Final Meeting Notes
FHWA Pavement Preservation Expert Task Group
May 14-15, 2009
Hotel Monteleone
New Orleans, LA

Introductions and Welcome

The meeting was called to order at 1:00pm on Thursday, May 14 by Mr. Jim Sorenson, FHWA. Mr. Sorenson stated that 19 years ago the FHWA formed working groups in 4 AASHTO regions and of the original group, there are only four remaining members affiliated with the Pavement Preservation Expert Task Group (PPETG): Mr. Sorenson, Mr. Denny Jackson, KBA, Mr. Ed Denehy, NYS DOT, and Mr. Larry Scofield, ACPA. Mr. Sorenson appreciates the work that the PPETG performs and feels the continual increase in membership shows the dedication towards pavement preservation.

Mr. Sorenson gave a special welcome to the Portland Cement Concrete Pavement group. He stated that it's important to have them involved in the PPETG group and he appreciates the effort they exerted to have so many members attend the meeting.

Mr. Jackson welcomed the new members and invited guests. The membership roster was circulated and attendees were asked to update or add their contact information.
ATTACHMENT 1 (PP ETG Members / PP ETF Members)
ATTACHMENT 1A PPETG Agenda, ATTACHMENT 1B ETF Agenda

Mr. Jackson asked for a motion to approve the meeting notes from the October 2007 meeting in Newport Beach, CA. Mr. Larry Rouen, Caltrans, made a motion to approve the notes, which was seconded by Mr. Jon Rice, NACE. Mr. Jackson asked that attendees review the list of Action Items from the Newport Beach meeting.
ATTACHMENT 2

Mr. Jackson then asked the Subcommittee Members break out into their individual groups to prepare an update and report on the long and short term goals outlined in the Strategic Plan. ATTACHMENT 3 – Updated Strategic Plan

Subcommittee Updates– Short and Long Term Goals

Portland Cement Concrete Pavement Subcommittee

Mr. Craig Hennings, ACPA, reported on the Portland Cement Concrete Pavement Subcommittee. Mr. Hennings wants more agency involvement and they will solicit additional members.

HANDOUT 1 ACPA Course Guide Mr. Steve Mueller, FHWA, noted that the cost to participate in the ACPA Education and Training webinars is $35.00 per participant per session. Mr. John Roberts, ACPA, added that interest in the webinar series is much greater than anticipated. In 2008, over 1000 people participated in the courses, and enrollment varied between 16 and 200 people per session. Mr. Mueller suggested that the course fee be waived if CEU's aren't awarded. Mr. Roberts stated that because of administrative costs, ACPA must charge for the webinars. Mr. Hennings stated that he could visit organizations and present the identical material at no cost to the organization or the participants.

Update on Activities of the Concrete Pavement Center

Dale Harrington stated that he represents the National Concrete Pavement Technology Center (CP Tech Center). HANDOUT 2 PRESENTATION The Center works with Industry, Federal, and State agencies. The CP Tech Center developed a FHWA Concrete Pavement Preservation Workshop HANDOUT 3, and there are 9 engineers that present it throughout the country. There is still availability to present it to a few more states and there is no cost associated with the course. The 10 module course can be presented in one day, but the participants will benefit if the training is covered over 1 ½ days.

Mr. Sorenson stated that FHWA has been working with CP Tech Center at Iowa State University to come up with an infrastructure-based course for professors to instruct students. FHWA is bringing professor training back within the next few weeks and is creating college level curriculum to assist the professors. Mr. Sorenson stated that the CP Tech Center must make the presentations interesting enough to spark demand. Mr. Mueller suggested that the instructors might benefit by participating in National Highway Institute's (NHI) Instructor Certification Program, which teaches effective teaching techniques. Mr. Hennings stated that IMC Construction has a construction training manual and that Mr. Harrington is teaching the course at no charge. Mr. Sorenson stated that FHWA needs to expand construction training and it's important for the same message to be relayed everywhere. There are many state agencies that don't have enough experience with PCC to train their own staff. Mr. Hennings stated that he taught a 6-hour "Just-in-Time" training in NV and CA.

Pavement Preservation: Acceptance and Implementation Subcommittee

Mr. Larry Galehouse, National Center for Pavement Preservation (NCPP), reviewed the strategic plan and feels that the long and short term goals need to be updated. He would like to see the aggregate industry involved in the ETG group. Mr. Galehouse stated that during the Southeastern Pavement Preservation Partnership (SEPPP) conference [held during the previous 2 days in New Orleans], there was discussion regarding some states being fragile and that a few bad jobs will throw them off track and certain pavement preservation processes won't be implemented if there are problems. He suggested that training and certification may ensure a better product. Mr. Chris Newman (FHWA) briefly addressed individual certification. Mr. David Peshkin, Applied Pavement Technology (APTech), has a project to determine what competencies and training should be required for certification. Mr. Newman pointed out that both DOT and industry want training, and that everyone wants a positive outcome. Mr. Sorenson pointed out that the owner agency and contactors would take identical training. Mr. Galehouse inquired about certification with a Disadvantaged Business Enterprise (DBE) and Mr. Newman pointed out there is a difference between individual and corporate certification. Mr. Newman stated that many DBE's are new corporations that are inexperienced and need training.

Mr. Galehouse discussed greenhouse gas and energy uses. He noted that in Canada, tax assessments are based off energy use. Mr. Galehouse stated we need to look at how to determine what is the most eco-efficient process, and which type of work is eco-friendly. There are different components: land use, energy consumption, etc. There are a number of various steps to determine efficiency. He pointed out that pavement management is one of the most important aspects of reducing energy. He added that we must know what the performance is for everything we do. Mr. Peshkin added that the University of Washington is looking into transferring "green" concepts to pavements. The ETG needs to see what others are doing and see what the other benefits are, and formulate them into the ETG format. Mr. Mueller added that New York, Washington, New Hampshire, and the University of Wisconsin have developed models. He said the FHWA needs to be involved with looking at the various studies. Mr. Mueller stated that pavement preservation movement wins whenever the "green" card is a factor. How do you certify a road as "green"? Mr. Peshkin suggested a task force to address the modules, training, promoting sustainability, and so on that could break out without the ETG that would address this issue. Mr. Mueller stated that there is already a similar committee on FALCON. Mr. Sorenson suggested six or eight folks work together and make a position paper. He feels there needs to be basic research, facts, and an evaluation. He suggested running a literature search and seeing what information is available. He added that the PCCP industry is interested in their industry's carbon footprint. Mr. Roberts added that Indiana is in the process of developing a way to determine the amount of impact on the environment. Mr. Sorenson stated that there are many studies underway, and a group needs to see what is being done and summarize it within 6 months. Mr. Roberts, Mr. Mueller, Mr. Chaignon, Colas, Inc., Mr. Fred Mello, BASF Corporation, and Colin Durante, Pavement Technology, Inc., will prepare a 3 to 5-page white paper.

Support Research Programs Subcommittee

Mr. Gregory, FHWA, reported that Todd Thomas has resigned from the PPETG and that Ms. Anita Bush, Nevada DOT (NDOT), has volunteered to Chair the Committee. Mr. Gregory outlined the following long and short term goals:

Short Term Goals

- Establish core group for the Research subcommittee for PPETG
- Continue to track the Pavement Notebook (NCHRP 1-46) to engage in the pavement preservation section.
    - Follow up with Larry Lockett
- Support the TRB Pavement Preservation Committee (headed by Larry Galehouse) and review TRB papers
    - Paper submitted August 1
    - Mid-to-late August for Review

- Submit to Larry each of our interests

Long Term Goals

- Coordinate with the AASHTO TSP2 to:
  - Monitor research and implementation from the TSP Roadmap
  - Review priorities of the TSP Problem Statements
  - Update problem statements.
- Integrate and monitor the research groups from each of the Regional Pavement Preservation Partnerships

Mr. Sorenson welcomed Ms. Bush, and noted that her experience will be beneficial to the PPETG.

Support PP Centers for Excellence and Regional and State Organizations Subcommittee

Ms. Lita Davis, Friend of the Committee, submitted the subcommittee report. HANDOUT 3A Ms. Davis reported that their committee had members at TRB's Annual Conference and they had identified the papers that were presented that involved pavement preservation. She stated that Mr. Craig Olson, APWA, has joined her subcommittee. She added that her committee is well balanced, with NACE and APWA represented. She would like to have a member from the aggregate industry represented. Ms. Davis continues to solicit photographs, documenting the different processes. It's important for future projects. Mr. Sorenson inquired about the status of the Speakers Bureau. Ms. Davis stated that during the April 2008 PPETG meeting, it was determined that the LTAP centers are being contacted by groups looking for speakers, and that the amount of work required to maintain an accurate list of available speakers isn't worth the committee's time.

Pavement Preservation Training and Certification Subcommittee

Mr. Newman stated that there are gaps in training and certification, both in the inspection and contractor workforces. He suggested coordinating certification with Agency QC and QA, but he needs to see what the other committees suggest. He acknowledges that there are many barriers regarding certification, depending if you represent Industry, Agency, a company, or an individual. He would like to find a way to take advantage of all the conversations, making a coordinating role to be involved at the partnership level. He would like to add a new long term goal to the strategic plan: to develop a draft guideline for agencies that want certification. He added that it won't be a Federal program, it will be a State program. He would like the ETG to identify the components of a certification program.

Mr. Jackson asked that the Subcommittee Chairs and Co-Chairs prepare and send an updated Strategic Plan to him as soon as possible. He added the pavement preservation movement is moving forward at a rapid pace, thanks to the dedication of the PPETG members.

Summary of the St. Louis CPTP Conference

Mr. Kurt Smith, APTech, reported the 2 ½ day conference had 150 attendees. PowerPoint HANDOUT 4 He stated that there is more movement towards preservation. He reviewed what is new and what remains the same (Slides 11 and 12) with regards to treatments, texturing, applications, and so on. Mr. Mueller pointed out that cross stitching isn't part of Caltrans maintenance treatments, and Mr. Rouen added that it isn't in the Caltrans Guide. Mr. Scofield suggested that, using a pooled fund study, Industry could pinpoint where and when seals should and should not be used.

Illinois DOT (IDOT) Concrete Overlay/Inlay Design Procedures

Mr. Matt Zeller, ACPA, presented Mr. Jeffery Roesler's PowerPoint presentation on Concrete Pavement Solutions for Lower Volume Roads. HANDOUT 5

Future Direction and Mission of the PPETG

There was general discussion regarding the future direction of the PPETG and the differences and similarities between the ETG and Emulsion Task Force (ETF). Ms. Simone Ardoin, LA DOTD, asked how to acquire data regarding different states' experience with new products. Mr. Sorenson stated that the information is not public domain and that having new products evaluated is costly for the manufacturer. He added that he expects FHWA's Division Offices to work with their own state to develop their own processes. He suggested that partnerships could be used as a sounding board for an emerging technology page. He added that emerging technology has got some proprietary specifications and warranty provisions and that they are starting to get some performance-based specifications. Mr. Mike Voth, FLHD, inquired how the ETG and ETF should interact. Mr. Sorenson stated the ETF is a high volume, short term, committee which he suspects will last between 5 and 10 years. He feels that SuperPave would have benefited with support from a group like the ETF. He added that the PCCP committee is a task force that is a more permanent in structure to the ETG. Mr. Peshkin stated that the ETG needs to meet twice a year and there must be earlier notification of meeting dates. Ms. Davis will provide a guideline for subcommittees to successfully utilize teleconferencing. HANDOUT 6 Mr. Sorenson concluded the discussion by stating there is still a tremendous amount of work to be done in pavement preservation, and that the strong FHWA representation on the ETG is important to show what information is available to all groups.

Mr. Sorenson posed the question of whether or not the ETG has grown to an unmanageable size and if it needs to be reorganized. Mr. Mueller stressed the importance of dedicated funding, and added that it's hard to have a healthy economy when your infrastructure is crumbling. Mr. Sorenson added that AASHTO has endorsed a revised version of *Rough Roads Ahead*, which will soon be available for download on the NCPP's website. HANDOUT 7 Mr. Rice feels that the ETG should work with other agencies so there is a consistent message. Discussion continued and the general consensus was that the ETG and their subcommittees should remain. Some selected comments were:

- Mr. Denehy: NYS DOT has benefited from the ETG and since his tenure as chair of the NEPPP is over, he'll be able to devote more time to his subcommittee.
- Mr. Mueller: Need more focus on dedicated funding. Preservation is the "green" movement. Mr. Mueller would also like more ideas from the Pavement Preservation Acceptance and Implementation Committee.
- Mr. Steve Varnedoe, NCPP: Would like the group to address performance accountability, and would like to establish a threshold for accountability.
- Ms. Tammy Sims, TX DOT: Would like to address pavement preservation techniques that promote road safety.

Mr. Sorenson added that the FHWA is looking at pavement treatments to improve roadway safety. He added that FHWA provides funding and staff support for the PPETG, and that the ETG provides its own direction. He suggested putting a focus group together and work with the ETF to address treatments that promote safety.

- Mr. Luis Rodriguez, FHWA: His main concern is the availability to coordinate efforts with regional partnerships. They are working on similar tasks and we need a process to coordinate efforts.

Mr. Sorenson stated that the regional partnerships are new and are self driven. He added that the ETG needs to support their work.

- Ms. Davis: Charter agencies fighting for funding out of the general fund. There is a need to identify different funding avenues, like safety.

Mr. Sorenson feels that there will not be additional funding for cities or counties. He added that at the federal level, the only available funds are from the gas tax. Beyond that, it varies from state to state and they want to stretch their funds.

Mr. Sorenson showed the WGN Channel 9 Chicago news interview with Mr. Peshkin. http://www.youtube.com/watch?v=u14Us9A3EQY. Mr. Jackson stated that 15 years ago, there would not have been any discussion regarding potholes but now the deterioration of the roadways has become a lead story.

Portland Cement Concrete Pavement (continued)

Status of Quiet Pavement Research

Mr. Scofield presented his PowerPoint presentation on Status of Quiet Pavement Research. HANDOUT 8 He added that AASHTO has a specification for test equipment, which is different in Europe. He stated that the noise level is gauged off surrounding dwellings, not the noise heard within the car. The contract with Purdue University looks at rehab and new construction, and will continue looking at different grinding patterns for the next 1 ½ years. Purdue University's challenge is how to add friction without increasing noise. Mr. Scofield noted that the Anisotropic Friction Behavior test should have been performed at 40 degrees, but due to the weather conditions, their tests were performed at 33 to 35 degrees.

SHRP R26: Preservation Approaches for High Traffic Volume Roadways

Mr. Peshkin presented his PowerPoint presentation on SHRP R26. HANDOUT 9 The survey was sent to all 50 states and overseas. Mr. Peshkin noted that that respondents to the survey determined what qualified as a low or high volume road. The object of the report is to determine guidelines. All the results for high traffic have been gathered and Mr. Peshkin would like industry input, which Mr. Moulthrop will coordinate. Mr. Peshkin added that when the agency was asked how long is the typical life of their pavements, that the answer is limited by the background and knowledge of the person completing the survey.

PP Training and Certification

American Recovery and Reinvestment Act Funding and Workforce Development
ATTACHMENT 4 Mr. Newman briefly discussed the attachments which addressed the Recovery Act. Additional information is available at http://www.recovery.gov/.
He stressed that inexperienced personnel would benefit from participating in training, conferences, and short courses.

Review of the TCC pavement Preservation Curricula Matrix

HANDOUT 10 Mr. Peshkin stated that the document is a draft version for asphalt and concrete pavements. The contract is still open and Mr. Newman wants input regarding possible additional competencies. Mr. Mueller asked about a formal review process and Mr. Newman advised him that the matrix would be included in these minutes and also sent out as a separate review document.
The website http://www.nttr.dot.gov/Home.aspx?AspxAutoDetectCookieSupport=1 has links to all training opportunities and is a great reference. Mr. Newman and Mr. Peshkin would like the members of the PPETG to look at the draft and see if additional information should be included. HANDOUT 10A Please send all comments to Mr. Peshkin.

Asphalt Emulsion Course, Phase 1

Mr. Newman stated that the scoping meeting is going on. Mr. Sorenson's vision is a week-long, in depth training course that is "hands on" instruction. This course should address the most effective use of treatments. Mr. Sorenson added that it's imperative that the participants understand what is taught in the laboratory. The upcoming Phase 1 meeting will identify who the target audience will be. States and Industry are working to make sure that the course addresses their needs. Development should start the beginning of 2010.

New Agenda Item: **Maintenance Leadership Academy**

The Maintenance Leadership Academy is a 4-week course that FHWA is developing with TCCC and NHI. HANDOUT 11 It's a blended course with self-directed and web-based studies. They are working with NHI to determine how to distribute individual modules. It will be piloted in Fall 2009 and they are negotiating with Texas to have it taught for the entire 4 weeks. It will be available for a national audience in Spring 2010. The course was written for the function of the Maintenance Supervisor and not just an Engineer's perspective. Mr. Peshkin added that personnel issues are addressed and Human Resources are included in the training matrix.

<u>Support Research Programs</u>

Phase 2-Crack Sealant Consortium Pooled Fund

Mr. Denehy discussed the Crack Sealant Consortium Pooled Fund Study. ATTACHMENTS 11, 11A, 11B It is an exhaustive study for crack sealant materials and will produce good guidelines for material selection and performance. The materials will be put in place in the field and will be monitored over a 4-year period. This is a Pooled Fund Study and it will result in industry supplying products that work. Mr. Denehy stated that all the tests are written to become an AASHTO standard. Mr. Sorenson added that in order to get it adopted it needs to be on the AASHTO agenda and someone needs to champion it as an interim specification. Mr. Sorenson feels the ETG can endorse the project and promote the pooled fund study. Mr. Galehouse and Mr. Varnedoe need to make the states aware of the study. The agenda for the AASHTO SCOM meeting in July may be closed. Mr. Rodriguez stated that Mr. Jack Springer will attend the August 2-7 AASHTO Subcommittee on Materials Conference. Mr. Sorenson added that Mr. Colin Franco, RI DOT, is also on the committee and can request that it be added to the agenda.

Final Contract Report- Draft
Development of Performance-Based Guidelines for Selection of Bitumionous-Based Hot-Poured Pavement Sealant: An Executive Summary Report ATTACHMENT 11C
Sealant and Specifications ATTACHMENT 11D
VOA Practice ATTACHMENT 11E

NCHRP 2010 Project Panel

Mr. Denehy reported that the NCHRP 2010 Project Panel list is available and there are several projects in the pavement preservation arena. ATTACHMENT 12 Mr. Denehy briefly pointed out some areas of interest.

NCHRP 14-17, Manual for Emulsion-Based Chip Seals for Pavement Preservation

Also, Mr. Denehy stated that he is the panel chair for NHCRP 14-17, a project to develop a manual on emulsion based chip seals for pavement preservation. Mr. Rodriguez is the

FHWA liaison. Colorado is doing the research, and is on phase 2, which was authorized in June 2007. The have been some subcontractor changes but the work in continuing and is expected to be completed at the end of 2009. Mr. Scott Schuler is the PM on the project.

OK Pavement Preservation Treatments

ATTACHMENT 5 Mr. Caleb Riemer, OK DOT, presented his PowerPoint presentation on *Quantifying the Costs and Benefits of Pavement Retexturing as a Pavement Preservation Tool*. HANDOUT 12 In addition to the material covered in the presentation, Mr. Riemer pointed out that they used the New Zealand standard, versus the ASTM Standard for Sand Circle. This was done because the New Zealand test had more coarse sand which stayed in place during high winds. Mr. Mueller challenged Mr. Riemer and asked if the ASTM standard needs to be modified to reflect different winds. Mr. Sorenson added that a lot of technology is researched overseas because there are fewer hurdles. Mr. Mueller stated that if there needs to be a change in test protocols, they should be changed. Mr. Riemer added that Ms. Dominique Pittenger, Broce Construction, is doing her Masters on the economic analysis. Mr. Mueller stated that the FHWA has Real Cost software that can help to perform life cycle cost analyses and will get Ms. Pittenger a copy. Mr. Sorenson mentioned that Mr. Peshkin prepared NCHRP 14-14 and that there was software with it. Mr. Peshkin added that it is an experimental design aimed at optimal timing. He added that there is a spreadsheet that can be used to perform benefit cost analyses. Mr. Sorenson wants Oklahoma to look at the parameters so the test results will be applicable to other states. Mr. Riemer asked how best to integrate the study into a National Program, other than working with DOT's. Mr. Sorenson suggested Mr. Riemer work with the ETG for peer review.

PP Training and Certification

Texas PP Center Services

Mr. Yetkin Yildirim, Texas Pavement Preservation Center, presented his PowerPoint presentation on the Texas Pavement Preservation Center. HANDOUT 13 Mr. Yildirim reiterated that they focus mainly on training but they do submit proposals to TX DOT for research projects. Information regarding training courses and additional information can be found at http://www.utexas.edu/research/tppc/.

TSP2 Update

Mr. Sorenson provided the TSP2 update and briefly discussed the handouts. ATTACHMENT 6, 7A, and 7B He stated that 34 states have provided funding to maintain the service desk at MSU and that AASHTO is moving into bridge preservation.

TSP2 is overseen by an owner/agency panel and is a program of technical services. Additional information is available at http://www.tsp2.org/news/welcome.php.

National Center for Pavement Preservation

Mr. Varnedoe stated that the NCPP website at http://www.pavementpreservation.org/ will be significantly enhanced this summer. He would like the ETG members to post their presentations on the website, where there are currently ~2000 documents. Their website has approximately 3000 users per month. There is an online survey, still under construction. The long term goal is to track ongoing research at NCHRP and at the State level for each item in the research roadmap. The regional partnerships are the key needed to drive the national agenda. Within the last few years, they were getting organized and developing working relationships. For example, Louisiana teamed up with Texas regarding chip seals. They are having formal discussions and facilitating contacts which have had a positive impact. As Mr. Rodriguez mentioned earlier, there needs to be coordination among the ETG and regional partnerships. Mr. Varnedoe stated that getting funding from states has been difficult this year and that AASHTO needs to see that there is value added. Mr. Denehy asked about getting financial accountability from AASHTO regarding the NCPP's use of state pooled funds. Mr. Sorenson pointed out that AASHTO funds are not intended to be regionalized, and that it is a nationwide program. He added that some states volunteered more money to benefit the country. However, there were some financial accountability issues at AASHTO which have since been rectified. Mr. Sorenson states the Mr. Varnedoe met with the AASHTO staff and helped organize their financial spreadsheets. Mr. Varnedoe stated that no AASHTO overhead is taken out of the money that is volunteered and that AASHTO allows states that haven't volunteered any money to still participate in the program. Mr. Denehy added that he was under the impression that funds not spent on meetings would be spent regionally. Mr. Varnedoe added that there are specific pooled fund projects that cover travel and technical support/help desk, and has all been rolled into one lump sum. ATTACHMENT 8 Legislative Packet

<u>Support PP Centers for Excellence and Regional and State Organizations</u>

Texas DOT "Underseal" Tack Coat for Asphalt Paving

Ms. Tammy Sims, TX DOT, presented her PowerPoint presentation. PowerPoint Presentation HANDOUT 14

NTPEP Crack Study

Mr. Jerry Geib, MN DOT, briefly discussed the NTPEP Crack Study. ATTACHMENT 9

KGO (Karl Gunnar Ohlson) III – Swedish Mix

Mr. Geib briefly discussed the performance of flow mixing technology. PowerPoint HANDOUT 15 He feels the technology of properly mixing asphalt and concrete is an area of interest. HANDOUT 15A, 15B, and 15C

Penn DOT 070507: Bituminous Overlay Strategies for PM on Interstate Roadways

Mr. Peshkin reported on this project that APTech is performing for PennDOT. So far the project has identified what the Districts and other agencies are doing for preservation of high volume HMA roadways. Mr. Peshkin reported that unfortunately, the personnel at PennDOT that were interested in this project are no longer involved. There is strong interest in maintaining an HMA overlay program, but not so much interest in other treatments. He added that personnel changes directly impact pavement preservation. Mr. Sorenson added that FHWA is willing to help, and that Mr. Peshkin should keep him informed and they'll go back to the Division office.

ADOT SPR 628: Evaluation of Maintenance Strategies for ADOT

Prior to retiring from ADOT, Mr. Scofield initiated a field study of maintenance strategies in Arizona. Mr. Peshkin reported on the current status of the follow-up evaluations and the report on the test sites that is under development. Mr. Scofield provided some insights on the initial data collection efforts that had been planned, using photologging technology and automated interpretation of the photo record. The report will address two of the three phases covered in the project: HMA surfaces and preservation treatments. The sealer/binder study is being reported on by Mr. Gayle King through the FPP/FHWA study. Unfortunately, some of the findings of this project provide another example of what happens when a champion stops being associated with a project before it is over.

NCHRP FY 2010, Caltrans Funded Projects

Mr. Rouen briefly discussed the NCHRP projects that Caltrans has been awarded. ATTACHMENT 10 Mr. Sorenson added that there is a call for panel members. If Caltrans put in the three problem statements, they may want to look at what is important to the ETG and make sure that NCHRP gets the right people on the panel. Mr. Larry Orcutt, Caltrans, would be the contract for any interested industry members that want to be put on the panel.

Pavement Preservation: Acceptance and Implementation

FHWA Polymer Modified Asphalt Emulsion Study

Mr. Gayle King, GHK, Inc., presented his PowerPoint Presentation of the FLH Field Study. ATTACHMENT 13 and 13A and PowerPoint Presentation HANDOUT 16 Mr. Sorenson suggested that the material be introduced to the AASHTO Material

Subcommittee and get it into their process. Mr. Franco already has it on the agenda. In addition to the material covered during the presentation, Mr. King gave special thanks to industry for reducing the costs for sample testing. Mr. Sorenson added that the research community, owner agencies, and contractors made the project work. And through the process, they came up with a straw man specification. He suggested that they take it to AASHTO, methods A & B, and let the research committee determine which method to do. He continued by saying that they need an AASHTO provisional specification approval. Mr. King invited any PPETG/ETF members to review the chip seal performance specification.

Little Red Book – A Quick Check of your Highway Network Health

Mr. Gregory stated that the Little Red Book has been updated. ATTACHMENT 14, 14A, 14B, and 14C There was a memo distributed to the field office notifying them that it has been updated. He added that for those just want to run the numbers, they can use the calculation formulas in the spreadsheet. It was noted that FHWA is continually updating their resources and that the Toolbox needs to be updated every 3 to 4 years. Mr. Gregory asked the ETG to provide input on updating the toolbox. Mr. Sorenson added that there have been 15,000 copies produced, and that the PowerPoint presentation is available on the NCPP and FHWA websites. Mr. Rice and Mr. Olson will work with their organizations and the LTAP Centers to inform the members of the updated Little Red Book.

Economic Benefits and Performance Specifications for Pavement Preservation

Mr. King asked Mr. Sorenson if we know the economic benefits for pavement preservation. Mr. Sorenson stated that the fundamental research projects aren't done to answer that question and that they are not in the position to answer economic issues. He added that the contracting community will be miles ahead, like with Hot-Mix Paving, once they take over their own performance and process. He continued by saying that FHWA will assist states in developing a maintenance and performance based contracts. The 2-day tutorial will be available in December 2009 and has had good feedback. He added that the issue is picking measureable criteria that can be used throughout the life of the project. The criteria will be determined between the owner agency and contractor. Mr. Sorenson feels that more research is needed, and suggested writing up a problem statement, getting it into the roadmap, and getting it solicited. There is a third course, Baselining and Benchmarking, and Stage 1 was reviewed the other day. This course teaches the fundamentals of performance contracts: what is measured, what are the criteria, and what are benchmarks. This is necessary to provide comfort to both contractor and the agency. Mr. Mueller asked if there were enough tools for performance contracting. One example was in Virginia. They didn't have a pavement management system and they knew they couldn't maintain their interstate system. They pull all their interstate contracts under maintenance contracts. Mr. Franco proposed to have performance contracting in Rhode Island starting in the Fall 2009. Mr. Sorenson added that FHWA is willing to sit down and assist in developing the specifications for the contract.

Green Effort and CA Presentation on Sustainability and Preservation

Mr. Mueller made a brief PowerPoint presentation. HANDOUT 17 (original version) HANDOUT 18 (abbreviated version.) Mr. Muller stressed the importance of thinking cradle to cradle and added that lifecycle cost doesn't mean just money, it also means how much can be recycled.

Emulsion Task Force Update
Pavement Preservation Emulsion Task Force Subcommittee List

Mr. Roger Hayner, Colas, Inc., made a brief PowerPoint presentation of the ETF's progress. HANDOUT 19 Mr. Mueller suggested that the ETF review an adhesion test that the University of Illinois performed. Mr. Sorenson added that the ETF has made great progress. Mr. Hayner added that the ETF has a July 15 conference call scheduled. Meeting Notes – Pending

HANDOUT 19A Mr. Hayner emailed the Emulsion Task Force Members the draft "Certifying Suppliers of Emulsified Asphalt" and solicited comments by June 1, 2009. For reference, Mr. Hayner also included the Combined States program requirement HANDOUT 19B, Kentucky DOT EASC requirements HANDOUT 19C, and Tennessee's SOP-3 for emulsion suppliers HANDOUT 19D.

Follow up: Mr. Hayner emailed the Emulsion Task Force Members the June 15, 2009 Draft AASHTO Standard Recommended Practice for Certifying Suppliers for Emulsified Asphalt. HANDOUT 19E

Meeting, Conferences, and Events

2010 International Pavement Preservation Conference

Mr. Sorenson reported the 2010 International PP conference is under development and that Caltrans has been working with the NCPP and others and approximately 80 problem statements are being reviewed (abstracts and papers). The group meets with Gary Hicks, California Pavement Preservation Center, via conference call on a monthly basis. They have invited 8 or 9 international organizations. Mr. Sorenson would like Larry to work with regional partnerships and arrange for a side bar meeting. Mr. Sorenson suggested that the conference might be good for the International Road Federation (IRF). The IRF is having a conference, *Preserving our Highway Infrastructure Assets*, on August 4-7, 2009 in Orlando, FL. HANDOUT 20 He added that there will be small FHWA representation since the conference conflicts with the AASHTO Subcommittee on Construction meeting in Chicago. Additional conferences are posted on the NCPP website.

Newsletter of the California Pavement Preservation Center March 2009 ATTACHMENT 15

In-Place Recycling Conference in Minnesota

This conference is tentatively scheduled for August 25-27 in Minnesota. The dates will be finalized once they've confirmed an onsite visit. They did a similar onsite visit in Salt Lake City, Utah in June 2008. They would like to organize the same type of onsite workshop in the Northeast and Southeast.

Next Meeting

Mr. Jackson suggested that the PPETG meet twice a year, in addition to conference calls. The next meeting is tentatively scheduled for Monday, November 16 and Tuesday, November 17, 2009 in Reno/Sparks, Nevada. Ms. Bush is willing to help with the arrangements. It was suggested that the Subcommittees meet for a longer period of time with separate break out rooms.

Before adjourning, Ms. Bush stated that she appreciated the invitation to join the PPETG and is excited to support pavement preservation. Mr. Riemer is excited to work with young pavement preservation engineers, and Ms. Ardoin was thrilled to attend the meeting in place of Janice Williams. Mr. Mueller concluded the solicited comments by stating the importance of picking up Environmental Stewardship.

The meeting adjourned at 3:00pm on Friday, May 15, 2009.

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Updated 7-27-09

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Expert Task Group on Pavement Preservation
They won't buy it if we can't build it *right*!
May 14-15, 2009
Hotel Monteleone
New Orleans, LA

Thursday, May 14, 2009

1:00 **Call to Order**

- Introductions – Jim Sorenson, FHWA
- Welcome to new PPETG members and guests – Denny Jackson, KBA
  Attachment 1 – Membership List
- Review and approval of meeting notes (October 2007) – Denny Jackson, KBA
- Action Items from Newport Beach – Denny Jackson, KBA
  Attachment 2
- First Words – Denny Jackson, KBA

Break out into Subcommittees 1 hour, 45 minutes

- ♦ **Break 2:45pm**

Attachment 3 – Strategic Plan

Subcommittee Updates- Short and Long Term Goals – Denny Jackson, KBA Moderator

- Pavement Preservation: Acceptance and Implementation – Mike Voth, FHWA 10 Minutes
- Support Research Programs – Joe Gregory, FHWA 10 Minutes
- Support PP Centers for Excellence and Regional and State Organizations – Lita Davis 10 Minutes
- PP Training and Certification – David Peshkin, APTech 10 minutes
- Portland Cement Concrete Pavement – Craig Hennings, ACPA 10 Minutes

- Portland Cement Concrete Pavement – Denny Jackson, KBA
  - Update on Activities of the Concrete Pavement Center – Dale Harrington, Snyder & Associates 10 Minutes
  - Summary of the St. Louis CPTP Conference – Kurt Smith, APTech 15 Minutes
  - IL DOT Concrete Overlay/Inlay Design Procedures – Matt Zeller, Concrete Paving Association of MN 10 Minutes

Discussion Item

- Future Direction and Mission of the PPETG – Lita Davis, Friend of the Committee 15 Minutes

Reception 5:00 – 6:00 Royal Ballroom A & B

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Friday, May 15, 2009

7:30 – 8:00 **Continental Breakfast, Royal Ballroom C & D**

Discussion Item

- Overnight Thoughts and Future Direction of the PPETG – Jim Sorenson, FHWA   8:00 – 8:15

- Portland Cement Concrete Pavement *(Continued from Day 1)*
  - Status of Quiet Pavement Research – Larry Scofield, ACPA
    15 Minutes
  - SHRP R26: Preservation Approaches for High Traffic Volume Roadways – Research Report Update - David Peshkin, APTech
    15 Minutes

- PP Training and Certification – Jim Sorenson, FHWA
  All Training and Certification Topics: 45 Minutes
  - American Recovery and Reinvestment Act Funding and Workforce Development – Chris Newman, FHWA
    Attachment 4
  - Review of the TCCC Pavement Preservation Curricula Matrix – Chris Newman, FHWA and David Peshkin, APTech
  - Asphalt Emulsion Course, Phase 1 – Chris Newman, FHWA

  Discussion Item
  - Contractor Certification Process – Chris Newman, FHWA and David Peshkin, APTech   15 Minutes

    Break   9:45 – 10:00

  - OK Pavement Preservation Treatments – Caleb Riemer, OK DOT and Dominique Pittinger, University of OK   10 minutes
    Attachment 5

- PP Training and Certification – *(Continued)*
  - Texas PP Center Services – Yetkin Yildirim, TPPC   10 Minutes
  - TSP2 Update – Jim Sorenson, FHWA   5 Minutes
    Attachment 6, 7A, and 7B
  - National Center Pavement Preservation – Larry Galehouse, NCPP
    Attachment 8 – Legislative Packet   10 Minutes

- Support PP Centers for Excellence and Regional and State Organizations – Denny Jackson, KBA
  - NTPEP Crack Study: ETG Support to AASHTO – Jerry Geib MN DOT   10 Minutes
    Attachment 9
  - KGO (Karl Gunnar Ohlson) III Swedish Mix – Jerry Geib, MN DOT   5 Minutes
  - Penn DOT 070507: Bituminous Overlay Strategies for PM on Interstate Roadways – Project and Status Update – David Peshkin, APTech   5 Minutes
  - ADOT SPR 628: Evaluation of Maintenance Strategies for ADOT – David Peshkin, APTech   5 Minutes
  - Texas DOT "Underseal" Tack Coat for Asphalt Paving – Tammy Sims, TX DOT   15 Minutes

- NCHRP FY2010, Funded Projects – Larry Rouen, Caltrans
  20 Minutes
  Attachment 10

- <u>Support Research Programs</u> – Denny Jackson, KBA

  - Phase 2-Crack Sealant Consortium Pooled Fund – Ed Denehy, NYS DOT   15 Minutes
    Attachments 11, 11A, and 11B
  - NCHRP 2010 Project Panel – Ed Denehy, NYS DOT
    Attachment 12

Lunch Break, Hunt Room Grill, Ground Floor   Noon – 1:00

- <u>Support Research Programs</u> – *(Continued)*

  - NCHRP 14-17, Manual for Emulsion-Based Chip Seals for PP – Ed Denehy, NYS DOT   15 Minutes

- <u>Pavement Preservation: Acceptance and Implementation</u> – Jim Sorenson, FHWA

  - FHWA Polymer Modified Asphalt Emulsion Study
    Next Steps: Additional testing, AASHTO Provisional Specifications, State Participation, and Funding Needs – Gayle King, GHK   20 Minutes
    Attachment 13 and 13A
  - "Little Red Book" – A Quick Check of your Highway Network Health – Joe Gregory, FHWA   5 Minutes
    Attachment 14, 14A, and 14B
  - Progress Towards Implementing R & D Roadmap through Pooled Fund TSP2, NCHRP, or Partnerships – Steve Varnedoe, NCPP
    15 Minutes
  - Economic Benefits and Performance Specifications for Pavement Preservation – Gayle King, GHK   5 Minutes
  - Green Effort and CA Presentation on Sustainability and Preservation– Steve Mueller, FHWA and Mark Ishee   10 Minutes

  - Emulsion Task Force Update – Roger Hayner, Colas   30 Minutes

    <u>Meetings, Conference, and Events</u>
    - 2010 International Pavement Preservation Conference – Jim Sorenson, FHWA
    - California Pavement Preservation Conference 2010 – Larry Rouen, Caltrans
      Attachment 15
    - In-Place Recycling Conference in MN

Next Meeting

Fall in Montana

- Last Words – Denny Jackson

Close-Out

Adjourn at 3:00

✯✯✯✯✯✯

AGENDA
PPETG Emulsion Task Force Meeting
May 14-15, 2009
Hotel Monteleone
New Orleans, LA

Thursday May 14th, 2009
Pontalba Room, 2nd Floor

1:00-2:00 p.m. ETF Individual Subcommittee Meetings

- Emulsion Testing & Residue Recovery Methods- Arlis Kadrmas, SEM Materials LP
- Residue Tests- Gayle King, GHK Inc.
- Aggregates, Mix Design & Performance Tests- TBD
- Approved Supplier Certification- Roger Hayner, Colas Inc.
- Inspection & Acceptance- Colin Franco, RIDOT

2:00-3:00 p.m. Subcommittee Reports to the Group

3:00-3:15 p.m. Break, ***In-Room Service***

3:15-5:00 p.m. Defining Path and Structure for Specification Development & Approvals

5:00-6:00 p.m. RECEPTION with PPETG
Royal Ballroom A & B, 1st Floor

Friday May 15th, 2009
Cathedral Room, 2nd Floor

7:30 a.m. Breakfast- PPETG Meeting Room
Royal Ballroom C & D, 1st Floor

8:00-11:00 a.m. Research Project Reviews

- Asphalt Research Consortium- Hussein Bahia/Andrew Hanz, Univ. Wisconsin Madison
- Chipseal Evaluation- Amy Epps Martin, Texas A&M Univ./Scott Schuler, Schuler Consultants
- Federal Lands Field Study Update- Gayle King/Laurand Lewandowski, PRI, et al

9:30-9:40 a.m. Break

- Emulsion Training Program Update- Mary Stroup-Gardiner

- AEMA Emulsion Handbook Review- Roger Hayner

- Federal Lands Polymer Modified Emulsions Handbook- Helen King
  See Handout 13

11:00 - 12:00 p.m. Action Item Review and Wrap-up

12:00- 1:00 p.m. Lunch Buffet
Hunt Room Grill, 1st Floor

1:00- 3:00 p.m. Join PPETG General Meeting

Attachment 2

Actions Items from April 2008 Newport Beach, CA Meeting

- Mr. Gregory will work with Mr. Galehouse to provide the NCPP link on the FHWA website.
- Mr. Peshkin will provide the ETG attendees a list of pavement preservation training.
- Mr. Rodriguez will provide Mr. Galehouse with a list of the FALCON members.
- Mr. Rodriguez will provide Mr. Sorenson the report on *Texas Chip Seals over Geotextile Fabric Project*.
- Mr. Scofield will provide Mr. Peshkin and Mr. Shatnawi the ACPA evaluation data on concrete joints that were sealed and unsealed.
- Mr. Mueller will help get local funding for the Texas Center.
- Mr. Chaignon will provide Mr. Hayner the US data on greenhouse data.
- Mr. Hennings will provide the ETG a task force member roster for the Portland Cement Concrete Pavement Committee.

