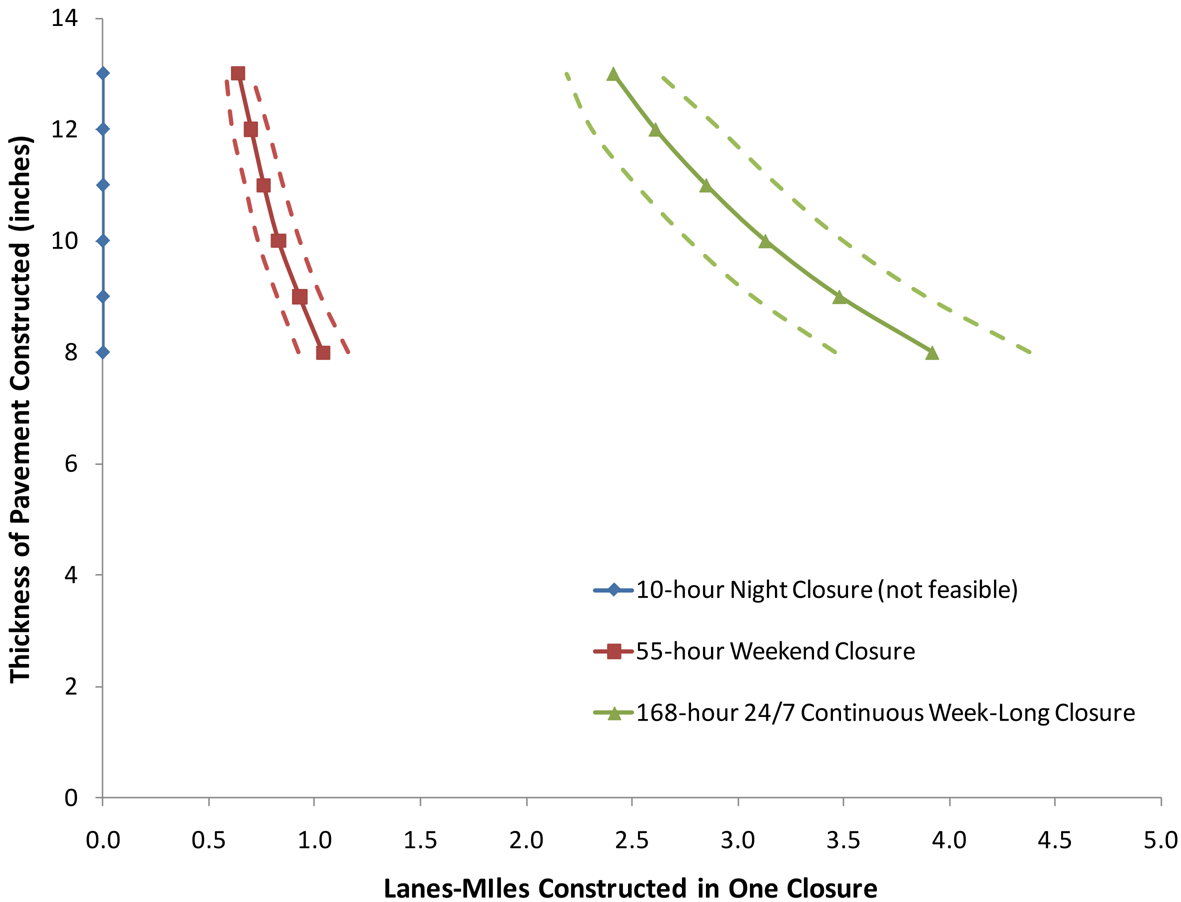


Figure C.10.1. Productivity estimates for remove and replace with PCC (fixed form) using sequential operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. (This option is not feasible using 10-hour night closures.)

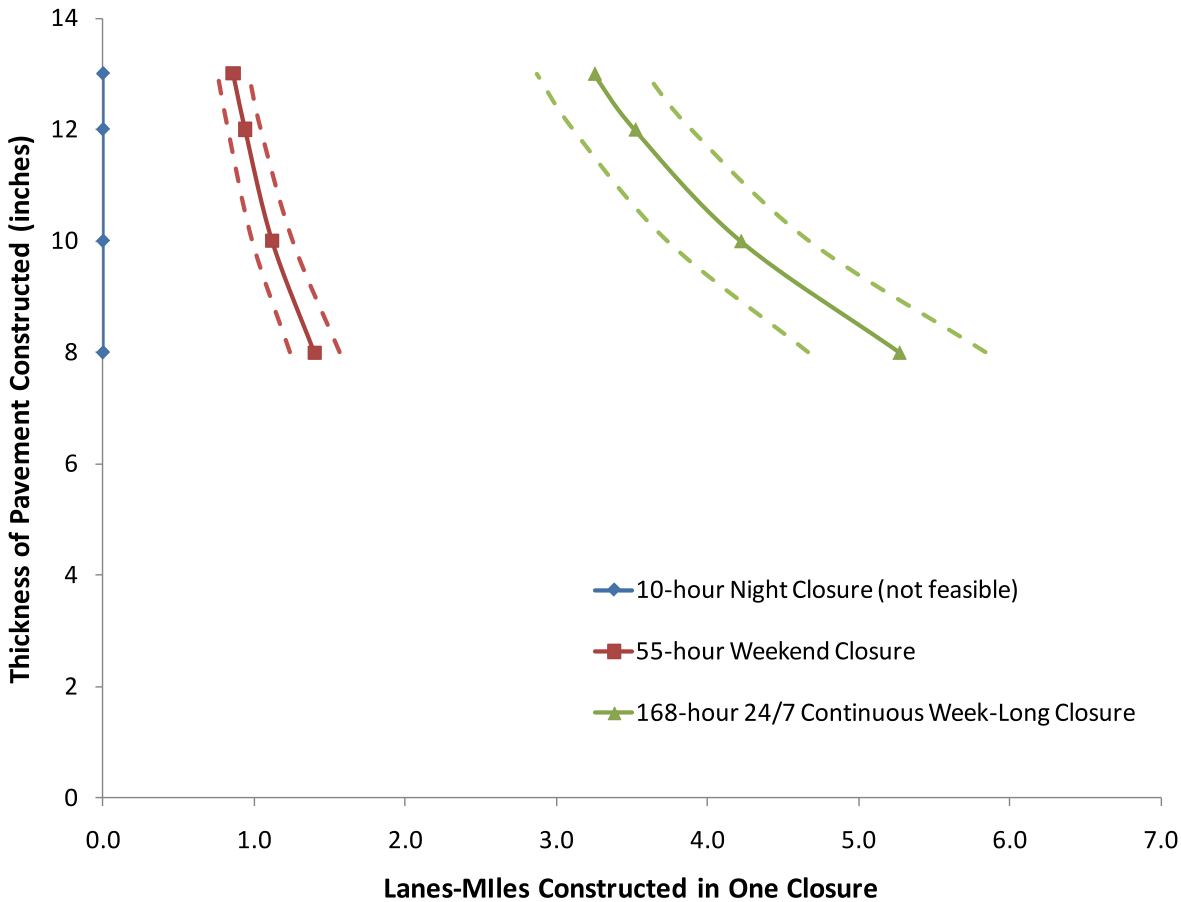


Figure C.10.2. Productivity estimates for remove and replace with PCC (slipform) using sequential operations. Solid lines indicate averages, and dashed lines indicate 95% confidence intervals. (This option is not feasible using 10-hour night closures.)

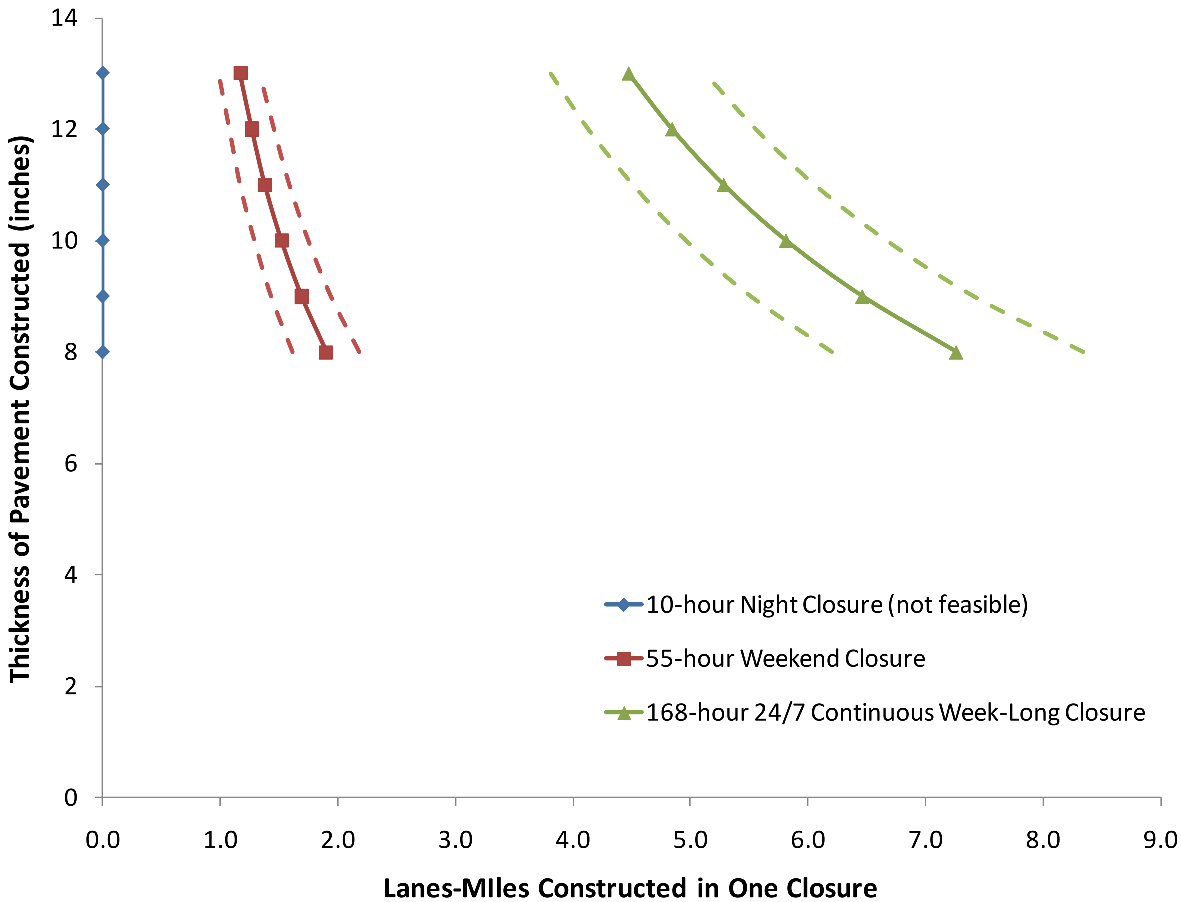


Figure C.10.3. Productivity estimates for remove and replace with PCC (fixed form) using concurrent operations. Solid lines indicate averages, and dashed lines indicate 95% confidence intervals. (This option is not feasible using 10-hour night closures. Doing demolition and paving concurrently results in significantly higher productivities than doing them sequentially.)