PAVEMENT PRESERVATION EXPERT TASK GROUP
STRATEGIC PLAN – Updated June 12, 2009

STATEMENT OF PURPOSE

"Provide expertise to advance pavement preservation"

GOALS

- Pavement preservation acceptance and implementation
- Support research programs
- Identify and address customer needs
- Support pavement preservation Centers for Excellence and Regional and State Organizations
- Integrate preventive maintenance into pavement management systems

Pavement Preservation Expert Task Group

Strategic Plan - Subcommittee Roster

Subcommittee	Members		
Pavement Preservation Acceptance & Implementation	**Larry Galehouse** **Jeff Forster** Chris Newman	Mike Voth Mark Ishee Francois Chaignon	Dominique Pittenger Jim Moulthrop Bill O'Leary
Support Research Programs	**Anita Bush** Joe Gregory Janice Williams Colin Franco	Russell Thielke Paul Montgomery Harold Von Quintus	Nick Burmas Caleb Reimer Simone Ardoin
Support Pavement Preservation Centers and Regional and State Organizations	**Lita Davis** Steve Mueller Larry Rouen Yetkin Yidirim	Janice Williams Delmar Salomon Bill O'Leary Craig Olson	Denny Jackson John Rice Tammy Sims
Pavement Preservation Training and Certification	**Chris Newman** **David Peshkin** Colin Durante John Vance	Ed Denehy Jerry Geib Luis Rodriguez	Larry Galehouse Jim Moulthrop Dennis Wofford
Portland Cement Concrete Pavement	**Craig Hennings** Angel Correa John Roberts Jim Tobin Matt Zeller Kurt Smith	Joe Gregory Wouter Gulden Delmar Salamon Mary Stroup-Gardner Larry Rouen	Steve Healow Larry Scofield Janice Williams Anita Bush Steve Varnedoe

Note: Names in boldface are Subcommittee Chairs or Co-Chairs

ACTIVITIES (SUBCOMMITTEES)

Pavement Preservation Acceptance & Implementation – Larry Galehouse and Jeff Forster, Co-Chairs

Short-Term Goals

- Encourage participation by the aggregate industry in ETG activities.

Ongoing Activities – Will provide progress on each of these ongoing activities for ETG meeting.

- ♦ Produce technical papers and provide articles of interest from the ETG for the Pavement Preservation Journal and other publications.
- ♦ Encourage new pavement preservation programs among local agencies and planning organizations. These customers include City, County, MPO's/ RPO's, NACE, and APWA.
- ♦ Encourage continued involvement of NACE and APWA.
- ♦ Promote Pavement Preservation by sending out toolboxes and literature to AASHTO, LTAP, APWA, NACE, TRB and Industry when new or revised information is available.
- ♦ Provide NCPP, our marketing agent, with customized marketing messages according to various customer needs.
- ♦ Review customer information already collected through existing surveys.
- ♦ Determine the current needs from the TSP2 help desk (at NCPP) by data mining questions types or information requests.
- ♦ Ascertain differences between existing surveys and current needs. Determine if a new survey is needed to address information gaps.

Long-Term Goals

- ♦ Draft a plan for implementation of needs and product technology for customers.
- ♦ Develop documented and validated benefits of pavement preservation for marketing purposes

Support Research Program – Anita Bush, Chair

Short-Term Goals

- ♦ Establish core group for the Research subcommittee for PPETG
- ♦ Continue to track the Pavement Notebook (NCHRP 1-46) to engage in the pavement preservation section
    - ▪ Follow up with Larry Lockett
- ♦ Support the TRB Pavement Preservation Committee (headed by Larry Galehouse) and review TRB papers
    - ▪ Paper submitted August 1
    - ▪ Mid-to-late August for Review
    - ▪ Submit to Larry each of our interests

Long-Term Goals

- Coordinate with the AASHTO TSP2 to:
    - Monitor research and implementation from the TSP Roadmap
    - Review priorities of the TSP Problem Statements
    - Update problem statements
- Integrate and monitor the research groups from each of the Regional Pavement Preservation Partnerships

<u>Support Pavement Preservation Centers and Regional and State Organizations – Lita Davis, Chair</u>

Short-Term Goals

- Develop and circulate a web-based survey to LTAP centers that work on pavement preservation. Purpose of the survey is to identify the types of training, and network of speakers, available on pavement preservation
- Review and comment on web sites for
    - FHWA Preservation
    - Iowa State University – Concrete Pavement Technology Program
    - Nation Association of County Engineers (NACE)
    - National Center for Pavement Preservation (NCPP)
- Develop a plan to improve information sharing between the Pavement Preservation Partnerships (PPP)
- Work with NCPP to identify and promote best practices developed from the state assessment surveys
- Promote the "red book" (<u>A Quick Check of Your History Network Health</u>) to APWA, LTAP, NACE and PPP to reach state and local agencies

Long-Term Goals

- Obtain pictures/videos of construction activities showing PP techniques for flexible and rigid pavements; forward for posting on NCPP as public domain
- Provide FP2 and NCPP with links to web sites for upcoming conferences and workshops
    - Solicit assistance from APWA and NACE
- Provide FP2 with links to PP award programs given by industry and non-profit associations (eg; AEMA, ARRA, ISSA, CPPC, CCSA, etc.)
- Develop a draft proposal to promote the distribution of PPP information, to local agencies, via State DOTs
- Develop a plan to open an alliance with Metropolitan Planning Organizations (MPOs) to promote pavement preservation

Pavement Preservation Training and Certification – David Peshkin and Chris Newman, Co-Chairs

Short-Term Goals

- ♦ Monitor and provide feedback and input to the available summaries of pavement preservation training programs. These include the TCCC and the content summarized by the National Transportation Training Resource (www.nttr.dot.gov).
    - Deadline for identifying existing training programs: Ongoing.
- ♦ Identify and prioritize training needs on the part of both industry and agencies and provide this input to the TCCC.
    - Deadline for identifying training needs: Ongoing

Long-Term Goals

- ♦ Coordinate the issue of certification across the other subcommittees and with the pavement preservation partnerships.
    - Identify how the certification program should work.
    - Who should be certified? Contractor or individuals?
    - What are agency interests in certifying either contractors of individuals? Can agency certification of their own inspectors and crews serve as a starting point?
    - What are industry interests and concerns?
    - What are you measuring?
    - What is relationship between performance-based specifications, warranties, and certification?
    - What defines the required training and certification required to assure a qualified product?
    - What entity is going to undertake certification? Who is responsible?
- Develop a straw man certification program.
    - What are the components?
    - How will it work

Portland Cement Concrete Pavement – Craig Hennings, Chair

Short-Term Goals

- ♦ Continue to refine survey results, re-solicit if needed
- ♦ Survey for CPR intervention points, use to calibrate MEPDG and refine education efforts

Long-Term Goals

- ♦ Educate agencies on cradle to grave management of PCCP
- ♦ Increase awareness of options to increase life of PCCP
- ♦ Increase awareness of CPR training opportunities by using National Center on Pavement Preservation website and regional websites
- ♦ Determine states that can contribute to pooled fund study on accelerated joint deterioration

2009 Webinars

Program and Course Guide

Revised 2nd Half 2009 Program for
August through December

The American Concrete Pavement Association
Education & Training

ACPA Education & Training
Making the world's best pavements even better!

The American Concrete Pavement Association's (ACPA's) highly acclaimed webinar series features an expanded program for 2009.

The webinars cover a broad range of topics presented in an interactive, online format.

Some of the transportation-construction community's most highly respected experts will present basic (60 minute) and intermediate (90 minute) webinars on a wide range of topics.

What's Covered?

The courses will cover virtually every aspect of design, construction, and rehabilitation of concrete pavements.

The program also will cover timely topics such as sustainability, energy, fuel efficiency, and more. There also will be application specific webinars that will focus on unique aspects of highways, or airports, roadways, or industrial/commercial pavements.

What's New for 2009?

Here are some of the highlights of our webinar program:

- Expanded from 16 to 36 webinars.
- More opportunities for interaction.
- Dynamic presentation materials.
- Special pricing on ACPA periodicals and resources.
- Special access and special invitations to ACPA sponsored education & training events, including our popular Tech Day event, scheduled for December 4, 2009.

Who Should Participate?

Almost anyone with involvement or an interest in pavements should attend. Our special 60-minute programs provide an overview of various topics; these webinars are ideal for busy people who need to know the basics.

Our 60- to 90-minute programs are designed for those who want an intermediate level course that covers various topics in greater depth.

ACPA's webinar program is designed for …

- Contractors
- Consultants
- Engineers
- Federal agencies (including FHWA and FAA)
- Military and Dept. of Defense contractors and engineers
- Municipal and county public works officials
- Metropolitan Planning Organization officials
- Academia

Some of our programs are also ideal for materials engineers, inspectors, and other agency personnel.

A few of our webinars are also well suited for facilities managers, architects, owners' representatives and other asset managers and planning professionals who need to enhance their knowledge of pavements.

Are you missing an opportunity?

If you are not an ACPA member, you may be missing an opportunity to receive valuable benefits, including tremendous savings on this and other programs. For membership information, contact Gerald Voigt, P.E. at 847-966-2272 or acpa@acpa.org.

ACPA Education & Training

Updated Schedule of ACPA Webinars

2nd half 2009 program—August through December

Number	Date	Time (Minutes)	Topic	Presenter
21	August 20	1 p.m. CST (60)	**Understanding COMPASS Mix Design Software**	Sabrina Garber, MSE., The Transtec Group
22	August 27	1 p.m. CST (90)	**Tips for Designing & Constructing Bonded Concrete Overlays**	Michael Ayers, Ph.D., American Concrete Pavement Association
23	**What You Should Know About Alternate Design/Alternate Bidding** (Rescheduled for October 19. Please see table on page 4 for details.)			
24	September 10	1 p.m. CST (90)	**The Truth About Admixtures for Concrete Pavements**	Michelle Wilson, Portland Cement Association
25	September 17	1 p.m. CST (90)	**A Guide to Reusing and Recycling Concrete Pavements**	Mark Snyder, Ph.D., P.E. ACPA-Pennsylvania Chapter
26	September 24	1 p.m. CST (60)	**How to Extend Pavement Life by Retrofitting Dowel Bars**	John Roberts, International Grooving & Grinding Association
27	October 1	1 p.m. CST (90)	**How to Troubleshoot Concrete Overlays**	Dale Harrington, P.E. Snyder & Associates and Mike Ayers, Ph.D., American Concrete Pavement Association
28	October 8	1 p.m. CST (60)	**All About Airport Pavement Specifications** *(Focus on P-501 Specs)*	Gary Mitchell, P.E., American Concrete Pavement Association
29	October 15	1 p.m. CST (90)	**Inside the Matrix: All About Testing Materials for Concrete Pavements**	Michael Ayers, Ph.D., American Concrete Pavement Association
30	October 22	1 p.m. CST (90)	**Identifying and Solving Materials Problems During Concrete Pavement Construction**	Peter Taylor, Ph.D., P.E., National Concrete Pavement Technology Center (CP Tech Center)

= Completed programs are highlighted in yellow. Look for our training-on-demand modules at www.acpa.org/bookstore.

Schedule continued on page 4...

Updated Schedule of ACPA Webinars

2nd half 2009 program—August through December

Schedule continued from page 3...

Number	Date	Time (Minutes)	Topic	Presenter
23 **(Rescheduled from Sept. 3.)**	Monday, October 19	1 p.m. CST (60)	**What You Should Know About Alternate Design/Alternate Bidding** (*Rescheduled from Thursday, Sept. 3.)*	Michael Ayers, Ph.D., American Concrete Pavement Association
31	October 29	1 p.m. CST (90)	**Contractors' Guide for Equipment Maintenance** *(No PDH credit*)*	Bob Leonard, GOMACO | Ron Meskis, Guntert & Zimmerman
32	November 5	1 p.m. CST (60)	**Tips and Techniques for Constructing Concrete Pavements in Urban Areas**	Michael Ayers, Ph.D., American Concrete Pavement Association
33	November 12	1 p.m. CST (60)	**A Guide to Fast-Track Pavement Construction**	Michael Ayers, Ph.D., American Concrete Pavement Association
34	November 19	1 p.m. CST (60)	**How to Perform Full-Depth Concrete Pavement Repairs**	John Roberts, International Grooving & Grinding Association
35	December 10	1 p.m. CST (60)	**Common Mistakes and Solutions to Concrete Curing**	Michael Ayers, Ph.D., American Concrete Pavement Association
36	December 17	1 p.m. CST (60)	**How to Repair Concrete Pavements in Urban Areas**	Michael Ayers, Ph.D., American Concrete Pavement Association

* *Please note: No test will be administered following webinar #31, and therefore, no Professional Development Hour (PDH) credit will be awarded.*

Rate Structure for the 2009 ACPA Webinars Series

Pricing and other important information.

Webinar Rates (approx. 60 minute webinars)

- $65 per webinar *(non-members)*
- $35 per webinar (members only)
- $25 per webinar (government employees)

Webinar Rates (approx. 90 minute webinars)

- $90 per webinar *(non-members)*
- $50 per webinar (members only)
- $35 per webinar (government employees)

Continued on page 5...

Registration

Register online at www.acpa.org. Click on the webinar link or ACPA E&T Program icon.

Terms and Conditions

- A $5 processing fee will be charged for each webinar canceled by the registrants or other company/ organization representatives.
- Professional development hours (PDH's) will be awarded only to individuals who score a 70% or higher on the ACPA-administered PDH exam. Persons who do not pass the exam may re-test.
- PDH's will be issued only to persons whose payments have been processed satisfactorily.
- Rates are applicable to all participants.
- Course materials are offered at the conclusion of each webinar. Course materials include a copy of the presentation, the PDH exam, and information about additional resources on each topic.
- Presentation materials (including watermarked copies of the presentations) are available to registrants at the conclusion of each webinar.
- The opinions, findings, and conclusions expressed by presenters and moderators during the ACPA webinar series, as well in associated information, is based on facts, tests, and authorities in the field of concrete pavements and related topics.
- The materials are intended for use by professional personnel competent to evaluate the applicability and limitations of the information and who will accept responsibility of the application of the material contained herein.
- ACPA and its partners and affiliates disclaim any liability for the application of the information presented in association with the webinars and cannot be held responsible for the accuracy of any of the sources, other than the work developed by the Association staff.

We're Here to Help

Your ACPA staff is available to assist you with any aspect of your webinar experience.

- Webinar Topics and Content — Michael Ayers, Ph.D., 217-621-3438 / mayers@acpa.org.
- Registration — Debbie Becker, 847-972-9802 / dbecker@acpa.org
- Billing — Marian Greco, 847-972-9834 / mgreco@acpa.org .
- General Questions — Bill Davenport, 847-972-9810 / bdavenport@acpa.org.

"Webinars 2009" (2nd Half 2009 Program: August through December) is the official program of the American Concrete Pavement Association's web-based seminar (webinar) program. For detailed descriptions of the webinars offered this year, as well as biographies of the presenters, please visit www.acpa.org. Follow the ACPA Education & Training link, or the logo shown above this paragraph.

Last update—September 3, 2009.

National CP Tech Center Update on Concrete Pavement Preservation Program

Expert Task Group on Pavement Preservation
May 14, 2009
New Orleans, LA

Concrete Pavement Preservation Program

- February 2008 FHWA and the CP Tech Center at ISU published:
  - Participant Workbook Manual
  - Instructors Guide
  - 222 page Preservation Manual
- The manuals serve as technical documents and presentation materials for the 1 -1 ½ day workshop.

- Committee of industry, State DOT and FHWA representatives
- Training provided through:
  - Manuals
  - Workshops
  - Web based Training

Concrete Pavement Preservation Program Objectives

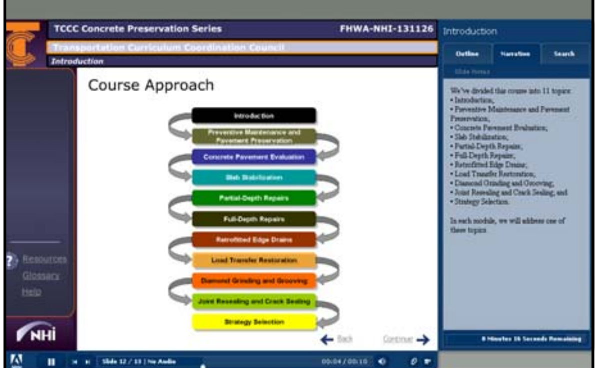
- Provide national training on Concrete Pavement Preservation techniques and practices.
- The reference manuals purpose is to provide the most up-to-date information available on
  - design,
  - construction, and
  - selection of cost-effective concrete pavement preservation strategies.
- It concentrates primarily on strategies and methods that are applicable at the project level, and not at the network level, where pavement management activities function and address such issues as prioritizing and budgeting.

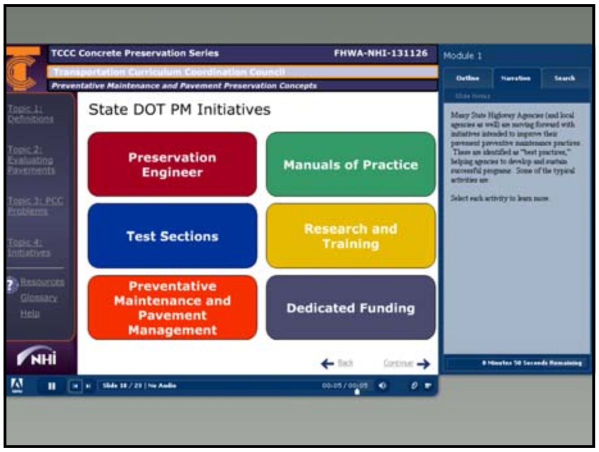
Concrete Pavement Preservation Workshops

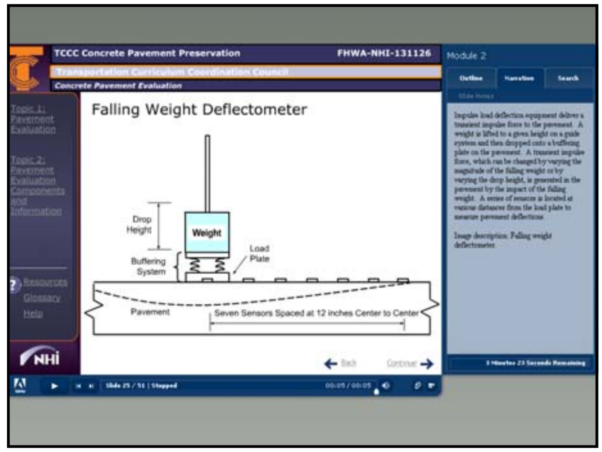
- Workshops conducted in Colorado, Kentucky, Iowa, Oklahoma & Missouri
- Workshops scheduled in Utah
- Two openings available for state workshops
- Detailed information presented on the following:
  1. Preservation Concepts
  2. Pavement Evaluation Techniques
  3. Slab stabilization and slab jacking
  4. Partial-depth repairs
  5. Full-depth repairs
  6. Retrofitted edge drains
  7. Load transfer restoration
  8. Diamond grinding and grooving
  9. Joint resealing and crack sealing
  10. Strategy Session

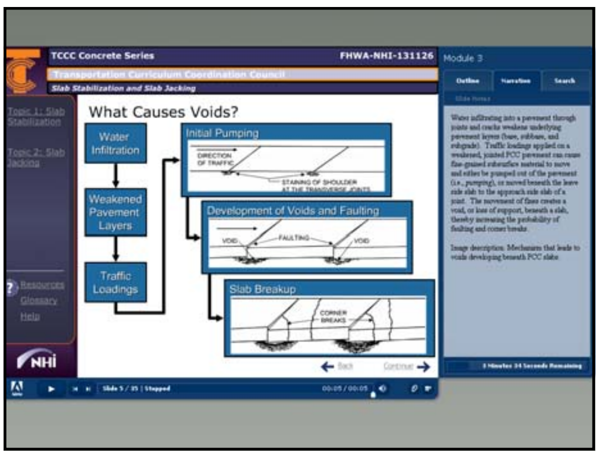
NHI/FHWA/TCCC/CP Tech Center Web Based Training

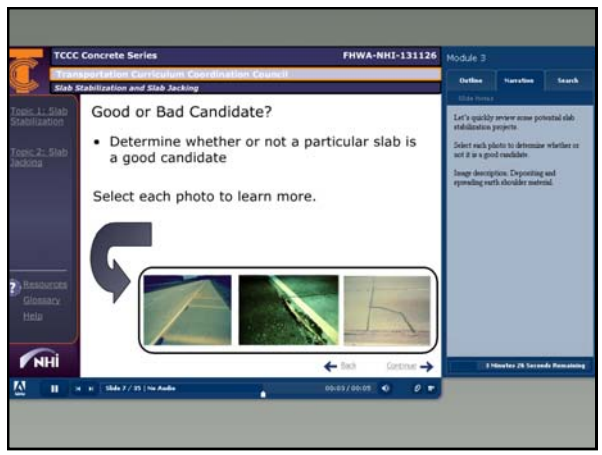
- 10 modules are developed from the preservation Instructors Guide for the web based training
- NHI/TCCC provides server space
- Project team is developing written dialogue for each module
- Iowa DOT provides technical services for the web based training
- Modules will be ready in the next few months

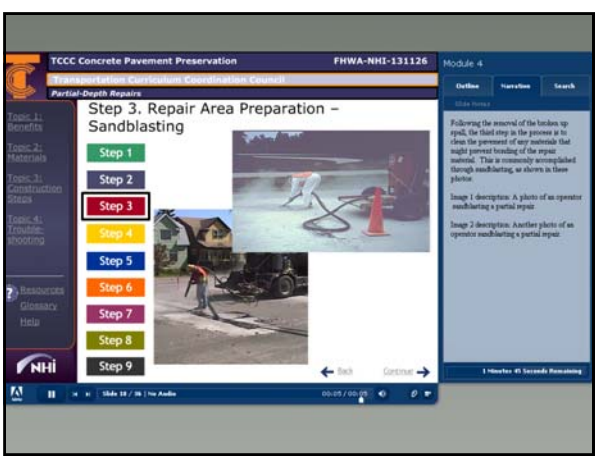












THANK YOU!

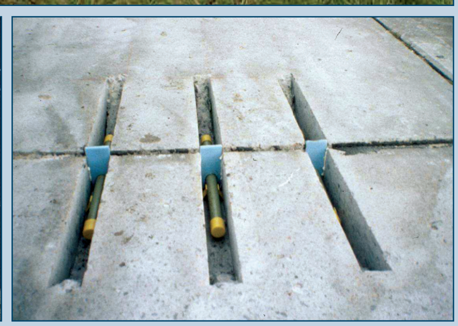
Dale S. Harrington
Representing the National Concrete Pavement Technology Center
dharrington@snyder-associates.com
515-964-2020

CONCRETE PAVEMENT PRESERVATION WORKSHOP

February 2008

Reference Manual

U.S. Department of Transportation
Federal Highway Administration



Prepared for

Federal Highway Administration Office of Pavement Technology
400 7th Street AW
HIPT 20
Washington, D.C. 20590

Prepared by

National Concrete Pavement Technology Center
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Ames, IA 50010-8664

Technical Report Documentation Page

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7. Author(s) Kurt D. Smith, Todd E. Hoerner, and David G. Peshkin	
9. Performing Organization Name and Address National Concrete Pavement Technology Center 2711 South Loop Drive, Suite 4700 Ames, IA 50010	10. Work Unit No. (TRAIS)
	11. Contract or Grant No.
	13. Type of Report and Period Covered Final January 2007 to February 2008
12. Sponsoring Agency Name and Address Federal Highway Administration 1200 New Jersey Avenue, SE Washington, DC 20590	14. Sponsoring Agency Code

15. Supplementary Notes

FHWA COTR: Gina Ahlstrom Project Manager: Dale Harrington.
Technical Advisory Panel: Gary Aamold, Angel Correa, Andy Gisi, Joe Gregory, Terry Kraemer, John Roberts, Wouter Gulden

16. Abstract

This document serves as the Reference Manual for the 1½-day FHWA workshop on concrete pavement preservation. The purpose of the document is to provide the most up-to-date information available on the design, construction, and selection of cost-effective concrete pavement preservation strategies. It concentrates primarily on strategies and methods that are applicable at the project level, and not at the network level, where pavement management activities function and address such issues as prioritizing and budgeting.

Detailed information is presented on seven specific concrete pavement preservation treatments: slab stabilization, partial-depth repairs, full-depth repairs, retrofitted edge drains, load transfer restoration, diamond grinding, and joint resealing. In addition, information is provided on pavement evaluation techniques and strategy selection procedures.

17. Key Words	18. Distribution Statement
concrete pavement, pavement preservation, pavement evaluation, slab stabilization, partial-depth repair, full-depth repair, edge drains, load transfer restoration, diamond grinding, joint sealing, life-cycle cost analysis	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classify. (of this report)	20. Security Classify. (of this page)	21. No of Pages	22. Price
Unclassified	Unclassified	222	

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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ACKNOWLEDGMENTS

The project team is grateful to the Federal Highway Administration (FHWA) for its generous support and sponsorship. In particular, we would like to thank Gina Ahlstrom, Office of Pavement Technology, who gave this project significant attention and energy during the course of the manual's development and publication.

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- Mr. Andy Gisi, Kansas Department of Transportation
- Mr. Joseph Gregory, Federal Highway Administration
- Mr. Wouter Gulden, American Concrete Pavement Association, Southeast Chapter
- Mr. Terry Kraemer, Diamond Surface, Inc.
- Mr. John H. Roberts, International Grooving & Grinding Association
- Mr. Paul Wiegand, National Concrete Pavement Technology Center, Iowa State University

Dale Harrington
Kurt Smith
Principal Investigators

CHAPTER 1. INTRODUCTION

1. OVERALL LEARNING OUTCOMES

This reference manual and the accompanying course materials have been prepared to provide guidance on the design, construction, and selection of concrete pavement preservation treatments. The overall learning outcomes of this training course are:

1. Define pavement preservation.
2. List the major components of the pavement evaluation process and the types of information gained from each.
3. Identify the purpose and suitable application of various concrete pavement preservation treatments.
4. Describe recommended materials and construction/installation practices for each preservation treatment.
5. List critical factors to consider in the selection of concrete pavement preservation treatments.

2. INTRODUCTION

The need for the effective management of transportation assets has never been greater. In an era of an aging infrastructure, ever-increasing traffic demands, and shrinking budgets, transportation agencies are continually being asked to "do more with less" in maintaining the condition of their facilities.

Pavements represent a large part of that transportation infrastructure, and the need for their effective management is just as acute. Pavements that are left to deteriorate without timely preservation or maintenance treatments are likely to require major rehabilitation and reconstruction much sooner, and those are costly and disruptive activities. Pavement preservation activities may be applied for a variety of reasons, including:

- Reduce water infiltration in the pavement structure.
- Prevent the intrusion of incompressibles into joints or cracks.
- Correct localized distress.
- Improve slab support conditions.
- Improve load transfer capabilities.
- Reduce roughness.
- Improve friction.

For concrete pavements, there are a variety of preservation treatments available to help agencies effectively manage their pavement network. However, in order for these treatments to be most effective, they must be:

- Applied to the right pavement at the right time.
- Effectively designed for the existing design conditions and prevailing design constraints.
- Properly constructed or installed using proven construction practices and procedures.

This document provides guidance on these and other critical concrete pavement preservation issues. The purpose of the document is to provide the most up-to-date information available on the design, construction, and selection of cost-effective concrete pavement preservation strategies. It concentrates primarily on strategies and methods that are applicable at the project level, and not at the network level, where pavement management activities function and address such issues as prioritizing and budgeting.

The intended audience for the accompanying training course is quite diverse, and includes design engineers, quality control personnel, contractors, suppliers, technicians, and trades people. While the course is aimed at those who have some familiarity with concrete pavements and pavement preservation, it should also be of value to those that are new to the pavement field.

3. DOCUMENT ORGANIZATION

This *Reference Manual* contains eleven chapters (including this one), which mirrors the sessions presented in the training course. These chapters include:

- Chapter 1. Introduction.
- Chapter 2. Preventive Maintenance and Pavement Preservation.
- Chapter 3. Concrete Pavement Evaluation.
- Chapter 4. Slab Stabilization.
- Chapter 5. Partial-Depth Repairs.
- Chapter 6. Full-Depth Repairs.
- Chapter 7. Retrofitted Edge Drains.
- Chapter 8. Load Transfer Restoration.
- Chapter 9. Grinding and Grooving.
- Chapter 10. Joint Resealing
- Chapter 11. Strategy Selection.

Chapter 2 provides general background information on pavement maintenance and pavement preservation, including an overview of anticipated benefits and current initiatives. This is followed by a chapter on pavement evaluation, which includes discussions on condition surveys, nondestructive testing, roughness and friction assessment, and materials and laboratory testing. These two chapters establish a strong foundation for the discussions on concrete pavement preservation treatments, which are covered in chapters 4 through 10. Each of these chapters shares the following elements:

- Learning Outcomes.
- Introduction.
- Purpose and Project Selection.
- Limitations and Effectiveness.
- Materials and Design Considerations.
- Construction.
- Quality Control.
- Troubleshooting.
- Summary.
- References.

Finally, chapter 11 describes factors to be considered in the selection of concrete pavement preservation strategies, and provides an approach to help identify suitable pavement preservation strategies for a given concrete pavement project.

4. COURSE MATERIALS

The materials for this course consist of two documents, this *Reference Manual* and the *Participant Workbook*. This *Reference Manual* is a stand-alone, technical document that has been developed to serve as a long-term reference for participants. It has been developed as a course textbook, following the same modular format as the course presentation material, and includes the most up-to-date technical information available at the time of its development. The *Reference Manual* contains complete detail about treatment design, construction, and inspection, and also includes references to sources of additional information.

The *Participant Workbook* has been developed to help participants to follow the presentations, and it contains the following information:

- General course information, including an introduction, learning objectives, and class schedule.
- Introduction to each training module.
- Copies of all visual aids shown by the instructors.

The *Reference Manual* and the *Participant Workbook* are meant to be used together, both during the course presentation and afterwards, as technical resources. While the *Reference Manual* has been developed to include detailed technical information on the design and construction of concrete pavement preservation treatments, the course is not taught directly from this document. Those who follow the course presentation with the *Participant Workbook* will find a useful place to note key concepts discussed during class and to jot down their own ideas that are triggered by those discussions.

5. ADDITIONAL INFORMATION

This *Reference Manual* presents a considerable amount of information on the design, construction, and selection of preservation treatments for concrete pavements. However, there are a number of topics that can not be given a complete treatment because of the scope of the document and overall space limitations. Numerous references are cited throughout the document to provide interested readers with additional (and more detailed) sources of information. Many of these references are available from the organizations listed in table 1-1.

Table 1.1. Sources of additional information.

Federal Highway Administration	
Office of Pavement Technology Room 3118 1200 New Jersey Avenue SE Washington, DC 20590 http://www.fhwa.dot.gov/pavement	Office of Infrastructure Research and Development Turner-Fairbank Highway Research Center 6300 Georgetown Pike McLean, VA 22101 www.tfhrc.gov
National Highway Institute 4600 North Fairfax Drive, Suite 800 Arlington, VA 22203 http://www.nhi.fhwa.dot.gov/home.aspx	National Center for Pavement Preservation 2857 Jolly Road Okemos, MI 48864 www.pavementpreservation.org
Industry	
American Concrete Pavement Association (ACPA) 5420 Old Orchard Road, Suite A100 Skokie, IL 60077 www.pavement.com	International Grooving & Grinding Association (IGGA) 12573 Route 9W West Coxsackie, NY 12192 www.igga.net
Other	
American Association of State Highway and Transportation Officials (AASHTO) 444 N. Capitol Street, NW, Suite 249 Washington, DC 20001 http://www.aashto.org	American Society of Civil Engineers (ASCE) 1801 Alexander Bell Drive Reston, VA 20191 http://www.asce.org

CHAPTER 2. PREVENTIVE MAINTENANCE AND PAVEMENT PRESERVATION CONCEPTS

1. LEARNING OUTCOMES

This chapter presents an overview of preventive maintenance and pavement preservation. Upon completion of this chapter, the participants will be able to accomplish the following:

1. Define pavement preservation and preventive maintenance.
2. Describe characteristics of suitable pavements for preventive maintenance.
3. Describe the importance of selecting the "right" treatment and placing it at the "right" time.
4. List some of the benefits of pavement preservation.

2. INTRODUCTION

In recent years, the FHWA has been a strong proponent and supporter of the concept of cost effectively preserving the country's roadway (pavement) network. This has helped to spur on a nationwide movement of pavement preservation and preventive maintenance programs. This is indeed a radically different approach to managing pavement networks than what has been used in the past. One of the big differences between past approaches and today's emphasis on preservation and preventive maintenance is that preservation focuses on being "proactive" rather than "reactive." The concept of adopting a proactive maintenance approach enables agencies to reduce the probability of costly, time consuming rehabilitation and reconstruction projects. One result is that the traveling public has benefited from improved safety and mobility, reduced congestion, and smoother, longer lasting pavements (Geiger 2005). This is the true goal of pavement preservation, a goal in which the FHWA, through its partnership with States, local agencies, industry organizations, and other interested stakeholders, is committed to achieve (Geiger 2005).

As a primer to the remaining chapters in this manual, this chapter introduces many of the pavement preservation concepts currently being promoted by the FHWA. Specifically, this chapter introduces common pavement preservation-related definitions, discusses the importance and benefits of conducting preventive maintenance, and describes recent initiatives and State Department of Transportation (DOT) experiences.

3. DEFINITIONS

During the evolution of pavement preservation concepts over the past decade, it has not been uncommon to hear questions such as the following:

- What is pavement preservation?
- What is the difference between "pavement preservation" and "preventive maintenance?"
- How does "preventive maintenance" differ from "corrective maintenance?"
- What characteristics make a treatment fit into the "preventive" category?

In order to promote a uniform understanding among all agencies, a 2005 memorandum clarified the Federal Highway Administration's pavement preservation-related definitions (Geiger 2005). The remainder of this "Definitions" section contains definitions taken verbatim from the 2005 memorandum "Pavement Preservation Definitions" (Geiger 2005).

Pavement Preservation—As defined by the *FHWA Pavement Preservation Expert Task Group*, pavement preservation is "a program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extends pavement life, improves safety and meets motorist expectations." This goal is achieved in practice through the application of preventive maintenance, minor rehabilitation (nonstructural), and some routine maintenance activities. The distinctive characteristics of pavement preservation activities are that they restore the function of the existing system and extend its service life, but do not increase its load-carrying capacity or strength.

Preventive Maintenance—In 1997, the *AASHTO Standing Committee on Highways* defined preventive maintenance as "a planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without significantly increasing the structural capacity)." Preventive maintenance is typically applied to pavements in relatively good condition and that have significant remaining service life. For concrete pavements, examples of preventive treatments include slab stabilization, partial-depth repairs, full-depth repairs, retrofitted edge drains, load transfer restoration (dowel bar retrofitting), diamond grinding and grooving, and joint resealing and crack sealing.

Pavement Rehabilitation—Pavement rehabilitation projects are those that extend the life of existing pavement structures either by restoring existing structural capacity. Most commonly, this is achieved by increasing pavement thickness to strengthen existing pavement sections in order to accommodate existing or projected traffic loadings.

Routine Maintenance—Routine maintenance consists of day-to-day activities that are scheduled by maintenance personnel to maintain and preserve the condition of the highway system at a satisfactory level of service. Examples of pavement-related routine maintenance activities include joint or crack sealing, cleaning of roadside ditches and structures, and maintenance of pavement markings. Depending on the timing of application, the nature of the distress, and the type of activity, certain routine maintenance activities may be classified as preservation. Routine maintenance activities are often "in-house" or agency-performed and are not normally eligible for Federal-aid funding.

Corrective Maintenance—Routine maintenance activities are performed in response to the development of a deficiency or deficiencies that negatively impact the safe, efficient operations of the facility and future integrity of the pavement section. Corrective maintenance activities are generally reactive, not proactive, and performed to restore a pavement to an acceptable level of service due to unforeseen conditions. Examples for concrete pavements might consist of partial-depth repairs of severely spalled joints or slab replacement at isolated locations.

Pavement Reconstruction—Reconstruction is the replacement of the entire existing pavement structure by the placement of the equivalent or increased pavement structure. Reconstruction usually requires the complete removal and replacement of the existing pavement structure. Reconstruction may utilize either new or recycled materials incorporated into the materials used for the reconstruction of the complete pavement section. Reconstruction is required when a pavement has either failed or has become functionally obsolete.

A general schematic indicating the relative timing of these different activities is shown in figure 2.1. Note that the pavement preservation area of the curve is the portion that includes preventive maintenance and some light rehabilitation.

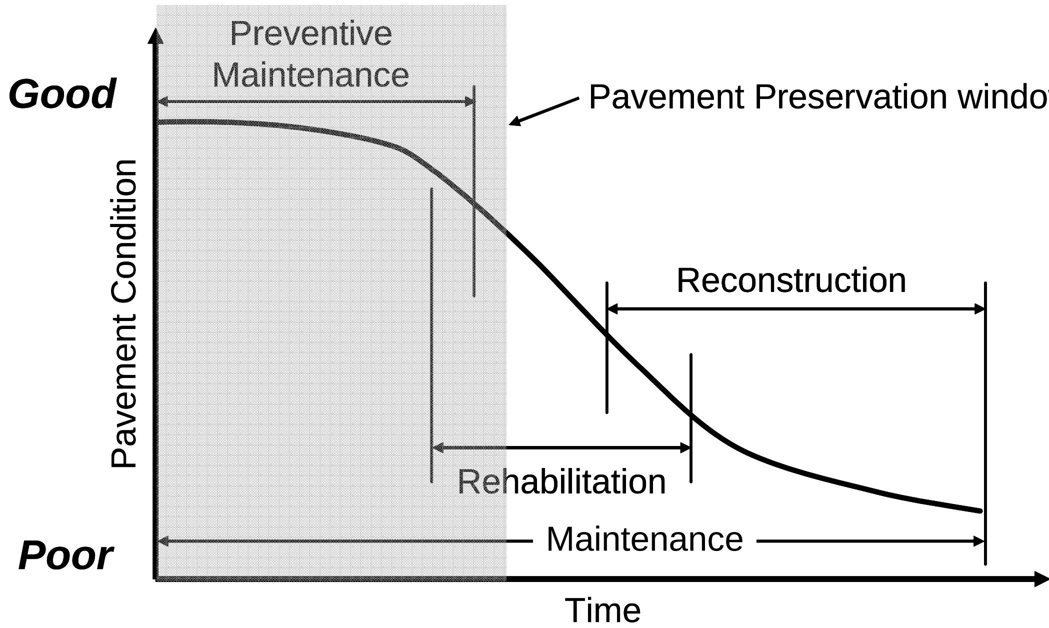

Figure 2.1. Representation of definitions of pavement preservation, rehabilitation, and reconstruction.

4. BENEFITS OF PREVENTIVE MAINTENANCE

Preventive maintenance is being embraced by more and more agencies because it is a logical approach to preserving assets that offers measurable benefits to the agency. Some of the benefits that have been cited as being important reasons for implementing or upgrading preventive maintenance programs include the following:

- Higher customer satisfaction.
- Better informed decisions.
- Improved strategies and techniques.
- Improved pavement condition.
- Cost savings.
- Increased safety.

Each of these benefits is discussed in more detail in the following sections.

Higher Customer Satisfaction

In the broadest sense, roads exist to serve the traveling public. Both nationwide surveys of customer satisfaction with highway systems (Coopers & Lybrand 1996) as well as many state-sponsored surveys (e.g. Washington [Dye Management Group 1996], California [Survey Research Center 1999], and Arizona [Dye Management Group 1998]), show that the public is interested in pavement conditions, and in seeing those conditions improved. Other concerns include maintaining or improving safety, and addressing congestion by constructing permanent rather than temporary repairs, and doing those repairs rapidly rather than over a prolonged closure.

Customer satisfaction is at the heart of successful preventive maintenance practices. From project selection to treatment selection to construction, a good preventive maintenance program will benefit users. Safer roads, faster repairs, and a pavement network in better condition that needs fewer repairs are logical outcomes of a preventive maintenance program.

Better Informed Decisions

Preventive maintenance programs rely on proper treatment selection and timing of the treatment to be successful. In order to select the right treatment for the right pavement at the right time, the following need to be known:

- What is the structure and condition of the existing pavement?
- What is the expected performance of the pavement?
- How will different treatments affect this performance?
- What other factors affect how the treatments will perform?

The availability of and accessibility to information is an essential part of the process of managing a successful preventive maintenance program. All of the successful programs have exploited the information that is available from a pavement management system (PMS) to help in the decision-making process. For example, Caltrans uses condition survey data from their PMS to prioritize projects and differentiate among the candidates for rehabilitation, routine maintenance, and capital preventive maintenance (CAPM) (Caltrans 1996). They program their preventive maintenance treatments or "CAPM" projects in the same way as rehabilitation and other projects. This relationship is critical because Caltrans recognizes that the placement of preventive maintenance treatments is highly dependent upon timing. They must be programmed and applied before the condition of the pavement warrants a more serious repair. At the same time, Caltrans has developed appropriate treatments for the different types of expected pavement condition and identified optimum times to apply these treatments.

Michigan DOT (MDOT) is another agency that has integrated its pavement management and preventive maintenance programs. In 1992, the DOT initiated the Michigan Preventive Maintenance Program (Galehouse 1998), with $8 million dedicated to highway preservation. During the period from 1992 through 1996, a total of $80 million was spent and almost 4,265 route km (2,650 route mi) of mainline pavement were treated. Using a module of their PMS to project long-term conditions and funding needs under different treatment scenarios, MDOT demonstrated that their preventive maintenance projects were more than six times as cost effective as rehabilitation or reconstruction.

Improved Preventive Maintenance Strategies and Techniques

One of the challenges to highway agencies and industry alike is to develop new and improved treatments to be used in preventive applications. Why are these needed? Conventional maintenance and rehabilitation treatments have evolved over the years to correct observed deficiencies, but may not be ideal for proactive applications in which life extension is expected.

Preventive maintenance treatments must provide a better level of performance. Preventive maintenance treatments are designed to be applied while the pavement is still in good condition and are meant to help to maintain the pavement at a high level of service. Treated pavements are smoother, have improved friction characteristics, and should last longer between rehabilitation or reconstruction. To be effective, these applications often require the use of high quality materials and quality control may play a much larger role than with other types of treatments. As a result, many of today's materials have been designed to provide the improved performance that users seek. While the initial treatment costs may be higher in some cases, the expected life of the treatment is going to be much greater than conventional applications. The net effect is that overall maintenance costs will be reduced.

As part of a changing attitude toward maintenance, higher quality, more durable materials are being evaluated by many agencies, along with new or improved application methods. Innovation in the development of these improved materials and treatment strategies has come from industry, agencies, and researchers.

Improved Pavement Condition

Agencies that have implemented preventive maintenance programs are not simply looking for a new way of doing the same old thing. The conventional approach most agencies take to manage their pavements consists of a combination of routine maintenance and rehabilitation. As previously described, routine maintenance is primarily a reactive process in which existing distresses are repaired; rehabilitation is typically programmed following the "worst first" principle, in which pavements are allowed to deteriorate until the worst one rises to the top of the capital projects list.

In contrast, preventive maintenance is a proactive approach intended to preserve a pavement and extend its useful performance period or cycle. The difference between these two approaches is substantial and central to the preventive maintenance concept. If pavements in good condition are kept in good condition longer, delaying the need for rehabilitation and reconstruction, then an obvious benefit is overall improved conditions. This is illustrated in figure 2.2.

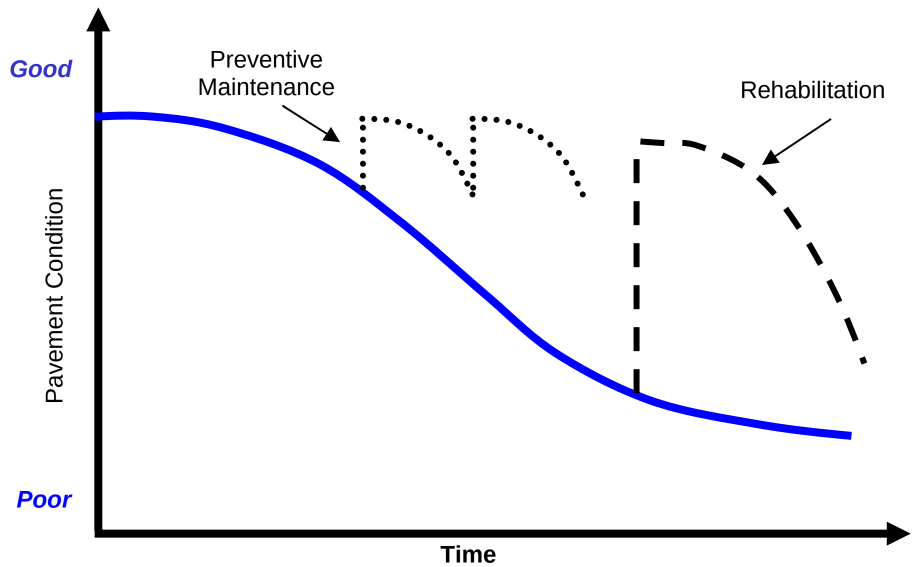

Figure 2.2. Illustration of typical effects of preventive maintenance and rehabilitation on pavement performance.

Cost Savings

From an agency standpoint, probably the most sought after benefit of preventive maintenance is financial. Saving money through a policy of preventive maintenance is certainly an intended benefit, but one that has been hard to prove. Nonetheless, a number of agencies have reported or projected cost savings from preventive maintenance strategies. These savings are both in the form of less expensive treatments and pavements with extended service lives, and are often used as the most persuasive argument to shifting pavement preservation strategies.

A reduction in user costs may also provide additional cost savings. These savings result from fewer delays, smoother roads (and lower vehicle operating costs), and enhanced safety (and thus lower crash-related costs). However, it must be noted that this analysis should not be reduced to the absurd level of applying frequent, very thin treatments; at some point the savings are offset by the disruption caused by more frequent treatment applications.

The agencies that have been active in preventive maintenance report that even after a relatively short time they are beginning to see the financial benefits of their practices. Michigan ($700 million over 5 years) and California (a 4:1 to 6:1 benefit with preventive maintenance treatments) specifically are reporting savings as they change the way that they take care of their pavements. Whatever the actual savings turn out to be, preventive maintenance treatments are (by their nature) less expensive than many alternatives. In addition, if these treatments can delay the need for more expensive repairs, agencies will see cost savings. An example of the savings documented by one agency in the 1990s is shown in figure 2.3, where the comparative costs of treatments applied at different times in the life of the pavement are represented.

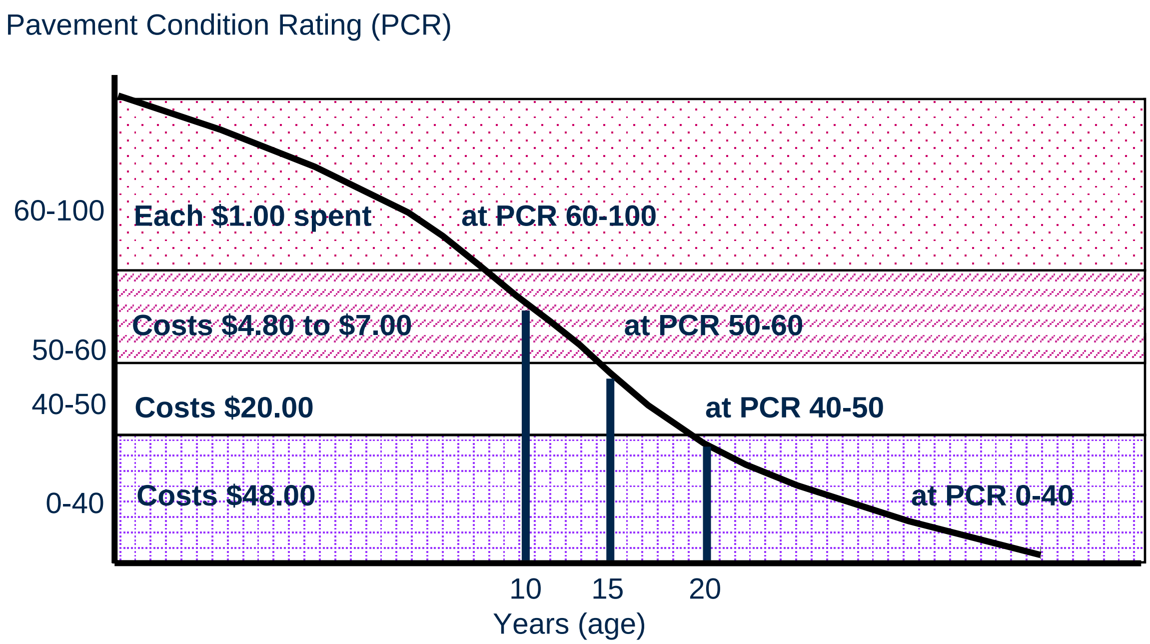

Figure 2.3. Comparison of treatment costs at different conditions/ages (Zimmerman and Wolters 2003).

Increased Safety

As noted above, most users cite safety as one of their fundamental expectations from the roads on which they travel. Safety is also an extremely important national priority, and the FHWA has recently established a Strategic Plan Goal to reduce fatal and injury crash rates 20 percent over 10 years. Recent work zone initiatives have been developed to improve safety in this very important area and it is also a high priority research area.

Preventive maintenance programs provide both implicit and explicit safety benefits that address this priority. Explicitly, today's treatments are specifically designed to provide safer surfaces. From better surface texture to fewer safety-related defects (such as spalling), the materials and treatments are expected to be an improvement over the treatments of the past.

Another explicit safety contribution of preventive maintenance treatments lies in the direct contribution of those treatments to safety measures. Pavement surface texture can have a positive effect on many roadway safety elements, most notably wet-weather surface friction. With a heightened interest in improving roadway safety, many studies are showing the impact that preventive treatments (such as diamond grinding) can have (Larson 1999).

The implicit safety benefits are obtained from keeping the pavement in better overall condition. Pavements with higher condition ratings are smoother and have fewer defects. These are conditions that contribute to safer operating conditions. Pavements in better overall condition also require fewer and less disruptive repairs.

None of these benefits stands alone. For any to be realized, the preventive maintenance treatment must be placed on a pavement that is a good candidate for preventive maintenance. The treatment must be properly designed, it must be properly constructed, and it must be properly maintained throughout its life.

5. RECENT INITIATIVES/STATE DOT EXPERIENCES

Many State Highway Agencies (and local agencies as well) are moving forward with initiatives intended to improve their pavement preventive maintenance practices. These are identified as “best practices,” helping agencies to develop and sustain successful programs. These include the following, discussed in greater detail below:

- Preservation Engineer.
- Manuals of Practice.
- Test Sections.
- Research and Training.
- Links Between Preventive Maintenance and Pavement Management.

Preservation Engineer. Perhaps 10 years ago this position didn’t exist in any public agency; today at least half a dozen SHAs either have a person whose specific title is Preservation Engineer or who is solely responsible for the agency’s preservation program. These include California, North Carolina, Minnesota, New York, and Louisiana, among others. This designation provides several benefits. In addition to having an individual who can help to improve preventive maintenance practices throughout the agency, it also helps to substantially raise the profile of preservation and preventive maintenance and thereby ensure that the programs are sustainable beyond the short term.

Manuals of Practice. A document that describes how to go about performing effective preventive maintenance can be a tremendous boon to an agency. These are often referred to as “manuals of practice” or “guides,” and typically include information about the various treatments in use locally, what they do, when they should be used, where they should be used, how they should be constructed, what benefits result from the proper use of the treatments, and so on. Examples include Caltrans’ Maintenance Technical Advisory Guide (Caltrans 2007), Nebraska’s Pavement Maintenance Manual (NDOR 2002), Ohio’s Pavement Preventive Maintenance Training Manual (ODOT 2001), and Colorado’s Preventive Maintenance Program Guidelines (CDOT 2004).

By having a manual or guide, agencies can communicate what constitutes accepted practice, what works well locally, and provide resources for additional information. While they may vary in content and complexity, these documents are a significant improvement over the limited guidance that was previously available to help individual agencies.

Test Sections. One of the barriers to more widespread acceptance of preventive maintenance is a lack of familiarity with what treatments are appropriate under what conditions. Previous nationwide studies (in the concrete area, the Strategic Highway Research Program’s [SHRP] SPS-4 studies) attempted to examine issues of maintenance effectiveness, but provided somewhat inconclusive results. By constructing test sections locally, using locally available or appropriate treatments perhaps applied to pavements of varying ages and stages of deterioration, an agency can develop a better understanding of what works well locally. Test sections can also supplement pavement performance information in a pavement management database to help improve treatment timing.

Research and Training. Another barrier to widespread adoption of preventive maintenance practices is a lack of knowledge about what treatments work, where they work, and when they should be applied. Furthermore, sometimes personnel involved in treatment decision-making don’t have a full understanding of why they should be doing preventive maintenance.

Targeted research and training are keys to breaking down this barrier. In addition to the research associated with test sections (described above), research into the use of locally available materials, construction methods, and programming issues can only help to improve practice. And because quite often preventive maintenance is so different from previous practice, training targeted at specific audiences will help to improve implementation efforts. Training is available from a number of industry sources, as well as the National Highway Institute (NHI) (www.nhi.fhwa.dot.gov), which has been offering several courses on pavement preservation and preventive maintenance since 1999. Some SHAs, such as California, Texas, and Ohio, have developed their own training programs, while others (Pennsylvania, North Carolina) have adapted NHI courses for local conditions.

Links Between Preventive Maintenance and Pavement Management. It should be clear that preventive maintenance is a different way of managing pavements for most agencies. Often change is met with resistance, especially if it can not be clearly demonstrated that the change is for the better. Most agencies already have pavement management systems (PMS), and most PMS have the ability to provide details of a pavement network's performance. Ideally, these PMS can also show where individual treatments have been placed and how they performed. Two agencies that have documented their preventive maintenance program performance with their PMS are Kansas and North Carolina. Kansas, in particular, has demonstrated how overall network conditions have improved since they started their preventive maintenance program in 2002 (see figure 2.4).

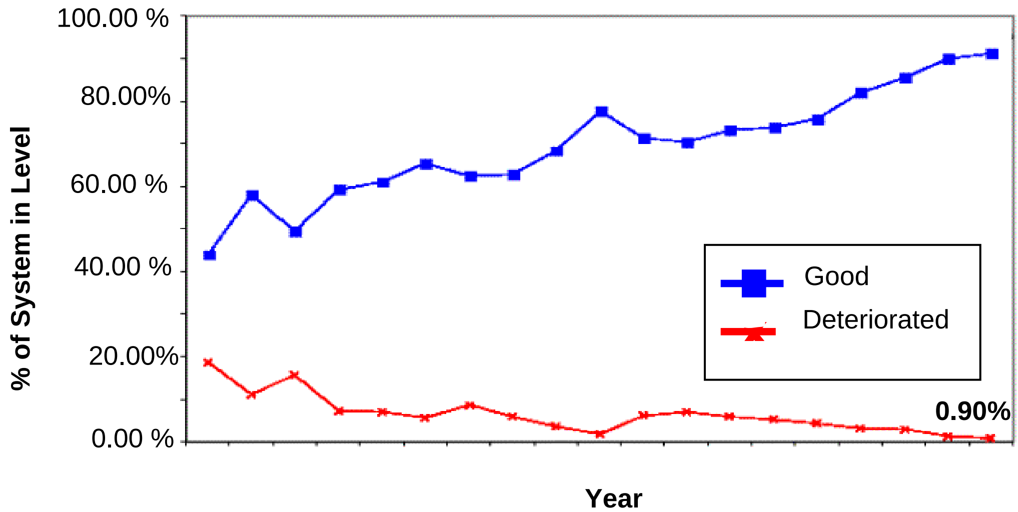

Figure 2.4. Change in network conditions in Kansas during preventive maintenance implementation.

6. SUMMARY

Preventive maintenance is by no means new concept, but as its use grows, more and more agencies are getting a better idea of what it means. While there are several refined definitions of what preventive maintenance means, the definition "keeping good roads in good condition" is as good as any.

There are many good reasons to implement a preventive maintenance program, and the forces that are at play in today's public agencies—tightened budgets, staff reductions, and greater public scrutiny of their decision making—almost require a preventive maintenance approach. However, the benefits of preventive maintenance will not be realized if sound practice in project evaluation and selection are not employed. The role of the timing of the treatment application, as well as the types of data collection that are required to help in the decision-making process, are briefly introduced. While these topics are covered in detail elsewhere (for example, Peshkin et al. 1999; Peshkin, Hoerner, and Zimmerman 2001), they are briefly introduced here.

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NOTES

CHAPTER 3. CONCRETE PAVEMENT EVALUATION

1. LEARNING OUTCOMES

This chapter presents a summary of the pavement evaluation process, including a description of the various pavement evaluation activities that are commonly conducted. Furthermore, this chapter describes how the results from the different pavement evaluation activities are brought together in an overall project evaluation. The results of the overall evaluation are used to assist in selecting cost-effective pavement preservation treatments. Upon completion of this chapter, the participants will be able to accomplish the following:

1. Describe the need for a thorough pavement evaluation and the uses of pavement evaluation data.
2. Name the common pavement evaluation components and what information is obtained from each.
3. Describe how pavement evaluation data are interpreted.

2. INTRODUCTION

Prior to selecting any preservation or rehabilitation treatment for a given pavement, it is important to conduct a thorough pavement evaluation to determine the causes and extent of pavement deterioration. This requires a systematic data collection effort and an analysis of the structural and functional condition of the pavement as well as several other factors. The approach to pavement evaluation described in this chapter is consistent with that presented in the *AASHTO Guide for Design of Pavement Structures* (AASHTO 1993), as well as that presented in the *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* (NCHRP 2004).

The size of a project often dictates the amount of time and funds that can justifiably be spent on pavement evaluation. Additionally, critical projects on major highways and projects subjected to high traffic volumes require more comprehensive and thorough pavement evaluations than those on low-volume highways. This is not because data collection is less important on lower volume highways, but because the effects of premature failures on the higher volume highways are much more serious.

Evaluating a pavement is similar to evaluating an automobile for repair. For example, prior to replacing a used car, the condition of the car, including its structural condition (e.g., motor, transmission, chassis), its functional condition (e.g., paint, interior, corrosion), and various individual components (e.g., speedometer, tires, windshield) should all be evaluated. The extent of deterioration can be assessed and either a cost-effective repair and preventive maintenance plan can be developed (combining the information from all of the different areas), or a decision made to replace the car. The consequences of neglecting to conduct such an evaluation could result in a very poor (and expensive) outcome.

3. DATA REQUIRED TO ACCOMPLISH A PAVEMENT EVALUATION

A thorough pavement evaluation requires the collection of a substantial amount of data about and from the existing pavement. These data can be divided into the following major categories:

- Pavement condition (e.g., distress, roughness, deflections).
- Shoulder condition.
- Pavement design (e.g., layer thicknesses, layer properties, structural characteristics, construction requirements).
- Materials and soil properties.
- Traffic volumes and loadings (current and projected).

- Climatic conditions.
- Drainage conditions.
- Geometric factors.
- Safety aspects (e.g., accidents, surface friction).
- Miscellaneous factors (e.g., utilities, clearances).

In many cases, the specific data to be collected under each of these general categories also depends upon the treatment alternatives being considered. For example, if grinding of a concrete pavement is to be considered, then the hardness of the aggregate and the faulting condition must be known. Similarly, if the addition of edge drains to a pavement project is being contemplated, the type and properties of the base and subbase materials must be determined. Table 3.1 provides a summary of suggested data collection items for various concrete treatment alternatives (AASHTO 1993). The data are classified as those that are "Definitely Needed," "Desirable," or "Not Normally Needed."

A thorough data collection effort serves the following important purposes in the overall pavement evaluation process:

- It provides the *qualitative* information needed to determine the causes of pavement deterioration, and to develop appropriate alternatives for repairing the deterioration and preventing its recurrence.
- It provides the *quantitative* information needed to make quantity estimates associated with different treatment alternatives, to assess the rate of deterioration of the pavement, and to perform life-cycle cost comparisons of competing treatment alternatives.

In pavement evaluation, the design engineer's objective is to make the most efficient use of data collection resources so that sufficient information can be obtained to identify feasible alternatives and to develop cost-effective designs.

4. PAVEMENT EVALUATION OVERVIEW

The activities included in a pavement evaluation will vary from project to project, depending on the type of project and its relative significance. Generally speaking, the overall pavement evaluation process can be broadly divided into the following general steps (Hoerner et al. 2001; NCHRP 2004):

1. Historical data collection and records review.
2. Initial site visit and assessment.
3. Field testing activities.
4. Laboratory materials characterization
5. Data analysis.
6. Final field evaluation report.

A brief introduction to each of these pavement evaluation steps are presented in the following sections, with more detailed discussions on specific field and laboratory testing activities included later in the chapter.

Table 3.1. Suggested data collection needs for concrete pavement treatment alternatives (AASHTO 1993).

Data Item	**Full-Depth Repair**	**Partial-Depth Repair**	**Overlay**	**Grinding**	**Recycling**	**Under-sealing**	**Slab Jacking**	**Sub-drains**	**Joint Resealing**	**Pressure Relief Joints**	**Load Transfer Restor-ation**	**Surface Treat-ment**
Pavement Design	X	X	X	X	X	X	X	X	X	X	X	X
Original Construction Data			*	*	*			*	*	*	*	
Age	*	*	*	*	*			*				*
Materials Properties	*	*	X	X	X	*		X				
Subgrade			X		X	*	*	X	X			
Climate			X		*	X		X	X	*		X
Traffic Loading and Volumes	X	X	X	X	X	*		X	*	*	X	X
Distress	X	X	X	X	X	X	X	X	X	X	X	X
Skid			*	*	*							*
Accidents			*	*	*							
NDT	*		X		*	X					X	*
Destructive Testing/ Sampling	X	X	X	*	X	*		X			*	
Roughness			*	*	*		*					*
Surface Profile			*	X			*					

KEY: X Definitely Needed * Desirable [*blank*] Normally Not Needed

Table 3.1. Suggested data collection needs for concrete pavement treatment alternatives (continued) (AASHTO 1993).

Data Item	**Full-Depth Repair**	**Partial-Depth Repair**	**Overlay**	**Grinding**	**Recycling**	**Under-sealing**	**Slab Jacking**	**Sub-drains**	**Joint Resealing**	**Pressure Relief Joints**	**Load Transfer Restor-ation**	**Surface Treat-ment**
Drainage	X		X	X	X	X		X	X			*
Previous Maintenance	*	*	*	*	*	*		*	*	*		*
Bridge Pushing			*						X	X		
Utilities	X		X		X	*	*	*				
Traffic Control Options	X	X	X	X	X	X	X	X	X	X	X	X
Vertical Clearances			X		X							
Geometrics			X		*							

KEY: X Definitely Needed * Desirable [*blank*] Not Normally Needed

Step 1: Historical Data Collection and Records Review

The first step of the evaluation process is to review the available historical records that are associated with the project. This process involves the collection of data from office files and other historical records that provide basic information needed for conducting the pavement evaluation. The goal is to collect as much information on the existing pavement as possible, such as original design data, construction information, subgrade data, materials testing data, traffic data, performance data, and so on. Possible data sources for this data collection effort are:

- Design reports.
- Construction plans/specifications (new and rehabilitation).
- Materials and soils properties from previous laboratory test programs and/or published reports.
- Past pavement condition surveys, nondestructive testing and/or destructive sampling investigations.
- Maintenance/repair histories.
- Traffic measurements/forecasts.
- Environmental/climate studies.
- Pavement management system reports.

The information gathered in this step can be used to divide the pavement into discrete sections with similar design and performance characteristics for the pavement evaluation.

Step 2: Initial Site Visit and Assessment

An initial site inspection is conducted to first gain a general knowledge of the performance of the pavement, and also to help determine the scope of the field testing activities to be conducted in step 3. As part of this activity, subjective information on distress, road roughness, surface friction, shoulder conditions, and moisture/drainage problems should be gathered. Unless high traffic volumes present a hazard, these data can be collected without any traffic control, through both “windshield” and observations from the roadside shoulder. In addition, an initial assessment of traffic control constraints, obstructions, right-of-way zones, presence of bridges and other structures, and general safety aspects should be made during this visit.

Information obtained from this initial site visit and assessment will be used to formulate the type and extent of field testing activities performed under step 3. For example:

- Distress observations may help identify the collection interval, the number of surveyors, and any additional measurement equipment that might be required.
- Roughness data may dictate the need for a more rigorous measurement program to address any differential sag/swell problems.
- Observations of moisture/drainage problems (e.g., pumping, corner breaks, standing water, and so on) may indicate the need for a more intensive deflection testing program or a more thorough investigation of subsurface drainage conditions.

Discussions with local design and maintenance engineers may also be beneficial to help gain a better understanding of the performance of the pavement.

Step 3: Field Testing Activities

Under this step, detailed field measurements and testing are conducted to better characterize the performance of the pavement. The specific field testing activities are guided by the information obtained from the initial site visit and assessment, and may include:

- Distress and drainage surveys—These surveys provide a visual indication of the structural condition of the existing pavement. The information gained from these surveys will have the greatest impact on the selection of the appropriate preservation or rehabilitation treatment, and consequently must be carefully performed.
- Nondestructive deflection testing—Deflection testing may be conducted on certain projects to evaluate the overall structural condition of the pavement or to assess the joint load transfer capabilities. The scope of the deflection testing program should be established by the design engineer during or after the initial site visit.
- Roughness and friction testing—This testing focuses on the functional performance of the pavement; that is, how well the pavement is meeting the rideability and safety needs of the traveling public.
- Field sampling and testing—Field sampling and testing activities serve several purposes, such as the confirmation of layer materials and thicknesses and the retrieval of cores and subsurface samples for later laboratory testing. Most pavement preservation projects will not require field sampling or testing programs.

Specific details associated with each of these different types of field testing activities are discussed later in this chapter.

Step 4: Laboratory Materials Characterization

Laboratory testing is a more limited component of a thorough project evaluation, and is not required on every project. When included as part of the pavement evaluation process, laboratory testing may be conducted to confirm or clarify certain results from the distress surveys or the deflection testing program, to provide additional insight into the mechanisms of distress, or to provide additional information needed for the identification of feasible treatment alternatives. Examples of information that can be determined from laboratory testing include:

- Concrete strength data.
- Stiffness of concrete and of bound layers.
- Petrographic testing and analysis of concrete layer.
- Resilient modulus of the unbound layers and of the subgrade.
- Density and gradation of underlying granular layers.

Clearly, the above information is not needed for most pavement preservation projects.

Step 5: Data Analysis

For each field data collection activity, there is a corresponding element of analysis required. For the pavement condition data, such as distress, roughness, and friction, the data can be plotted along the project to illustrate varying conditions. If prepared in bar chart form, these profile plots can depict both the extent and severity at each measurement interval. Slab cracking, corner breaks, faulting, and spalling are candidate distresses that can be expressed in these types of illustrations; continuous plots of roughness or friction can also be displayed. In addition, areas of poor drainage or significant changes in topography (cut/fill sections) can also be overlaid on the strip charts.

Pavement condition information provides critical insight into a pavement's structural and functional performance, and helps define when pavement preservation activities may or may not be appropriate. Table 3.2 presents examples of both general trigger and limit values for different pavement performance indicators. Trigger values define the point when pavement preservation may be appropriate, whereas limit values define the point at which the pavement is in need of major structural improvements. Many agencies have developed their own trigger and limit values for pavements within their jurisdiction.

Nondestructive deflection testing on concrete pavements can be used in a number of ways, including development of pavement deflection profiles, backcalculation of layer properties, determination of load transfer capabilities, and evaluation of the potential for voids at slab corners. And, as discussed above, deflection data can be analyzed to help assess the structural capacity of the pavement.

Step 6: Final Field Evaluation Report

The final step in the evaluation process is the preparation of the field evaluation report, which summarizes the results of the data collection and analyses. In addition, any critical non-pavement factors, such as shoulder condition, ditches, right-of-way, curves, bridges, ramps, and traffic patterns, should be identified as part of the report. Ultimately, this information will be used in the identification and selection of appropriate treatments.

5. PAVEMENT DISTRESS AND DRAINAGE SURVEYS

Section 4 provided an overview of the pavement evaluation process, and the remaining sections of this chapter describe the specific field testing components of the process, namely pavement distress and drainage surveys; deflection testing; roughness and surface friction testing; and material sampling and laboratory testing.

As described previously, project-level pavement distress surveys are the first step in the overall pavement evaluation process, and serve as the cornerstone for evaluating the suitability of the pavement to receive preservation treatments. These surveys record visible distresses on the surface of the pavement, and are performed to:

- Document pavement condition.
- Identify types, quantities, and severities of distress observed distress.
- Group areas of pavement exhibiting similar performance.
- Gain insight into causes of deterioration (e.g., structural vs. functional).
- Identify additional testing needs.
- Identify potential treatment alternatives.
- Identify repair areas and repair quantities.

Pavement distress is any visible defect or form of deterioration on the surface of a pavement, and is the most basic measure of the performance of an existing pavement. In order to fully describe pavement distress, the following three factors must be considered:

- <u>Type</u>—The type of distress is determined primarily by similar mechanisms of occurrence and appearance. By identifying the types of distress, a great deal of information can be inferred regarding the underlying causes of deterioration.
- <u>Severity</u>—The severity of distress represents the criticality of the distress in terms of progression; more severe distresses will require more extreme rehabilitation measures.
- <u>Amount</u>—The quantity of each type and severity level must be measured and expressed in convenient terms.

Table 3.2. Example critical trigger and limit values (adapted from ACPA 1997).

Pavement Type and Performance Measure	First Value = Trigger Value / Second Value = Limit Value[3]		
	High (ADT>10,000)	Medium (3,000<ADT<10,000)	Low (ADT<3,000)
Jointed Plain Concrete Pavement (Joint Space < 20 ft)[1]			
Structural Measurements			
Low-High Severity Fatigue Cracking (% of slabs)	1.5 / 5.0	2.0 / 10.0	2.5 / 15.0
Deteriorated Joints (% of joints)	1.5 / 15.0	2.0 / 17.5	2.5 / 20.0
Corner Breaks (% of joints)	1.0 / 8.0	1.5 / 10.0	2.0 / 12.0
Average Transverse Joint Faulting (in)	0.10 / 0.50	0.10 / 0.6	0.108 / 0.7
Durability Distress (severity)	Medium-High		
Joint Seal Damage (% of joints)	> 25 / —		
Load Transfer (%)	< 50 / —		
Skid Resistance	Minimum Local Acceptable Level / —		
Functional Measurement			
IRI (in/mi)	63 / 158	76 / 190	89 / 222
PSR	3.8 / 3.0	3.6 /2.5	3.4 / 2.0
California Profilograph (in/mi)	12 / 60	15 / 80	18 / 100
Jointed Reinforced Concrete Pavement (Joint Space < 20 ft)[2]			
Structural Measurements			
Med-High Severity Trans. Cracking (% of slabs)	2.0 / 30.0	3.0 / 40.0	4.0 / 50.0
Deteriorated Joints (% of joints)	2.0 / 10.0	3.0 / 20.0	4.0 / 30.0
Corner Breaks (% of joints)	1.0 / 10.0	2.0 / 20.0	3.0 / 30.0
Average Transverse Joint Faulting (in)	0.16 / 0.50	0.16 / 0.60	0.16 / 0.70
Durability Distress (severity)	Medium-High		
Joint Seal Damage (% of joints)	> 25 / —		
Load Transfer (%)	< 50 / —		
Skid Resistance	Minimum Local Acceptable Level / —		
Functional Measurement			
IRI (in/mi)	63 / 158	76 / 190	89 / 222
PSR	3.8 / 3.0	3.6 /2.5	3.4 / 2.0
California Profilograph (in/mi)	12 / 60	15 / 80	18 / 100
Continuously Reinforced Concrete Pavement			
Structural Measurements			
Failures (Punchouts, Full-depth Repairs) (no./mi)	3 / 10	5 / 24	6 / 39
Durability Distress (severity)	Medium-High		
Skid Resistance	Minimum Local Acceptable Level / —		
Functional Measurement			
IRI (in/mi)	63 / 158	76 / 190	89 / 222
PSR	3.8 / 3.0	3.6 /2.5	3.4 / 2.0
California Profilograph (in/mi)	12 / 60	15 / 80	18 / 100

[1] Assumed slab length = 15 ft

1 mi = 1.609 km; 1 m = 3.281 ft; 1 in = 25.4 mm

[2] Assumed slab length = 33 ft

[3] Values should be adjusted for local conditions. Actual percentage repaired may be much higher if the pavement is restored several times.

Because excess moisture in the pavement structure contributes to the development of so many pavement distresses, it is helpful during the distress survey to also conduct a drainage survey. In a drainage survey, visual signs of poor drainage are noted and can be coupled with information from materials sampling testing and nondestructive deflection testing to provide some insight into the overall drainability of the pavement structure. Unless moisture-related problems are recognized and corrected where possible, the effectiveness of any pavement preservation activity will be reduced.

The remainder of this section presents many of the important details associated with conducting both distress and drainage surveys. The first section discusses the importance of using a distress identification manual to standardize the way distresses are interpreted by raters in the field. Next, separate sections are used to present the guidelines associated with conducting distress and drainage surveys, respectively. Finally, guidance is provided on how to interpret the collected distress and drainage data.

Distress Survey Procedures

To be consistent in how the distress type, severity, and amount are determined during a distress survey, distress measurement protocols need to be adopted by the agency conducting the surveys. In recent years, significant progress has been made in the standardization of distress survey procedures. Several procedures are available at the national level, and most state highway agencies have developed their own survey procedures to assess the condition of their pavement structures.

In the FHWA's long-term pavement performance (LTPP) program, a detailed distress survey procedure and standardized distress definitions are available (Miller and Bellinger 2003). This document describes the appearance of each distress type, depicts the associated severity levels (where defined), and describes the standard units in which the distress is measured. Figures and photographs of the distress type at various levels of severity are also provided to aid the surveyor in the distress identification process. Table 3.3 summarizes the distresses defined for concrete pavements in that manual, and also notes whether the distresses are primarily traffic related or climate/materials related. Because this manual was developed for the LTPP program, the manual is more research oriented, and consequently requires that the pavement distress data be collected in considerable detail and at high levels of precision.

Another common pavement distress survey procedure is the pavement condition index (PCI) procedure developed by the Army Corps of Engineers (Shahin and Walther 1990). Extensive work went into the development of a numerical index value that is used to represent the pavement's structural integrity and its surface operational condition based on the observed distress. The resulting index, the pavement condition index, ranges from 0 (failed pavement) to 100 (perfect pavement) and accounts for the types of distress, the severity of the distresses, and the amount or extent of the distresses; the associated effects of these factors are combined into a composite PCI value through established "weighting factors" so that it more accurately reflects the overall performance of the pavement (Shahin and Walther 1990). The PCI procedure is intended primarily for network-level pavement management purposes, not only in documenting the current condition of the pavement but also in developing prediction models to forecast future pavement condition (Shahin and Walther 1990). However, the methodology is sufficiently comprehensive and flexible enough that it can be used in project-level analyses.

Guidelines for Conducting Manual Distress Surveys

Although modern technology has made automated distress data collection a more feasible alternative at the network level, manual distress surveys are still preferred at the project level. A manual distress survey is a "walking" survey of the pavement in which the entire limits of the project are evaluated and all distresses are measured, recorded, and mapped. Manual distress surveys serve as a cornerstone in the documentation of pavement condition and in the development of feasible treatment alternatives.

Table 3.3. Concrete distress types defined in LTPP *Distress Identification* manual (Miller and Bellinger 2003).

Distress Type	Unit of Measure	Severity Levels?	Primarily Traffic/Load	Primarily Climate/Materials
Cracking				
Corner Breaks	Number	Yes	X	
Durability Cracking	Number of Slabs, Sq. Meters	Yes		X
Longitudinal Cracking	Meters	Yes	X	X
Transverse Cracking	Number, Meters	Yes	X	X
Joint Deficiencies				
Transverse Joint Seal Damage	Number	Yes		X
Longitudinal Joint Seal Damage	Number	No		X
Spalling of Longitudinal Joints	Meters	Yes		X
Spalling of Transverse Joints	Number, Meters	Yes		X
Surface Defects				
Map Cracking	Number, Sq. Meters	No		X
Scaling	Number, Sq. Meters	No		X
Polished Aggregate	Square Meters	No	X	
Popouts	Number/Sq. Meter	No		X
Miscellaneous Distresses				
Blowups	Number	No		X
Transverse Const. Joint Deterioration	Number	Yes		X
Faulting of Transverse Joints / Cracks	Millimeters	No	X	
Lane-to-Shoulder Dropoff	Millimeters	No		X
Lane-to-Shoulder Separation	Millimeters	No		X
Patch/Patch Deterioration	Number, Sq. Meters	Yes	X	
Punchouts	Number	Yes	X	
Water Bleeding and Pumping	Number, Meters	No	X	

Equipment needed for a manual distress survey is readily available and should include (Van Dam et al. 2002a):

- Hand odometer (measuring wheel) or tape measure for measuring distances.
- Stringline or straightedge for measuring rut depth and/or dropoff.
- Small scale or ruler for fine measurements.
- Marking paint or lumber crayon to mark distresses or record stationing along project.
- Mid- to full-sized vehicle.
- Faultmeter or other means for measuring joint/crack faulting and lane-shoulder dropoff.
- Data collection forms or sheets.
- Clipboard and pencils.

- Agency-approved distress-identification manual.
- Camera or videotape for capturing representative distresses and conditions.
- Hard hats and safety vests.
- Traffic control provisions.

Elements of a manual distress survey are described in the following sections.

Pre-Survey Activities

Prior to any fieldwork, a preliminary records review should be conducted on the project. This should include information needed to assist in the conduct of the field surveys, such as general location information, general structural design information (pavement type, layer thicknesses, subgrade type, and so on), traffic information, and data from any previous distress surveys. Ideally, complete historical information on the project is desirable, although it may not always be available.

Arrangements for the provision of traffic control should also be made prior to any fieldwork. Although some of the work can be performed from the shoulder, the pavement surveyor must be allowed on the pavement with the freedom to closely inspect the entire pavement. In addition, any sampling and testing activities that will be conducted will require complete access to the pavement. All traffic control arrangements should be scheduled as far in advance as possible and should adhere to the guidelines provided in the Manual on Uniform Traffic Control Devices (MUTCD) (FHWA 2003) or the agency's governing requirements.

Windshield Survey

As a first step in the pavement distress survey, a "windshield survey" should be conducted in which the entire project is driven in each lane in both directions at posted speed limits to get an overall "feel" or impression for how the pavement is performing. This is also the easiest way to get a measure of the overall rideability of the pavement. During these passes, any swells, depressions, or other sources of discomfort should be recorded and their location noted by milepost. Also, significant changes in overall pavement condition or performance over the length of the project should be noted.

Detailed Distress Survey

A manual distress survey typically uses a two-person crew that walks along the shoulder of the entire project, noting all distresses and recording physical measurements (crack widths, faulting, drop-offs, and so on) as needed. In most cases, both travel lanes and shoulders are included in the survey. As previously described, if the project is on a busy roadway, traffic control should be provided for the safety of the survey crew.

The data collection forms that are used to record the distresses can be easily developed to fit an agency's objectives for distress surveys. These should be developed with the intended use of the data in mind in order to minimize future work. In addition, it is generally recommended that mapping of the project be conducted in order to help identify critical repair areas. An example field survey form used in the FHWA LTPP program is provided in figure 3.1 (Miller and Bellinger 2003).

Some agencies have also used portable, hand-held computers to aid in the collection of distress data. Users can input distress quantities and amounts directly into the computer, which can then be downloaded for further evaluation. These can be convenient for reducing paperwork and are also effective in reducing transcription errors; some models also allow mapping of actual distresses. Field surveys using computers may proceed a little slower than surveys with data collection forms, but the time is made up in the processing of the data.

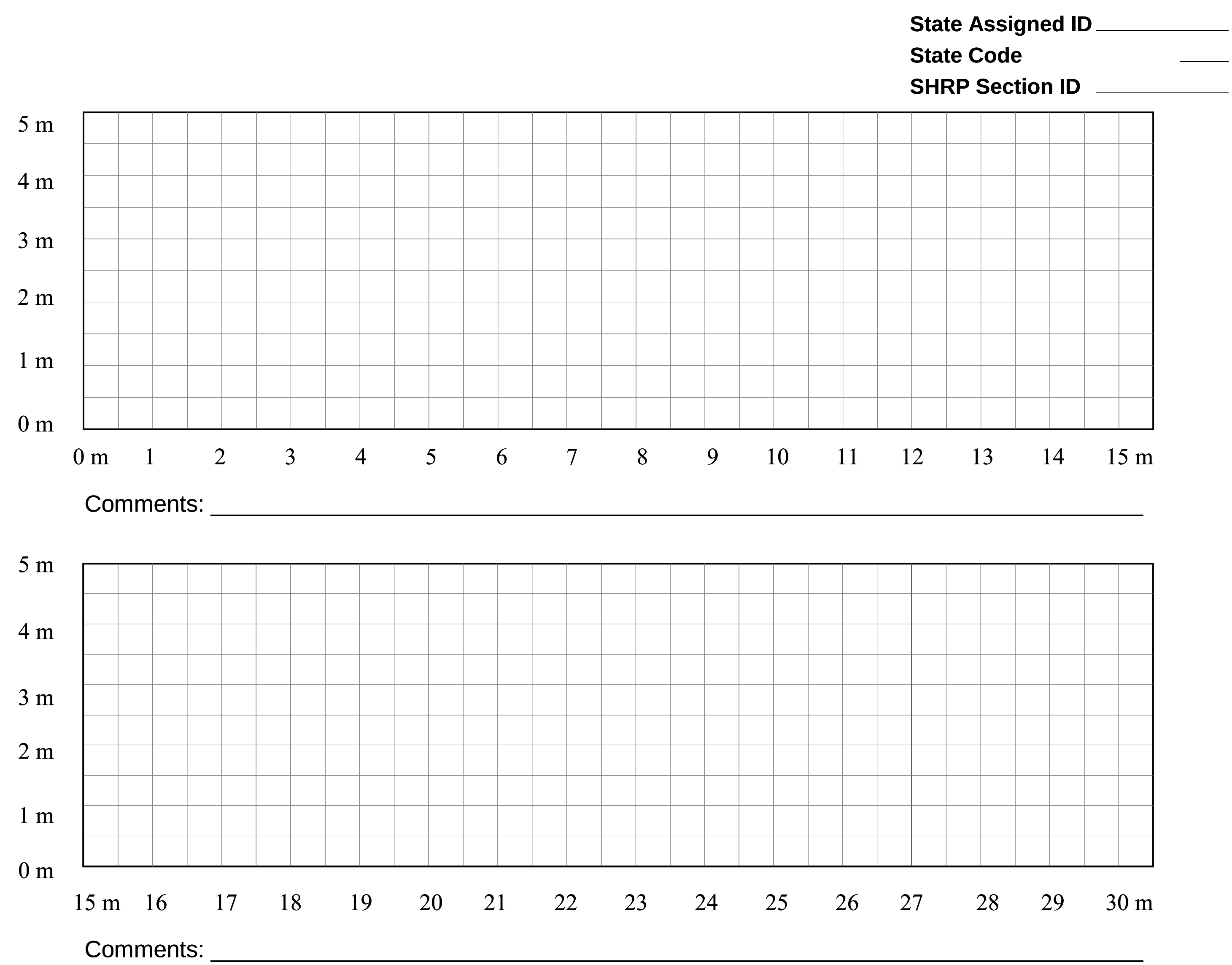

Figure 3.1. Example LTPP field data collection form (Miller and Bellinger 2003).