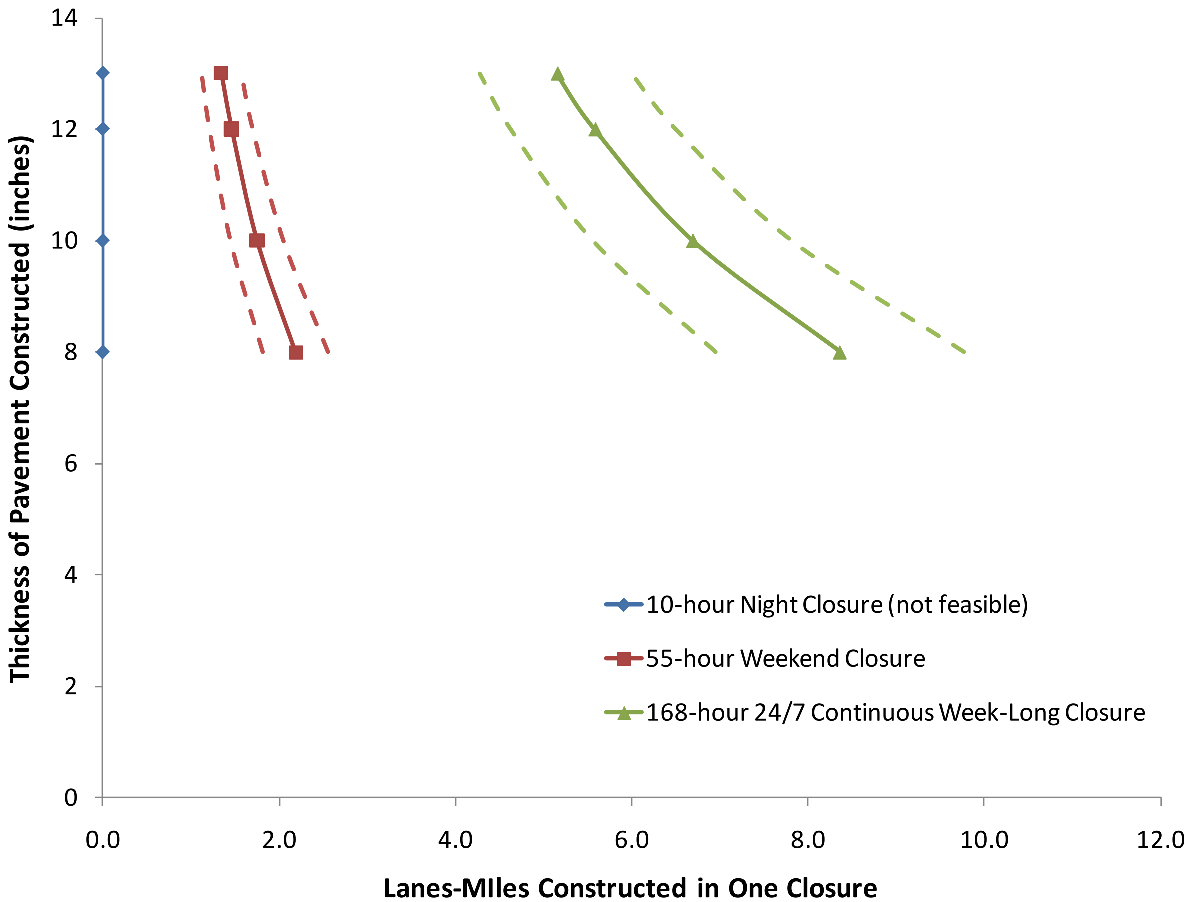


Figure C.10.4. Productivity estimates for remove and replace with PCC (slipform) using concurrent operations. Solid lines indicate averages and dashed lines indicate 95% confidence intervals. (This option is not feasible using 10-hour night closures. Doing demolition and paving concurrently results in significantly higher productivities than doing them sequentially.)

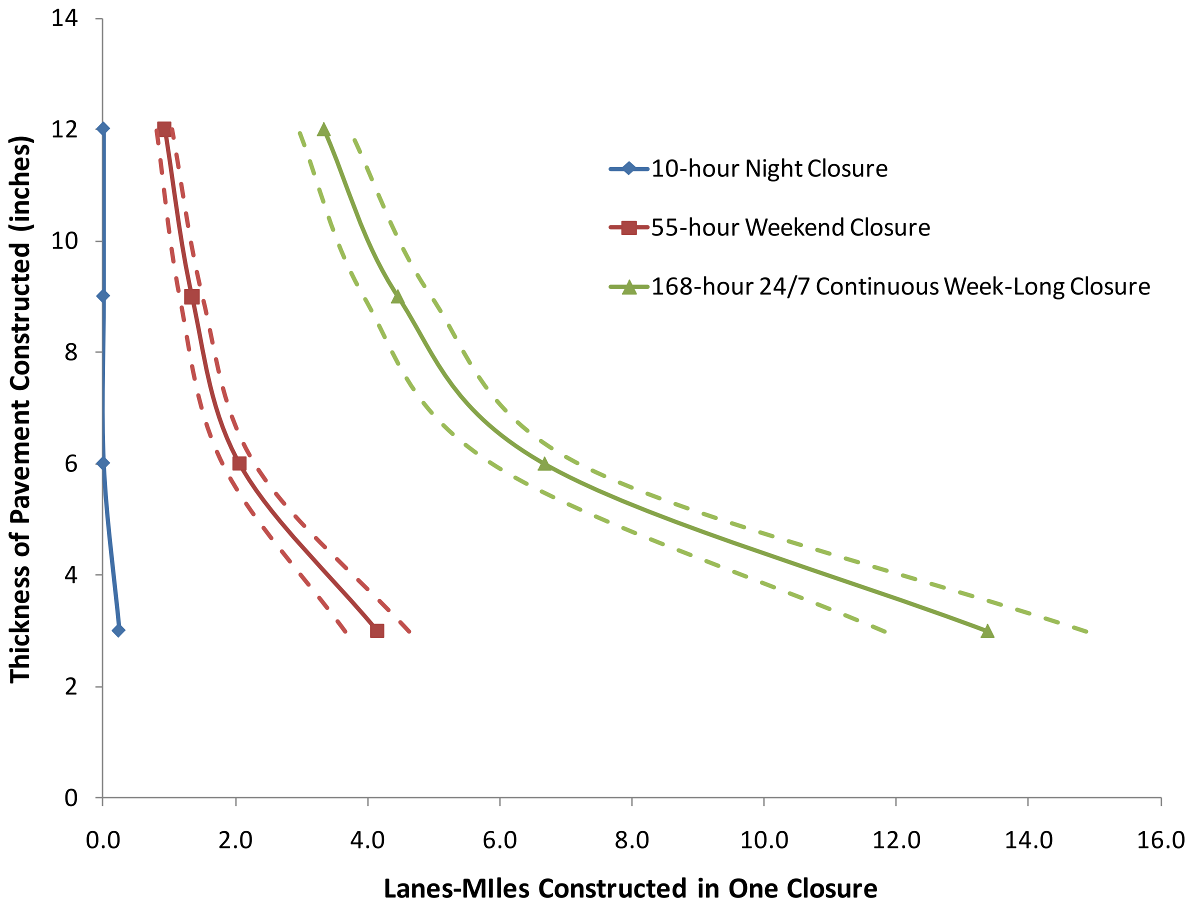


Figure C.10.5. Productivity estimates for remove and replace with HMA. Solid lines indicate averages, and dashed lines indicate 95% confidence intervals.

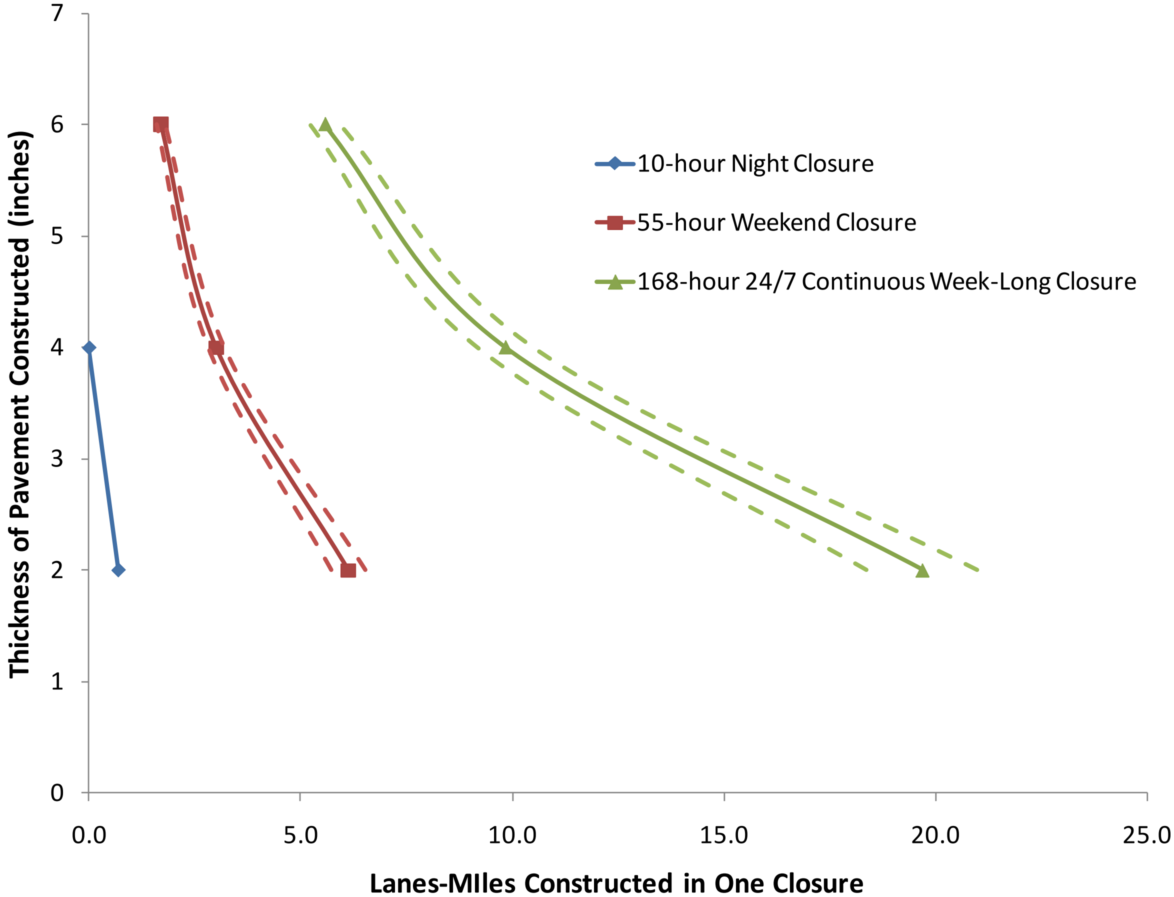


Figure C.10.6. Productivity estimates for mill and fill with HMA. Solid lines indicate averages, and dashed lines indicate 95% confidence intervals.

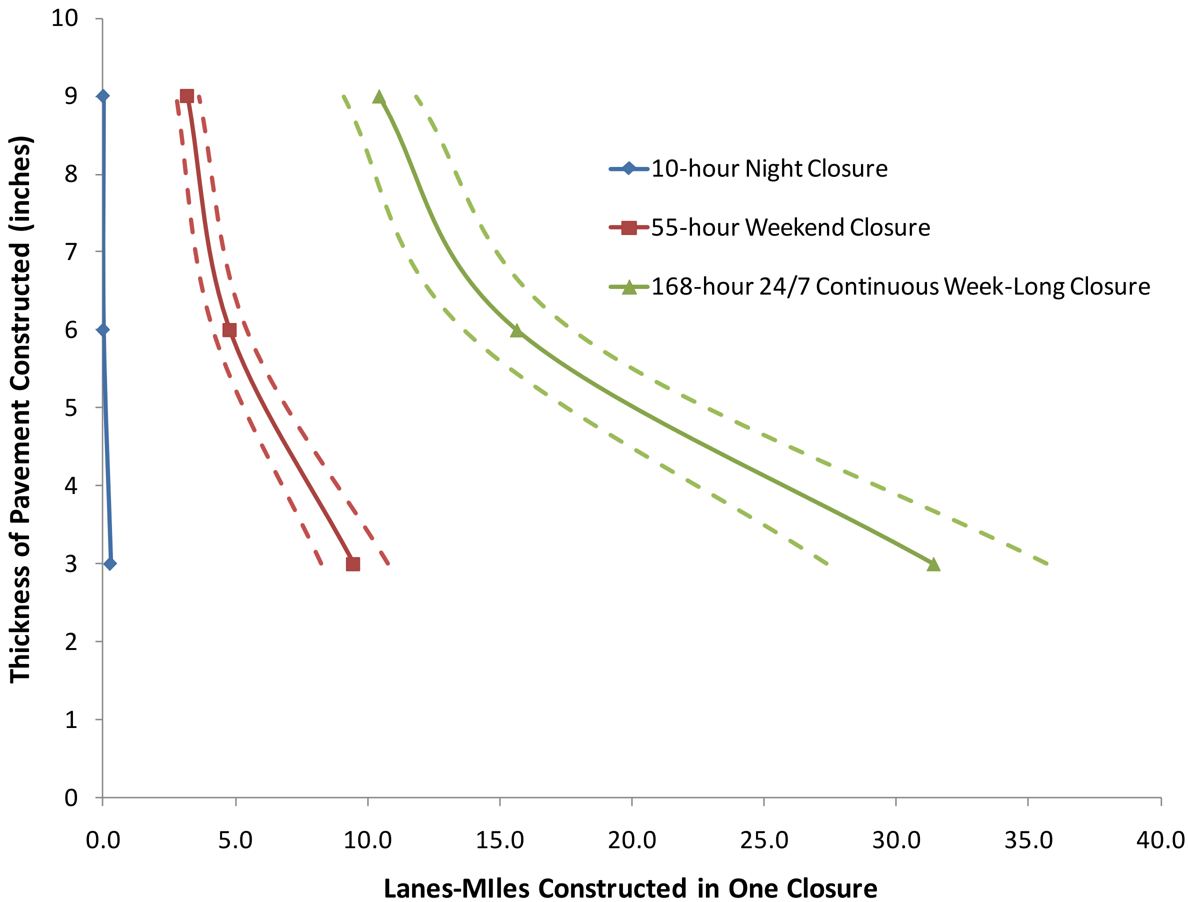


Figure C.10.7. Productivity estimates for crack, seat, and overlay. Solid lines indicate averages, and dashed lines indicate 95% confidence intervals.

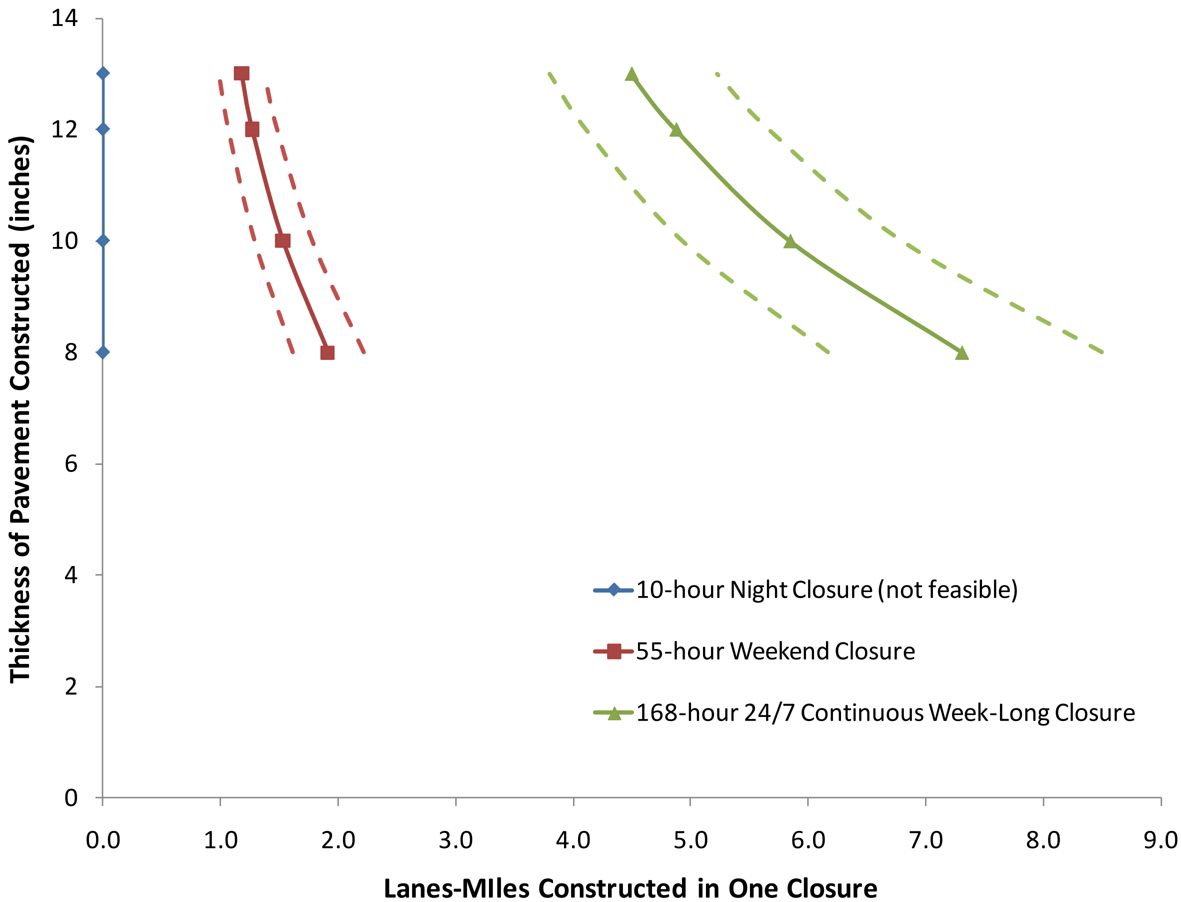


Figure C.10.8. Productivity estimates for PCC unbounded overlay. Solid lines indicate averages, and dashed lines indicate 95% confidence intervals.

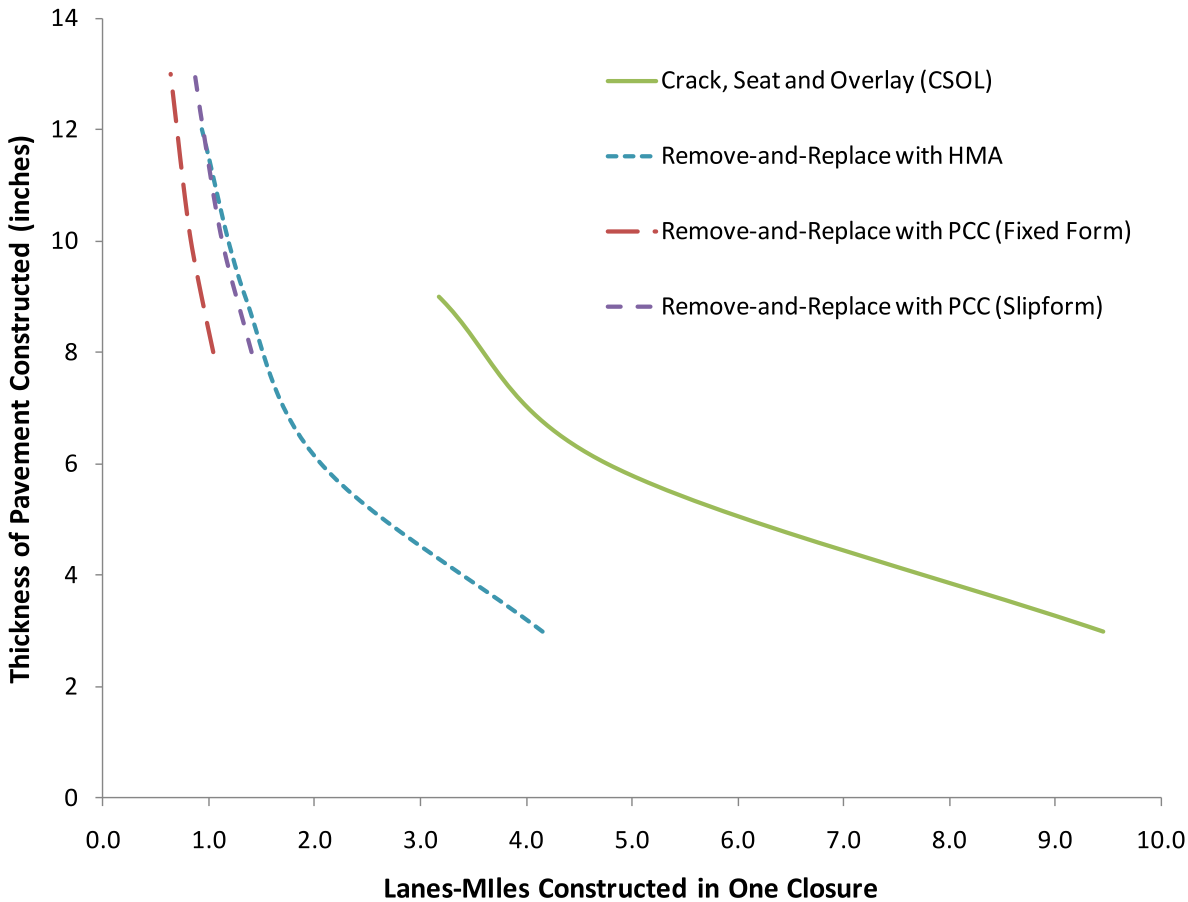


Figure C.10.9. A productivity comparison of PCC remove and replace (both fixed form and slipform), HMA remove and replace, and crack, seat, and overlay.