At the conclusion of the distress survey, it is recommended that a complete photo or video summary of the project be performed (Van Dam et al. 2002a). The purpose of this photo summary is to fully and completely document the condition of the pavement, as well as to record the prevailing foundation and drainage characteristics of the roadway.

Guidelines for Conducting Pavement Drainage Surveys

As part of a pavement distress survey, it is also important to assess the overall drainage conditions of the existing pavement. This is because poor drainage conditions have long been recognized as a major cause of distress in pavement structures, and unless moisture-related problems are identified and corrected where possible, the effectiveness of any preservation treatment will be reduced. Thus, the purposes of conducting a drainage survey are to:

- Identify the presence of moisture-related distresses.
- Document the prevailing drainage conditions of the pavement (e.g., cross slopes, cut/fill areas, depth and condition of ditches).
- Assess the condition and effectiveness of edge drains (if present).

The detection of possible drainage problems as evidenced from a drainage survey may suggest the need for an in-depth analysis of the drainability of the pavement structure. A computer program called DRIP (<u>D</u>rainage <u>R</u>equirements <u>I</u>n <u>P</u>avements) is available from FHWA that can assist in such an analysis (Mallela et al. 2002).

The drainage survey is conducted at the same time as the manual distress survey. Particular attention should be given to the severity and extent of those distresses that are known to be moisture-related to help assess the degree to which excess moisture may be contributing to the pavement deterioration. The location of these distressed areas should be clearly noted. In addition, the following drainage-related items should be noted as part of the drainage survey:

- <u>Topography of the project</u>. The overall topography and the approximate cut/fill depth should be noted along the length of the project to help determine if more distresses occur in certain cut or fill areas.
- <u>Transverse slopes of the shoulder and pavement</u>. These should be evaluated to ensure that they are not ponding water or preventing the effective runoff of moisture from the surface. Typical recommendations for pavement surface drainage are a minimum 2 percent cross slope for mainline pavements and a 3 percent cross slope for shoulders (Anderson et al. 1998).
- <u>Condition of the ditches</u>. The condition of the ditches (and the embankment material adjacent to the shoulder) should be noted along the length of the project to see if they are clear of standing water, debris, or vegetation that might otherwise impede the flow of water. The presence of cattails or willows growing in the ditch is a sign of excess moisture.
- <u>Geometrics of the ditches</u>. The depth, width, and slope of the ditches should be noted along the length of the project to ensure that they facilitate the storage and movement of water. It is generally recommended that ditches be 1.2 m (4 ft) below the surface of the pavement, be at least 1 m (3 ft) wide, and have a minimum slope of 1 percent.
- <u>Condition of drainage outlets (if present)</u>. These should be assessed over the entire length of the project to ensure that they are clear of debris and set at the proper elevation above the ditchline. The overall condition of the outlets and headwall (if present) should also be assessed, and the presence or absence of outlet markers noted.
- <u>Condition of drainage inlets (if present)</u>. Many urban projects incorporate drainage inlets to remove surface water, and these should also be inspected over the length of the project. These should be free-flowing and clear of debris.

If edgedrains are present, their effectiveness should be evaluated by observing their outflow either after a rainfall or after water is released from a water truck over pavement discontinuities. Another way of assessing the effectiveness of edgedrains is through the use of video inspections (Daleiden 1998; Christopher 2000), in which a camera attached to a pushrod cable is inserted into the drainage system at the outlets. In this way, any blockages, rodents' nests, or areas of crushed pipes can be located. Several states have adopted video edgedrain inspection as part of new pavement construction.

All of the information collected from the drainage surveys should be marked and noted on strip maps, and then examined together to obtain a visual picture of what moisture is doing to the pavement, where any moisture damage is occurring, and what factors are present that allow this moisture damage to occur.

While it is beyond the scope of this course, there are established procedures that can be used to analyze a pavement system estimate the time it takes to drain water from the pavement to a prescribed level of saturation. The DRIP computer program, mentioned previously, can be used to conduct a detailed drainage analysis and design (Mallela et al. 2002).

Collective Evaluation of Distress and Drainage Survey Results

Upon completion of the distress and drainage surveys, the critical distresses and drainage conditions should be summarized for the project. One useful way of summarizing the results is through a strip chart that shows the occurrence of various distresses over the length of the project. Primary distresses such as slab cracking are often plotted, but other important performance parameters such as roughness and surface friction can also be included. And when other important pavement evaluation information, such as deflections, soil types, and traffic volumes, are added to the strip chart, a more complete picture of the overall pavement condition is obtained and additional insight into possible causes of distress is gained. In addition, a strip chart can also assist in identifying particularly troublesome areas for more detailed materials and pavement testing.

An example strip chart is shown in figure 3.2. This figure plots the severity of concrete slab cracking along the length of the project. Three different slab cracking "conditions" are noted over the length of the project, and it is observed that the worst condition (condition 1) occurs in an area with high traffic levels and a silty clay subgrade. The "best" performance is observed in an area with low traffic levels and a granular subgrade. Other factors, such as cut and fill areas, depth of ditches, and condition of drainage outlets (if present) could also be added to the strip chart to provide additional insight.

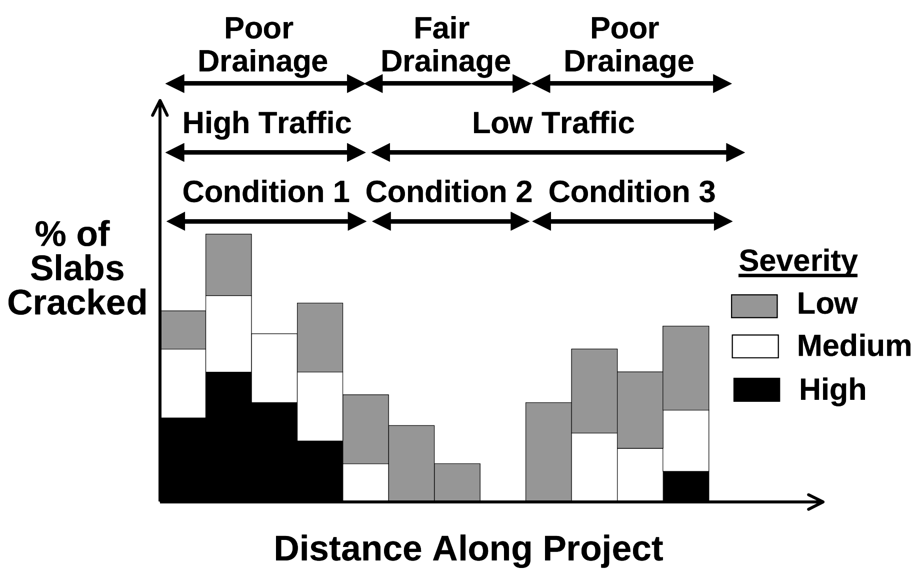

Figure 3.2. Example project strip chart.

A complete summary of the uses of the information obtained from the pavement distress and drainage surveys are listed below:

1. Distresses and other deficiencies requiring repair can be identified and corresponding repair quantities can be estimated. If there is a delay between the conduct of the field survey and the construction, a follow-up survey may be needed to ensure that contract quantities are still valid.
2. An overall examination of the data along the project will reveal if there are significantly different areas of pavement condition along the project. In addition, the inner lanes of multilane facilities may exhibit significantly less distress or lower severity levels of distress than the outer lane.
3. The condition survey data provide permanent documentation of the condition of the existing pavement. This lends itself to several uses, including the monitoring of the pavement performance over time, the comparison of pavement performance before and after treatment, and the development of performance prediction models.
4. The data provide an excellent source of information with which to plan structural and materials testing, if required.
5. The data provide valuable insight into the mechanisms of pavement deterioration and, consequently, the type of treatment alternative that may be most appropriate.
6. If time-series condition data are available (that is, performance data collected on a pavement at different points in time), then information can be obtained regarding the time that the various deficiencies began to appear and their relative rates of progression. Such information can be extremely valuable in identifying causes of condition deficiencies and in programming appropriate treatment alternatives.

6. DEFLECTION TESTING

Pavement deflection testing is an extremely valuable engineering tool for assessing the uniformity and structural adequacy of existing pavements. Over the years a variety of deflection testing equipment has been used for this purpose, from simple beam-like devices affixed with mechanical dial gauges to more sophisticated equipment using laser-based technology. Nevertheless, all pavement deflection testing equipment basically operates in the same manner, in that a known load is applied to the pavement and the resulting surface deflection measured.

For concrete pavements, deflection data can be analyzed to provide a wealth of information about the existing pavement structure, including the following:

- Concrete elastic modulus and modulus of subgrade reaction (*k*-value), and their variability along a project.
- Seasonal variation in base and subgrade stiffness and pavement response.
- Load transfer efficiency across joints and cracks.
- Presence of voids under slab corners and edges.

The remainder of this section provides general information on deflection testing equipment and procedures, as well as the methods used to interpret the deflection testing results.

Concrete Pavement Response

Pavement deflections represent an overall "system response" of the pavement structure and subgrade soil to an applied load. When a load is applied at the surface, all layers deflect, creating strains and stresses in each layer, as illustrated in figure 3.3. Critical pavement stresses develop when the concrete slab is loaded along the outside longitudinal edge or when the concrete slab is loaded at the corner.

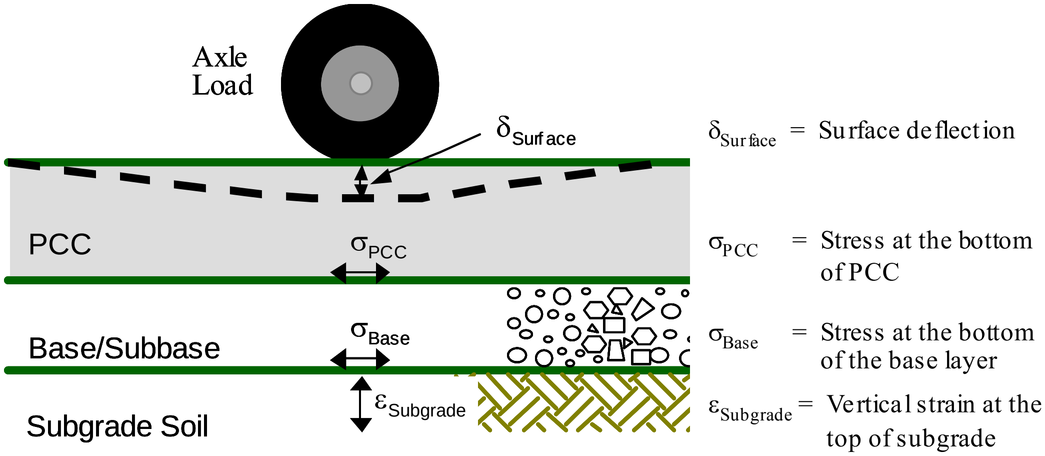

Figure 3.3. Illustration of pavement responses to moving wheel loads.

Deflection Testing Equipment

At present, there are many different commercially available deflection testing devices. These devices are generally grouped into the following categories, based on the type of loading imparted on the pavement (static, steady-state vibratory, and impulse).

- Static load deflection equipment—Static load deflection equipment measures the maximum deflection response of a pavement to static or slowly applied loads. The most commonly used static deflection device is the Benkelman Beam.
- Steady-state vibratory load deflection equipment—Steady-state dynamic load deflection devices apply a static preload and a sinusoidal vibration to the pavement with a dynamic force generator. A series of sensors is located at fixed intervals to measure the resulting deflection. The most commonly used devices in this category are the Dynaflect and Road Rater.
- Impulse load deflection equipment— Under the category of impulse load deflection devices is the falling weight deflectometer (FWD), which is the most common deflection-measuring device in use today. As shown in figure 3.4, the FWD releases a known weight from a given height onto a load plate resting on the pavement structure, producing a load on the pavement that is similar in magnitude and duration to that of a moving wheel load. A series of sensors are located at fixed distances from the load plate, so that a deflection basin can be measured. Variations in the force applied to the pavement are obtained by varying the weights and the drop heights; force levels from 13 to over 222 kN (3,000 to over 50,000 lbf) can be applied, depending on the equipment.

  Developed in the 1970s, the FWD emerged in the 1980s as the worldwide standard for pavement deflection testing. Commercial impulse load deflection devices include the Dynatest, KUAB, JILS, and Phonix FWDs. Numerous factors make the FWD the equipment of choice for pavement deflection testing, including the following:

  – The ability to better simulate a moving wheel load.
  – The ability to measure deflections at varying loads.
  – The ability to measure load transfer efficiency and identify voids beneath the slab.
  – The ability to record a deflection basin.
  – The speed, repeatability, and robustness of the equipment.

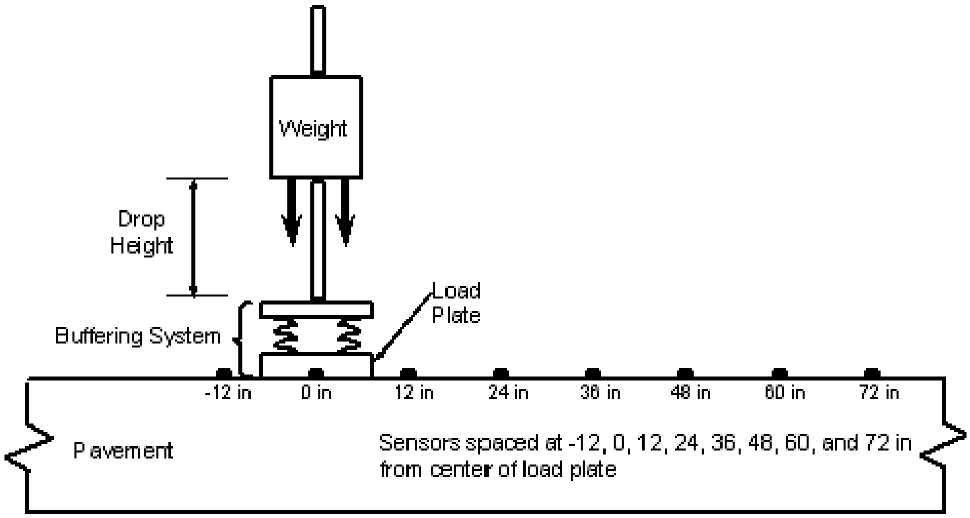

Figure 3.4. Schematic of FWD device.

In the last decade, considerable work has been conducted on the development of deflection-measuring equipment capable of collecting continuous deflection data along the length of a project. Continuous deflection profiles provide the following advantages over discrete deflection measurements:

- The entire length of the pavement project can be investigated. Thus, there is no danger of missing critical sections and no uncertainty about a test section being representative of the entire pavement system.
- The spatial variability in deflections due to pavement features such as joints, cracks, patches, and changing constructed or subgrade conditions are identified.
- More efficient testing and measurement operations, as there is no time lost stopping and starting.

Currently, two such devices are under development, the Rolling Dynamic Deflectometer (RDD) and the Rolling Wheel Deflectometer (RWD), with each still in the prototype stage. More information on these devices is provided by Bay and Stokoe (2000) and by Grogg and Hall (2004).

Factors That Influence Measured Deflections

There are a number of factors that affect the magnitude of measured pavement deflections, which makes the interpretation of deflection results difficult. To the extent possible, direct consideration of these factors should be an integral part of the deflection testing process so that the resultant deflection data are meaningful and representative of actual conditions. The major factors that affect pavement deflections may be grouped into categories of pavement structure (type and thickness), pavement loading (load magnitude and type of loading), and climate (temperature and seasonal effects). Each of these is discussed in the following sections.

Pavement Structure

As previously described, the deflection of a pavement represents an overall system response of the surface, base, subbase, and subgrade. Thus, the properties of the surface layer (thickness and stiffness) and of the supporting layers (thickness and stiffness) all affect the magnitude of the measured deflections. Generally speaking, “weaker” or “thinner” systems will deflect more than “stronger” or “thicker” systems under the same load, with the exact shape of the deflection basin related to the stiffness of the individual paving layers. As a general rule, pavements exhibiting greater deflections typically have shorter service lives. Many other pavement-related factors can affect deflections, including the following:

- Testing near joints, edges, or cracks can produce higher deflections than testing at interior portions of the slab.
- Random variations in slab and base layer thicknesses can create variabilities in deflection.
- Variations in subgrade properties and the presence of underlying rigid layers (such as bedrock or a high water table level) can provide significant variability in deflections.
- Voids beneath slab edges and corners cause increased deflections.
- For concrete pavements constructed over a stabilized base, the condition of the concrete–base interface can significantly affect deflections. If the interface is effectively bonded (no slip at the interface), the pavement deflection will be significantly less than if the interface is unbonded.
- Material-related distress, such as alkali-silica reactivity (ASR) and D-cracking, can increase the magnitude of slab deflections.

The coefficient of variation for deflection measurements along a project is typically 20 to 30 percent, and sometimes higher. To ensure that obvious pavement factors such as presence of cracks, visible material deterioration, or visible structures do not falsely indicate variability in pavement deflections, care must be taken during deflection testing to avoid testing over such features.

Pavement Loading

Load Magnitude

One of the most obvious factors that affects pavement deflections is the magnitude of the applied load. Most modern deflection equipment can impose load levels from as little as 13 kN (3,000 lbf) to over 245-kN (55,000-lbf), and it is important that appropriate load levels be targeted for each application. For example, for most highway pavement testing, a nominal load level of 40 kN (9,000 lbf) is often used since this is representative of a standard 8,160-kg (18,000-lb) axle load. An important reason for selecting test loads as close as possible to the design loads is the nonlinear deflection behavior exhibited by many pavements. This is shown in figure 3.5, in which the pavement exhibits a deflection of 0.028 mm (0.001 in) under a 4.4-kN (1,000-lbf) loading, and a deflection of 0.35 mm (0.014 in) under a 40-kN (9,000 lbf) load. Had the 40-kN (9,000 lbf) deflection been projected based on the 4.4-kN (1,000 lbf) load, a deflection of 0.25 mm (0.01 in) would have been erroneously projected.

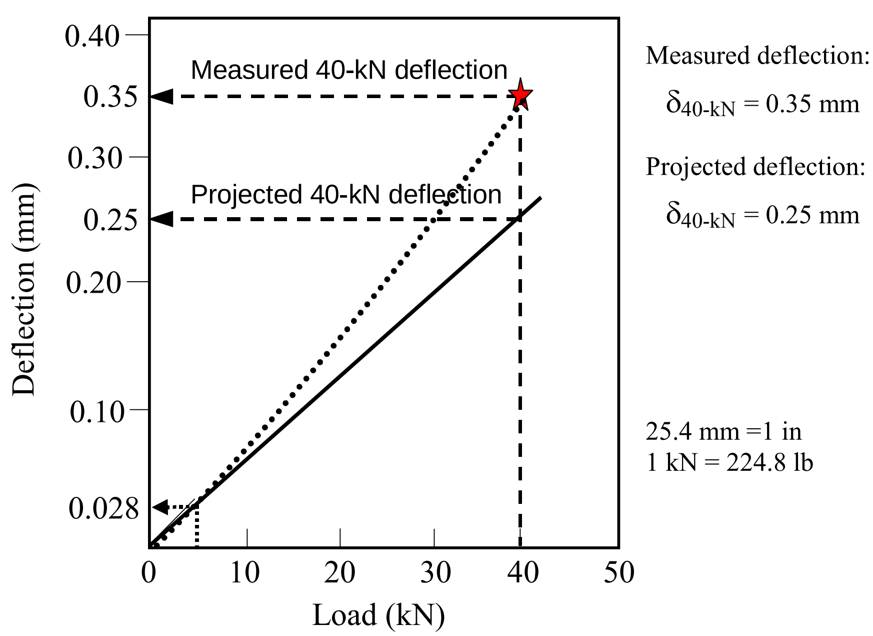

Figure 3.5. Pavement deflection as a function of dynamic load.

Type of Loading

Pavement deflection response can also be affected by the type of loading; a slow, static loading condition produces a different response than a rapid, dynamic loading condition. In general, the more rapid the loading (the shorter the load pulse), the smaller the deflections; this is why the static load devices (such as the Benkelman Beam) tend to produce deflections larger than those produced by dynamic loading devices (such as the FWD).

Climatic-Related Factors

Pavement Temperature

Concrete pavement deflections are affected by temperature, in both basin and joint/corner testing. Differences in temperature between the top and bottom of the slab cause the slab to curl either upward (slab surface is cooler than the slab bottom) or downward (slab surface is warmer than the slab bottom). If basin testing is conducted when the slab is curled down, or if the corner testing is conducted when the slab is curled up, the slab could be unsupported and greater deflections may result. Figure 3.6 shows the effect of daily temperature variations on the backcalculated modulus of subgrade reaction (k-value).

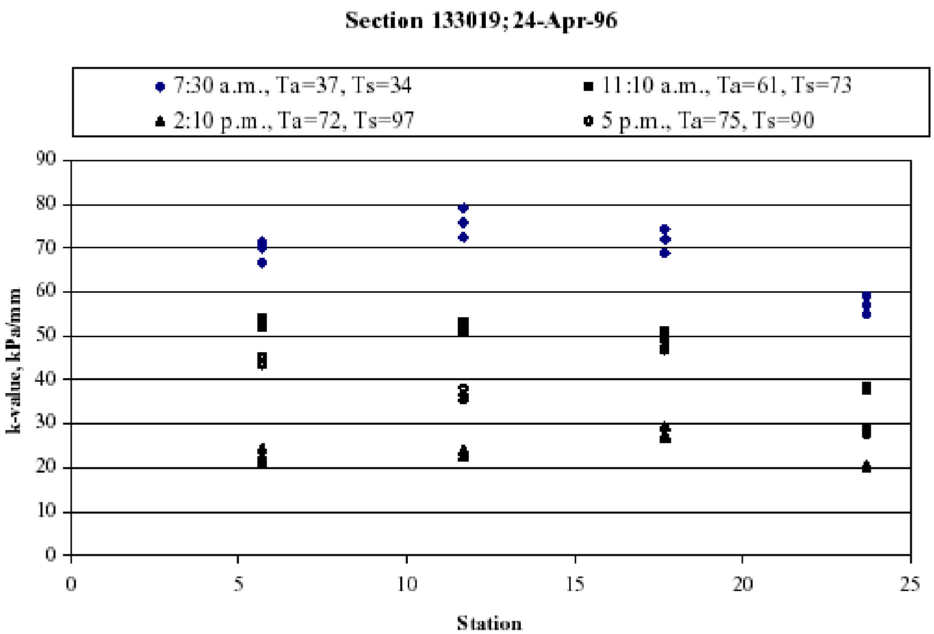

Figure 3.6. Variation in backcalculated k-value due to variation in temperature gradient (Khazanovich and Gotlif 2003).

Temperature also affects joint and crack behavior in concrete pavements. Warmer temperatures cause the slabs to expand and, coupled with slab curling effects, may “lock-up” the joints. Deflection testing conducted at joints when they are locked-up will result in lower joint deflections and higher load transfer efficiencies that are misleading of the overall load transfer capabilities of the joint. Figure 3.7 shows the variation in computed load transfer efficiencies throughout the day, with the higher values computed from data collected in mid-afternoon (Khazanovich and Gotlif 2003). Because of these effects, it is normally recommended to conduct FWD testing early in the morning or during cooler periods of the year.

Testing Season

Seasonal variations in temperature and moisture conditions also affect pavement deflection response. As an example, figure 3.8 shows the average load transfer efficiencies over a 2-year period (Khazanovich and Gotlif 2003). As a general trend, the LTE parallels the surface temperature, generally decreasing with the decreases in the surface temperature and increasing with increases in the surface temperature.

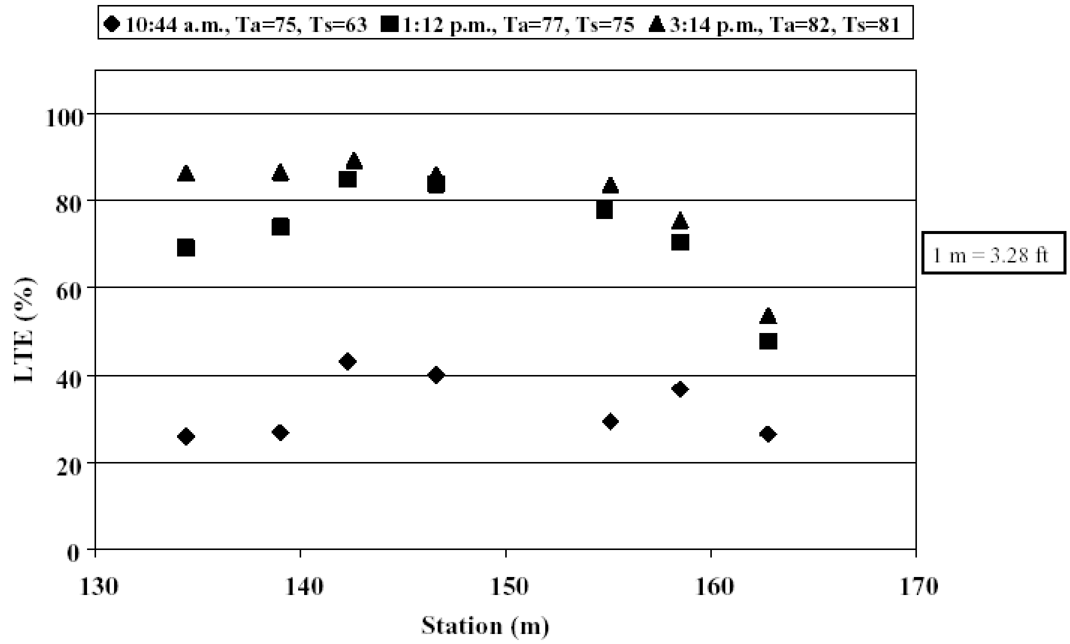

Figure 3.7. Daily variation in the calculated load transfer efficiencies (leave side of joint) (Khazanovich and Gotlif 2003).

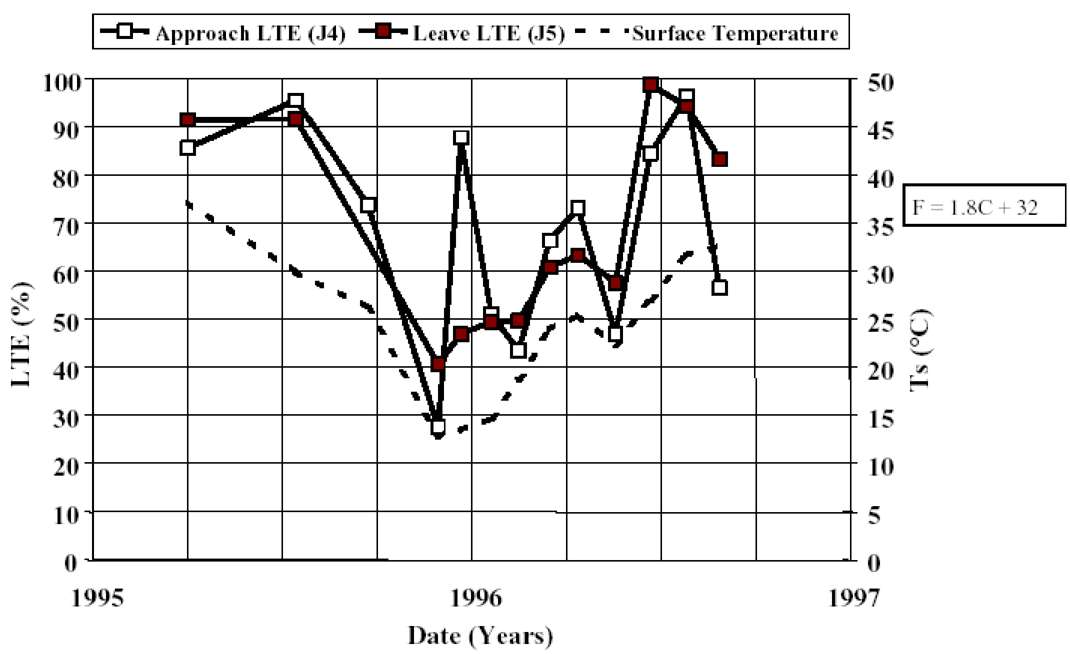

Figure 3.8. Seasonal variation in LTE and concrete surface temperature (Khazanovich and Gotlif 2003).

Guidelines for Conducting Deflection Testing

Deflection testing is normally not required for most pavement preservation candidate projects. One exception is the need to assess the load transfer capabilities of the pavement, particularly when significant pumping and faulting is observed as part of the pavement distress survey. In some cases, however, deflection testing may be needed to ensure that the existing pavement is still structurally adequate to receive pavement preservation treatments (and not structural treatments, such as an overlay).

Guidelines on conducting pavement deflection testing—if required—is presented in the following sections. Generally, deflection testing should be completed prior to any destructive testing to assist in locating areas where such sampling is required. In addition, it is recommended that pavement deflections be measured at a time that best represents the effective year-round condition. In climates where frost penetrates into the subgrade, the best time to conduct deflection testing is shortly after the spring thaw, after the soil has regained some of its strength. Testing during the spring thaw is not recommended because it is likely to produce overly conservative results. The deflection survey should never be conducted when the pavement or subgrade soil is frozen, because misleading information will be obtained (Darter, Hall, and Kuo 1995; Hall, Darter, and Kuo 1995).

Testing Locations and Frequency

For a project-level analysis, deflections should be measured at 30- to 150-m (100- to 500-ft) intervals. On multiple-lane facilities, it is normally sufficient to take measurements only in the outer or truck lane, but it may be desirable to take measurements in one or more additional lanes if the extent of load-associated distress varies greatly across lanes. On two-lane highways, the profiles in each direction should be staggered. For example, if deflections are spaced every 30 m (100 ft) in one direction, they should be placed between those measurements in the opposite direction.

One recommended testing plan for jointed concrete pavements is shown in figure 3.9. Test locations include the mid-slab (at least 1.5 m [5 ft] away from any crack or joint) and at the slab corner (with the load plate placed as close as possible to the corner of the slab). Corner tests should be conducted at the approach and leave corners for void detection (loss of support). However, since every evaluation project is different, there is no all-inclusive testing standard that will work for all cases. Additional guidance on deflection testing locations and frequency is available from FHWA (2006b).

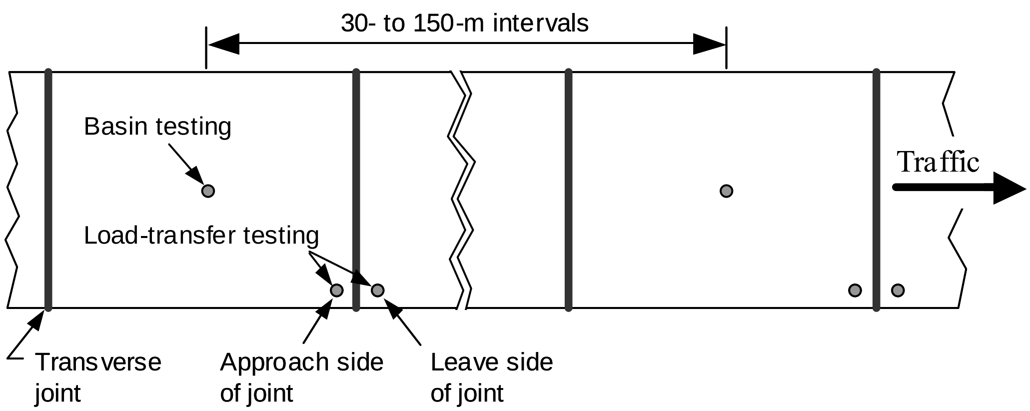

Figure 3.9. Recommended deflection testing locations for jointed concrete pavements.

Continuously reinforced concrete pavements (CRCP) should be tested in the outer wheelpath with the center of the load approximately 0.6 m (2 ft) from the edge of the slab. The load should be placed between cracks and not over a crack, although it may be desirable to test the load transfer across deteriorated cracks. Testing at the edge of the CRCP slab may also be conducted to identify the presence of voids.

Intensive Deflection Testing

If further information is needed to ascertain either the cause or extent of certain distress types (e.g., voids, loss of load transfer, soft areas), an intensive deflection test program may be conducted. Specific intensive testing areas along the project can be selected within each kilometer and deflection measurements taken at close intervals within these areas. These tests should be closely coordinated with any coring tests that may be conducted at the same time.

Temperature Measurements

Knowing the temperature of the pavement at the time of testing is useful in interpreting the deflection data, particularly as it pertains to the evaluation of load transfer efficiencies, the effects of curling on backcalculated moduli, and the evaluation of potential "built-in" curling during pavement construction. Pavement temperature information can be obtained by drilling holes of varying depths in the concrete, filling the bottom of these holes with glycerin or any other suitable liquid to the appropriate depth, and recording the temperature of the fluid. It is desirable to obtain temperatures at the pavement surface, mid-depth, and bottom of the concrete layer at a regular interval (e.g., every 15 minutes) using an automated data logger. At a minimum, the air and pavement surface temperatures should be recorded at each test location (ASTM D 4695). Many deflection testing devices automatically record the air and pavement surface temperatures during testing.

Interpretation of Deflection Testing Data

Pavement deflection data can be used and interpreted in a number of ways to help characterize the overall pavement condition. Several ways that deflection data are used and interpreted are discussed below.

Assessment of the Uniformity of the Support Conditions Along the Project

The maximum pavement deflection measured at each location can be plotted as shown in figure 3.10 to graphically evaluate the variation along the project. The deflections should be referenced directly to stationing so that they can be related to the distress, drainage, materials, and subgrade surveys. This information is very helpful in identifying subsections within the project and also indicating locations where distress, poor moisture conditions, cut/fill, and other conditions may be adversely affecting the pavement.

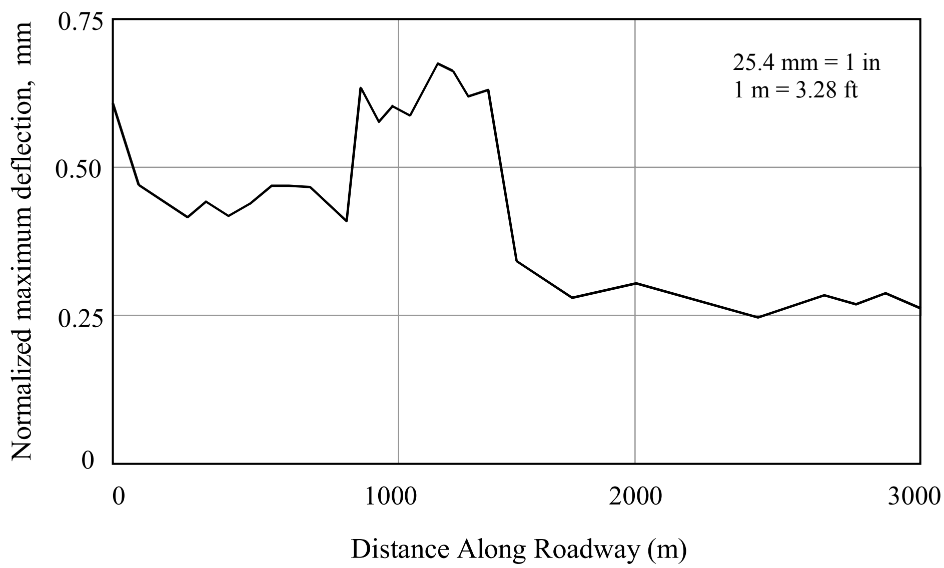

Figure 3.10. Illustration of deflection variation along a project.

Backcalculation of Concrete and Subgrade Layer Properties

"Backcalculation" is the process whereby the fundamental engineering properties of the pavement structure (concrete elastic modulus) and underlying subgrade soil (*k*-value) are estimated based on measured surface deflections. While the details of the procedures used to compute these backcalculated concrete elastic modulus and subgrade *k*-value values are outside of the scope of this course, more detailed information on the backcalculation methods for concrete pavements are contained in published reports by Hall (1992); AASHTO (1993); Hall et al. (1997); Khazanovich (2000); and Khazanovich, Tayabji, and Darter (2001). It is important to note that all existing backcalculation methods for concrete pavements share the following limitations:

- Slab curling due to temperature gradients can significantly influence deflection response of concrete pavements, but none of the existing methods account for the effects of slab curling.
- Base modulus values cannot be backcalculated directly. Currently, the base layer can only be considered by assuming either a bonded or unbonded interface and converting the two-layer system into an equivalent single layer.
- Backcalculated concrete modulus values are highly sensitive to slab thickness used in the backcalculation; even random variability in slab thickness can cause significant variations in the backcalculated concrete modulus values. Therefore, accurate pavement thickness information is essential to minimize random error.

Evaluation of Joint and Crack Load Transfer

Load transfer is the ability of a joint or crack to transfer the traffic load from one side of the joint or crack to the next. Although load transfer can be defined in a number of ways, it is commonly expressed in terms of the deflections measured at the joint or crack:

$$LTE = \frac{\delta_U}{\delta_L} \cdot 100\% \tag{3.1}$$

where:

LTE	=	Load transfer efficiency, percent.
δ_U	=	Deflection on unloaded side of joint or crack, mm (mils).
δ_L	=	Deflection on loaded side of joint or crack, mm (mils).

Figure 3.11 illustrates the concept of deflection load transfer. It should be noted that different LTE values may be obtained depending on which side of the joint is loaded, so both sides of the joint should be tested and the lowest value used. Furthermore, temperatures will significantly affect the LTE results, and it generally recommended that load transfer testing be conducted at temperatures below 21 ºC (70 ºF). Generally speaking, the following guidelines may be used to define different levels of LTE (NCHRP 2004):

- Excellent: 90 to 100 percent.
- Good: 75 to 89 percent.
- Fair: 50 percent to 74 percent.
- Poor: 25 to 49 percent.
- Very Poor: 0 to 24 percent.

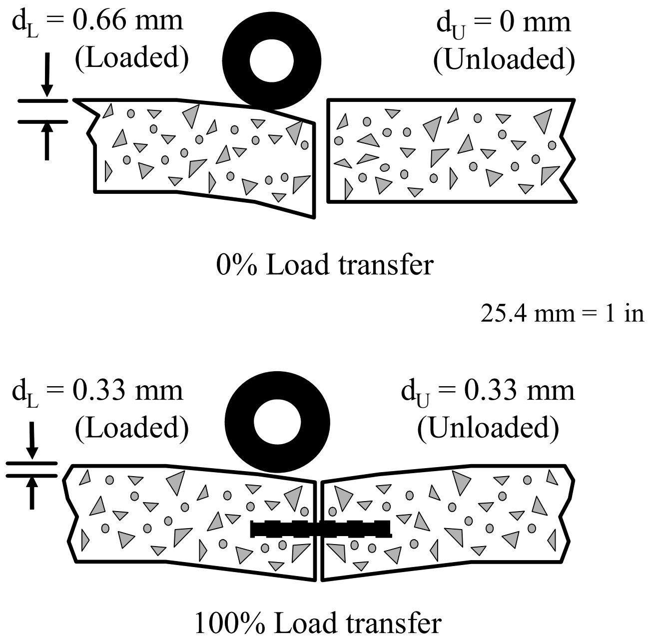

Figure 3.11. Concept of deflection load transfer.

<u>Identification of Locations of Loss of Support (Voids)</u>

Loss of support beneath slab corners (and edges) is the result of high deflections, excess moisture, and an erodible base or subbase material beneath the slab. The FWD can be used to conduct a series of load tests at suspected joint corners to help determine if there is significant loss of support. This is shown in figure 3.12, in which load sweeps were conducted on both the approach and leave sides of a transverse joint. After conducting the testing, a load vs. deflection plot is generated, and when the lines are extrapolated back toward the origin, the approach side projects very close to the origin (suggesting full support) where the leave side projects over 10 mils away from the origin (suggesting a void). The 1993 AASHTO Guide (AASHTO 1993) provides a summary of the available procedures for using the FWD to determine loss of support beneath concrete pavements.