(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, 2008a), as follows, and they are summarized in Table C.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 HCM (TRB 2000). 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 regionwide impacts of extended roadway closures (e.g., closing a freeway for

several months). It is not likely that a travel demand model would be built for the specific purpose of work zone traffic analysis. Rather, an existing model may be used if available and warranted.

3. *Traffic signal optimization tools*. 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. These models can simulate both large geographic areas as well as specific corridors. They do not, however, possess the detail to model more detailed 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 C.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 micromodels	Data intensive
Microscopic	CORSIM, VISSIM, PARAMICS	Can model small details, good communication tool	Data intensive

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

- *Work zone characteristics*. The expected level of impact a work zone will have on travelers, which includes 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, which includes 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 money 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 C.10.10 and 10.11), then assigns traffic counts to that network. It can coarsely simulate traffic variations between days of the week and months of the year by applying multiples to standard ADT inputs. It can simulate multiple lane closures over time, model traffic over an entire week (Figure C.10.12), and display various traffic impact metrics (Figure C.10.13). These capabilities are helpful because they allow QuickZone to show differences in traffic impacts between nights and days, weekends and weekdays, and seasons (e.g., summer vs. fall work). The user guide explains the algorithm that 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 because of tedious data entry and difficult input troubleshooting if outputs are suspect.

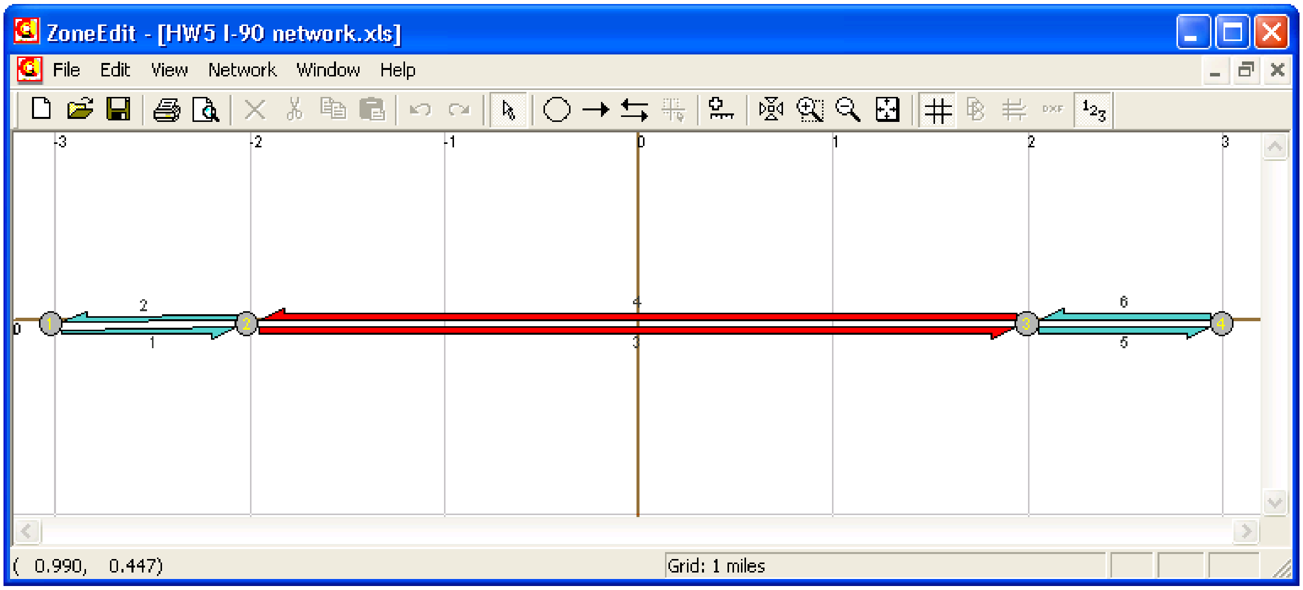


Figure C.10.10. A simple network that works quite well in QuickZone.

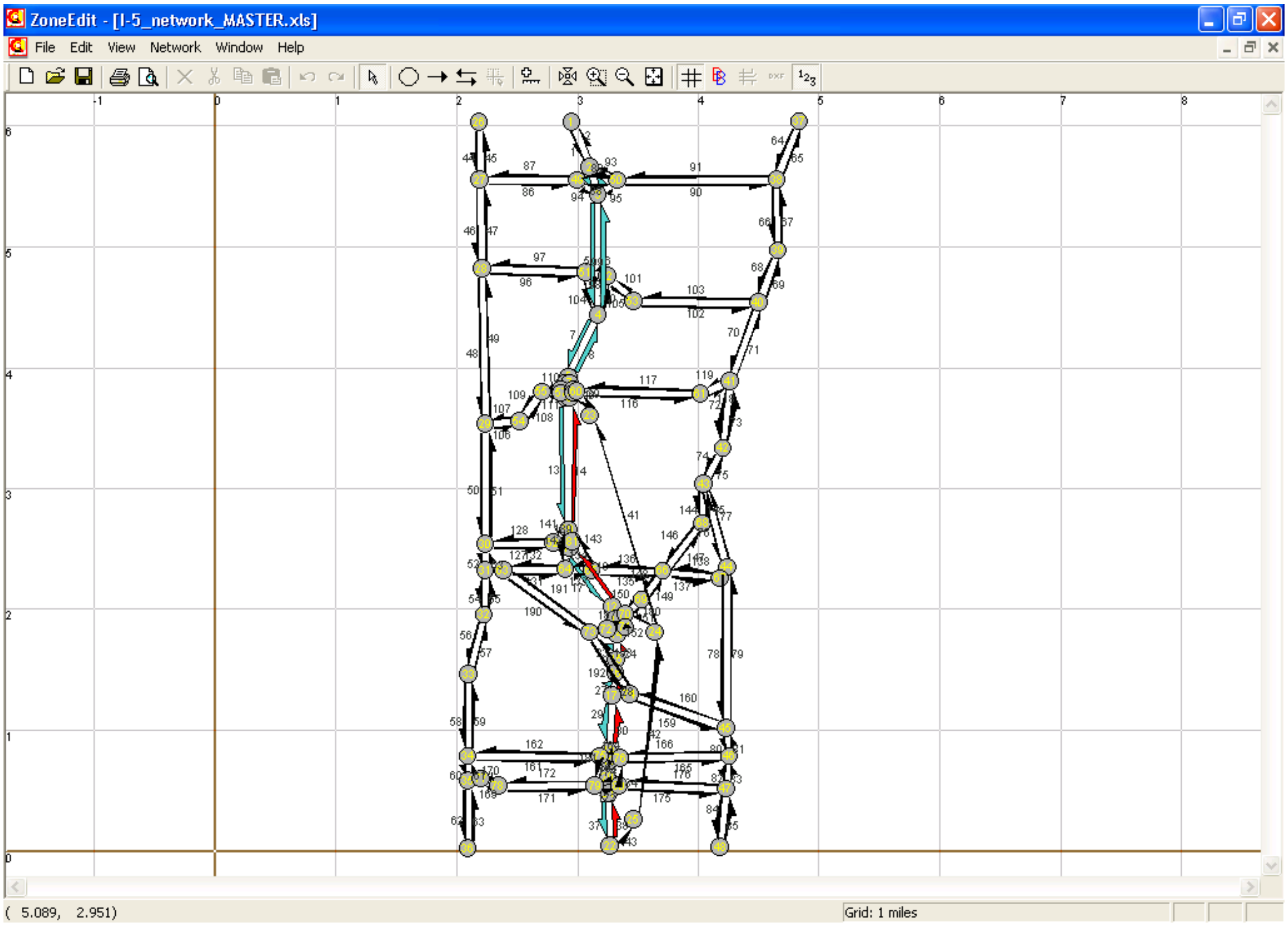


Figure C.10.11. A complex network simulation in QuickZone (I-5 in the Seattle, Washington, area is shown). (This network simulation exposed several program bugs, was unwieldy to process and required tedious troubleshooting to make operational. This level of complexity is not recommended.)