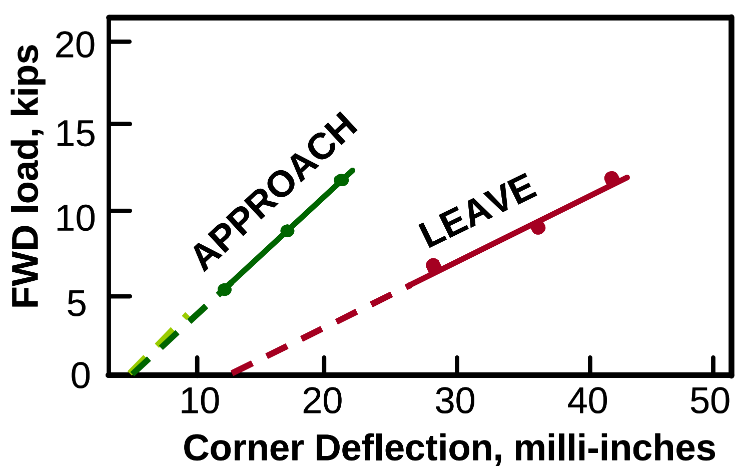

Figure 3.12. Example void detection plot using FWD data.

7. ROUGHNESS AND SURFACE FRICTION TESTING

As part of the pavement evaluation process, it is important to assess a pavement's functional performance, which refers to how well the pavement is providing a smooth, safe ride to the traveling public. Two easily measurable characteristics that give an indication of a pavement's functional condition are roughness and surface friction. Excessive roughness can create user discomfort and irritation and can lead to increased vehicle operating costs, user delay, and crashes. Inadequate surface friction can also contribute to crashes, especially under wet weather conditions.

Definitions

This section defines a number of important roughness- and friction-related terms. For convenience, these definitions are presented in alphabetical order.

- Pavement Roughness. In its broadest sense, pavement roughness is defined as "the deviations of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage" (Sayers 1985). Surface irregularities that influence pavement roughness can generally be divided into those that are built into the pavement during construction (e.g., bumps or depressions) and those that develop after construction as the result of developing distresses (e.g., cracking or faulting). Pavement roughness is now commonly expressed in terms of the international roughness index (IRI).
- Pavement Surface Friction. Pavement surface friction (sometimes referred to as "skid resistance") is the force developed at the tire-pavement interface that resists sliding when braking forces are applied to the vehicle tires (Dahir and Gramling 1990). Surface friction is largely influenced by the pavement's texture (described in more detail below) and surface drainage characteristics. Adequate surface drainage (i.e., cross slope) influences pavement surface friction by assisting water runoff from the pavement surface.
- Pavement Texture. The feature of the road surface that ultimately determines most of the tire/road interactions including wet friction, noise, splash and spray, rolling resistance, and tire wear is pavement texture (Henry 2000). Pavement texture is typically divided into categories of microtexture, macrotexture, and megatexture based on wavelength and vertical amplitude characteristics (Henry 2000; Gothié 2000):
  - Microtexture—wavelengths of 1 μm to 0.5 mm (0.00004 to 0.02 in) with a vertical amplitude ranging between 1μm and 0.2 mm (0.00004 to 0.008 in). Microtexture is the surface "roughness" of the individual coarse aggregate particles and of the binder, and contributes to friction through adhesion with vehicle tires). For concrete surfaces constructed for speeds under 80 km/h (50 mi/h), microtexture is usually all that is needed to provide adequate stopping in wet weather conditions (Hibbs and Larson 1996).
  - Macrotexture—wavelengths of 0.5 mm to 50 mm (0.02 to 2 in) with a vertical amplitude ranging between 0.1 mm and 20 mm (0.004 to 0.8 in). Macrotexture refers to the overall texture of the pavement, which in concrete pavements is controlled by the surface finish (tining). For concrete pavements constructed for speeds greater than or equal to 80 km/h (50 mph), good macrotexture is needed to reduce the water film thickness and prevent hydroplaning (Hibbs and Larson 1996). The difference between microtexture and macrotexture (and the relative different degrees of each) is illustrated in figure 3.13.
  - Megatexture—wavelengths of 50 mm to 500 mm (2 to 20 in), with a vertical amplitude ranging between 0.1 mm and 50 mm (0.004 to 2 in). This level of texture is generally a characteristic or a consequence of deterioration of the surface.

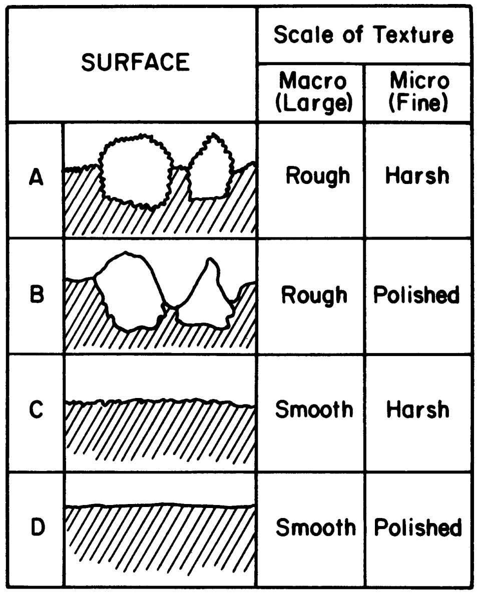

Figure 3.13. Illustration of the differences between microtexture and macrotexture (Shahin 1994).

- <u>Present Serviceability Rating (PSR)</u>. An indicator of pavement roughness based on the subjective ratings of users. The PSR is expressed as a number between 0 and 5 with the smaller values indicating greater pavement roughness. Specifically, the following rating scale applies:
  - 0–1: Very poor
  - 1–2: Poor
  - 2–3: Fair
  - 3–4: Good
  - 4–5: Very good

Roughness Surveys

Roughness surveys are an important part of the pavement evaluation process. They can be conducted subjectively (through windshield surveys) or objectively (with roughness-measuring equipment). Regardless of the method used to determine roughness, the primary purpose of the survey is to identify areas of severe roughness on a given project, as well as to provide some insight into its cause. Roughness surveys can also be useful in determining the relative roughness between projects and in gauging the effectiveness of various treatments.

<u>Types of Roughness Surveys</u>

Windshield Surveys

A simple windshield survey is often an adequate and valid means of subjectively assessing pavement roughness. A trained surveyor who is familiar with the vehicle they are driving should easily be able to assess pavement roughness, particularly if broad categories of roughness (e.g., not rough, slightly rough, moderately rough, very rough) are all that is desired from the evaluation. In addition to giving a subjective rating, additional notes should be taken that indicate the estimated sources of the roughness (i.e., roughness due to *surface distress* such as transverse cracking, corner breaks, faulting, spalling, versus roughness due to *differential elevations* [e.g., swells and depressions]).

Roughness Testing

Objective roughness testing is conducted using commercially available roughness-measuring equipment. Modern roughness-measuring equipment used on pavement evaluation projects can typically be divided into two general categories: *response-type road roughness measuring systems* (RTRRMS) and *inertial road profiling systems* (IRPS). The primary difference between these two categories is that RTRRMS measure vehicle response to pavement roughness while IRPS measure actual pavement profiles. Most highway agencies are using IRPS to monitor the roughness of their pavement network.

To be of most use for the evaluation of a project, it is recommended that the roughness equipment traverse the project in each lane and obtain a representative roughness index for each 0.16 km (0.1 mi) increment. Roughness equipment that only measures one wheelpath should measure the right wheelpath in the direction of traffic for the outer and inner lanes. Special efforts should be made to ensure that the equipment is properly calibrated before its use to eliminate potential equipment deviations over time (Sayers and Karamihas 1998).

A particular concern when testing on concrete pavements is the effect of daily temperature cycles on the measured roughness (Gillespie et al. 1999). On days where the air temperature changes significantly throughout the day, slab curling effects may be introduced that cause significant variations in the measured pavement profile over the course of the day. These effects are more noticeable on short-jointed concrete pavements, and will result in the highest level of roughness occurring in the early morning hours when the slabs are more likely to be curled up. Thus, for project-level profiling, several repeat runs of the project at different times during the day may be necessary to quantify the temperature effects.

Types of Roughness Indices

The roughness index to be used on a project is very much dependent on the type of method and type of equipment used to collect the roughness data. One important aspect to remember in selecting an appropriate roughness index is that, ideally, it should be strongly correlated with user response. All roughness indices can be grouped into three general categories: subjective ratings, mechanical filter-based numerics, and profile-based numerics (Paterson 1987). Each of these roughness index types is discussed in more detail in the following sections.

Subjective Ratings (Serviceability)

Subjective roughness assessments determined while conducting a windshield survey are typically expressed as ratings of *serviceability*. The concept of serviceability was developed at the AASHO Road Test that was conducted in the late 1950s (Carey and Irick 1960; Highway Research Board 1962) and, as previously mentioned, is based on a scale of 0 to 5. The PSR was used in the development of the AASHO pavement design procedure and remains an integral part of the current AASHTO procedures for both new pavement design and overlay design (AASHTO 1993).

Mechanical Filter-Based Indices

RTRRM systems (such as the Mays Ride Meter or BPR Roughometer) measure the cumulative relative displacement between the axle and the vehicle body and then average that value over some distance of roadway; the resulting roughness index is reported in terms of vertical deviation over distance of roadway traveled (e.g., m/km [in/mi]). Examples of mechanical filter-based indices include the Mays Ride Number (MRN) and the Profile Index (PI) (measured with a California-type profilograph).

Summary numerics measured by response-type systems, calibrated to a profile or other numeric in some cases, are reported to not correlate well with user response to roadway roughness (Smith et al. 1997). This poor correlation can be attributed to the inability of response-type systems to measure and sufficiently weight the surface profile wavelengths that are most related to user response and to overall variability within these systems (Smith et al. 1997).

Profile-Based Indices—Mechanical System Simulation

Profile-based pavement smoothness indices are generally obtained by either simulating the response of an RTRRM system as it traverses the profile, or by separating (filtering) and weighting the spectra of wavebands that make up the road surface profile (Smith et al. 1997). One example of such an index is the International Roughness Index (IRI), which has become the most widely used statistic to describe pavement roughness. The IRI is a property of the true pavement profile, and as such can be measured with any valid profiler (Sayers and Karamihas 1998). Furthermore, the IRI provides a common numeric scale of measuring roughness that can be correlated to roughness measurements obtained from both response-type and inertial-based profiler systems (Sayers 1990).

The IRI scale ranges from 0 m/km to 20 m/km (0 in to 1267 in/mi), with larger values indicating greater roughness. The approximate break point between “rough” and “smooth” concrete pavements is often considered to be 2 m/km (125 in/mi). FHWA has presented guidelines in which an “acceptable” ride quality for highway pavements is defined by an IRI range of 0 to 2.7 m/km (0 to 170 in/mi) (FHWA 2006a). The specific FHWA guidelines that relate IRI levels to condition and PSR are presented in table 3.4. IRI is computed in accordance with ASTM Standard E-1926.

Table 3.4. Relationship between IRI and condition (FHWA 2006a).

Ride Quality Terms*	All Functional Classifications	
	IRI Rating (in/mi)	**PSR Rating**
Good	< 95	≥ 3.5
Acceptable	≤ 170	≥ 2.5
Not Acceptable	> 170	< 2.5

* The threshold for “Acceptable” ride quality used in this report is the 170 IRI value as set by the FHWA Performance Plan for the NHS. Some transportation agencies may use less stringent standards for lower functional classification highways to be classified as acceptable.

Surface Friction Surveys

The importance of maintaining adequate pavement surface friction is evident as pavement safety continues to be a major concern of most highway agencies around the world. There are more than one million deaths and 50 million injuries annually on highways and roads worldwide, with more than 40,000 deaths and 3 million injuries annually in the U.S. alone (Larson, Scofield, and Sorenson 2005). Research indicates that about 14 percent of all crashes occur in wet weather, and that 70 percent of these crashes are preventable with improved pavement texture/friction (Larson, Scofield, and Sorenson 2005).

Two primary causes of wet weather crashes are 1) uncontrolled skidding due to inadequate surface friction in the presence of water (hydroplaning) and 2) poor visibility due to splash and spray (Snyder 2006). Moreover, inadequate friction contributes to accidents in dry weather as well, especially in work zones and intersections, where unusual traffic movements and braking action are common.

Historically, pavement friction has been measured directly with different friction-measuring devices and expressed as a single number index (e.g., “skid number”) (Henry 2000). Recent research, however, has indicated that a single number index for evaluating the friction characteristics of a pavement can be misleading, and it is now realized that in order to adequately assess pavement friction characteristics, information on the pavement’s macrotexture characteristics are also important.

Types of Friction Measuring Equipment

There are four basic types of full scale devices used to obtain direct measurements of pavement surface friction. These include locked wheel, side force, fixed slip, and variable slip testers. Each of these equipment types are described in more detail below.

Locked Wheel Testers

Locked wheel testing devices simulate emergency braking conditions for vehicles without antilock breaks (i.e., a 100 percent slip condition). Today, the majority of agencies in the United States measure pavement friction with an ASTM locked-wheel trailer in accordance with ASTM E 274 (Henry 2000). In this procedure, the locked-wheel trailer is towed on a pavement that has been wetted with a specified amount of water, and then a braking force is applied. Testing can be done with either a ribbed (treaded) or blank (smooth) tire, but measurements using the blank tire are reportedly better indicators of the pavement's macrotexture (Dahir and Gramling 1990).

Measurements made with the locked-wheel trailer are reported as a "skid number," that is, the measured value of friction times 100. Skid numbers are reported in the form of: SN(Test Speed [in mph]) followed by an R if a ribbed tire was used or and S if a smooth tread tire. If the test speed is expressed in km/h it is enclosed in parentheses. For example, if a ribbed tire was used in a locked-wheel trailer test at a test speed of 80 km/h (50 mph), the skid number would be reported as SN(80)R or SN50R (metric and English units, respectively).

Side Force Testers

Side force testers are designed to simulate a vehicle's ability to maintain control in curves. They function by maintaining a test wheel in a plane at an angle (the yaw angle) to the direction of motion, while the wheel is allowed to roll freely (i.e., a 0 percent slip condition) (Henry 2000). The developed side force (cornering force) is then measured perpendicular to the plane of rotation. An advantage of these devices is that they measure continuously through the test section while locked wheel devices only sample the friction over the distance while the wheel is locked (the wheel is typically locked for only one second before the brake is released) (Henry 2000). Examples of specific side force testing equipment include the MuMeter and the Sideway-force Coefficient Routine Investigation Machine (SCRIM), both of which originated in the United Kingdom.

Fixed Slip Testers

The fixed and variable slip methods are used to simulate a vehicle's ability to brake while using antilock brakes. Fixed slip devices operate at a constant slip, usually between 10 and 20 percent slip (i.e., the test wheel is driven at a lower angular velocity than its free rolling velocity) (Henry 2000). As with the side force testers, the largest advantage of using a fixed slip tester is that these testers can also be operated continuously over the test section without excessive wear of the test tire. Examples of specific fixed slip testing devices are the Griptester and the SAAB Friction Tester.

Variable Slip Testers

Variable slip testers are similar to fixed slip devices, except that instead of using one constant slip ratio during a test, the variable slip devices sweep through a predetermined set of slip ratios (in accordance with ASTM Standard E 1859) (Henry 2000). An example of a specific variable slip device is the Norsemeter Road Analyzer and Recorder (ROAR) (although this device has not typically been used in the United States for friction testing).

Friction Testing Procedures

The pavement friction should be measured at uniform increments along the project in each traffic lane. As a minimum, most states test in the left wheel path of the driving lane (under normal conditions, this is the location where the surface friction is minimum). The increments should be tied into the mile post markers so that intersections, interchanges, curves, and hills can be identified. Sharp curves are particularly important to consider.

Measuring Pavement Surface Texture

In recent years, it has been recognized that measuring pavement surface texture is necessary to accurately represent a pavement's true functional characteristics. As described previously, pavement texture is primarily divided into three categories: microtexture, macrotexture, and megatexture. While all three are known to influence the pavement's functional performance, it is the surface macrotexture that is most often assessed with texture measuring methods. Traditionally, the sand patch test has been used to assess pavement macrotexture, which produces an indicator of surface texture known as the mean texture depth (MTD). To provide adequate surface friction, the average MTD should be 0.8 mm (0.03 in) with a minimum of 0.5 mm (0.02 in) for any individual test (Hibbs and Larson 1996).

In the past decade, advances in laser technology and computational power have led to the development of systems that measure pavement longitudinal profile at traffic speeds (Henry 2000). Analysis of this data can be used to compute a mean profile depth (MPD), which can be used to estimate the more traditional MTD measurement. The MPD is measured using modern high-speed vehicle-mounted laser-based measuring devices or with portable devices such as the circular track meter (CTMeter).

Evaluation of Roughness, Friction, and Texture Survey Results

Any collected roughness, friction, and texture data should be evaluated in much the same way as pavement condition survey data. These measured data should be summarized so that a clear picture of the existing functional condition can be obtained by those involved in making design decisions. As with condition survey data, strip charts can be a useful way of showing the various condition deficiencies along the project. These should include the location and severity of roughness and surface friction characteristics along the project by lane. Strip charts can also aid in the selection of sites for further detailed materials and pavement testing.

When selecting an appropriate treatment alternative, it is also important to recognize the visible pavement distresses that are also indicative of potential roughness or friction problems. For example, common distresses that greatly influence concrete pavement roughness include:

- Cracking (corner breaks, durability, longitudinal, and transverse) and crack deterioration.
- Transverse joint faulting.
- Transverse joint spalling.
- Punchouts.
- Patch deterioration.

Surface conditions that are indicative of potential surface friction problems include:

- Smooth macrotexture that may be the result of inadequate finishing texturing.
- Polishing caused by soft aggregate.
- Inadequate pavement cross slopes that result in slow runoff of water from the pavement surface.

It is informative to view these poor friction conditions in conjunction with wet-weather crash data to see if there are any correlations. Overall, the combined results obtained from the roughness and friction assessments can be used to determine if functional improvements are needed.

8. FIELD SAMPLING AND TESTING

Introduction

Because they should be in relatively good condition, most pavement preservation candidate projects will not require field sampling or testing as part of the pavement evaluation process. Some exceptions to this might be indications of materials-related distress in the concrete, the presence of peculiar distresses, or areas suggestive of poor support conditions.

When conducted, the primary purposes of field sampling and testing are to help observe subsurface pavement conditions, verify pavement layer types and thicknesses, and retrieve samples for later laboratory testing and analyses. Many different field and laboratory tests are available to determine the subgrade and paving material properties, especially those that are linked to pavement performance. The types and amount of material sampling and testing is primarily dependent upon the following factors:

- Observed pavement distress. The type, severity, extent, and variation of visible distress on a pavement greatly affect the locations and amount of field sampling and testing. If the distress is uniformly spread over the project, sampling is most likely conducted in a random (objective) manner. Otherwise, sampling can be targeted in areas of high distress concentrations.
- Variability. The variability along the project site will affect the amount of material and sampling required. Projects with greater variability in material properties will require a greater amount of testing in order that this variability can be properly characterized and accounted for.
- Traffic volume. The locations and number of allowable samples may be limited on higher trafficked roadways due to worker and driver safety concerns. Such lane closure restrictions and safety related issues are typically not an issue on roadways with lower traffic volumes.
- Economics. Most agencies have a limited budget that determines the types and amount of sampling and testing that can be conducted for a given project. Engineering judgment must be used to determine a sampling and testing plan that minimizes the amount of testing required to adequately assess a pavement's condition, while staying within the provided budget constraints.

The typical field sampling techniques, in situ field testing methods, and standard laboratory testing procedures used in a detailed material investigation are discussed in this section.

Common Field Sampling and Testing Methods

Coring

By far, the most common field sampling method is *coring*, which is the process of cutting cylindrical material samples (cores) from an in-place pavement. Coring is accomplished with the use of a hollow, cylindrical, diamond-tipped core barrel attached to a rotary core drill. The drill is anchored (either to the pavement or to a coring rig) and held perpendicular to the pavement surface while the rotating core barrel is used to slowly cut into the pavement surface. Cores are drilled and retrieved from the pavement and tested in accordance with ASTM C-42, *Standard Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete* and ASTM C-823, *Standard Practice for Examination and Sampling of Hardened Concrete in Construction*.

Coring is most often used to determine/verify layer types and thicknesses, as well as to provide samples (concrete slab and stabilized layers only) for strength testing and possible petrographic examination. A visual inspection of retrieved cores can also provide valuable information when trying to assess the causes of visual distress or poor pavement performance. Cores are particularly useful at identifying material consistency problems such as honeycombing in concrete.

Cores are commonly cut with diameters of 50-, 100-, or 150-mm (2-, 4-, or 6-in), the selection of which depends on the purpose. If thickness verification is all that is needed, 50-mm (2-in) diameter cores are sufficient. Strength testing is most commonly conducted on 100-mm (4-in) diameter cores; however, a 150-mm (6-in) diameter core is recommended when the maximum aggregate size is greater than 38-mm (1.5 in). Although 100-mm (4-in) diameter cores can be used for petrographic testing, 150-mm (6-in) diameter cores are often preferred.

If desired, material samples of subsurface layers (i.e., subgrade soil, subbase, and base) can be obtained from the core holes. Other specialized testing may also be conducted at these locations, such as split-spoon (split-barrel) sampling and Shelby (push) tubes. More details on all of these material-sampling methods are available elsewhere (Hoerner et al. 2001).

Dynamic Cone Penetrometer (DCP)

The DCP is a device for measuring the in situ strength of paving materials and subgrade soils. The principle behind the DCP is that a direct correlation exists between the "strength" of a soil and its resistance to penetration by solid objects (Newcomb and Birgisson 1999). In the last decade, the DCP has gained widespread popularity, largely because it is fast, easy to use, and provides reliable estimates of CBR (Laguros and Miller 1997).

The DCP consists of a cone attached to a rod that is driven into the soil by the means of a drop hammer that slides along the penetrometer shaft (Newcomb and Birgisson 1999). Figure 3.14 shows a schematic of the DCP apparatus (US Army 1989). The test is performed by driving the cone into the pavement/subgrade by raising and dropping the 8 kg (16.7 lb) hammer from a fixed height of 57.5 cm (22.6 in). Earlier versions of the DCP used a 30º cone angle with a diameter of 20 mm (0.8 in) (Newcomb and Birgisson 1999). More recent versions of the DCP use a 60º cone angle and also have the option of using a 4.6-kg (10 lb) hammer for weaker soils (Newcomb and Birgisson 1999).

During a DCP test, the cone penetration (typically measured in mm or inches) associated with each drop is recorded. This procedure is completed until the desired depth is reached. A representative DCP penetration rate (PR) (mm or inches of penetration per blow) is determined for each layer by taking the average of the penetration rates measured at three defined points within a layer: the layer midpoint, midpoint minus 50 mm (2 in), and midpoint plus 50 mm (2 in). DCP penetration rates can be used to identify pavement layer boundaries and subgrade strata, and to estimate the CBR values of those individual layers.

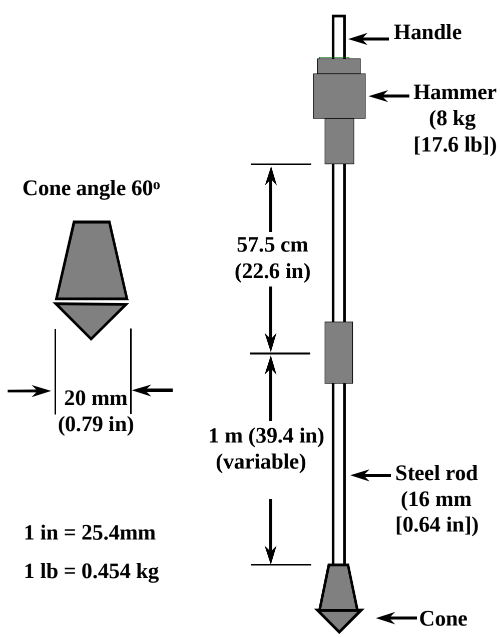

Figure 3.14. Dynamic cone penetrometer (US Army 1989).

DCP results have been correlated with the CBR for a broad range of material types (including fine-grained soils and gravel). The most commonly used empirical correlations express CBR as a function of the DCP Penetration Index (DPI), defined as penetration in millimeters per blow (Newcomb and Birgisson 1999). One of the most widely used correlations between DPI and CBR is the following developed by Webster, Grau, and Williams (1992) for the manual DCP:

$$CBR = \frac{292}{DPI^{1.12}} \tag{3.2}$$

where:

CBR = California Bearing Ratio.
DPI = DCP Penetration Index (measured in mm per blow).

Recent research has also resulted in variations of this equation that are applicable for heavy and lean clays (Webster, Brown, and Porter 1994). These new correlations are illustrated in figure 3.15.

Another example of an empirical relationship between CBR and DPI is the following relationship used in Norway (Newcomb and Birgisson 1999):

$$CBR = 2.57 - 1.25 \times \log DPI \tag{3.3}$$

Automated DCPs are now being developed in which the hammer is picked up and dropped automatically. Research results have indicated that CBR values computed using automated DCP results (obtained using the Israeli automated DCP) are about 15 percent greater than CBR values computed using DPI from the manual DCP (Newcomb and Birgisson 1999).

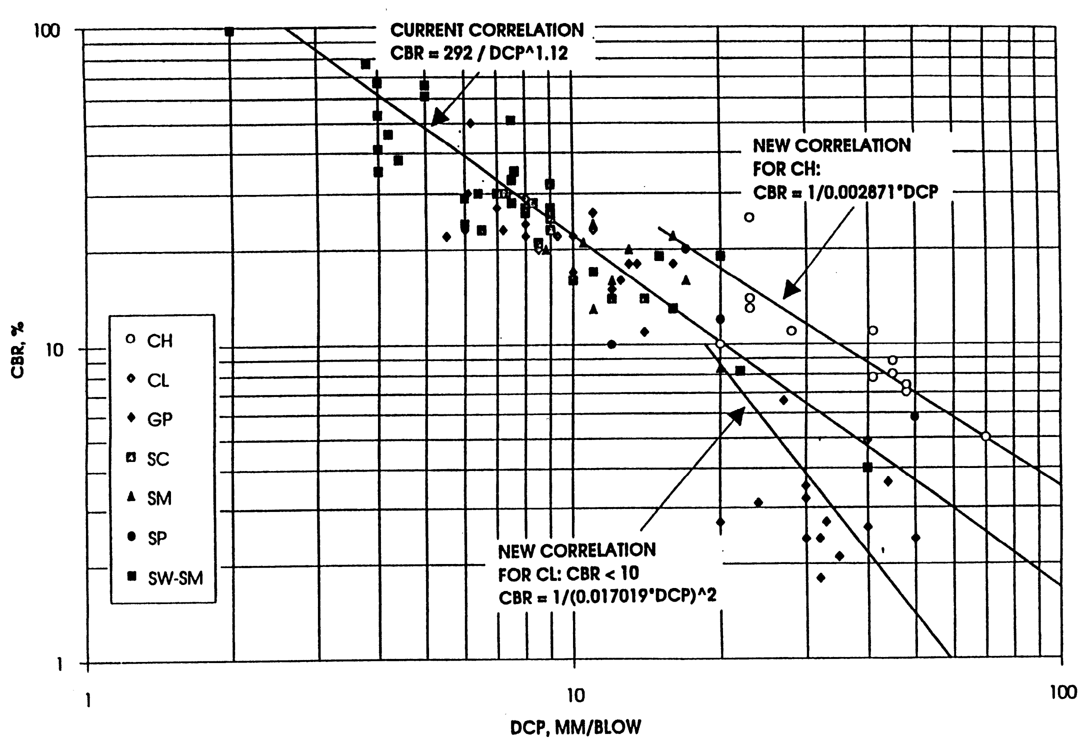

Figure 3.15. Correlations between DPI and CBR (Webster, Brown, and Porter 1994).

Standard Penetration Test (SPT)

The SPT is one of the most common in situ geotechnical tests used all over the world. The SPT consists of driving a standard split-spoon sampler into the ground with blows from a 63.5-kg (140-lb) hammer. The number of blows associated with each 150 mm (6 in) is recorded. Penetration through the first 150 mm (6 in) of soil is considered to be a seating drive. The sum of the number of blows required for the second and third 150 mm (6 in) of penetration (i.e., 150 to 450 mm below the starting elevation) is termed the "standard penetration resistance," or the "N-Value." This measure of resistance to soil penetration is correlated with the relative density, the unit weight, the angle of internal friction, the undrained shear strength, and the elastic modulus of a soil (Kulhawy and Mayne 1990).

There are many published correlations between SPT and important mechanical soil properties such as undrained shear strength, unconfined compressive strength, angle of internal friction, and relative density (Kulhawy and Mayne 1990). An example of a general relationship for fine-grained soils that relates undrained shear strength and N-value (measured in blows per meter) is illustrated in table 3.5 (Kulhawy and Mayne 1990). This information was further generalized into the following relationship:

$$s_u / p_a \approx 0.06N \qquad (3.4)$$

where:

s_u = undrained shear strength.
p_a = atmospheric pressure (included to make the resulting number dimensionless and thus independent of the units of measure.)
N = number of blows per meter.

Table 3.5. Approximate relationship between undrained shear strength and N-value as determined from the Standard Penetration Test method.

N-Value (blows/m)	Consistency	Approximate s_u/p_a
0 – 6	Very soft	< 1/8
6 – 12	Soft	1/8 – ¼
12 – 24	Medium	1/4 – ½
24 – 45	Stiff	1/2 – 1
45 – 90	Very stiff	1 – 2
> 90	Hard	> 2

The primary advantages of the SPT include its availability, past experience with the method (large experience database), and that fact that it is relatively quick and simple to perform; primary disadvantages of the SPT include its many potential sources of error (such as the method of winding the hammer rope around the cathead on the drill rig) and its inaccuracy in soils containing coarse boulders, cobbles, or coarse gravel (Newcomb and Birgisson 1999).

Common Laboratory Testing Methods

This section presents some of the common laboratory testing methods used in the evaluation of pavement layer materials. The types of tests discussed here are divided into general categories of material characterization, material strength and strength-related testing, and special concrete materials evaluation.

Material Characterization (for Subsurface Layer Materials)

Collected material samples (e.g., soil samples and granular base samples) are often subjected to a series of standard laboratory tests such as soil classification, gradation, moisture content, and density. These tests are primarily run to show whether the properties of the materials have changed since construction. Original construction records containing original test results may be compared with the present condition of each material to determine if any significant changes have occurred that may be suggestive of a problem in the material. The results of these tests should be used in conjunction with other material tests (e.g., strength-related testing) in order to fully characterize the properties of a material. Some general correlations relating soil classification to traditional measures of subgrade support or strength are provided in figure 3.16 (PCA 1992).

Strength and Strength-Related Testing

The ability of a pavement structure to adequately carry repeated traffic loadings is very much dependent on the strength, stiffness, and deformation-resistance properties of each layer. Strength tests, or tests that are indicative of material strength, have long been a popular method of assessing the quality of a pavement layer. However, measures of elastic or resilient modulus have a greater significance because of their effect on the way pavements respond to load. The types of tests used depend on the type of material making up a given layer (stabilized or unstabilized) and the function of the layer (surface, base, subbase, or subgrade soil material).

There are various laboratory testing methods that are used to measure material strength, stiffness, or its ability to resist deformation or bending. Some of the more common tests used in the assessment of paving materials are described in the following sections.

California Bearing Ratio (CBR)

The CBR test measures the resistance of an unbound soil or base or subbase sample to penetration by a piston with an end area of 1,935 mm^2 (3 in^2) being pressed into the soil at a standard rate of 1.3 mm (0.05 in) per min. A schematic of the test and typical data are shown in figure 3.17. The load resulting from this penetration is measured at given intervals and the resulting loads at sequential penetrations are compared to the penetration recorded for a standard, well-graded crushed stone. The ratio of the load in the soil to the load in the standard material (at 2.5 mm [0.1 in] penetration), multiplied by 100, is the CBR of the soil. CBR values will typically range from 2 to 8 for silts and clays up to 50 to 70 (or more) for granular bases and high-quality crushed stones (PCA 1992).

The CBR test is an empirical test that has been used extensively in pavement design. The major advantages of this test are the simple equipment requirements and the database available for correlating results with field performance. Drawbacks of this test are that it is sensitive to specimen preparation and it does not provide an intrinsic material property.

Hveem Resistance Value

A Hveem Stabilometer measures the transmitted horizontal pressure associated with the application of a vertical load (PCA 1992). In accordance with ASTM D-1560, the test consists of enclosing a cylindrical sample (100 mm [4 in] in diameter and 6 mm [0.25 in] tall) in a membrane and loading it vertically over the full face of the sample to a given pressure. The resulting horizontal pressure is measured and used to calculate the Resistance Value (R-value), which gives an indication of the stiffness of the material. The R-value method has been used most frequently in several western States, but is an empirical test method and does not represent a fundamental soil property.

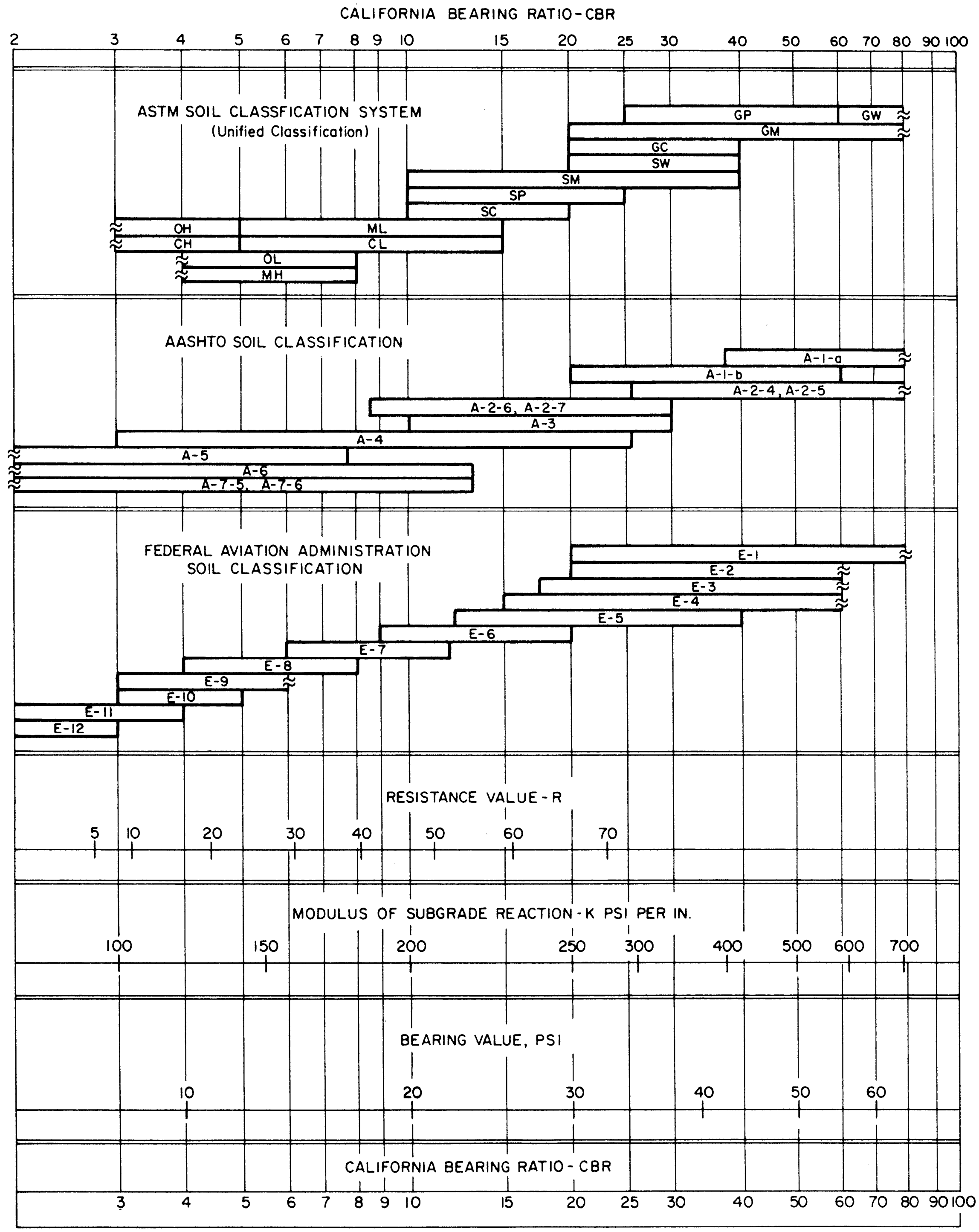

Figure 3.16. Approximate correlations between soil classification and subgrade soil parameters (PCA 1992).

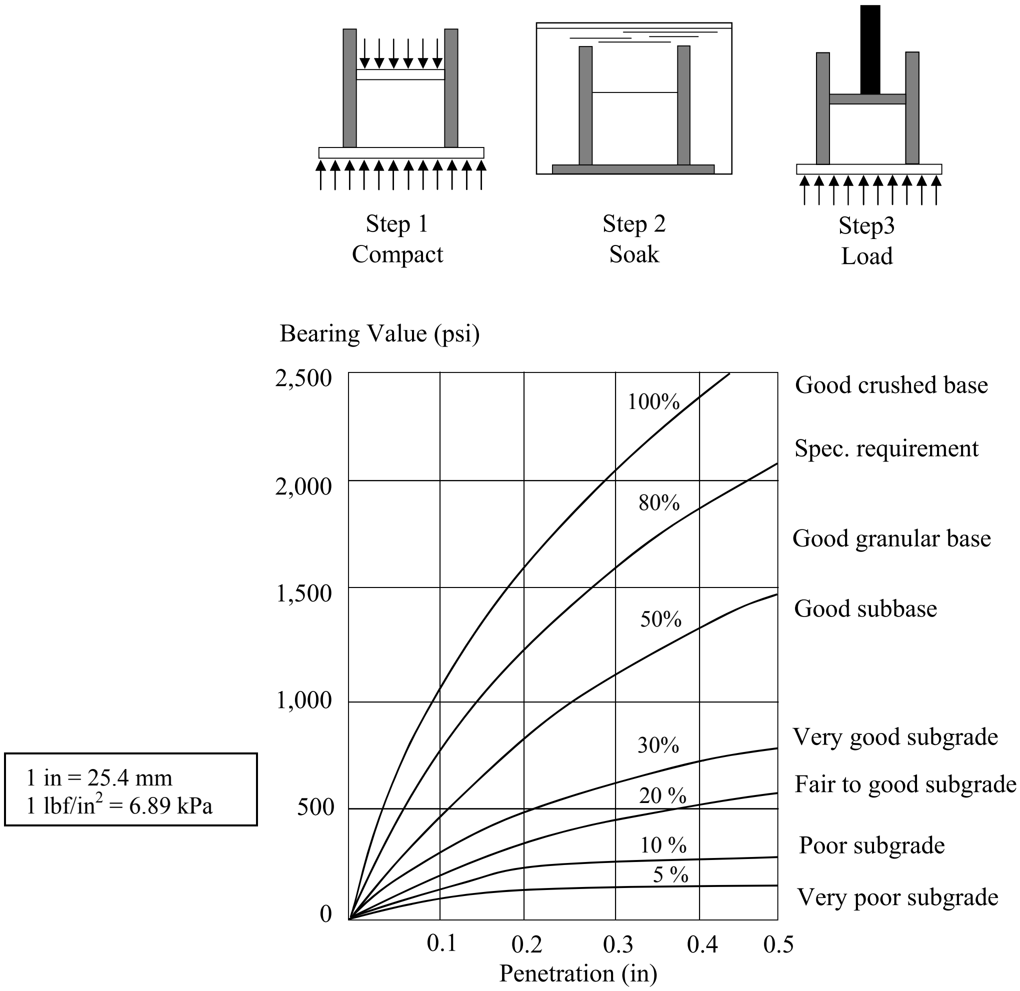

Figure 3.17. CBR testing procedures and load penetration curves for typical soils (Oglesby and Hicks 1982).

Triaxial Strength Testing

The triaxial test is a compressive strength test in which a soil (or unbound material) sample is placed in a triaxial cell and a confining pressure is applied to the sample in the chamber prior to the test. The confining pressures are applied to simulate the confining conditions of the materials in place. A vertical, axial load is then applied to the sample until it fails. Several samples are tested under several confining pressure levels to develop a relationship between the vertical load at failure and the associated confining pressure. The test procedure is described in ASTM D-2850.

Resilient Modulus

The resilient modulus test provides a material parameter that more closely simulates the behavior of the material under a moving wheel. In the laboratory, the resilient modulus test is conducted by placing a compacted material specimen in the triaxial cell, as shown in figure 3.18. The specimen is subjected to an all-around confining pressure, σ_3 or σ_c, and a repeated axial stress (deviator stress), σ_D, is applied to the sample. The number of times the axial load is applied to the sample varies, but typically ranges from 50 cycles to 200 cycles. During the test, the recoverable axial strain, ε_r is determined by measuring the recoverable deformations across the known gauge length. The test is run at various combinations of deviator stress and confining pressure, which vary depending on the type of material being tested (i.e., fine grained or coarse grained).

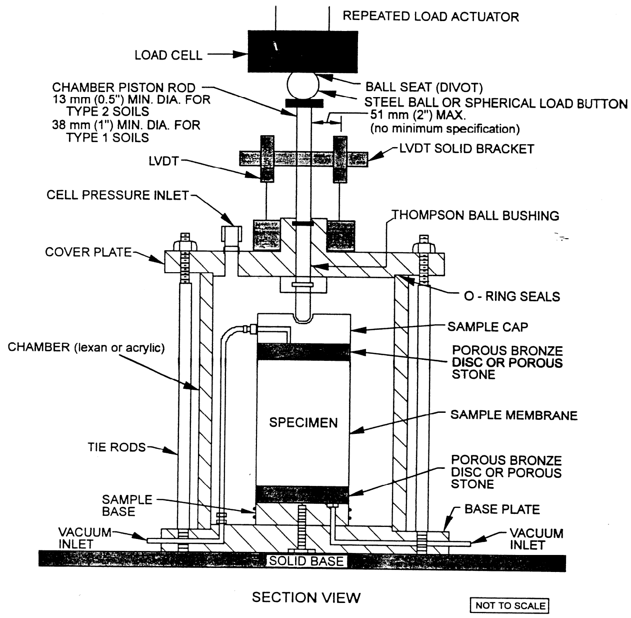

Figure 3.18. Subgrade resilient modulus test apparatus.

Resilient modulus testing is performed on subgrade soils and on unbound base/subbase materials in accordance with one of two current procedures: AASHTO T307-99, *Determining the Resilient Modulus of Soils and Aggregate Materials*, or AASHTO T292-96, *Resilient Modulus of Subgrade Soils and Untreated Base/Subbase Materials*. AASHTO T307-99 is based on LTPP Protocol P-46.

Since not all agencies are familiar with the resilient modulus test and the resultant values, it is useful to consider correlations between some of the various material strength indicators. Approximate relationships between resilient modulus and CBR and R-value are given below. However, these correlations should be taken only as general indicators and should be applied with extreme caution.

- Resilient Modulus vs. CBR:

$$M_R = B * CBR \tag{3.5}$$

where:

M_R = Resilient Modulus, lbf/in^2.
CBR = California Bearing Ratio.
B = Coefficient = 750 – 3000 (1500 for CBR < 10).

- Resilient Modulus vs. R-value:

$$M_R = A + B(R) \tag{3.6}$$

where:

M_R = Resilient modulus, lbf/in^2.
R = Resistance value obtained using the Hveem Stabilometer.
A = Constant = 772 – 1155 (1000 for R < 20).
B = Constant = 369 – 555 (555 for R < 20).

Unconfined Compressive Strength

A very popular test on concrete and other cement- and lime-treated materials is the unconfined compressive strength test. The popularity of this test method is primarily because it is an easy test to perform, and many of the desirable characteristics of concrete are qualitatively related to its strength (Neville 1996). The unconfined compression test can be performed on all stabilized materials used in pavement construction.

For concrete core samples, the test is run in accordance with ASTM C-39, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. The test can be performed on cores obtained for slab thickness determination.

Elastic Modulus Testing

Elastic modulus testing is sometimes conducted on concrete cores samples to help validate FWD results and as an input into many overlay design procedures. Elastic modulus testing is conducted in accordance with ASTM C-469, *Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*.

Indirect Tensile Strength

The indirect tension test, also called the splitting tensile test, can be used to determine the tensile strength of concrete cores or any stabilized pavement layer. The procedure is described in ASTM C-496, *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens*. The test involves applying a vertical load at a constant rate of deformation (1.3 mm [0.05 in] per min) on the diameter of a cylindrical sample (as shown in figure 3.19). The sample will fail in tension along the vertical diameter of the sample and the indirect tensile strength is calculated from the following equation:

$$\sigma_t = \frac{2P_{ult}}{\pi LD} \tag{3.7}$$

where:

σ_t	=	Indirect tensile strength, Pa (lbf/in^2).
P_{ult}	=	Vertical compressive force at failure, N (lbf).
L	=	Length of sample, m (in).
D	=	Diameter of sample, m (in).

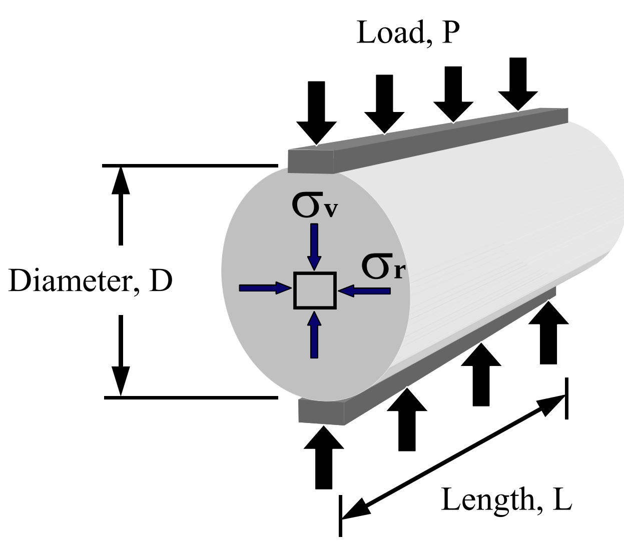

Figure 3.19. Indirect tension test (Mindess and Young 1981).

This test is particularly valuable for pavement evaluation purposes as it is performed on cores taken from the pavement. As with the compression testing, this test can be performed on cores obtained for slab thickness determination.

Special Concrete Materials Evaluation Tests

In some cases, an existing concrete pavement may be exhibiting materials-related distresses (MRD) that is compromising the performance of the pavement. Materials-related distresses are those distresses that develop due to the concrete's inability to maintain its integrity (changes in concrete microtexture) when subjected to changes in physical (environmental) and chemical mechanisms. MRD is generally visible as cracking or a degradation of the concrete such as scaling or spalling, often accompanied by some type of staining or exudate.

The occurrence of MRD is a function of many factors, including the constituent materials (aggregate, cement, admixtures) and their proportions, the pavement's location (maritime or inland), the climatic conditions (temperature, moisture) to which it is subjected, and the presence of external aggressive agents (e.g., roadway deicing chemicals) (Van Dam et al. 2002a). It is not uncommon for combinations of these factors to result in the occurrence of multiple types of MRD in a given pavement section. When multiple MRD types develop together, the process of determining the exact cause(s) of material failure is often complicated. Table 3.6 summarizes details of the most common MRD types, including information regarding their causes, typical time of appearance, and prevention (Van Dam et al. 2002a).

When MRD is suspected of playing a role in the premature deterioration of concrete, laboratory tests are essential to help understand the underlying mechanisms at work (Van Dam et al. 2002b). Typical laboratory methods used to characterize PCC microstructure include optical microscopy (OM), staining tests, scanning electron microscopy (SEM), analytical chemistry, and x-ray diffraction (XRD).

Optical microscopy using the stereo microscope and the petrographic microscope are recognized as the most versatile and widely applied tools for diagnosing causes of MRD. Staining tests are effective at identifying certain types of MRD. Electron microscopy is becoming more prevalent, especially for chemical identification of reaction products and other secondary phases using energy dispersive spectroscopy (Van Dam et al. 2002b). Analytical chemistry is an effective method of determining some of the key parameters of the concrete (e.g. water-to-cement ratio [w/c], chloride content). XRD is applicable in some cases, however, it is not widely used in the analysis of PCC.

9. SUMMARY

This chapter presents guidelines and procedures on conducting an overall pavement project evaluation. A thorough pavement evaluation is absolutely essential to the identification of appropriate and cost-effective solutions to the observed problems. Many premature failures can be attributed to a lack of understanding about the cause or extent of pavement deterioration.

A thorough pavement evaluation begins with the collection and review of all available historical data associated with a given project. This includes reviewing original design data, construction information, subgrade data, performance data, and so on. A collective review of this data often provides an engineer with valuable insight into why the pavement is performing the way it is.

A pavement distress survey is the first and most fundamental pavement evaluation procedure. As part of the survey, pavement distress is defined in terms of type, severity, and amount in order to fully characterize the condition of the existing pavement. By knowing the type of distress, insight as to whether the distress is primarily load-related or primarily materials/climate-related can be gained, which in turn will assist in the selection of the appropriate treatment alternative.

Table 3.6. Summary of key materials-related distresses (Van Dam et al. 2002a).

Type of MRD	Surface Distress Manifestations and Locations	Causes/ Mechanisms	Time of Appearance	Prevention or Reduction
MRD Due to Physical Mechanisms				
Freeze-Thaw Deterioration of Hardened Cement Paste	Scaling, spalling or map-cracking, generally initiating near joints or cracks; possible internal disruption of concrete matrix.	Deterioration of saturated cement paste due to repeated freeze-thaw cycles.	1–5 years	Addition of air-entraining agent to establish protective air void system.
Deicer Scaling/Deterioration	Scaling or crazing of the slab surface with possible alteration of the concrete pore system and/or the hydrated cement paste leading to staining at joints/cracks.	Deicing chemicals can amplify freeze-thaw deterioration and may interact chemically with cement hydration products.	1–5 years	Provide minimum cement content of 335 kg/m^3, limit water–cement ratio to no more than 0.45, and provide a minimum 30-day "drying" period after curing before allowing the use of deicers.
Freeze-Thaw Deterioration of Aggregate (D-cracking)	Cracking parallel to joints and cracks and later spalling; may be accompanied by surface staining.	Freezing and thawing of susceptible coarse aggregates results in fracturing and/or excessive dilation of aggregate.	10–15 years	Use of non-susceptible aggregates or reduction in maximum coarse aggregate size.
MRD Due to Chemical Mechanisms				
Alkali–Silica Reactivity (ASR)	Map cracking over entire slab area and accompanying expansion-related distresses (joint closure, spalling, blowups).	Reaction between alkalis in the pore solution and reactive silica in aggregate resulting in the formation of an expansive gel and the degradation of the aggregate particle.	5–15 years	Use of non-susceptible aggregates, addition of pozzolans to mix, limiting total alkalis in concrete, minimizing exposure to moisture, addition of lithium compounds.
Alkali–Carbonate Reactivity (ACR)	Map cracking over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Expansive reaction between alkalis in pore solution and certain carbonate/dolomitic aggregates which commonly involves dedolomitization and brucite formation.	5–15 years	Avoid susceptible aggregates, significantly limit total alkalis in concrete, blend susceptible aggregate with quality aggregate or reduce size of reactive aggregate.
External Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area, ultimately resulting in joint or surface deterioration.	Expansive formation of ettringite that occurs when external sources of sulfate (e.g., groundwater, deicing chemicals) react with the calcium sulfoaluminates.	1–5 years	Use *w/c* below 0.45, minimize tricalcium aluminate content in cement, use blended cements, use pozzolans.
Internal Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Formation of ettringite from internal sources of sulfate that results in either expansive disruption in the paste phase or fills available air voids, reducing freeze-thaw resistance.	1–5 years	Minimize internal sources of slowly soluble sulfates, minimize tricalcium aluminate content in cement, avoid high curing temperatures.
Corrosion of Embedded Steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel.	Chloride ions penetrate concrete, resulting in corrosion of embedded steel, which in turn results in expansion.	3–10 years	Reduce the permeability of the concrete, provide adequate concrete cover, protect steel, or use corrosion inhibitor.

Drainage surveys are performed as part of a pavement distress survey in order to assess the overall drainage conditions of the existing pavement. This is because poor drainage conditions have long been recognized as a major cause of distress in pavement structures, and unless moisture-related problems are identified and corrected where possible, the effectiveness of any treatment will be reduced.

A number of other field testing procedures are available for evaluating an existing pavement, although they may not commonly be needed for candidate pavement preservation projects. These procedures include deflection testing, smoothness and friction testing, and field sampling and testing.

Deflection testing is often conducted as part of a pavement evaluation program to assess the uniformity and structural adequacy of existing pavements. For concrete pavements, deflection data can be analyzed to provide a wealth of information about the existing pavement structure, including the concrete elastic modulus and modulus of subgrade reaction (k-value), seasonal variations in these values, load transfer efficiencies, and the presence of voids under slab corners and edges. Over the years a variety of deflection testing equipment has been used, with the falling weight deflectometer (FWD) established as the current worldwide standard.

In addition to determining a pavement's *structural* condition, it is also important to assess a pavement's *functional* characteristics. Functional considerations are those pavement characteristics that identify how well the pavement is providing a smooth, safe ride to the traveling public. Measurable characteristics that give an indication of a pavement's functional condition include roughness, surface friction, and surface texture. Common methods and equipment used to assess these functional characteristics are also included in this chapter.

Finally, it may be necessary to conduct a more detailed investigation of the in-place materials within a pavement structure. This additional material property data is commonly used to calibrate/verify distress and deflection data, provide material information where NDT data is not available, and help determine the causes of any observed pavement deficiencies. Many of the more commonly used in situ field tests and laboratory test methods are described in this chapter.

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NOTES

CHAPTER 4. SLAB STABILIZATION AND SLAB JACKING

1. LEARNING OUTCOMES

This chapter covers the use of two different pavement preservation treatments: slab stabilization (also known as undersealing), which is performed to restore support beneath concrete slabs, and slab jacking, which is conducted to physically lift a depressed slab back to the elevation of adjacent slabs. The participants will be able to accomplish the following upon successful completion of this chapter:

1. List benefits of slab stabilization and slab jacking.
2. Describe recommended materials and mixtures.
3. Identify recommended construction activities.
4. Identify typical construction problems and remedies.

2. INTRODUCTION

Pumping and loss of support occurs beneath concrete pavements due to the presence of three factors: an erodible base or subbase, excessive moisture, and significant slab deflections. Poor support conditions can lead to faulting and corner breaks, and can be a major contributor to the accelerated deterioration of the pavement. Slab stabilization has been used to restore support to slabs by filling voids, thereby reducing deflections and retarding the development of additional pavement deterioration.

Settlements sometimes occur on PCC pavements in areas of poor foundation support. Such settlements not only provide riding discomfort, they also can create large stresses in the slab that can lead to cracking. In some cases, these slabs can be raised back to their original elevation by pressure inserting a material beneath the settled slabs and raising them back to the desired elevation. This process of raising slabs is referred to as slab jacking.

3. PURPOSE AND PROJECT SELECTION

Slab stabilization should be performed only at joints and working cracks where loss of support is known to exist. Stabilizing slabs where loss of support does not exist is not only wasteful, it may even be detrimental to pavement performance (Crovetti and Darter 1985; Wu 1991). To be most effective, it is important that slab stabilization be performed prior to the onset of pavement damage due to loss of support (Wu 1991; ACPA 1994). However, because loss of support is caused by several factors, slab stabilization alone is not sufficient to eliminate the problem; the underlying mechanisms themselves must also be addressed in order to ensure the longevity of the treatment (ACPA 1994; Hoerner et al. 2001).

As differentiated from slab stabilization, slab jacking consists of the pressure insertion of a grout or polyurethane material beneath the slab to slowly raise the slab until it reaches a smooth profile. Ideal projects for slab jacking are pavements that exhibit localized areas of settlement. Settlements can occur anywhere along a pavement profile, but most usually are associated with fill areas, over culverts, and at bridge approaches. Slab jacking is not recommended for repairing faulted joints, as this is more effectively addressed through diamond grinding.

4. LIMITATIONS AND EFFECTIVENESS

Slab Stabilization

Over the years, a number of state highway agencies have experienced mixed results with slab stabilization. One of the biggest issues has been the ability to accurately identify the presence of voids beneath the slab. When slab stabilization has been conducted where no voids exist, the pumping of the material beneath the slab can induce stress points and actually increase the rate of pavement deterioration.

On the other hand, some agencies have shown that slab stabilization can be an effective technique when performed under the right conditions. For example, a 2000 study conducted by the Missouri DOT concluded that (Donahue, Johnson, and Burks 2000):

- Slab stabilization and diamond grinding can be an effective CPR technique under the right conditions.
- Evidence of widespread pumping and highly plastic fine-grained subgrade soils with high in-situ water contents should eliminate a concrete pavement from being a candidate for undersealing/diamond grinding.
- Retrofitting edge drains provide little, if any, additional benefit to undersealing/diamond grinding.
- Slab stabilization/diamond grinding should not be expected to provide more than 5 years of reasonable service to a concrete pavement with high cumulative ESALs.
- Slab stabilization/diamond grinding may provide 10 years or more service to a concrete pavement with low cumulative ESALs.

Overall, the effectiveness of slab stabilization is greatly dependent on the selection of an appropriate project and careful quality control of the construction process.

Slab Jacking

The effectiveness of slab jacking is highly dependent upon closely monitoring the amount of lift being performed at any one location. It is very important that the slab not be lifted more than 6 mm (0.25 in) at a time to prevent the development of excessive stresses in the slab. Where careful monitoring has been conducted, slab jacking has been effective at leveling out isolated depressed areas (such as over culverts) and at bridge approach slabs.

5. MATERIALS AND DESIGN CONSIDERATIONS

Determining the Repair Area

Slab Stabilization

For slab stabilization, the first step in the process is locating the areas of voids beneath the slab. The following techniques have been used to determine whether loss of support has occurred beneath a concrete pavement surface:

- Visual observations. Faulting of transverse joints and cracks, pumping, corner breaks, and shoulder drop-off all indicate that loss of support has occurred (ACPA 1994). Figure 4.1 shows the progression of deterioration in nondoweled concrete pavements as it occurs in four stages (Darter, Barenberg, and Yrjanson 1985). Ideally, slab stabilization should be conducted at the third stage before the onset of slab cracking.
- Deflection data. Deflection data can be used not only to determine whether loss of support has occurred, but also to make estimates of the quantity of grouting material required to adequately fill the voids. Several deflection-based void detection methods are available and have been used by a number of highway agencies. Deflections can be measured using an FWD or by using a loaded truck with dial gauges placed on the slab corners.
- Other nondestructive (NDT) methods. Other NDT methods have been used for void detection, including ground penetrating radar (GPR) and infrared thermography. Recent improvements in GPR equipment and data interpretation techniques have enabled the detection of air-filled voids as small as 6 mm (0.25 in) thick (the detection of water-filled voids is more difficult) (Morey 1998; Maser 2000).

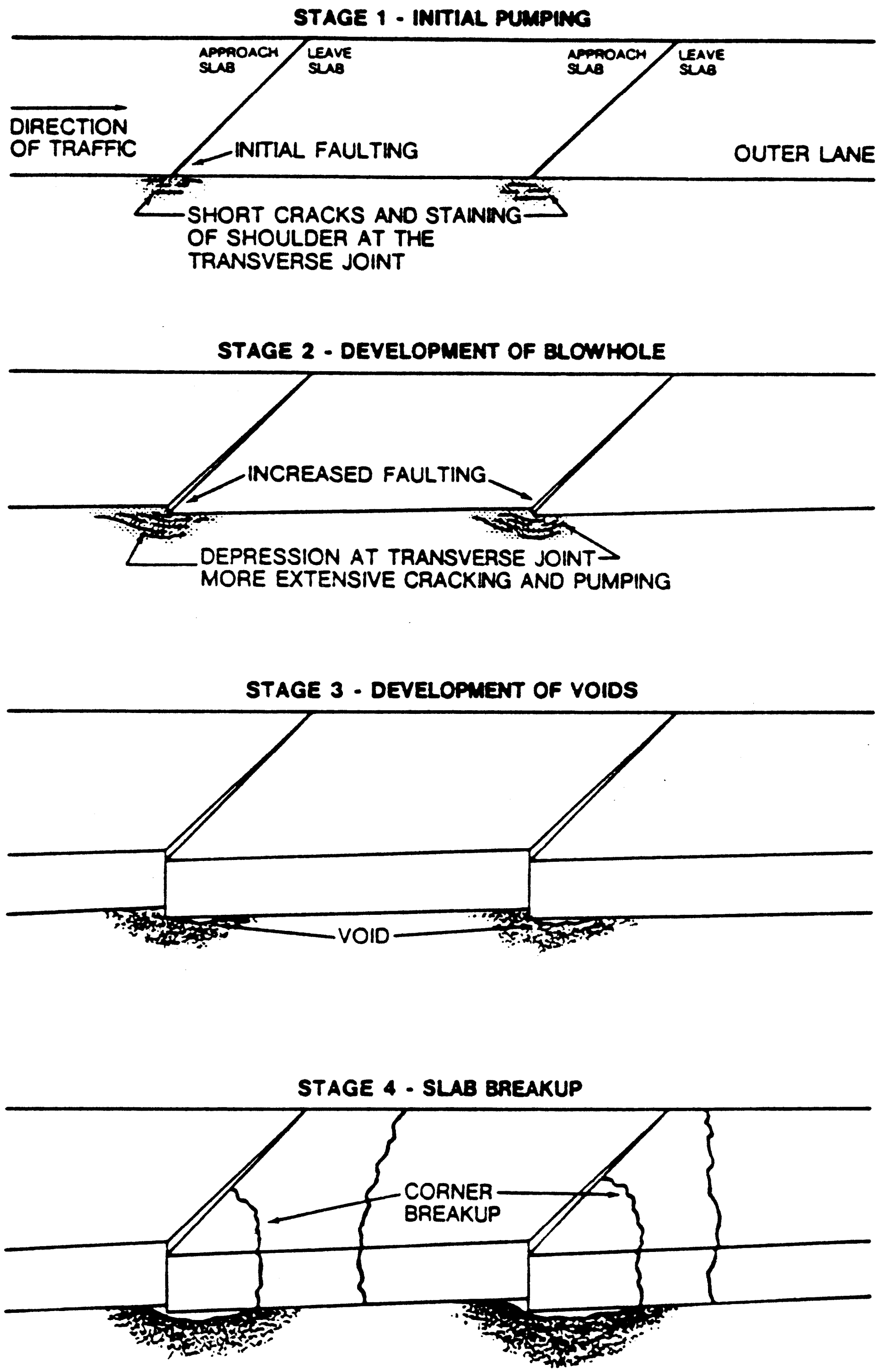

Figure 4.1. Typical stages in the deterioration of a concrete pavement (Darter, Barenberg, and Yrjanson 1985).

Many agencies use a maximum corner deflection criterion to determine if a void is present. Table 4.1 summarizes some available agency-defined maximum corner deflection values that are used to trigger the need for slab stabilization. However, specifications based on a single corner deflection may not always provide reasonable estimates of the presence of a void. This is because the variation in load transfer from joint-to-joint can cause considerable variation in corner deflections. The deflections can be measured using an FWD or by using a loaded trucks with dial gauges placed on the slab corners.

Table 4.1. Maximum corner deflection criteria used by selected States for assessing the presence of voids (Taha et al. 1994).

State	Maximum Corner Deflection, mm (in)
South Dakota	0.25 (0.010)
Florida	0.38 (0.015)
Pennsylvania	0.50 (0.020)
Oregon	0.64 (0.025)
Georgia	0.76 (0.030)
Texas	0.50 (0.020)
Washington	0.89 (0.035)

Another deflection-based method of identifying the presence of voids measures and plots the profile of both the approach and leave corner deflections. An example of this procedure is shown in figure 4.2, in which deflection measurements are recorded at a constant load at both the approach slab corner and the leave slab corner (Darter, Barenberg, and Yrjanson 1985). As voids first form under the leave corner, it is normal to find that the approach corner deflection is less than the leave corner deflection. If this difference is great, then the presence of a void is likely (Darter, Barenberg, and Yrjanson 1985). The procedure recommends the identification of a corner deflection value above which slab stabilization is warranted. For example, in figure 4.2, a reasonable value might be 0.5 mm (0.02 in).

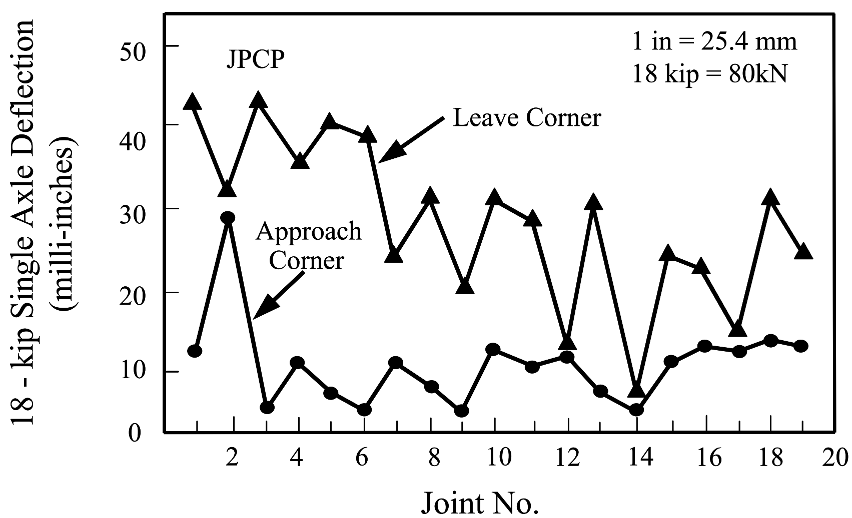

Figure 4.2. Example profile of corner deflections (Darter, Barenberg, and Yrjanson 1985).

Another void detection method is based on measuring the magnitude of the corner deflection at three different load levels (Crovetti and Darter 1985). Typically, load levels of 27, 40, and 63 kN (6, 9, and 14 kips) have been used to develop load versus deflection plots for each test location (Crovetti and Darter 1985). Load versus deflection plots passing through or very near the origin on these charts suggest that full support exists under the slab corner.

Still another procedure that has been used to identify voids is the epoxy/core test method. In this procedure, a hole is drilled in the slab at a suspected void location and then a low viscosity, two-part epoxy is poured into the hole, which fills any void that might be present. After the epoxy hardens, a core is taken over the injection hole and examined to note the existence of a void (Chapin and White 1993).

Slab Jacking

Slab jacking should be considered for any condition that is the result of nonuniform support. These conditions often result in localized dips or depressions that adversely affect the rideability of the pavement. Common areas include slabs over culverts or bridge approach slabs, both typically the result of poor or inadequate compaction of the underlying fill. Localized settlements may also occur over embankment areas.

Selecting an Appropriate Injection Hole Pattern

Slab Stabilization Hole Pattern

After identifying any voids that would benefit from slab stabilization, the next step is to determine the optimal locations of grout insertion holes (i.e., the hole pattern). The pattern is dependent on a number of factors, including the following (Darter, Barenberg, and Yrjanson 1985):

- Pavement type (i.e., jointed-plain concrete pavement [JPCP], jointed-reinforced concrete pavement [JRCP], or continuously-reinforced concrete pavement [CRCP]).
- Transverse joint spacing (jointed pavements).
- Estimated size and shape of the detected void.
- The flowability of the material being used.
- Location of cracks and joints near void.
- Slab condition.

Holes should be placed as far away from nearby cracks and joints as possible, but still within the area of the identified void. Moreover, the holes should be placed close enough to achieve a flow of grout from one insertion hole to another when a multiple hole pattern is used. Figure 4.3 illustrates recommended initial trial hole patterns for different void locations on jointed concrete pavements. It is noted that in some cases the slab stabilization may be needed only on the leave (downstream) side of the joint, whereas in other cases slab stabilization may be needed on both the approach (upstream) and leave sides. A typical hole spacing for CRCP is shown in figure 4.4.

Slab Jacking Hole Pattern

The best location of holes for a given site can only be determined by experienced personnel. Holes should be spaced not less than 305 mm (12 in) nor more than 457 mm (18 in) from a transverse joint or slab edge (MnDOT 2006). In addition, holes should be spaced 1.8 m (6 ft) or less center-to-center, so that less than 2.32 to 2.78 m^2 (25 to 30 ft^2) of the slab is raised by grouting a single hole (MnDOT 2006). A greater number of holes may be required if the slabs are cracked. Figure 4.5 illustrates the location of holes in a triangular pattern, correcting a settlement over two lanes. The holes are spaced, as nearly as possible, equidistant from one another, as the grout tends to flow in a circular pattern from each hole. Holes in adjacent slabs should follow the same arrangement.

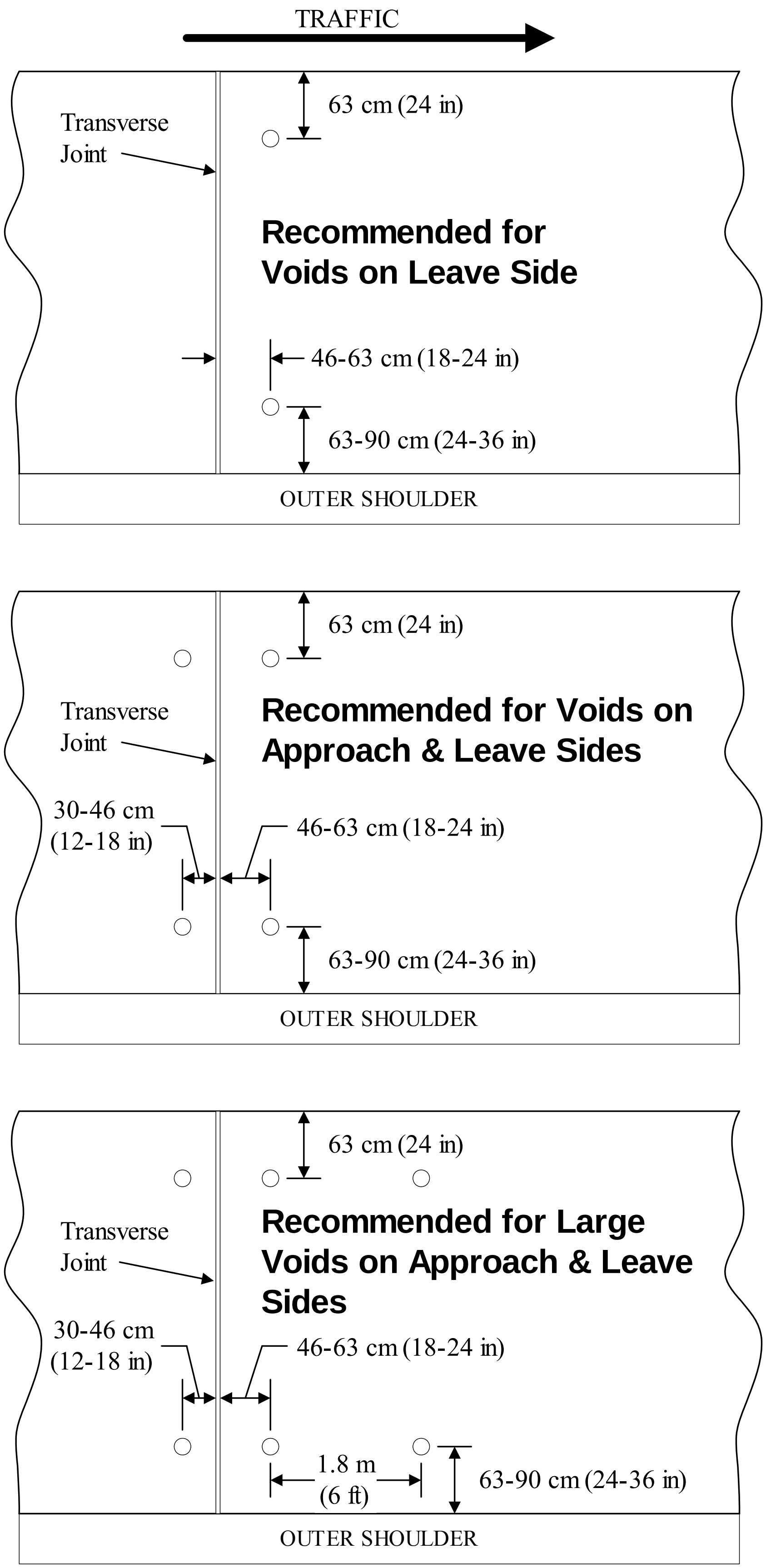

Figure 4.3. Typical hole pattern for jointed concrete pavements (Darter, Barenberg, and Yrjanson 1985).

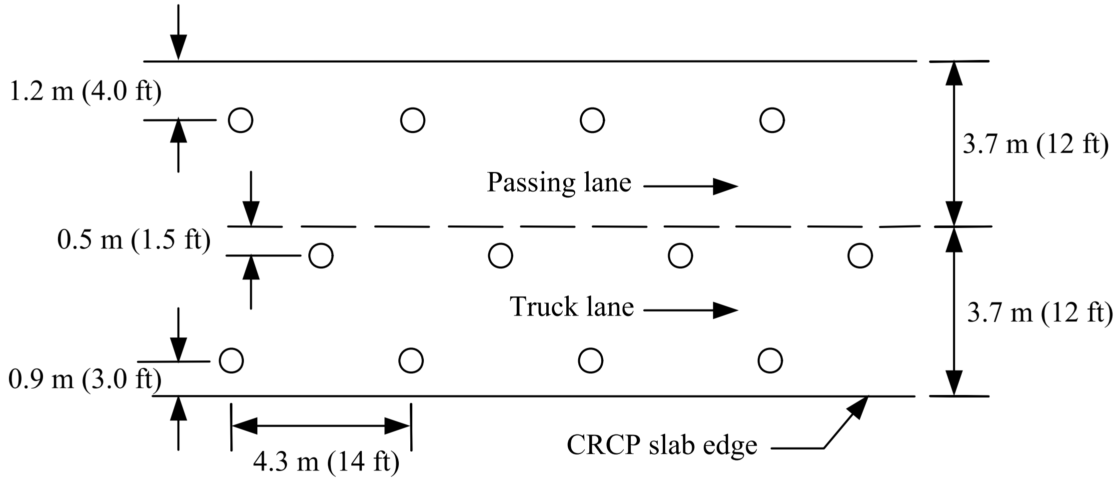

Figure 4.4. Hole pattern used on a continuously reinforced concrete pavement (Barnett, Darter, and Laybourne 1980).

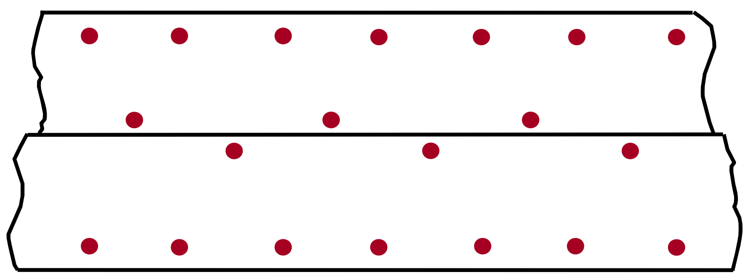

Figure 4.5. Pattern of grout pumping holes used to correct a settlement.

Selecting an Appropriate Material

The material chosen for slab stabilization must be able to penetrate into very thin voids while having the strength and durability to withstand pressures caused by traffic, moisture, and temperatures. Many different slab stabilization materials have been used, with pozzolan-cement grout and polyurethane being the most common. Other materials used less frequently include asphalt cement, limestone dust-cement grouts, and silicone rubber foam (Taha et al. 1994).

Materials used for slab jacking are typically slightly stiffer than those used for slab stabilization. Cement grout and polyurethanes are two materials commonly used for slab jacking.

Cement Grout Mixtures

Historically, the more common cement-based grout mixtures included pozzolanic-cement and limestone-cement grout (Taha et al. 1994). The typical flow cone time for limestone grouts is 16 to 22 seconds, whereas the typical time for flyash grouts is in the 10 to 16 second range (for comparison, water has a flow cone time of 8 seconds) (ACPA 2003).

The following is a typical mix design for a pozzolanic-cement grout for use in slab stabilization (ACPA 2003; ACCA 2003):

- One part by volume portland cement type I or type II (type III may be specified if there is a need for early strength).

- Three parts by volume pozzolan (Class F flyash; it may be possible to reduce the cement component if Class C flyash is used). Pozzolans shall conform to the requirements of ASTM C 618, if used, and limestone dust shall comply with AASHTO M 17 for mineral fillers.
- Water (usually about 1.5 to 3.0 parts) to achieve required fluidity.
- If ambient temperatures are below 10 ºC (50 ºF), an accelerator may be used (if approved).
- A minimum compressive strength (typically 4.1 MPa [600 lbf/in^2] at 7 days) is normally required to ensure the durability of the grout. The ultimate strength of the grout will typically be much higher (on the order of 10 to 28 MPa [1,500 to 4,000 lbf/in^2]).
- Additives, superplasticizers, water reducers, and fluidifiers as needed.

Overall, a thorough testing regimen should be instituted to ensure the suitability of the grout prior to the start of any slab stabilization project. The contractor should be able to verify chemical and physical properties of the pozzolan or limestone, 1-, 3-, and 7-day compressive strength tests, flow cone results, time of initial set, and shrinkage/expansion results.

Cement grouts used for slab jacking are typically slightly stiffer than that used for slab stabilization procedures, generally having flow cone times of 16 to 30 seconds. Pozzolan and fly ash based grouts generally consist of three to seven parts fine aggregate (or a mixture of aggregate and pozzolans or flyash) to one part portland cement, with enough water to produce the desired consistency (MnDOT 2006).

Polyurethane

In recent years, polyurethane materials have seen increased use as a slab stabilization and slab jacking material. Polyurethane materials are made of two liquid chemicals that combine under heat to form a strong, light-weight, foam-like substance. After being injected beneath the pavement, a reaction between the two chemicals causes the material to expand and fill any existing voids (ACPA 1994). For slab stabilization purposes, the polyurethane density is about 64 kg/m^3 (4 lb/ft^3) and the compressive strength ranges from about 0.4 to 1.0 MPa (60 to 145 lbf/in^2) (ACPA 1994). One laboratory study indicated that the injected polyurethane will consistently penetrate openings as small as 6.4 mm (0.25 in) and will penetrate some openings as small as 3.2 mm (0.125 in) (Soltesz 2000).

The URETEK Method™ is a well-known and patented process that uses high-density polyurethane foam for slab stabilization and slab jacking applications. The URETEK material expands up to 20 times its original liquid volume (thereby effectively filling any surrounding voids), is insensitive to the presence of water, and cures to a strong and durable state (URETEK 2007). According to the manufacturer, the URETEK 486 polyurethane foam system will have a free-rise density of 48 to 51 kg/m^3 (3 to 3.2 lb/ft^3), with a minimum compressive strength of 0.28 MPa (40 lbf/in^2) (URETEK 2007).

States that have used polyurethane materials have experienced mixed results. In Louisiana, a study was conducted in which polyurethane was used to stabilize continuously reinforced concrete pavement (CRCP), jointed concrete pavement (JCP), and bridge approach slabs (Gaspard and Morvant 2004). The initial results of this study found the material to be an effective method of leveling CRCP and bridge approach slabs, but the JCP results were not as positive. Although it was determined that the polyurethane did fill the voids, the material did not appear to provide much support to the joints as the joints were observed to be deflecting under traffic loadings (although the authors did note that the load transfer devices in this pavement were not functioning) (Gaspard and Morvant 2004).

Other states, including Oregon (Soltesz 2000), Missouri (Donahue, Johnson, and Burks 2000), Kansas (Barron 2004), and New Mexico (URETEK 2007) have had good experience with their initial projects using polyurethane foam for slab stabilization. For slab jacking, Wisconsin (Abu al-Eis and LaBarca 2007) reported that the lifting process was successful and that trial projects are performing well after 1 year of service, but also indicated shortcomings in the ability to estimate material quantities.

6. CONSTRUCTION CONSIDERATIONS

Slab Stabilization

Step 1: Drilling of Injection Holes

Any hand-held or mechanical drill that produces clean holes with no surface spalling or breakouts on the underside of the slab is acceptable for creating the injection holes (ACPA 1994). Specifically, for portland cement-based grout projects, any pneumatic or hydraulic rotary percussion drill that is capable of cutting 38-to 51-mm (1.25- to 2.0-in) diameter holes through the slab are suitable. A general specification recommends limiting the downward pressure on any drill to 90 kg (200 lb) to avoid conical spalling at the bottom of the slab (ACPA 1994). When large pieces of the underside of the slab spall, these pieces can potentially block the void and make it impossible to fill.

For polyurethane slab stabilization, hand-held electric-pneumatic rock drills are typically used to drill the injection holes (ACPA 1994). For these procedures, the maximum hole diameter should not exceed 15 mm (0.625 in) (ACPA 1994).

A quick check of whether or not the hole should be grouted may be made by pouring water into the drill hole (note that the water does not create a problem as it is displaced when grout is pumped into the hole). If the hole does not take water, there is no void and therefore no need to grout. When it is determined that there is no void, the hole can be filled with an acceptable patching material and the operation can proceed to the next hole.

While the typical injection hole pattern is determined during the design process, the location of the injection holes may need to be adjusted in the field in order to effectively fill each void. If the flow is easily achieved, the hole spacing may be increased. Conversely, if good flow is not achieved before maximum back pressure is reached, the hole spacing should be reduced.