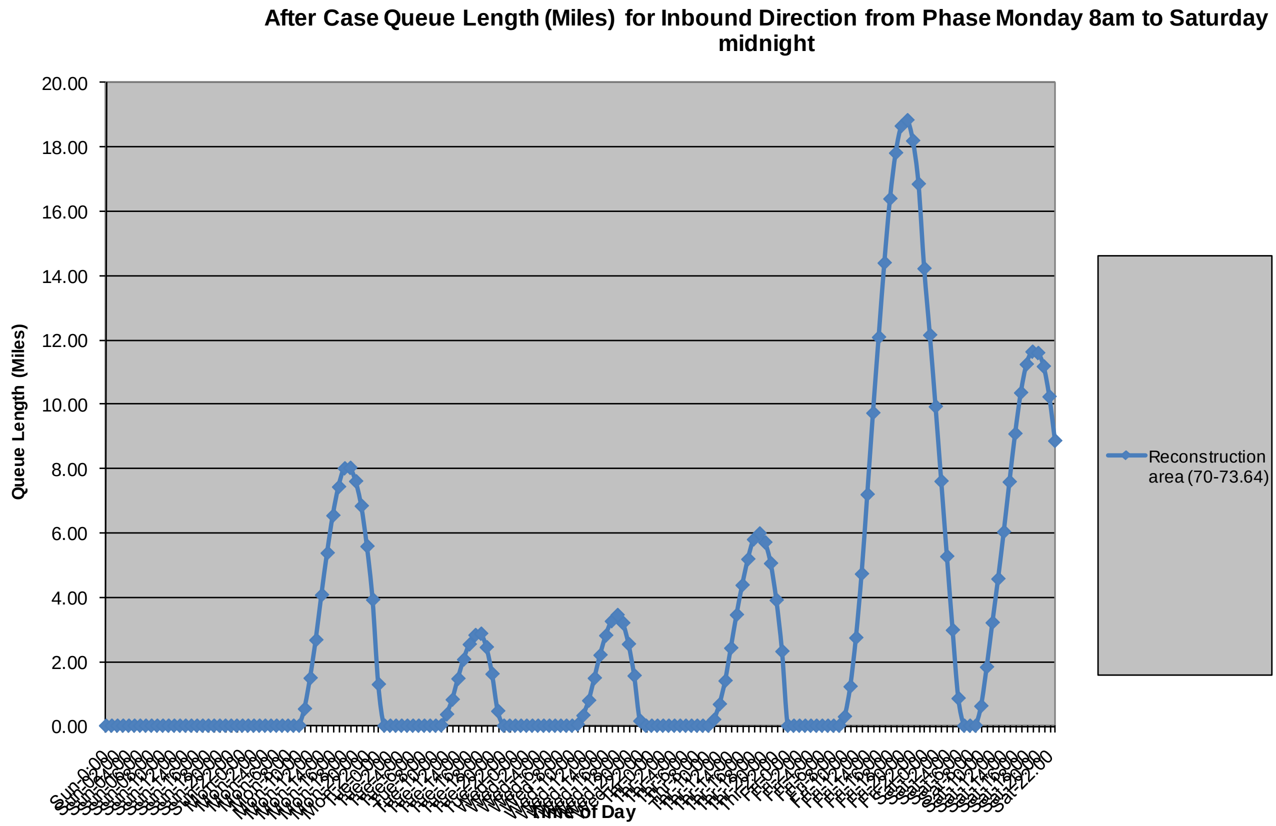


Figure C.10.12. Unedited QuickZone 2.0 simulation output chart for a one-week time period. (Note that the automatic graph labeling on the horizontal axis is unreadable; however, this can be corrected by editing the graph in Excel.)

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 C.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 that include 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 C.10.14). Traffic can be entered by hourly count, or ADT can be entered and then distributed over 24 hours by 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) when 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 C.10.15 and C.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.

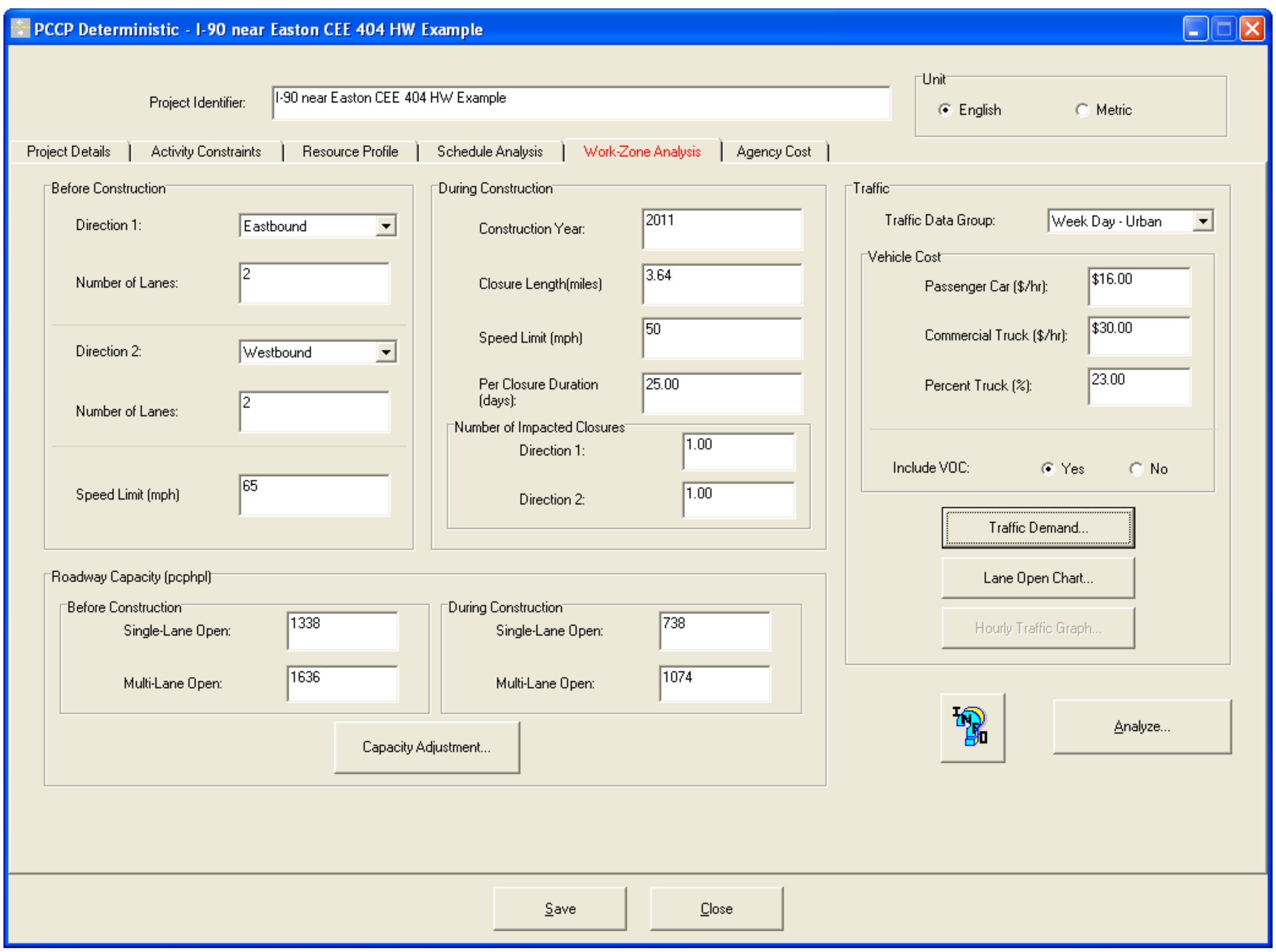
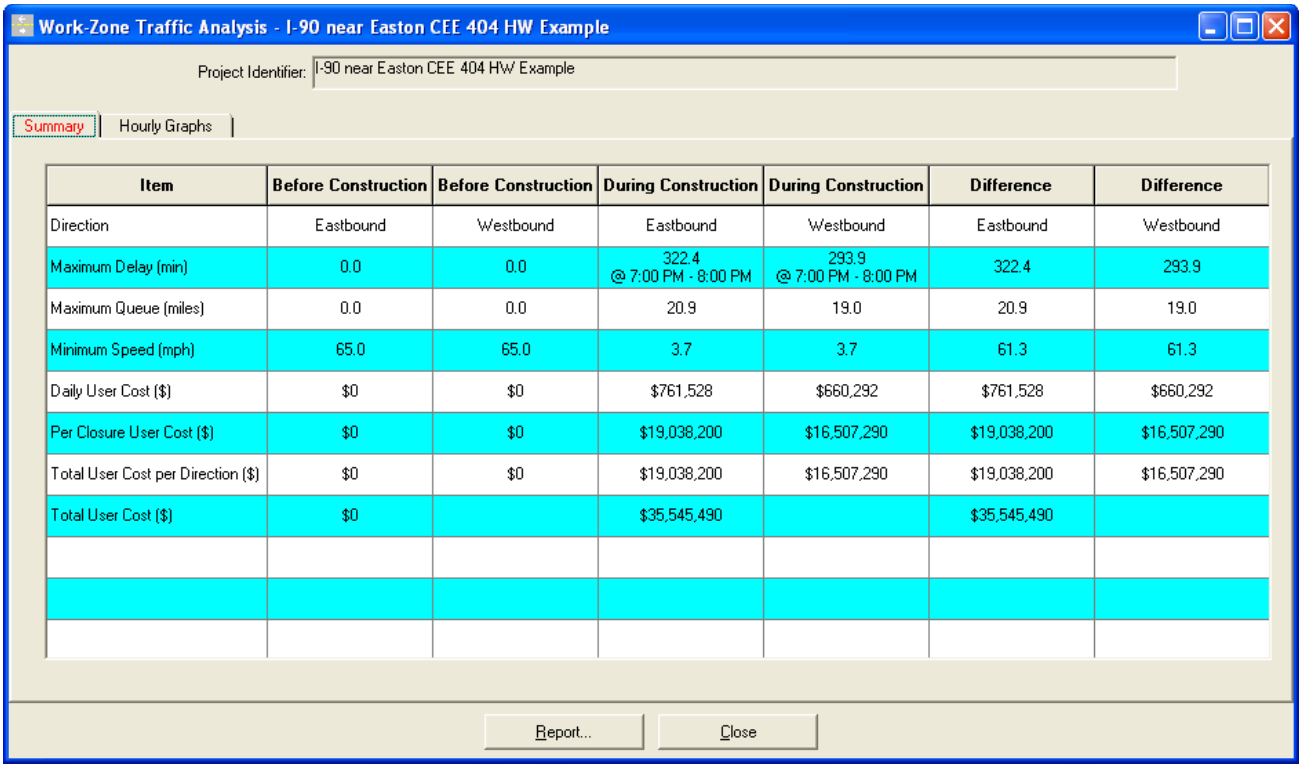


Figure C.10.14. CA4PRS work-zone analysis input screen.



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

Figure C.10.15. CA4PRS work-zone analysis summary results screen showing available traffic impact metrics.