Step 2: Material Preparation

Most slab stabilization contractors use mobile, self contained equipment that carries all of the tools and materials needed for slab stabilization (ACPA 1994). As past procedures typically utilized labor-intensive, small batch mixers with bagged materials, these modern systems have been found to reduce both labor and materials costs by as much as 30 to 50 percent (ACPA 1994). The differences in preparing cement-based and polyurethane materials are discussed in this section.

Cement Grout Mixtures

For cement-grout mixtures, a grout plant that is capable of accurately measuring, proportioning, and mixing the material by volume or weight is used. When working with pozzolan-cement grouts, it is recommended that contractors use colloidal mixing equipment. Colloidal mixers provide the most thorough mixing for pozzolan-cement grouts, as the material stays in suspension and resists dilution by free water (ACPA 1994). Two of the more common types of colloidal mixers include (ACPA 1994):

- Centrifugal pump mixer. This mixer pulls grout through a mixing chamber at high pressure and velocity.
- Shear blade mixer. For this mixer type, blades rotate at 800 to 2,000 revolutions per minute.

Whenever possible, contractors should avoid using paddle-type drum mixers with cement-pozzolan grouts (ACPA 1994). This is because the low agitation of these mixers makes it very difficult to thoroughly mix the grout. Paddle-drum mixers are, however, effective at thoroughly mixing limestone dust grouts (AASHTO 1993). Conveyors, mortar mixers, or ready-mix trucks should not be used to mix any type of stabilization material as these mixers require adding too much water for fluidity and the solids tend to agglomerate and clump in the mix (ACPA 1994).

Polyurethane

When using polyurethane foam, all material is stored, proportioned, and blended within a self-contained pumping unit. All handling and usage of these materials should be in accordance with the material manufacturer's instructions and specifications.

Step 3: Material Injection

Because the injection procedures differ slightly by material type, the injection procedures associated with each material type are described separately below.

Injection of Cement-Grout Mixtures

It is recommended that positive-displacement injection pumps, or non-pulsing progressive-cavity pumps, be used during the slab stabilization process. It is important that the pump be capable of maintaining low pumping rates and injection pressures. Specifically, a pump should work well if maintaining pressures between 0.15 and 1.4 MPa (25 and 200 lbf/in^2) during grout injection (ACPA 2003). Maintaining a lower pumping rate (ideally about 5.5 liters [1.5 gallons] per minute) and lower pumping pressure ensure better placement control and lateral coverage, and usually keeps the slab from rising (AASHTO 1993). Typical pumping pressures are in the 275 to 413 kPa (40 to 60 lbf/in^2) range (ACPA 2003).

Portland cement-based grouts are typically injected using a grout packer in order to prevent material extrusion or backup during injection. Two types of grout packers are used, depending on the size of the hole. Drive packers are pipes that taper and fit snugly into the injection hole by tapping with a small hammer (ACPA 2003). Drive packers are generally used with 25 mm (1.0 in) diameter holes. Expandable packers consist of a threaded inner pipe, a thin-walled steel outer sleeve, and a short rubber sleeve at the bottom (near the nozzle) that expands to fill the hole during injection (ACPA 2003). Expanding rubber packers require 1.5-in or larger diameter holes (ACPA 2003).

The injection equipment should include either a return hose from the injection device (packer or tapered nozzle) to the material storage tank, or a fast-control reverse switch to stop grout injection quickly when slab movement is detected on the uplift gauge (ACPA 2003). A grout-recirculation system also helps eliminate the problem of grout setting in the injection hoses because the grout circulates back to the pump after pumping ceases (Darter, Barenberg, and Yrjanson 1985).

After grouting has been completed, the packer is withdrawn and the hole is plugged immediately with a temporary wooden plug. When sufficient time has elapsed to permit the grout to set, the temporary plug is removed and the hole is sealed flush with an acceptable patching material.

Slab stabilization should not be performed when the ambient temperature is below 4 ºC (40 ºF). Unless a fast setting material is used, traffic should be kept off of a stabilized slab for at least 3 hours after grouting to allow adequate curing of the grout (Darter, Barenberg, and Yrjanson 1985).

Injection of Polyurethane

Polyurethane grouting operations use slightly different injection equipment from those described above. Instead of large grout packers, plastic nozzles that screw onto the hoses deliver the material into the holes (ACPA 1994).

Slab Jacking

Procedures must be developed to monitor the raising of the slab and to ensure that the profile meets the desired grade. The taut stringline method (illustrated in figure 4.6) is an excellent way to not only control the pumping sequence, but also to achieve the proper grade.

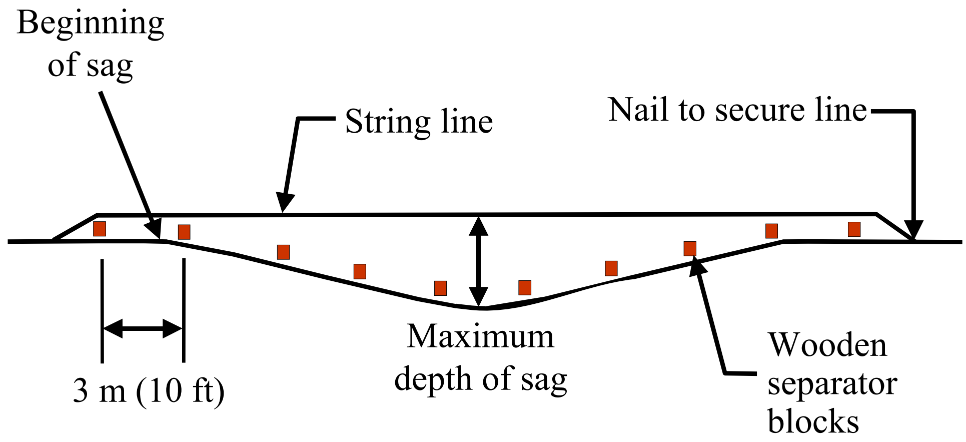

Figure 4.6. Stringline method of slab jacking.

In the stringline method, small wooden blocks, 19 mm (0.75 in) high, are set on the pavement surface along the outer and inner edges and a stringline is secured at least 3 m (10 ft) from each end of the depression. As material pumping proceeds, the exact amount of rise at each point within the sag can be observed, allowing the pumping at specific holes to be carefully controlled. This method can consistently achieve profiles within tolerances of 6- to 9-mm (0.25- to 0.38-in).

Various agencies have different techniques for the raising of the slab. A typical procedure is described below:

- After all preliminary work has been completed (holes drilled, relief opening cut if needed), the pavement is ready to be raised. The slab must be raised only a very small amount at each hole at a time. A good rule is not to raise a slab more than 6 mm (0.25 in) while pumping in any one hole. No portion of the slab should be more than 6 mm (0.25 in) higher than any other part of the slab (or an adjacent slab) at any time. The entire working slab and all those adjacent to it must be kept in the same plane, within 6 mm (0.25 in), throughout the entire operation to avoid cracking.
- Pumping should be done over the entire section so that no great strain is developed at any one place. If, for example, pumping was started at either end of a dip, the tension on the top surface will be increased, and the slab will undoubtedly crack. However, if pumping is started at the middle where the tension is on the lower surface, lifting will tend to reduce it, and the slab can be raised an appreciable amount without any damage. As the section is brought back to its original profile, the pumping is extended farther and farther in either direction until the entire dip is at the desired elevation.
- Care must be taken not to flatten the middle out completely. This will cause a sharp bend and will cause cracking. The middle section naturally must be raised faster than the ends of the dip, but lifting should be conducted in such a manner as to avoid sharp bends.
- An example of a suggested slab jacking pumping sequence that provides a general guideline for obtaining satisfactory results is presented below. It must be remembered that this sequence must be modified to meet the specific needs of a given project.

   a. Figure 4.7 shows a plan view of a dip. Pumping should begin in the middle of the dip, shown as point A. The hole where the material is initially pumped will take more material than those at either side, due to the shape of the dip. Pumping should always begin at the outside holes, followed by the inside row of holes.

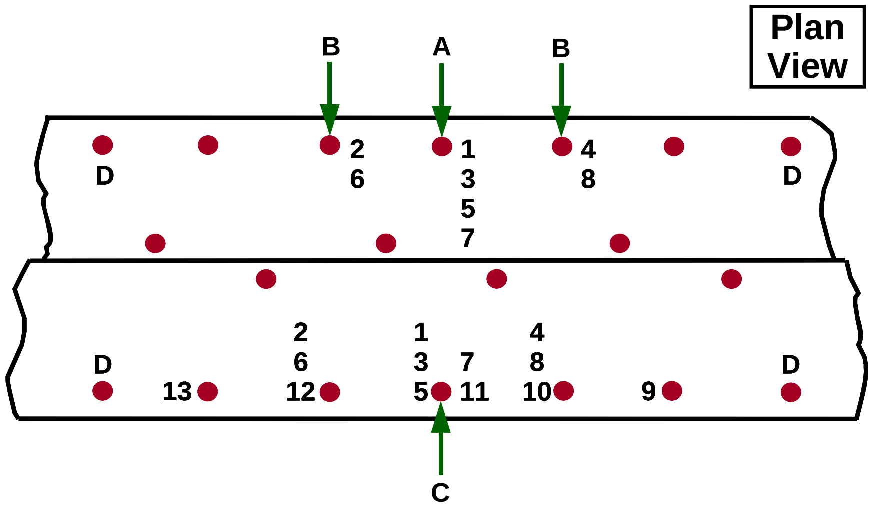

Figure 4.7. Order of grout pumping used to correct a settlement.

b. Pumping at point B relieves the strain that may have resulted from lifting the slab at point A. The third hole to be grouted will be at point A again, and then material is pumped following steps 4 to steps 8 shown in figure 4.7. This results in material being pumped four times into the middle hole at point A and twice at the hole on either side at points B. If the same amount of material was pumped at each time and traveled the same distance away from the hole, the slab would be raised twice as much at the middle hole as at the other two.

c. The line of holes in the middle of the pavement is pumped after the outer row, using the same sequence. If both sides of the slab were at about the same elevation, the next pumping is at the outer side of the adjoining slab at point C, following the same sequence, with additional pumping conducted further from the center of the dip, as shown in figure 4.7 (i.e., grout applications 9, 10, 11, 12, and 13). Pumping is continued in this order until the slab has been brought to the desired elevation.

d. Pumping should never be performed along a series of holes back and forth across the slab; instead, work always proceeds along the length of the slab to avoid cracking. A concrete slab can withstand more twisting than transverse bending.

e. The last hole at each end of a dip, shown as point D in figure 4.7, should not be used until the slab is at the desired grade. A very thin grout, similar to that use for slab stabilization, may be used to ensure complete filling of the thin wedge-shaped opening that was created at this part of the dip.

Holes should be plugged with tapered wooden plugs immediately after pumping in the hole has been completed to retain the pressure of the grout and to prevent any return flow of the mixture (MnDOT 2006). When the entire slab jacking operation is complete, the temporary plugs are removed and filled with an appropriate patching material.

7. QUALITY CONTROL

Slab Stabilization

The purpose of slab stabilization is to fill existing voids and not to raise the slab. Close inspection is required by the contractor and the inspector during the stabilization operation, as lifting of the slabs can create additional voids and may lead to slab cracking. The success of the slab stabilization operations is highly dependent upon the skill of the contractor.

The grout injection should start with a low pumping rate and pressure and should be pumped until one of the following conditions occurs (Darter, Barenberg, and Yrjanson 1985):

- The maximum allowable pressure of 0.69 MPa (100 lbf/in^2) at the grout plant is obtained. Note that a short surge up to 1.38 MPa (200 lbf/in^2) can be allowed when starting to pump in order for the grout to penetrate the void structure, if necessary.
- The slab lift exceeds 3 mm (0.125 in).
- Grout is observed flowing from adjacent holes, cracks, or joints.
- Grout is being pumped unnecessarily under the shoulder, as indicated by lifting.
- More than about 1 minute has elapsed (any longer than this indicates the grout is flowing into a cavity).

The uplift for any given slab corner should be monitored using a modified Benkelman Beam or other similar device that is capable of detecting 0.025 mm (0.001 in) of uplift movement.

The effectiveness of slab stabilization can be determined only by monitoring the subsequent performance of the pavement. The best early indication of effectiveness is obtained by measuring slab deflections before and after grouting to determine if the magnitude of the deflection has been significantly reduced by the process. Figure 4.8 shows measured corner deflections (before and after grouting) for an example joint. If the retesting still indicates a loss of support, the slabs should be regrouted using new drilled holes. ACPA guidelines recommend that if voids are still present after three attempts to stabilize the slab, other methods of repair should be considered (e.g., full-depth repair) (ACPA 2003).

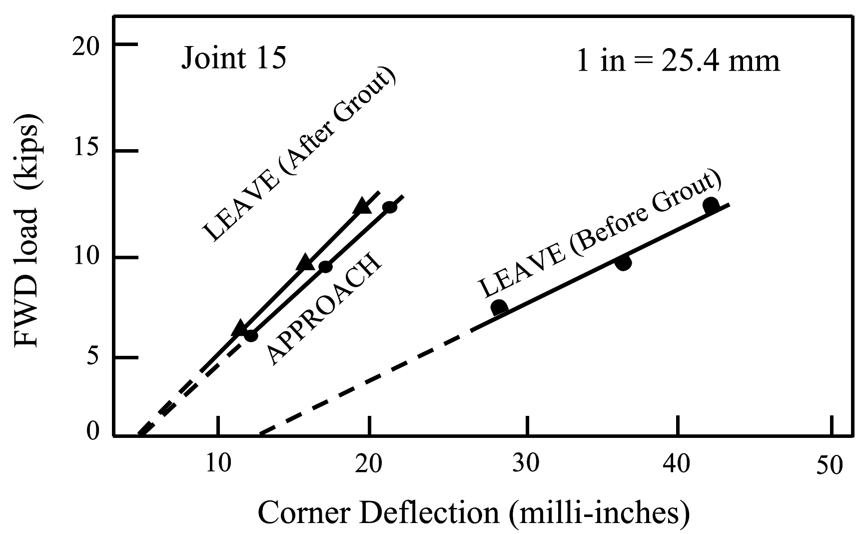

Figure 4.8. Example of load versus deflection plot before and after slab stabilization (Darter, Barenberg, and Yrjanson 1985).

Slab Jacking

The primary concern on slab jacking is excessively raising the slab, which can induce stress concentrations in the slab and produce cracking. Therefore, it is critical that the slab be raised no more than 6 mm (0.25 in) when pumping at each hole. Moreover, during the lifting process, no portion of the slab should be more than 6 mm (0.25 in) higher than any other part of the slab (or an adjacent slab) at any time. The entire working slab and all those adjacent to it must be kept in the same plane, within 6 mm (0.25 in), throughout the entire operation to avoid cracking.

Pumping is generally recommended to start at the middle of the depressed slab. This will help to reduce the tension that has developed at the top of the slab. As the section is brought back to its original profile, the pumping is extended farther and farther in either direction.

The effectiveness of the slab jacking process can be assessed both visually and from an examination of the pavement profile. Figure 4.9 shows the profile of a bridge approach slab, both before and after the slab jacking operation.

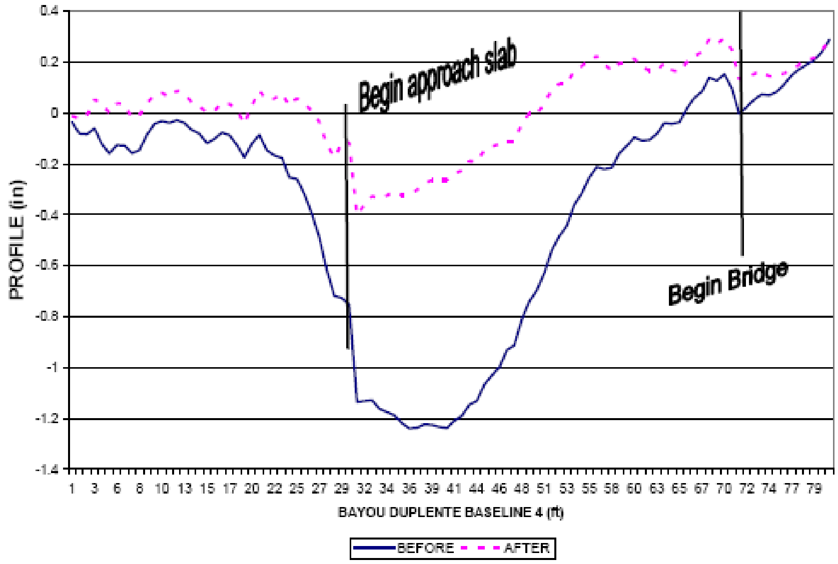

Figure 4.9. Profile of bridge approach slab before and after slab jacking (Gaspard and Morvant 2004).

8. TROUBLESHOOTING

Some of the more common problems that a contractor or inspector may encounter in the field during a slab stabilization project are shown in table 4.2. Typical causes and recommended solutions associated with known problems are also provided.

9. SUMMARY

Slab Stabilization

Loss of support from beneath concrete pavement slabs is a major factor contributing to pavement deterioration. Slab stabilization is defined as the insertion of a material beneath the slab or subbase to fill voids, thereby reducing deflections and associated distresses. However, because loss of support is caused by several factors, slab stabilization is often done in conjunction with other rehabilitation activities in order to address the causes of the voids (ACPA 1994). Commonly used slab stabilization materials include cement-based mixtures (cement-limestone dust slurry and cement-pozzolan slurry) and polyurethane. Since slab stabilization is not intended to lift the slab, it is very important to monitor slab lift during the material injection process in order to avoid overgrouting the slab, and associated slab damage. An experienced contractor and proper inspection are essential to a successful project.

Slab Jacking

In areas of localized settlements or depressions, slab jacking can be used to lift the slab and reestablish a smooth profile. This is accomplished through the pressure injection of a material beneath the slab and carefully monitoring the lift at different insertion holes until the desired profile is obtained. Slightly stiffer cement grouts than those used for slab stabilization are required for slab jacking. During slab jacking, the stringline method can be used effectively to control slab movement. Careful monitoring of slab lift is essential to minimize the development of slab stresses.

Table 4.2. Potential slab stabilization-related problems and associated solutions.

Problem	Typical Cause(s)	Typical Solution(s)
The combination of 1) no evidence of grout in any adjacent hole, joint, or crack after 1 minute, and 2) no registered slab movement on the uplift gauge.	Grout is flowing into a large washout cavity.	Stop the injection process. The cavity will have to be corrected by another repair procedure.
High initial pumping pressure does not drop after 2 to 3 seconds.	Spalled material at bottom of hole may be blocking entrance to void.	Material blockages may sometimes be cleared by pumping a small quantity of water or air into the hole to create a passage that will allow grout to flow into the void. If this activity does not solve the problem, it is possible that the hole was drilled outside of the boundaries of the void.
Testing after <u>one</u> properly performed grouting still indicates a loss of support.	The void was not adequately filled. The first assumption should be that the selected hole pattern did not provide complete access to the void.	Regrout the void using different holes from those that were initially used.
Testing after <u>two</u> properly performed groutings (i.e., after regrouting) still indicates a loss of support.	The void is still not adequately filled. After regrouting, has been attempted, the assumed typical causes are: ▪ The second selected hole pattern still did not provide complete access to the void. ▪ The void may be deeper in the pavement layer system.	One of the following may apply: 1. If it is suspected that the selected hole pattern did not adequately locate the boundaries of the void, the contractor may choose to drill holes at additional locations. 2. If the contractor is confident that the boundaries of the void have been established, the injection holes may have to be extended into the subgrade.
Uplift gauge exceeds the maximum specified slab lift (typically 0.125 in).	Overgrouting.	Overgrouting a void can cause immediate cracking or, as a minimum, increase the potential for long term slab cracking. The solution to this problem is determined by the governing agency specification. If slab damage is immediately observed, the contractor will most likely be responsible for replacing the slab at no cost to the agency.
Grout extrudes into a working transverse joint or crack.	This typically indicates that the void is filled or that the hole has been drilled too close to a joint or crack.	The presence of incompressible material in a joint or crack can increase the probability of spalling or blow-ups. For a joint, the solution is to restore the joint reservoir and joint sealant. For a crack, the solution is to rout or saw and seal the crack.

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CHAPTER 5. PARTIAL-DEPTH REPAIRS

1. LEARNING OUTCOMES

This chapter describes recommended procedures for the partial-depth repair of concrete pavements. Upon completion of this chapter, the participants should be able to accomplish the following:

1. List benefits and appropriateness of using partial-depth repairs.
2. List the advantages and disadvantages of different repair materials.
3. Describe recommended construction procedures.
4. Identify typical construction problems and appropriate remedies.

2. INTRODUCTION

Partial-depth repairs are defined as the removal of small, shallow areas of deteriorated concrete that are then replaced with a suitable repair material. These repairs restore structural integrity and improve ride quality, thereby extending the service life of pavements that have spalled or distressed joints. Partial-depth repairs of spalled joint areas also restore a well-defined uniform joint sealant reservoir prior to joint resealing.

Partial-depth repairs are an alternative to full-depth repairs in areas where slab deterioration is located primarily in the upper one-third of the slab, and the existing load transfer devices (if any) are still functional. When applied at appropriate locations, partial-depth repairs can be more cost-effective than full-depth repairs. The costs of a partial-depth repair are largely dependent upon the size, number, and location of the repair areas, as well as the materials used. Lane closure time and traffic volume also affect production rates and costs.

Several resources are available on partial-depth repairs, including ACPA's *Concrete Pavement Field Reference: Preservation and Repair* manual (ACPA 2006), the reference manual for the NHI training course *PCC Pavement Evaluation and Rehabilitation* (Hoerner et al. 2001), and the SHRP manual on concrete pavement rehabilitation (Yu et al. 1994).

3. PURPOSE AND PROJECT SELECTION

Partial-depth repairs replace deteriorated concrete only, and most repair materials can not accommodate the movements across working joints and cracks, load transfer devices, or reinforcing steel without experiencing high stresses and material damage. As a result, they are appropriate only for certain types of concrete pavement distresses that are confined to the top one-third of the slab. Distresses that have been successfully corrected with partial-depth repairs include:

- Spalling caused by the intrusion of incompressible materials into the joints.
- Spalling caused by poor consolidation or inadequate curing.
- Spalling caused by localized areas of scaling, weak concrete, clay balls, or high steel.
- Surface scaling or deterioration that is limited to the upper one-third of the slab and caused by reinforcing steel that is too close to the surface or by an inadequate air void system.
- Spalling caused by the use of joint inserts.

Concrete pavement distresses that are <u>not</u> candidates for partial-depth repairs include:

- Spalling caused by dowel bar misalignment or lockup.
- Spalling of transverse or longitudinal cracks caused by shrinkage, fatigue, or foundation movement.
- Spalling caused by D-cracking or reactive aggregate.

If several severe spalls are present along a transverse joint, it may be more cost effective to perform a full-depth repair at the joint rather than installing a series of individual partial-depth spall repairs (ACPA 2006).

4. LIMITATIONS AND EFFECTIVENESS

The performance of partial-depth repairs has been excellent on many projects. Over the years, several studies have demonstrated that properly designed and constructed partial-depth repairs can provide satisfactory performance (McGhee 1981; Snyder et al. 1989; Good-Mojab, Patel, and Romine 1993). However, many partial-depth repair projects have exhibited premature failures, and these have often been attributed to improper construction and placement techniques and not to material deficiencies (Wyant 1984; Jiang and McDaniel 1993). In general, when sound construction practices and a durable material are used, partial-depth repairs can last 5 to 15 years, or longer; when poor materials or workmanship are encountered, partial-depth repairs may fail in as little as 2 to 3 years (ACPA 2006).

5. DESIGN AND MATERIALS CONSIDERATIONS

When designing a project with partial-depth repairs, the design engineer needs to determine and mark repair boundaries within the project, and then select a repair material that is appropriate for the project. The first part of this section describes the steps and techniques used to determine and mark individual repair boundaries. The second part of the section focuses on patch repair materials including specific discussions of different available materials commonly used in partial-depth repairs, what to consider when selecting the material for a given project, the types of bonding agents that can be used, and the costs associated with the different material types.

Determining Repair Boundaries

The first step in the design process is to conduct a field survey of the project to determine repair boundaries for the partial-depth repairs. The actual extent of deterioration in a concrete pavement is often greater than what is visible at the surface. In early stages of spall formation, weakened planes may exist in the slab with no signs of deterioration visible at the surface. During the survey, the extent of deterioration should be determined by “sounding” the concrete with a solid steel rod, chains, or a ball peen hammer. Areas yielding a sharp metallic ringing sound are judged to be acceptable, while those emitting a dull or hollow thud sound are delaminated or unsound (ACPA 2006).

All weak and deteriorated concrete must be located and removed if the repair operation is to be effective. The repair boundaries should extend 75 mm (3 in) beyond the detected delaminated or spalled area to ensure removal of all unsound concrete (ACPA 2006). A minimum repair length of 250 mm (10 in) and a minimum repair width of 100 mm (4 in) are recommended (Wilson, Smith, and Romine 1999b). The repair area should also be kept square or rectangular in shape, and in line with the existing joint pattern to avoid irregular shapes that could cause cracks to develop in the repair material (ACPA 2006). If repair areas are closer than 600 mm (24 in) apart, they should be combined to help reduce costs and eliminate numerous small patches (ACPA 2006). All selected patch boundaries should be clearly marked on the pavement by the survey crew.

Repair Material Types

Repair materials for partial-depth repairs are generally classified into three categories: cementitious, polymeric, and bituminous. The specific material selection depends on available curing time, ambient temperature, cost, and the size and depth of the repairs. Because of the multitude of factors that go into the selection process, it is impossible to specify a single repair material for all applications. When the cost of time delays to motorists and the safety hazards to motorists and maintenance crews are considered, many projects, particularly in high traffic volume areas, require that repairs be opened to traffic within a few hours. To meet these challenges, a wide variety of rapid-setting and high-early-strength proprietary materials has been developed (Patel, Mojab, and Romine 1993; Smoak, Husbands, and McDonald 1997 ACI 2006). The remainder of this section introduces the specific material types included within each of these material categories and presents any mix-related concerns associated with each.

Cementitious Materials

Cementitious materials include conventional portland cement-based products, gypsum-based (calcium sulfate) products, magnesium phosphate, and high alumina (calcium aluminate) cements.

Portland Cement Concrete

High-quality portland cement concrete (PCC) is generally accepted as the most appropriate material for the repair of existing concrete pavements. Typical mixes combine Type I, II, or III portland cement with coarse aggregate not larger than one-half the minimum repair thickness (a 9.5 mm [0.375 in] maximum size is often used). The material should be a low-slump mixture of air-entrained concrete having a water-cement ratio not exceeding 0.44. Type I (GU) portland cement concrete can be used when the patch material can be protected from traffic for at lest 24 hours (ACPA 2006). For faster setting materials such as Type III (HE) cements, patches can be opened as soon as the material can withstand loads without plastic deformation (ACPA 2006).

Type I portland cement, with or without admixtures, is more widely used than most other materials because of its relatively low cost, availability, and ease of use. Rich mixtures (up to eight bags of cement, or 446 kg/m^3 [752 lb/yd^3]) gain strength rapidly in warm weather, although the rate of strength gain may be too slow to permit quick opening to traffic in cool weather. Insulating layers can be used to retain the heat of hydration and reduce curing time.

Several proprietary portland cement-based repair materials are available to achieve high-early strength for critical full-depth repairs. One such material, 4x4™ concrete, was developed in response to a California Department of Transportation requirement of having its full-depth repairs achieve a flexural strength of 2.8 MPa (400 lbf/in^2) in 4 hours. The material is easy to place and achieves exceptional early strength, and has been approved for use by a number of highway agencies (BASF 2006).

Gypsum-Based Concrete

Gypsum-based (calcium sulfate) repair materials gain strength rapidly and can be used in any temperature above freezing. However, gypsum concrete may not perform well when exposed to moisture and freezing weather (ACPA 1998). Additionally, the presence of free sulfates in the typical gypsum mixture may promote steel corrosion in reinforced pavements (Good-Mojab, Patel, and Romine 1993).

Magnesium Phosphate Concrete

Magnesium phosphate concretes set very rapidly and produce a high-early-strength, impermeable material that will bond to clean dry surfaces. However, this type of material is extremely sensitive to water, either on the substrate or in the mix (even very small amounts of excess water can reduce strength). Furthermore, magnesium phosphate concrete is very sensitive to aggregate type (for example, some limestones are not acceptable) (Good-Mojab, Patel, and Romine 1993). In hot weather (i.e., above 32 ºC [90 ºF]), many commonly available mixes experience short setting times (e.g., 10 to 15 minutes).

High Alumina Concrete

Calcium aluminate cements gain strength rapidly, have good bonding properties (on a dry surface), and very low shrinkage. However, due to a chemical conversion that occurs in calcium aluminate cement, particularly at high temperatures during curing, strength loss over time is likely to occur; consequently, these materials are not recommended for use as a patching material (ACPA 1998).

Polymer-Based Concretes

Polymer-based concretes are formed by combining polymer resin (molecules of a single family or several similar families linked into molecular chains), aggregate, and an initiator. Aggregate is added to the resin to make the polymer concrete more thermally compatible with the concrete (which would otherwise lead to debonding), to provide a wearing surface, and for economy. The main advantage of polymers is that they set much quicker than most of the cementitious materials. However, they are expensive and can be quite sensitive under certain field conditions. Polymers used for pavement repairs can be classified into four categories: epoxies, methacrylates, polyester-styrenes, and urethanes.

Epoxy Concrete

Epoxy concrete repair materials are impermeable and have excellent adhesive properties. When used, it is important that the epoxy concrete be compatible with the concrete in the pavement. Differences in the coefficients of thermal expansion between the repair material and the concrete can cause repair failures, but the use of larger aggregate increases the volume stability and helps reduce the likelihood of debonding (ACPA 1998). Deep epoxy repairs must frequently be placed in multiple lifts to control heat build-up.

Methyl Methacrylate Concrete

Methyl methacrylate (MMA) concretes and high molecular weight methacrylate (HMWM) concretes have long working times, high compressive strengths, and good adhesion. Furthermore, they can be placed over a wide range of temperatures, from 4 to 54 ^{o}C (40 to 130 ^{o}F) (ACPA 1998). However, many methacrylates are volatile and may pose a health hazard to those exposed to the fumes for prolonged periods (Krauss 1985).

Polyester-Styrene Concrete

Polyester-styrene polymers have many of the same properties as methyl methacrylates, except that they have a much slower rate of strength gain, which limits their usefulness as a rapid repair material. Polyester-styrene polymers generally cost less and are used more widely than methyl methacrylates (Krauss 1985).

Polyurethane Concrete

Polyurethane repair materials generally consist of a two-part polyurethane resin mixed with aggregate (ACPA 1998). Polyurethanes are generally very quick setting (90 seconds), which makes a very quick repair. Some polyurethanes claim to be moisture-tolerant; that is, they can be placed on a wet substrate with no adverse effects. These types of materials have been used for several years with variable results (Krauss 1985).

Other Polymeric Materials

There are a number of other polymeric materials available for partial-depth repairs, most of which exhibit rapid strength gain and a high degree of impermeability. Furthermore, some of these materials exhibit certain elastic properties that allow them to be placed across a joint without the need for an insert to maintain the joint.

Bituminous Materials

Bituminous materials are often used as temporary repair materials on concrete pavements. They have the advantage of being relatively low in cost, widely available, easy to place with small crews, easy to handle, and can be opened to traffic almost immediately. However, because the joint cannot be re-established when using bituminous mixtures, and proper repair techniques are not typically utilized, they are not recommended for permanent repairs. Bituminous patches are often used prior to overlaying, particularly when the existing concrete pavement is too D-cracked or otherwise deteriorated to permit full-depth repairs. Results from the Federal Highway Administration's (FHWA) long-term monitoring of partial-depth repairs showed that bituminous repair materials performed well for a period of 3 to 4 years, but generally experienced rapid failure after a point where the bituminous material had oxidized and become more brittle (Wilson, Smith, and Romine 1999a).

Repair Material Selection Considerations

Selection of the proper material should include an evaluation of the material properties. Currently, the most widely reported property used for selection is the strength of the material at a given time (i.e., when the patch can be opened to traffic). However, other factors also play a role in the short- and long-term performance of the patch. Two of the more critical factors are the shrinkage characteristics and coefficient of thermal expansion of the material. Drying shrinkage of most repair materials is greater than normal concrete, and when the material is restrained can induce a tensile stress as high as 6,900 kPa (1,000 lbf/in^2) (Emmons, Vaysburd, and McDonald 1993). Differential expansion between the repair material and the surrounding concrete can also be detrimental.

Another important property of the repair material is freeze-thaw durability. A study of the properties of repair materials found that the freeze-thaw durability of many materials is unacceptable, especially under severe exposure conditions (Smoak, Husbands, and McDonald 1997).

Materials with rapid strength gain characteristics may be particularly susceptible to durability problems because of the accelerated nature of the material and the reduced curing times. The composition of modern cements is such that they gain higher strengths earlier, but have a lower long-term strength gain; this may affect the long-term durability of the concrete (Van Dam et al. 2005). And, depending on the application, early opening times may be desired, which can significantly reduce the available curing time. The early strength criterion and enhanced durability may be most effectively achieved by using high-quality materials, by reducing the w/c, and by increasing the aggregate volume as long as workability is maintained (Van Dam et al. 2005).

The FHWA/SHRP Manual of Practice, *Materials and Procedures for Rapid Repair of Partial-Depth Spalls in Concrete Pavements*, states that premature partial-depth patch failures can be attributed to a number of material-related causes, including (Wilson, Smith, and Romine 1999b):

- Incompatibilities between the climatic conditions during repair replacement and the materials or procedures used.
- Thermal incompatibility between the repair material and the pavement.
- Extreme climatic conditions during the life of the repairs that are beyond the capabilities of the repair material.
- Inadequate cure time prior to opening repairs to traffic.
- Incompatibility between the joint bond breaker and the joint sealant material.

Many highway agencies maintain a qualified products list that can be consulted to help identify appropriate partial-depth repair materials.

Bonding Agents

PCC materials generally require the placement of a bonding agent to enhance the bond between the repair material and the existing pavement. Sand-cement grouts have proven adequate when used as bonding agents with concrete repair materials, provided the repairs are protected from traffic for 24 to 72 hours. Excellent results have been obtained with 7-sack Type III mixes using a sand-cement grout bonding agent, with a cure period of 72 hours before opening to traffic. The recommended mixture for the sand-cement grout consists of one part sand and one part cement by volume, with sufficient water to produce a mortar with a thick, creamy consistency. Epoxy bonding agents have been used successfully with both PCC and proprietary repair materials to reduce the repair closure time to 6 hours or less. Not all repair materials require a bonding agent to promote adhesion, however. Proprietary mixes will specify what type of bonding agent, if any, should be used.

Material Cost Considerations

Material costs, mechanical properties, workability, and performance vary greatly between the different repair materials. Generally, the more rapid setting the material, the more expensive the product. Other material selection factors to consider include durability and reliability; in many cases, the conventional materials, while they do not gain strength as rapidly, are much more reliable.

6. CONSTRUCTION CONSIDERATIONS

The construction and installation of partial-depth repairs involves the following steps:

1. Repair dimension selection.
2. Concrete removal.
3. Repair area preparation.
4. Joint preparation.
5. Bonding agent application.
6. Patch material placement.
7. Curing.
8. Diamond grinding (optional).
9. Joint resealing.

A simplified overview of this repair process is illustrated in figure 5.1, with a detailed description of these steps provided in the following sections. A number of manuals that describe the construction procedures for partial-depth repairs are available (Patel, Mojab, and Romine 1993; Wilson, Smith, and Romine 1999b; ACPA 1998; ACPA 2004; ACPA 2006).

Step 1: Repair Dimension Selection

The first step in the repair process is to determine the boundaries for the partial-depth repairs. As previously described, this is often accomplished by "sounding" the concrete with a solid steel rod, a heavy chain, or a ball peen hammer to determine unsound areas. The repair boundaries should then be clearly marked, keeping in mind the minimum repair dimension requirements of 250 mm (10 in) long and 100 mm (4 in) wide. Repair boundaries should also be at least 75 mm (3 in) away from the unsound areas. If there is a significant amount of time between the condition survey and the construction process, the repair boundaries should be verified by the construction crew to ensure that the extent of the unsound material has not expanded.

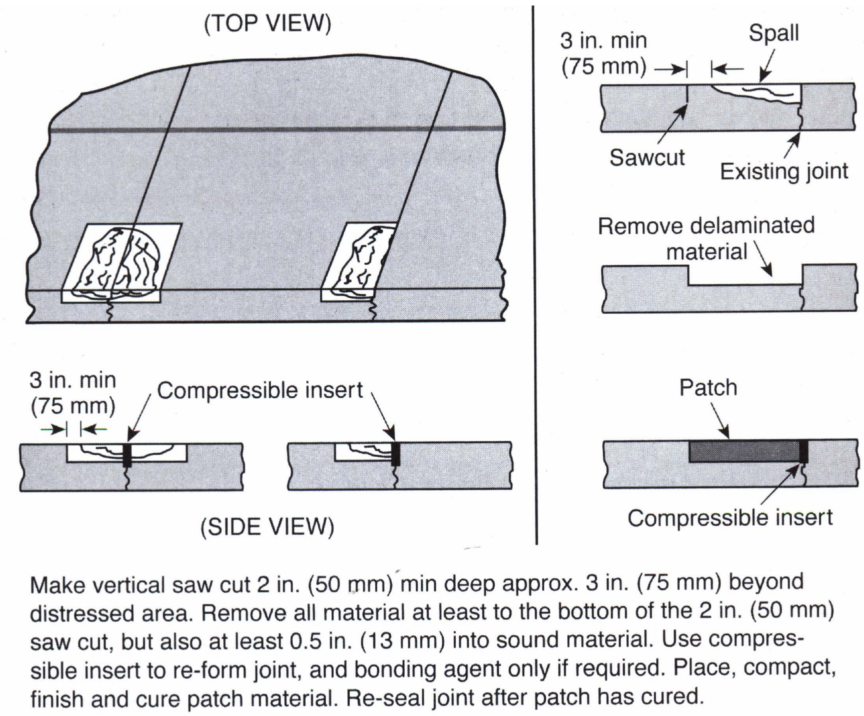

Figure 5.1. Partial-depth repair details (ACPA 2006).

Step 2: Concrete Removal

The second step of the construction process is the removal of the unsound material. During this step, it is important to remember that partial-depth patches should always be limited to the top one-third of the slab. In addition, patches should be at least 50 mm (2 in) deep for the sake of weight and volume stability and should never come in contact with dowel bars. If dowel bars do become exposed during the patching process, a full-depth repair must be used (Wilson, Smith, and Romine 1999b).

The removal of the deteriorated concrete may be accomplished using one of the following four methods:

- Saw-and-patch.
- Chip-and-patch.
- Mill-and-patch.
- Clean-and-patch.

Details of these material removal methods are discussed in the following sections.

Saw-and-Patch Procedure

The most frequently used method employs a diamond-bladed saw to outline the repair boundaries. The saw cut should be 50 mm (2 in) deep (see figure 5.1). The cut boundary should be straight and vertical to provide a vertical face and square corners. Vertical boundaries reduce the spalling associated with thin or feathered concrete along the repair perimeter. For large repairs, removal of the unsound concrete may be facilitated by sawing the pavement marked for removal in a shallow criss-cross or waffle pattern.

After sawing, removal of the unsound concrete is usually accomplished using a light jackhammer with a maximum weight of 7 kg (15 lb); a jackhammer with a maximum weight of 14 kg (30 lb) may be allowed if damage to sound pavement is avoided (Wilson, Smith, and Romine 1999b). Removal begins near the center of the repair area and proceeds toward (but not to) the edges. Care must be taken to avoid fracturing the sound concrete below the repair and undercutting or spalling repair boundaries. Removal near the repair boundaries must be completed with lighter (4.5 to 9 kg [10 to 20 lb]) hammers, particularly in the area of the repair borders. Even hammers of this size fitted with gouge bits can damage sound concrete. Jackhammers for removing unsound concrete should be operated at no greater than a 45 degree angle from the pavement. Carefully operated, small hammers with spade bits have been used successfully to remove unsound concrete without fracturing the underlying sound concrete.

Chip-and-Patch Procedure

The chip-and-patch procedure differs slightly from the saw-and-patch procedure in that the patch boundaries are not sawed. The deteriorated concrete in the center of the patch is removed using a lightweight jackhammer with a maximum weight of 7 kg (15 lb); however, a jackhammer up to 14 kg (30 lb) may be allowed if damage to the sound pavement is avoided (Wilson, Smith, and Romine 1999b). The material near the patch edge is then removed using either the light jackhammer or hand tools. Work should again progress from the inside of the patch toward the edges, and the chisel point should always be directed toward the inside of the patch (Wilson, Smith, and Romine 1999b).