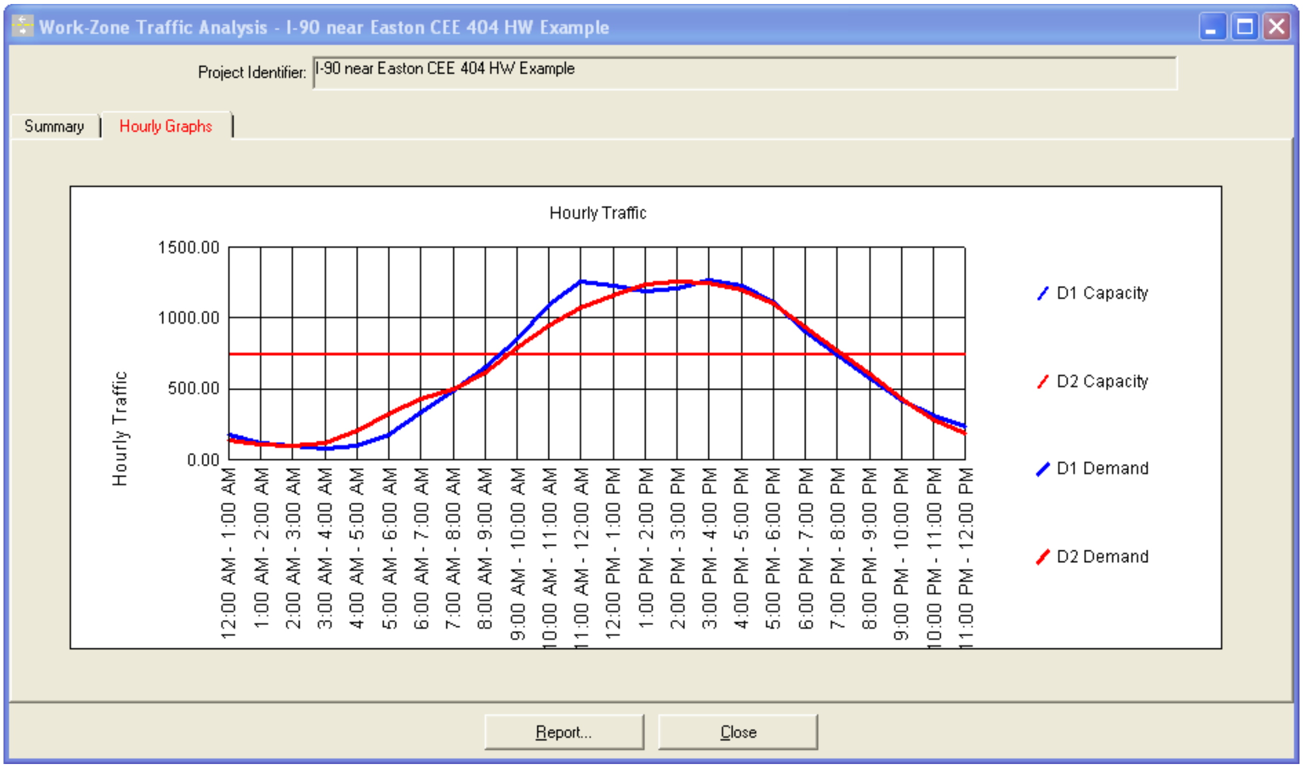


Figure C.10.16. CA4PRS work-zone analysis hourly traffic results graph showing demand vs. capacity.

10.4 References

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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 United States and are increasingly being addressed through the National Environmental Policy Act (NEPA) as recent White House Council on Environmental Quality guidance shows (Sutley, 2010). As this scrutiny increases, there will likely be more tools to help in analysis. It also seems plausible that once industry has a fair idea what energy, emissions, and other resources are associated with roadway construction, it will begin to adopt (either voluntarily or by regulation) efficiency standards associated with these items similarly 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

(i) Introduction

An LCA attempts to identify inputs and outputs of a system that are relevant to the environment from its inception to its ultimate disposition. This means that an LCA includes everything from gathering raw materials to the point at which those materials are returned to the environment. This collection of all processes from “cradle to grave” allows an 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 are 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 the following:

- *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 come from many different sources beyond these and range from personal observation to national databases, which can lead to problems when comparing one LCA with another. For instance, the CO_2 associated with HMA production is not a universal constant; rather, it 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), especially at the local project level. At the very least, an LCA should clearly identify its data sources.
- *Missing data*. There are many industrial processes where some, if not all, relevant data are not known, recorded, or made available for public use. For instance, the amount of fugitive dust on site associated with pavement construction is not generally known. Or, the exact chemical makeup of an asphalt modifier may be a trade secret that the manufacturer is not willing to divulge.
- *Outdated data*. Sometimes, data exist but are outdated. Over time, processes change, equipment improves, raw material sources change, etc.
- *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 these data are used it cannot distinguish between a mix using all virgin materials and one using 25% RAP, for instance.
- *Setting boundaries*. An LCA that attempts to account for all processes associated with a system can quickly become intractable. For example, one could account for the slipform paver and its energy use and emissions associated with a concrete pavement. One could also account for the energy and emissions associated with manufacturing that slipform paver. However that leads to

potentially considering the energy and emissions associated with the manufacture of the machines that made the paver and so on. For another example, one can choose whether to include the effect of pavement stiffness on the rolling resistance it offers to vehicles that travel on 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. However, one might question whether including this effect alone is a realistic assessment of the pavement's impact on vehicle operations. Because of these boundary issues (what processes are included and excluded), every LCA has a defined boundary (that should be explicitly stated) that details which processes are included and which are not. Inclusions and exclusions are often not consistent between LCAs and can be controversial.

- *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. For instance, ISO 14044 says that feedstock energy (energy associated with burning the material in a product when that material is not used as an energy source but could be) should be included in the analysis. In essence, one must assign the energy that is involved in burning the asphalt cement in an HMA pavement to the energy use of that pavement even though that asphalt cement is almost certainly never going to be burned. This is significant since asphalt has a significant amount of energy stored in it (Santero, 2009).

Despite these limitations, an 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 an LCA because it may significantly affect the results. Ultimately, boundary locations are somewhat subjective, which can lead to difficulty in comparing one LCA's results to another. Process-based LCAs are desirable because they can be done in enough detail so that they include processes that can differentiate between two options (e.g., using warm mix asphalt or not). They are problematic because of the subjective boundary and difficulty in obtaining data on specific processes.

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 activity in the selected economy and depicts the economic interaction of industries (sectors) in a nation (or a region) by showing how the output of each sector is used as input for another. The system boundary is inherently the whole country's entire economy. Interactions are represented by monetary value in a matrix form, called the 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).

(iii) 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 the following:

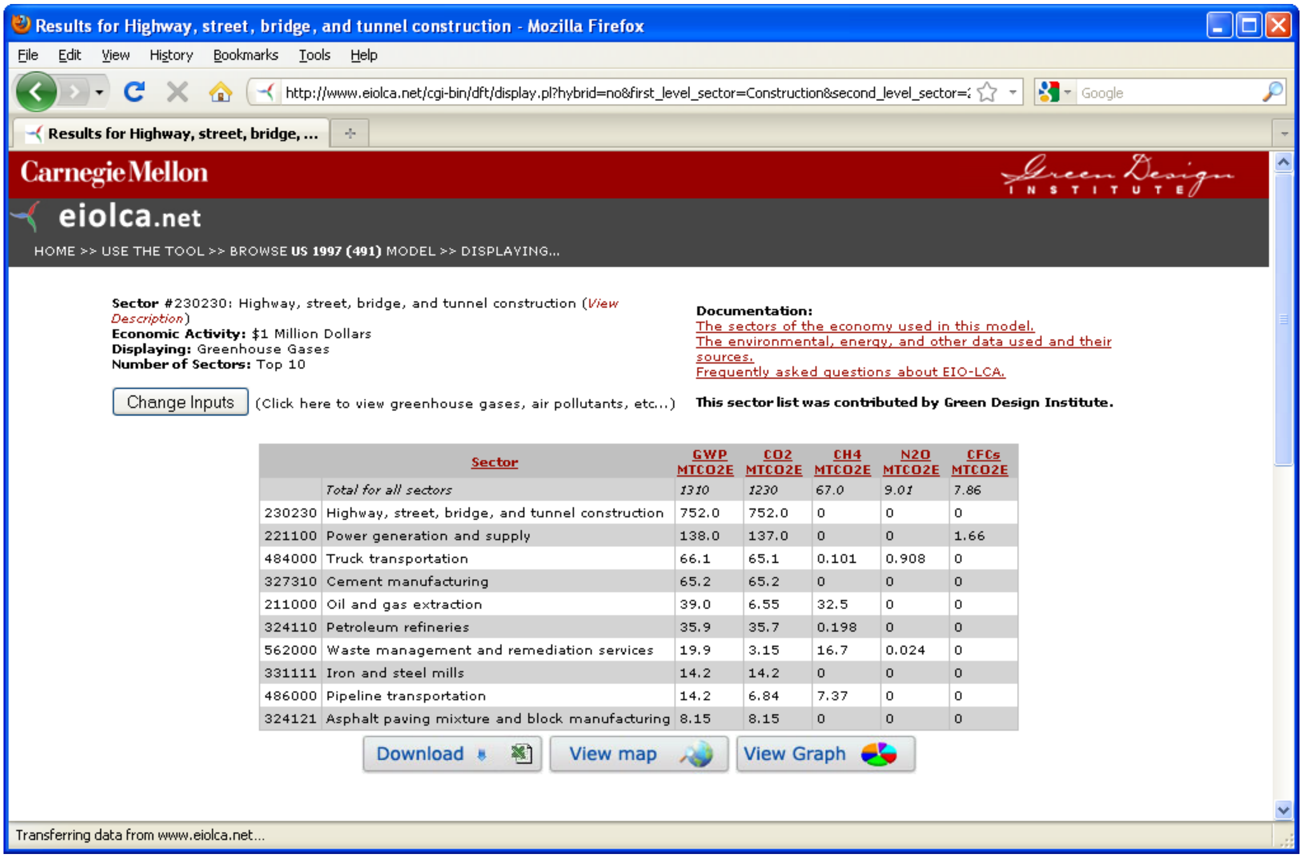
- *System scope*. Most LCAs tend to address the pavement structure only and not include other road features (e.g., striping, guardrails, etc.). Analysis periods are usually 40-50 years.
- *Relation of roadway construction to traffic use*. A good rule-of-thumb is that the energy expended in initial construction of a new roadway is roughly equivalent to the energy used by traffic on the facility over 1-2 years.
- *Relation of roadway construction to operations*. Operations are defined as those equipment, actions, and operations that happen on a routine basis necessary to ensure proper and safe roadway use. They include items such as lighting, traffic signals, deicing, sanding, drawbridge actions, toll booths, etc. Construction energy ranges from about 25% to 100% of operations energy.
- *Total energy use*. It can be loosely stated that energy expenditures per lane-mile of pavement are typically on the order of 3-7 TJ depending upon the pavement section, maintenance activities, and LCA scope.
- *CO_2 emissions*. It can be loosely stated that CO2 emissions per lane-mile of pavement are typically on the order of 200-600 tonnes 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% to 80% of energy use and 60% to 90% of CO_2 emissions.
- Construction accounts for less than 5% of energy use and CO_2 emissions.
- Transportation associated with construction accounts for about 10% to 30% of energy use and about 10% of CO_2 emissions.
- Maintenance activities account for a broad range of about 5% to 50% of energy and CO_2 emissions.

11.3 Analysis Tools

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

EIO-LCA. 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 C.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	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 C.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, 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. 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.

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 but can do so for pavement, earthwork, and clearing and grubbing. The IRF plans to expand this tool to address the entire roadway (i.e., beyond just the pavement to include signs, striping, guardrail, etc.).

Project Emissions Estimator (PE2). An online application developed at Michigan Technological University (Mukherjee and Cass, 2012). PE2 is a web-based (http://www.construction.mtu.edu/cass_reports/webpage/plca_estimator.php) tool that can be used to "estimate and benchmark the CO_2 footprint of highway construction projects" (Mukherjee and Cass, 2012). The underlying inventory for PE2 comes from 14 highway projects in Michigan, and the estimator has features that are tailored to use with Michigan DOT project information.

Athena Impact Estimator for Highways. An LCA computer program for roadway construction, use, and rehabilitation (Athena Institute, 2013). It is available for free on the Athena website ((http://www.athenasmi.org/our-software-data/impact-estimator-for-highways). Athena appears to be based on Canadian data and is designed for use in Canada.

Roadprint. An online application developed with funding from Federal Lands Highway (FLH), the Oregon DOT, and WSDOT (Lin, 2012). Currently available at http://clients.paviasystems.com/wfl, this online tool performs a process-based LCA on pavement materials production, materials transport, and on-site construction. It does not address any environmental inputs or outputs associated with vehicles driving on the pavement surface. Roadprint's development is comprehensively documented (Lin, 2012), and there are plans to expand it to address the entire roadway (i.e., beyond just the pavement to include signs, striping, guardrail, etc.). Roadprint is available for use for free and is designed to produce LCA results in about 15 minutes for users who have basic knowledge of road construction and no knowledge of LCA processes.

(i) Example LCA and findings using Roadprint

This section describes a case study in which actual construction project data were used as input data for an LCA conducted with Roadprint. Several potential changes to the construction materials are investigated to determine their effects on energy consumption and greenhouse gas emissions over a 50-year analysis period.

Project description and data. A local collector road in Kailua, Hawaii, is scheduled for repaving. The work involves removing 4 in. of HMA with a milling machine and an additional 2 in. of base material, then replacing it with two layers of HMA: a 4-in. base course and a 2-in. surface course. This job is equivalent to 11.87 lane-miles (assumed lane width is 12 ft) of paving. 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

- Milling: 79,386 yd^2 of 6-in.-deep milling

Assumed processes. A 2-in. mill and fill for surface renewal is assumed every 10 years (year 10, 20, 30, and 40). Quantities for each mill and fill quantities are assumed as follows:

- Surface course: 7,913 tons of HMA
    - 5.5% asphalt by total weight of mix
    - No recycled material in the mix
- Milling: 79,386 yd^2 of 2-in. deep milling

Materials locations. Materials for both the initial construction and mill and fills 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
- Glass cullet: assumed to come from a source 30 miles from the job site
- Ground tire rubber: assumed to come from a source 30 miles from the job site

Results. Results from this analysis are (Figure C.11.2):

- 62.34 TJ of life-cycle energy consumption (5.25 TJ/lane-mile)
    - Including 141.03 TJ of feedstock energy (at 40.20 MJ/kg of asphalt cement); this total rises to 203.37 TJ of life-cycle energy consumption (17.13 MJ/lane-mile).
- 3,482 Mg (tonnes) of life-cycle CO_2 equivalent emissions (293 tonnes/lane-mile).

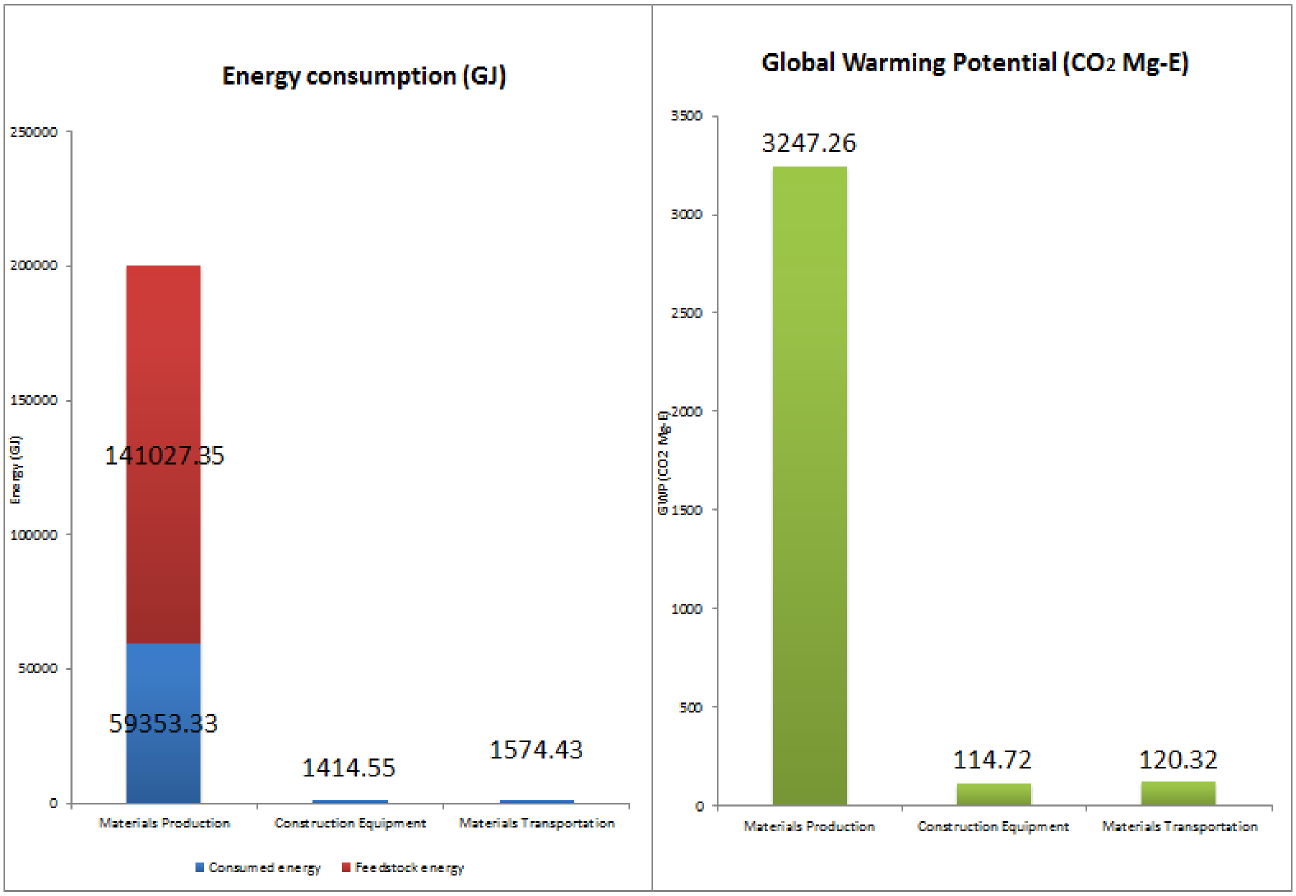


Figure C.11.2. Example Roadprint output graphs showing total energy consumption (left) and global warming potential in terms of CO2e (right).

Table C.11.1. Example Roadprint Output Table Showing an Overall Contribution Analysis by Impact Assessment Categories

	Energy Consumption With feedstock		Energy Consumption w/o feedstock		Global Warming Potential		Acidification		Photochemical Smog		Eutrophication		Human Health Criteria Air	
Unit	Energy (GJ)		Energy (GJ)		GWP(CO_2 Mg-E)		Kg SO2		Kg NOx		Kg PO4		milli - DALYs/Kg	
Material Production	200381	99%	59353	95%	3247	93%	14042	76%	9484	85%	964	84%	287	80%
Equipment	1415	1%	1415	2%	115	3%	704	4%	1063	10%	123	11%	19	5%
Transportation	1574	1%	1574	3%	120	3%	3805	21%	553	5%	67	6%	52	15%
Total	**203370**		**62342**		**3482**		**18550**		**11100**		**1153**		**358**	

Table C.11.2. Example Roadprint Output Table Showing a Contribution Analysis on Materials Production

Material	Material Production					
	Energy (GJ)		Energy (GJ)		GWP(CO_2 Mg-E)	
	Value	%	Value	%	Value	%
HMA/WMA	41465.3	20.7%	41465.3	70%	1835.4	56.5%
PCC	0.0	0.0%	0.0	0%	0.0	0.0%
Virgin Aggregate	3818.4	1.9%	3818.4	6%	242.6	7.5%
Sand and Gravel	0.0	0.0%	0.0	0%	0.0	0.0%
Bitumen	13951.0	7.0%	13951.0	24%	1089.6	33.6%
Feedstock	141027.4	70.4%	0.0	0%	0.0	0.0%
Cement	0.0	0.0%	0.0	0%	0.0	0.0%
RAP/RAC to plant	0.0	0.0%	0.0	0%	0.0	0.0%
Aggregate substitutes	118.7	0.1%	118.7	0%	79.6	2.5%
Steel	0.0	0.0%	0.0	0%	0.0	0.0%

Note: The second "energy (GJ)" column assesses contributions while ignoring feedstock energy.

Table C.11.3. Example Roadprint Output Table Showing a Contribution Analysis on Transportation

Material	Transportation			
	Energy (GJ)		GWP(CO2 Mg-E)	
	Value	%	Value	%
HMA/WMA	906.3	57.6%	69.3	57.6%
PCC	0.0	0.0%	0.0	0.0%
Virgin Aggregate	0.0	0.0%	0.0	0.0%
Sand and Gravel	0.0	0.0%	0.0	0.0%
Bitumen	238.0	15.1%	18.2	15.1%
Cement	0.0	0.0%	0.0	0.0%
RAP/RAC to plant	0.0	0.0%	0.0	0.0%
Aggregate substitutes	225.7	14.3%	17.2	14.3%
Steel	0.0	0.0%	0.0	0.0%
Rap/RAC Collection	204.5	13.0%	204.5	13.0%

Alternative investigation. Several alternatives to the previous baseline scenario were investigated to determine potential changes if these alternatives were pursued. These options were

- *All virgin material*. Remove the glass cullet from the base course.
- *Glassphalt base*. Include 10% glass cullet by weight of mix in place of base layer aggregate. This is the baseline scenario calculated previously.

- *15% surface / 15% base RAP*. Include 15% RAP in the surface and base courses.
- *20% surface / 40% base RAP*. Include 20% RAP in the surface course and 40% RAP in the base course. This is the maximum RAP percentage allowed by Hawaii DOT specifications.
- *Warm-mix asphalt (WMA).* Use WMA instead of HMA. This assumes a 15% reduction in energy and CO_2 emissions from the HMA manufacturing process only.
- *Stone matrix asphalt (SMA).* Use an SMA surface course at 6.5% asphalt by total weight of mix that allows a surface life of 17 years. This results in resurfacing at years 17 and 34 only.
- *Quiet pavement*. Use an open-graded friction course to reduce tire-pavement noise. This mix is at 10% asphalt by total weight of mix (20% of the asphalt binder is ground tire rubber), and the surface life is assumed to be 8 years. This results in five total resurfacings during the 50-year analysis period
- *Ultimate*. Use a combination of an SMA surface course, no glass in the base course, 40% RAP in the base course, and WMA for both courses in order to reduce the energy and GHG footprint to the maximum extent possible given current standards.

Figures C.11.3 through C.11.6 show the percentage change from the baseline practice in terms of energy consumption.

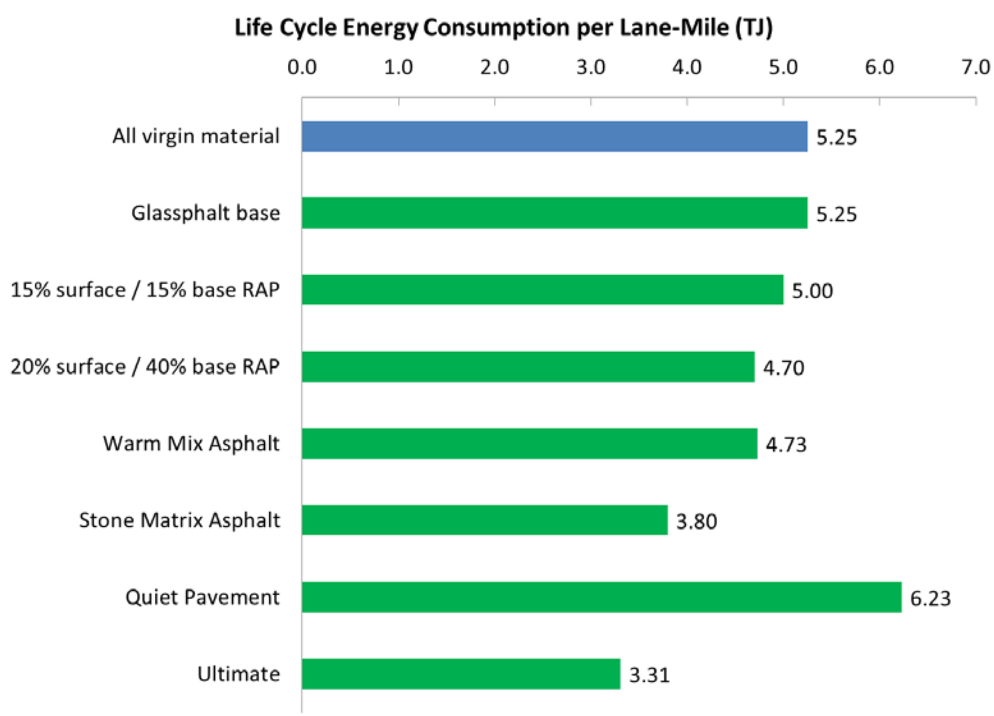


Figure C.11.3. Life-cycle energy consumption for the current practice and eight alternate scenarios for the example LCA.

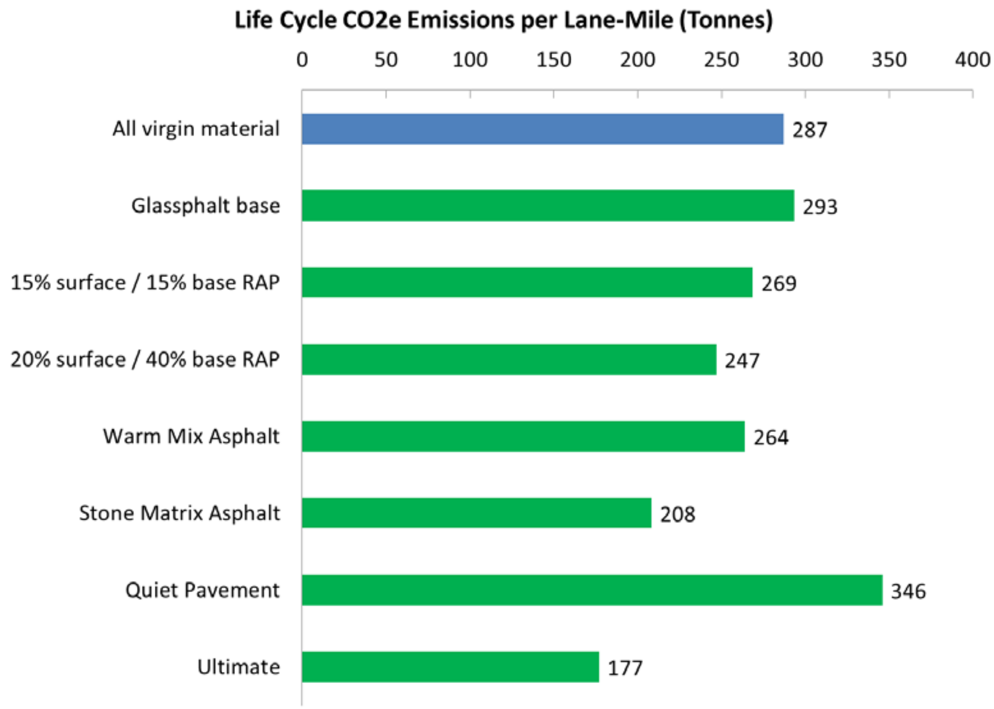


Figure C.11.4 Life-cycle CO_2 equivalent emissions for the current practice and eight alternate scenarios for the example LCA.

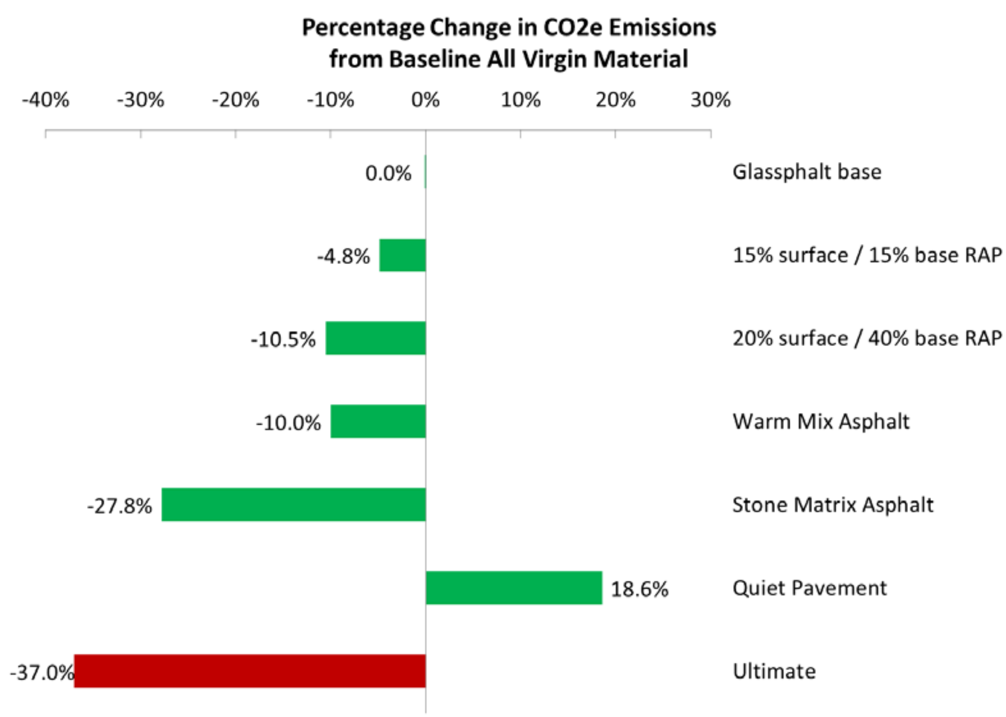


Figure C.11.5. The percentage change from the baseline value of energy consumption for a number of alternate scenarios for the example LCA.

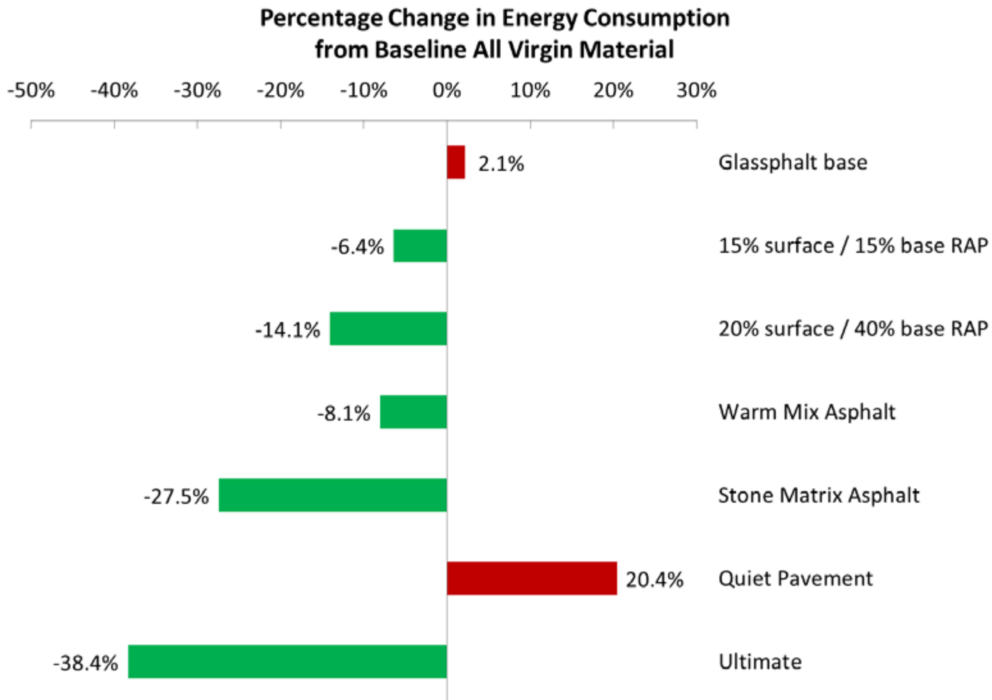


Figure C.11.6. The percentage change from the baseline value of CO2e emissions for a number of alternate scenarios for the example LCA.

General conclusions. Some general conclusions that can be reached for this example are:

- Extending service life can be the biggest single influence in energy used and CO_2 emitted by the pavement. The biggest single improvement came with the use of SMA and the assumed increase in surface life from 10 to 17 years.
- Often, a combination of options can produce an even greater savings in energy used and CO_2 emitted by the pavement. Several of the options can be combined on one project to provide even bigger savings.
- The inclusion or exclusion of the glass cullet makes very little difference in energy used and CO_2 emitted by the pavement. While the inclusion of glass cullet was, at one time, mandated in Hawaii (where practical), its inclusion has little impact on energy and emissions.
- The use of quiet pavement (and its assumed shorter surface life of 8 years) results in the consumption of more materials and a corresponding increase in energy and emissions. Therefore, in this scenario the use of quiet pavement represents a sacrifice in energy and emissions in order to achieve less tire-pavement noise.

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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 C.12.1 shows typical layer moduli for several material conditions. Table C.12.2 shows information about rubblized PCC, and Table C.12.3 gives information about crack and seat renewal.

Table C.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 C.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
Rubblized PCC modulus	Range: 40,000 to 700,000 psi
Typical Rubblized PCC modulus	50,000 to 150,000 psi

Table C.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
Modulus of break and seat PCCP	Range: 250,000 to 2,000,000 psi

12.3 References

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