Mill-and-Patch Procedure

A few states have successfully used carbide-tipped milling machines for concrete removal (Zoller, Williams, and Frentress 1989). Standard milling machines with cutting heads of 300 to 450 mm (12 to 18 in) have proven efficient and economical, but they must be affixed with a mechanism that will stop penetration of the milling head at a preset depth. As shown in figure 5.2, the milling operation can proceed either across lanes or parallel to the pavement centerline; milling across lanes is effective for spalling along an entire joint, and produces a rectangular-shaped repair area, whereas milling parallel to the centerline is effective for smaller, individual spalls, and produces a dish-shaped repair area. Milling produces a very rough, irregular surface that promotes a high degree of mechanical interlock between the repair material and the existing slab. Milling may be more suitable for concrete pavements containing softer coarse aggregates.

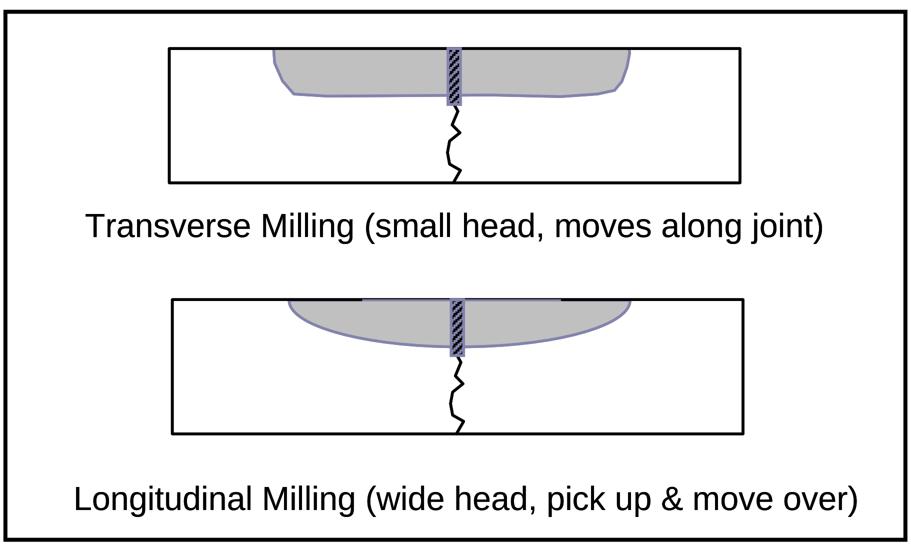

Figure 5.2. Transverse and longitudinal milling options.

Clean-and-Patch Procedure

The clean-and-patch procedure is used to perform emergency repairs under adverse conditions (Wilson, Smith, and Romine 1999b). The procedure consists of removing deteriorated or loose concrete with hand tools or a light jackhammer (only used if the area is large and the cracked concrete is held tightly in place). The loosened material is then swept away with stiff brooms. Such a procedure should only be used if a spall is hazardous to highway users and the climate is so adverse that no other procedure can be used (Wilson, Smith, and Romine 1999b).

Step 3: Repair Area Preparation

Following removal of the concrete, the surface of the repair area must be prepared to provide a clean, irregular surface for the development of a good bond between the repair material and the existing slab. Dry sweeping, sandblasting, and compressed airblasting are normally sufficient for obtaining an adequately clean surface. Sandblasting is a highly recommended step as it is very effective at removing dirt, oil, thin layers of unsound concrete, and laitance. High-pressure water may also be used to remove contaminants, but sandblasting usually produces better results. The compressed air used in the final cleaning must be free of oil, since contamination of the surface will prevent bonding. This can be checked by placing a cloth over the air compressor nozzle and visually inspecting for oil.

With any cleaning method, the prepared surface must be checked prior to placing the new material. Any contamination of the surface will reduce the bond between the new material and the existing concrete. If a finger rubbed along the prepared surface picks up any loose material (e.g., dust, asphalt, slurry), the surface should be cleaned again. If there is a delay between cleaning and repair placement, the surface may also have to be cleaned again.

Step 4: Joint Preparation

The most frequent cause of failure of partial-depth repairs at joints is excessive compressive stresses on the repair material. Partial-depth repairs placed directly against transverse joints and cracks will be crushed by the compressive forces created when the slabs expand and insufficient room is provided for the thermal expansion. Failure may also occur when the repair material is allowed to infiltrate the joint or crack opening below the bottom of the repair, resisting slab movement and thereby preventing the joint or crack from functioning. These damaging stresses may also develop along longitudinal joints or at lane-shoulder joints.

Placing a strip of polystyrene, polyethylene, asphalt-impregnated fiberboard, or other compressible material between the new concrete and the adjoining slab (see figure 5.3) will reduce the risk of such failures. Such an insert is typically referred to as a *bond breaker* (or joint reformer). This insert must be placed so that it prevents intrusion of the repair material into the joint opening. Failure to do so can result in the development of compressive stresses at lower depths that will damage the repair. The insert will also guard against damage due to deflection of the joint under traffic. It is recommended that the compressible insert extend 25 mm (1 in) below and 76 mm (3 in) beyond the repair boundaries.

Prior to its placement, the insert is typically scored at an appropriate depth prior to placement. Once the scored bond breaker has been placed in the clean joint, and the patch has been installed and has cured or set, the top strip (above the scoring line) is removed. The removal of the top strip provides a clean surface and a preformed joint reservoir that is ready for the installation of the joint sealant (Wilson, Smith, and Romine 1999b).

Some partial-depth repairs have been successfully constructed on both sides of a joint without transverse joint forms by sawing the transverse joint to full depth as soon as the repair material has gained sufficient strength to permit sawing. Timing is absolutely critical in this operation, because any closing of the joint before sawing will fracture the repair. To avoid cracking, joints must be formed with compression-absorbing materials in partial-depth repairs placed across joints and cracks.

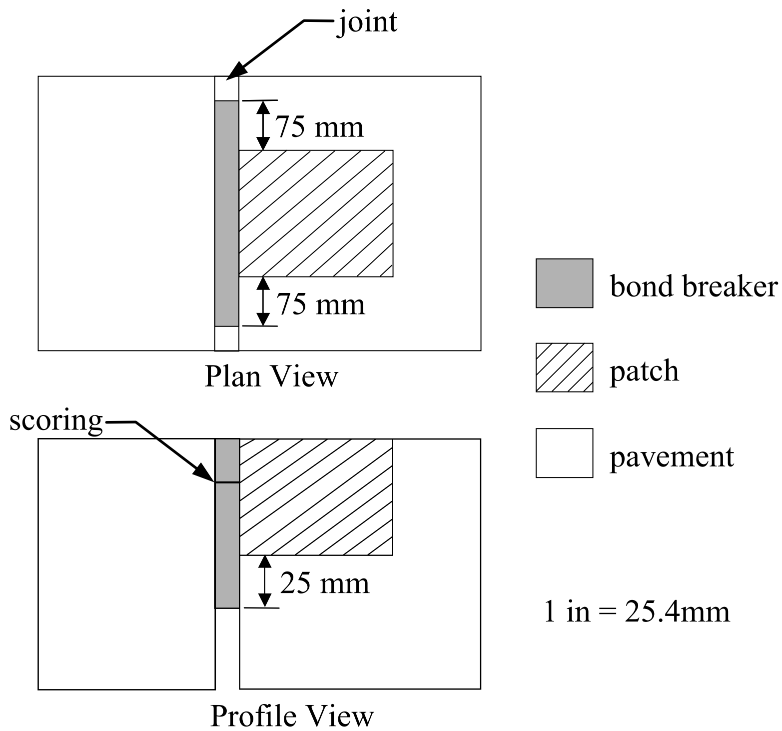

Figure 5.3. Compressible insert placement (Wilson, Smith, and Romine 1999b).

Partial-depth repairs placed at the centerline joint directly in contact with the adjacent lane frequently develop spalling because of curling stresses. This can be prevented by placing a polyethylene strip (or other thin bond-breaker material) along the centerline joint just prior to placement of the repair material. If a repair is to be placed along the outer edge of a lane, it must be formed along the lane/shoulder joint. If the repair material is allowed to flow into the shoulder, it may form a "key" that will restrict longitudinal movement and damage the repair.

Certain proprietary "flexible" or "elastic" repair materials may have sufficient compressibility to accommodate joint movements without the need for a compressible insert. The manufacturers of these products should be consulted for appropriate joint treatment.

Step 5: Bonding Agent Application

Portland Cement Concrete Repair Materials

After the surface of the existing concrete has been cleaned, and just prior to placement of the repair material, the surface may be coated with a bonding agent to ensure complete bonding of the repair material to the surrounding concrete. The type of bonding agent used depends on the bond development requirements for traffic opening times.

The existing surface should be in a saturated surface-dry condition prior to the application of cement grouts. When using epoxies or other manufactured grouts, the manufacturer's directions should be followed closely. Thorough coating of the bottom and sides of the repair area is essential. This may be accomplished by brushing the grout onto the concrete, although spraying may be appropriate for large repair areas. Excess grout or epoxy should not be permitted to collect in pockets. The grout should be placed immediately before the repair material so that the grout does not set before it comes into contact with the repair material. Any bonding material that is allowed to set must be removed by water jet or sandblasting and then fresh material reapplied before continuing.

Rapid-Setting Proprietary Repair Materials

Bonding agents for proprietary repair materials should be those recommended by the manufacturer for the placement conditions. Many proprietary repair materials do not require the use of a bonding agent.

Step 6: Patch Material Placement

Repair Material Mixing

The volume of material required for a partial-depth repair is usually small (0.014 to 0.056 m^3 [0.5 to 2.0 ft^3]). Ready-mix trucks and other large equipment cannot efficiently produce such small quantities, since maximum mixing times for a given temperature would be easily exceeded, resulting in waste of material. Small drum or paddle-type mixers with capacities of up to 0.056 m^3 (2.0 ft^3) are often used. Based on trial batches, repair materials may be weighed and bagged in advance to facilitate the batching process. Continuous feed mixers are also popular.

Careful observation of mixing times and water content for prepackaged rapid setting materials is important because of the quick setting nature of the materials. Mixing beyond the amount of time needed for good blending reduces the already short time available for placing and finishing the material.

Placement and Consolidation of Material

Portland cement concrete and most of the rapid-setting proprietary repair materials should not be placed when the air temperature or pavement temperature is below 4 ºC (40 ºF). Additional precautions, such as the use of warm water, insulating covers, and longer cure times, may be required at temperatures below 13 ºC (55 ºF). Some polymer concretes and bituminous mixes may be installed under adverse conditions of low temperatures and wet substrates with reasonable success; however, even these materials will perform better when installed under more favorable environmental conditions.

Some epoxy concretes may require that the material be placed in lifts not exceeding 50 mm (2 in) due to their high heat of hydration. The time interval between placing additional layers should be such that the temperature of the epoxy concrete does not exceed 60 ºC (140 ºF) at any time during hardening.

Almost all repair materials require consolidation during placement. Failure to properly consolidate concrete results in poor repair durability, spalling, and rapid deterioration. Consolidation provides a more dense mixture by releasing trapped air from the fresh mix, thereby contributing to the overall performance of the patch. Three common methods of achieving consolidation follow:

- Use of internal vibrators with small heads (less than 25 mm [1 in] in diameter).
- Use of vibrating screeds.
- Rodding or tamping and cutting with a trowel or other hand tool.

The internal vibrator and the vibrating screed give the most consistent results. The internal vibrator is often more readily available and is used most often, although very small repairs may require the use of hand tools.

The placement and consolidation procedure begins by slightly over-filling the area with repair material to allow for a reduction in volume during consolidation. The vibrator is held at a slight angle (15 to 30 degrees) from the vertical and is moved through the repair in such a way as to vibrate the entire repair area. The vibrator should not be used to move material from one place to another within the repair as this may result in segregation. Adequate consolidation is achieved when the mix stops settling, air bubbles no longer emerge, and a smooth layer of mortar appears at the surface.

On very small repairs, the mix can be consolidated using hand tools. Cutting with a trowel seems to give better results than rodding or tamping. The tools used should be small enough to easily work in the area being repaired.

Screeding and Finishing

Partial-depth repairs are usually small enough so that a stiff board can be used to screed the repair surface and make it flush with the existing pavement. The materials should be worked toward the perimeter of the repair to establish contact and enhance bonding to the existing slab. At least two passes should be made to ensure a smooth repair surface. Partial-depth repairs typically cover only a small percentage of the pavement surface and have little effect on skid resistance. Nonetheless, the surface of the repair should be textured to match that of the surrounding slab as much as possible.

The patch/slab interface should be sealed with a one-to-one cement grout in order to form a moisture barrier over the interface and to impede delamination of the patch (ACPA 2006). Delamination of the patch can also start to occur if water at the interface freezes in cold weather (ACPA 2006). Sawcut runouts extending beyond the patch perimeter at patch corners also can be filled with grout to help prevent moisture penetration that may negatively affect the bond (ACPA 2006). In lieu of grout, the sawcut runouts can be sealed with the material used to seal the adjacent joint or crack.

Step 7: Curing

Because partial-depth repairs have large surface areas in relation to their volumes, moisture can be lost quickly. Thus, curing is an important component of the construction process and must be effectively conducted in order to prevent the development of shrinkage cracks that may cause the repair to fail prematurely.

Curing Methods

For PCC materials, the most common curing method is to apply a white-pigmented curing compound as soon as water has evaporated from the repair surface. This will reflect radiant heat while allowing the heat of hydration to escape, and will provide protection for several days. Some agencies require that curing compound be applied at 1.5 to 2 times the normal application rate to prevent shrinkage cracks in the repairs. Moist burlap and polyethylene may also be used, and in cold weather the use of insulating blankets or tarps may be required to help retain heat. Curing of proprietary repair materials should be conducted in accordance with the manufacturer’s recommendations.

Opening to Traffic

It is important that the partial-depth repair attain sufficient strength before it is opened to traffic. Generally, compressive strengths in the range of 13.8 to 21.7 MPa (2,000 to 3,000 lbf/in^2) are required by many agencies before the partial-depth repair is opened to traffic.

Step 8: Optional Diamond Grinding

Rehabilitation techniques such as partial-depth repair may result in increased roughness if not finished properly. This is typically due to differences in elevation between the repair areas and the existing pavement. It is often desirable to blend partial-depth repairs into a concrete pavement with diamond grinding, which leaves a smooth surface that matches the surrounding pavement.

Step 9: Joint Resealing

The final step in the partial-depth repair procedure is the restoration of joints. This is accomplished by resawing the joint to a new shape factor, sandblasting and airblasting both faces of the joint, inserting a closed cell backer rod, and applying the sealer. More detailed information on joint resealing can be found in Chapter 10 (*Joint Resealing*).

7. QUALITY CONTROL

The combination of proper design procedures and sufficient construction quality control (QC) is extremely important to achieving well performing partial-depth repairs. On many projects where QC inspections have been known to be less stringent, performance has typically been found to be unsatisfactory. Some of the common causes of failure include inappropriate use, lack of bond, compression failure of the patch (due to failure to re-establish the joint), variability in the effectiveness of repair material, improper use of repair materials, insufficient consolidation, and incompatibility in thermal expansion between the repair material and the original slab.

This section summarizes key portions of a recently developed checklist that has been compiled to facilitate the successful design and construction of good performing partial-depth repairs (FHWA 2005). Although these procedures do not necessarily ensure the long-term performance of a specific repair, the checklist topics are intended to remind both the agency and contractor personnel of those specific design and construction topics that have the potential of influencing the performance of the repair. These checklist items are divided into general categories of preliminary responsibilities, equipment inspections, weather requirements, traffic control, and project inspection responsibilities.

Preliminary Responsibilities

As a first step of the QC process, agency and contractor personnel should collectively conduct a review of the project documentation, project scope and intended construction procedures, and material usage and associates specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, inspectors, and construction personnel. Specific checklist items for this review are summarized below.

Document Review

As a first step, review the following project-related documents:

- Bid/project specifications and design.
- Applicable special provisions.
- Agency application requirements.
- Traffic control plan.
- Manufacturer’s specific installation instructions for chosen patch material(s).
- Manufacturer’s material safety data sheets (MSDS).

Project Review

In an attempt to maximize the efficiency of the field construction process, review the following project scope-related items:

- Verify that pavement conditions have not significantly changed since the project was designed and that a partial-depth repair is still appropriate for the pavement.
- Verify that the estimated number of partial-depth repairs agrees with the number specified in the contract.
- Agree on quantities to be placed, but allow flexibility if additional deterioration is found below the surface.
- Some partial-depth repairs may become full-depth repairs if deterioration extends below the top one-third of the slab. Make sure that the criteria for identifying this change are understood.

Materials Checks

A number of material-related checks are recommended prior to the start of a partial-depth repair project. Specifically, agency and contractor personnel should collectively verify that:

- The selected patch material is of the correct type and meets specifications.
- The patch material is obtained from an approved source or is listed on the agency Qualified Products List as required by the contract documents.
- The patch material has been sampled and tested prior to installation as required by the contract documents.
- Additional or extender aggregates have been properly produced and meet requirements of contract documents.
- Material packaging is not damaged so as to prevent proper use (for example, packages are not leaking, torn, or pierced).
- Bonding agent (if required) meets specifications.
- Curing compound (if required) meets specifications.
- Joint/crack re-former material (compressible insert) meets specifications (typically polystyrene foam board, 12 mm [0.5 in] thick).
- Joint sealant material meets specification requirements.
- Sufficient quantities of materials are on hand for completion of the project.

Equipment Inspections

A second step in the QC process involves the inspection of all equipment that will be utilized in the construction of partial-depth repairs. Ensuring that construction equipment is in good working order will help avoid construction-related problems during the construction process. The following items should be checked or verified as part of the equipment inspection process prior to the start of a partial-depth repair project.

Concrete Removal Equipment

- Verify that concrete saws are of sufficient weight and horsepower to adequately cut the existing concrete pavement to the depth required along the patch boundaries as required by the contract documents.
- Verify that the concrete saws and blades are in good working order.
- Verify that pavement milling machines are power operated, self-propelled, cold milling machines capable of removing concrete as required by the contract documents.
- Verify that milling machines used for concrete removal are equipped with a device that allows them to stop at pre-set depths to prevent removal of more than the top third of the slab and to prevent damage to embedded steel.
- Verify that the maximum rated weight of removal jackhammers is 14 kg (30 lbs).

Patch Area Cleaning Equipment

- Verify that the sandblasting unit is adjusted for correct sand rate and that it is equipped with and using properly functioning oil/moisture traps.
- Verify that air compressors have sufficient pressure and volume capabilities to clean patch area adequately in accordance with contract specifications.

- Verify that air compressors are equipped with and using properly functioning oil and moisture filters/traps. This can be accomplished by passing the air stream over a board, and then examining for contaminants.
- Verify that the volume and pressure of waterblasting equipment (if used) meets the specifications.

Mixing and Testing Equipment

- Verify that auger flights and paddles within auger-type mixing equipment are kept free of material buildup that can result in inefficient mixing operations.
- Ensure that volumetric mixing equipment such as mobile mixers are kept in good condition and are calibrated on a regular basis to properly proportion mixes.
- Verify that the concrete testing technician meets the requirements of the contract documents for training/certification.
- Ensure that material test equipment required by the specifications is all available on-site and in proper working condition (equipment typically includes slump cone, pressure-type air meter, cylinder molds and lids, rod, mallet, ruler, and 3-m [10-ft] straightedge).

Placing and Finishing Equipment

- Verify that a sufficient number of concrete vibrators (25 mm [1 in] diameter or less) is available on-site and in proper working condition.
- Verify that all floats and screeds are straight, free of defects, and capable of producing the desired finish.

Other Equipment

- Ensure that a steel chain, rod, or hammer is available to check for unsound concrete around the patch area.
- Verify that grout-application brushes (if necessary) are available.

Weather Limitations

Immediately prior to the start of the construction project, the following weather-related concerns should be checked:

- Review manufacturer installation instructions for requirements specific to the patch material being used.
- Ensure that air and surface temperature meets manufacturer and contract requirements (typically 4 °C [40 °F] and above) for concrete placement.
- Ensure that patching does not proceed if rain is imminent.

Traffic Control

The developed traffic control plan should be reviewed by field personnel prior to construction. Specifically, the following pre- and post-construction traffic-related items should be verified:

- Verify that the signs and devices used match the traffic control plan presented in the contract documents.
- Verify that the set-up complies with the Federal or local agency *Manual on Uniform Traffic Control Devices* (MUTCD) or local agency procedures.
- Verify that traffic control personnel are trained/qualified according to contract documents and agency requirements.

- Verify that unsafe conditions, if any, are reported to a supervisor.
- Ensure that traffic is not opened to the repaired pavement until the patch material meets strength requirements presented in the contract documents.
- Verify that signs are removed or covered when they are no longer needed.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors can greatly increase the chances of obtaining well performing partial-depth patches. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Patch Removal and Cleaning

- Ensure that the area surrounding the patch is checked for delamination and unsound concrete using steel chain, rod, or hammer.
- Ensure that the boundaries of unsound concrete area(s) are marked at least 75 mm (3 in) beyond the area of deterioration.
- Verify that concrete is removed by either (1) sawcutting the boundaries and jackhammering interior concrete; or (2) using a cold milling machine.
- Verify that concrete removal extends at least 50 mm (2 in) deep and does not extend below one-third of the slab depth, and that load transfer devices are not exposed.
- Verify that, after concrete removal, the patch area is prepared by sandblasting or waterblasting.
- Verify that the patch area is cleaned by air blasting. A second air blasting may be required immediately before placement of patch material if patches are left exposed for a period of time longer than that specified in the contract documents.

Patch Preparation

- Ensure that the patch is effectively sandblasted (or waterblasted) to remove any dirt, debris, or laitance.
- Ensure that compressible joint inserts (joint/crack re-formers) are inserted into existing cracks/joints in accordance with contract documents. Joint inserts are typically required to extend both below and outside patch area by 12 mm (0.5 in).
- When a patch abuts a bituminous shoulder, ensure that a wooden form is used to prevent patch material from entering the shoulder joint.
- Ensure that the bonding agent (epoxy- or cement-based) is placed on the clean, prepared surface of existing concrete immediately prior to the placement of patch material as required by the contract documents. If the bonding agent shows any sign of drying before the patch material is placed, it must be removed by sandblasting, cleaned with compressed air, and re-applied.
- Verify that cement-based bonding agents are applied using wire brush, and epoxy bonding agents are applied using soft brush.

Placing, Finishing, and Curing Patch Material

- Verify that quantities of patch material being mixed are relatively small to prevent material from setting prematurely.
- Verify that the fresh concrete is properly consolidated using several vertical penetrations of the surface with a hand-held vibrator.

- Verify that the surface of the concrete patch is level with the adjacent slab using a straightedge in accordance with contract documents. The material should be worked from the center of the patch outward toward the boundary to prevent pulling material away from the patch boundaries.
- Verify that the surface of the fresh patch material is finished and textured to match the adjacent surface.
- Verify that the perimeter of the patch and sawcut runouts (if saws are used) are sealed using grout material. Alternatively, sawcut runouts can be sealed using joint sealant material.
- Verify that adequate curing compound is applied to the surface of the finished and textured, fresh patch material in accordance with contract documents.
- Ensure that insulation blankets are used when ambient temperatures are expected to fall below 4 °C (40 °F). Maintain blanket cover until concrete attains the strength required in the contract documents.

Resealing Joints and Cracks

- Verify that the compressible inserts are sawed out to the dimensions specified in the contract documents when the patch material has attained sufficient strength to support concrete saws.
- Verify that joints are cleaned and resealed according to contract documents.

Clean Up Responsibilities

- Verify that all concrete pieces and loose debris are removed from the pavement surface and disposed of in accordance with contract documents.
- Verify that mixing, placement, and finishing equipment is properly cleaned for the next use.

8. TROUBLESHOOTING

As mentioned previously, poor performing partial-depth repairs are typically attributed to inappropriate use, improper design, or improper construction and placement techniques. While paying close attention to the checklist items in the previous section attempts to minimize any design or construction-related problems, construction problems do sometimes develop in the field. Some of the more typical problems that are encountered either during or after construction are summarized in table 5.1. Typical causes and recommended solutions accompany each of the identified potential problems.

9. SUMMARY

Partial-depth repairs are an excellent tool for restoring rideability and the overall integrity of a concrete pavement. A broad range of products is available for these types of repairs, and the selection of the proper material is dependent upon the specific project requirements. Each material will call for different handling and mixing steps. However, all of the products require the same surface preparation steps. Taking the time to properly prepare the repair area, following the manufacturers' recommendations when placing the materials, and paying attention to weather concerns during placement and curing, will all contribute to the long-term performance of the partial-depth repair.

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Table 5.1. Potential partial-depth repair-related construction problems and associated solutions (FHWA 2005; ACPA 2006).

Problem	Typical Cause(s)	Typical Solution(s)
Deterioration found to extend beyond the original repair boundaries.	This is an unforeseen problem as the true amount of deterioration is not actually known until the concrete is removed.	The first solution is to extend the limits of the repair area into the surrounding sound concrete. However, if the deterioration is found to extends significantly deeper than expected (i.e., below 1/3 the depth), a full-depth repair should be placed instead of the partial-depth repair.
Dowel bar exposed during concrete removal.	Concrete deterioration extends deeper than originally believed or improper concrete removal techniques.	A full-depth repair should be used instead of the scheduled partial-depth repair.
Reinforcing steel exposed during concrete removal.	If the steel is located in the upper third of the slab, exposing the steel is most likely unavoidable. If steel is exposed below the upper third of the slab, this indicates that either the concrete deterioration extends deeper than originally believed or improper concrete removal techniques are being used.	If the steel is in the upper third of slab, the steel should be removed to the edges and the placement of the patch placement should continue as planned. However, if the exposed steel is below the upper third of the slab, a full-depth repair should be used instead of the scheduled partial-depth repair.
Patch material flows into joint or crack.	When the patch material flows into the joint or crack, it is most commonly the result of one of the following: ▪ Joint insert not extending far enough into adjacent joint/crack and below patch. ▪ Incorrectly selected insert size for the joint/crack width.	When this problem is observed, there are two solutions: either remove and replace the patch, or mark the joint for sawing as soon as it can support a saw without raveling the mix. If patch material is allowed to infiltrate a crack it should be removed and replaced.
Patch cracking or debonding of patch material.	An in-place partial-depth repair that fails prematurely by cracking or by debonding from the prepared area is typically attributed to one of the following causes: ▪ Joint insert not used or used improperly. ▪ Incorrect joint insert size for joint/crack width or insert not installed correctly. ▪ Patch area not cleaned immediately prior to grouting/concrete placement. ▪ Grout material dried out before concrete placement. ▪ Curing compound not applied adequately. ▪ Patch material susceptible to shrinkage. ▪ Patch placed during adverse environmental conditions.	If the patch fails prematurely due to one of these causes, the only practical solution is to replace the distressed patch. However, it is important to try and determine the cause of the premature failure in order to avoid repeating the same mistakes.

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CHAPTER 6. FULL-DEPTH REPAIRS

1. LEARNING OUTCOMES

This chapter describes procedures for cast-in-place full-depth repair (FDR) of existing concrete pavements. The techniques for both jointed plain or jointed reinforced concrete pavements (JPCP and JRCP) and continuously reinforced concrete pavements (CRCP) are discussed. Upon successful completion of this chapter, the participant will be able to accomplish the following:

1. List benefits of full-depth repairs.
2. Describe primary design considerations in terms of dimensions, load transfer, and materials.
3. Describe recommended construction procedures.
4. Identify typical construction problems and remedies.

2. INTRODUCTION

Concrete pavements exhibiting various types of structural distresses may be candidates for full-depth repairs. When appropriately used, full-depth repairs are an effective means of restoring the rideability and structural integrity of deteriorated concrete pavements and, therefore, extending their service life. Typical distresses that can be addressed using full-depth repairs include transverse cracking, corner breaks, longitudinal cracking, deteriorated joints, blowups, and punchouts. Full-depth repairs are also often used to prepare distressed concrete pavements for a structural overlay.

Long-lasting full-depth repairs are dependent upon many items, including appropriate project selection, effective load transfer design, and effective construction procedures. This chapter focuses on proper techniques that can be used to design and construct well-performing concrete full-depth repairs on both jointed concrete pavements (JCP) and CRCP.

3. PURPOSE AND PROJECT SELECTION

Full-depth repairs are cast-in-place concrete repairs that extend through the full thickness of the existing concrete slab. As previously described, full-depth repairs are used to restore the rideability of the pavement, to prevent further deterioration of distressed areas, or to prepare the pavement for an overlay. Because full-depth repair involves complete removal and replacement of deteriorated areas, this technique can be used to address a wide variety of concrete pavement distresses, as described below.

Jointed Concrete Pavements

Table 6.1 provides a summary of the JCP distresses and severity levels that can be successfully remedied using full-depth repairs. In determining the need for full-depth repairs, consideration must be given to the extent of distress within a project. Good candidates for the application of full-depth repairs are concrete pavements in which deterioration is limited to the joints or cracks, if the deterioration is not widespread over the entire project length. Concrete pavements exhibiting severe structural distresses throughout the entire length of the project are more suited for a structural overlay or reconstruction. In evaluating the appropriate rehabilitation strategy, consideration should be given to the deterioration that may have taken place since the distress survey, especially if a significant amount of time has passed (e.g., 1 year or more).

Full-depth repairs typically represent a large cost item in any pavement project. Because of the high cost of full-depth repairs, the lack of adequate funds, and increasing repair quantities, some agencies may not repair all of the distressed areas that should be addressed. This results in either continued deterioration of the distressed area, or if an overlay is placed, premature failure of the overlay.

Table 6.1. JCP distresses addressed by full-depth repairs (Hoerner et al. 2001).

Distress Type	Severity Levels That Require Full-Depth Repair
Transverse Cracking	Medium, High
Longitudinal Cracking	Medium, High
Corner Break	Low, Medium, High
Spalling of Joints	Medium[1], High
Blowup	Low, Medium, High
D-Cracking (at joints or cracks)[2]	Medium[1], High
Reactive Aggregate Spalling[2]	Medium[1], High
Deterioration Adjacent to Existing Repair	Medium[1], High
Deterioration of Existing Repairs	Medium[1], High

[1] Partial-depth repairs can be used if the deterioration is limited to the upper one-third of the pavement slab.

[2] If the pavement has a severe material problem (such as D-cracking or reactive aggregate), full-depth repairs may only provide temporary relief from roughness caused by spalling. Continued deterioration of the original pavement is likely to result in redevelopment of spalling and roughness.

NOTE: Highways with low traffic volumes may not require repair at the recommended severity level.

CRCP

Table 6.2 provides a summary of the CRCP distresses and severity levels that can be successfully remedied using full-depth repairs. Punchouts are the most common structural distress on CRCP that are addressed with full-depth repairs.

Table 6.2. Candidate CRCP distresses addressed by full-depth repairs (Hoerner et al. 2001).

Distress Type	Severity Levels That Require Full-Depth Repair
Punchout	Low, Medium, High
Deteriorated Transverse Cracks [1]	Medium, High
Longitudinal Cracking	Medium, High
Blowup	Low, Medium, High
Construction Joint Distress	Medium, High
Localized Distress	Medium[2], High
D-Cracking (at cracks)[3]	High
Deterioration Adjacent to Existing Repair	Medium[2], High
Deterioration of Existing Repair	Medium[2], High

[1] Typically associated with ruptured steel.

[2] Partial-depth repairs can be used if the deterioration is limited to the upper one-third of the pavement slab.

[3] If the pavement has a severe material problem (such as D-cracking or reactive aggregate), full-depth repairs may only provide temporary relief from roughness caused by spalling. Continued deterioration of the original pavement is likely to result in redevelopment of spalling and roughness.

NOTE: Highways with low traffic volumes may not require repair at the recommended severity level.

4. LIMITATIONS AND EFFECTIVENESS

Although full-depth repairs can be designed and constructed to provide good long-term performance, the performance of full-depth repairs is very much dependent on their appropriate application and the use of effective design and construction practices. Although inconsistent performance of full-depth repairs has been documented over the years (Darter, Barenberg, and Yrjanson 1985; Snyder et al. 1989), most of the performance problems can be traced back to inadequate design (particularly poor load transfer design), poor construction quality, or the placement of these repairs on pavements that are too far deteriorated. For example, a study in Pennsylvania on the performance of various pavement restoration activities revealed that the life of full-depth repairs was about 5 years (Stoffels, Kilareski, and Cady 1993). However, the researchers acknowledge that many of these repairs were placed on pavements that had deteriorated beyond the point at which full-depth repairs are expected to provide long-lasting performance, and noted that pavement sections with less than 5 percent patching demonstrated good performance (Stoffels, Kilareski, and Cady 1993). Furthermore, the poor performance of the repairs was traced to "socketing" of the dowel bars placed in the repair, a condition in which oval-shaped gaps develop around the dowel bars.

If properly designed and constructed, full-depth repairs can restore the pavement to "like new" condition in a near-permanent manner, but project selection is very important to obtain the desired performance. Important points for consideration in selecting this repair technique include the following:

- If the existing pavement is structurally deficient, or is nearing the end of its fatigue life, a structural overlay is needed to prevent continued cracking of the original pavement.
- If the original pavement has a severe materials-related problem (e.g., D-cracking or reactive aggregate), full-depth repairs may only provide temporary relief from roughness caused by spalling. Continued deterioration of the original pavement is likely to result in redevelopment of spalling and roughness.
- Additional joints introduced by full-depth repairs add to the pavement roughness. Diamond grinding should be considered after the repairs are made to produce a smooth-riding surface.
- Non-deteriorated cracks in JPCP may be repaired by retrofitting dowels or tie bars in lieu of full-depth repair.

Overall, the effectiveness of full-depth repairs depends strongly on the installation of the repairs at the appropriate time in the life of the pavement and on the proper design and installation of the load transfer system.

5. MATERIALS AND DESIGN CONSIDERATIONS

This section presents the materials and design considerations for full-depth repairs of JCP, as well as special considerations for full-depth repairs of CRCP. For each pavement type, guidance is provided on selecting repair locations and boundaries, selecting repair materials, restoring load transfer, and determining when to open the pavement to traffic.

Selecting Repair Locations and Boundaries

Jointed Concrete Pavements

The first step in the installation of full-depth repairs is the selection of the repair boundaries. Distressed areas must be identified and marked, with special consideration given to those areas of extensive distress that might require complete slab replacement. This is accomplished by a trained crew performing a condition survey for the entire project in all lanes. A follow-up survey should be performed immediately prior to construction to verify the quantity of repair work needed, because additional pavement deterioration is likely to have occurred since the previous pavement inspection.

JRCP often exhibit deteriorated joints, as well as mid-panel cracks that deteriorate under repeated heavy traffic loadings. Additionally, some intermediate cracks deteriorate because of “frozen” or locked doweled joints, which force the cracks to absorb the movements the doweled joints are designed to accommodate. These cracks soon lose their aggregate interlock under repeated heavy traffic loadings. Some projects will actually have joints with very little deterioration but one or more intermediate cracks in each slab opened wide and essentially acting as joints.

On JPCP, all structural cracks are candidates for full-depth repair. The rate at which the cracks deteriorate depends on traffic, climate, and pavement structure. The types of JCP distresses that can be successfully addressed through full-depth repairs are presented in table 6.1. Each agency should examine these recommendations and modify them as needed to develop a table that more closely reflects local conditions.

Sizing the Repair

After the repair locations are identified, the boundaries of each repair must be determined. This is typically performed by the project engineer at, or just before, construction. Repair dimensions play a major role in repair performance: the agency is interested in limiting the dimensions to control repair costs. However, it is equally important that the repair boundaries extend to include all of the significant deterioration in the slab and underlying layers (including the subgrade). The extent of deterioration beneath the slab surface may be identified through coring and deflection studies. Where the pavement has a materials-related distress, the deterioration at the bottom may extend as much as 1 m (3.3 ft) or more beyond the visible boundaries of deterioration at the surface (see figure 6.1).

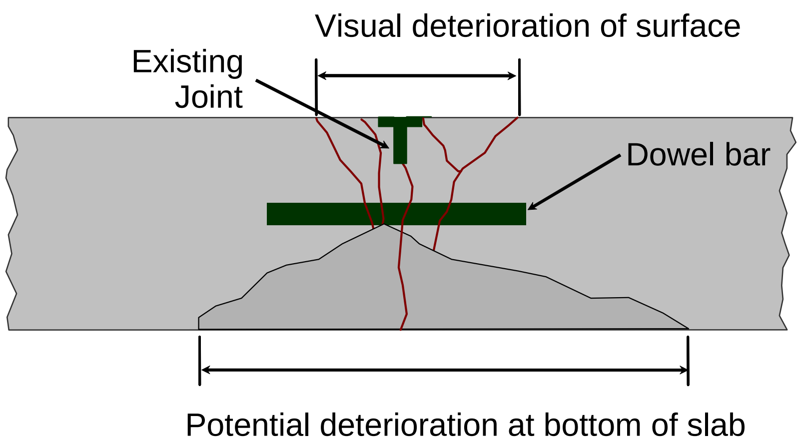


Figure 6.1. Illustration of potential extent of deterioration beneath a joint.

To minimize the potential for premature repair failure, the following minimum repair dimensions are recommended:

- Doweled Repair. When load transfer is provided, a minimum repair length of 1.8 m (6 ft) (in the longitudinal direction) is effective in minimizing rocking, pumping, and breakup of the slab (Correa and Wong 2003). Although partial-width slab replacements have been used successfully by a few agencies, full-width replacements are preferred because boundaries are well defined and the patch is more stable.
- Nondoweled Repair. The minimum recommended repair lengths are 1.8 to 3.0 m (6 to 10 ft) for pavements exposed to low truck traffic volumes (ACPA 1995).

Engineering judgment is required in selecting repair boundaries, particularly in areas exhibiting several types of distresses. The recommended minimum guidelines for determining repair boundaries for JCP are the following (Correa and Wong 2003; ACPA 2006):

- Saw full-depth a minimum of 0.6 m (2 ft) from any joints.
- Use straight-line sawcuts, forming rectangles in-line with the jointing pattern.
- Extend the patch boundary to the joint if the boundary is within 1.8 m (6 ft) of an existing joint.
- Connect patches to make one large patch if the patches are 2.4 to 3.6 m (8 to 12 ft) from each other in a single lane. This alternative requires two sawcuts instead of four, as well as one removal instead of two. Table 6.3 provides guidelines for determining the maximum distance between full-depth repairs to maintain cost-effectiveness.
- Make two additional cuts if the patch is a utility cut. The cuts should be 150 to 300 mm (6 to 12 in) beyond the limits of the excavation and made after the trench has been backfilled.

Table 6.3. Maximum distance between full-depth repairs to maintain cost-effectiveness (Correa and Wong 2003: ACPA 2006).

Pavement Thickness, mm (in)	**Patch or Lane Width, m (ft)**	
	3.3 (11)	**3.6 (12)**
150 (6)	4.9 (16)	4.6 (15)
175 (7)	4.3 (14)	4.0 (13)
200 (8)	3.6 (12)	3.3 (11)
225 (9)	3.3 (11)	3.0 (10)
250 (10)	3.0 (10)	2.7 (9)
275 (11)	2.7 (9)	2.4 (8)
300 (12)	2.4 (8)	2.4 (8)

Note: if patches are closer than the distances listed, they should be combined into one repair.

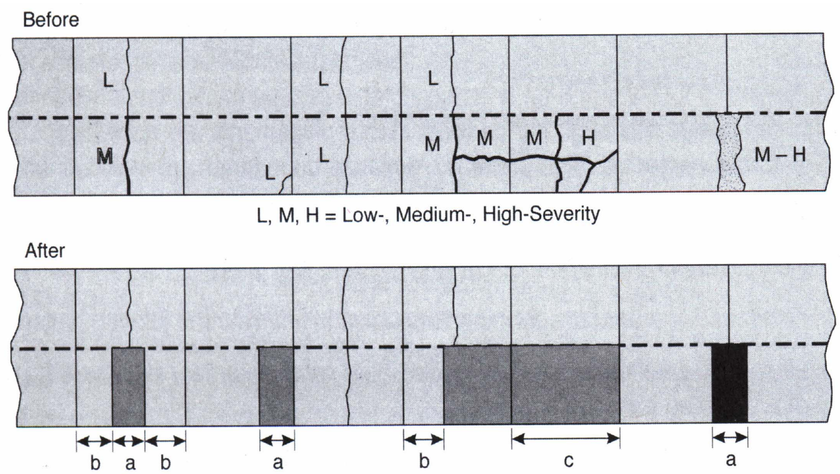
Figure 6.2 illustrates an example of how to select repair boundaries when multiple distresses of different severities are present. Note that not all distresses require a full-depth repair.

Large Area Removal and Replacement

In some situations, the existing distress is so extensive that the repair of every deteriorated area within a short distance (e.g., 3 to 9 m [10 to 30 ft]) is either very expensive or impractical. Repair costs can be reduced by simply removing and replacing larger areas of concrete. On JCP, this is called "slab replacement." A separate pay item should be set up for this type of repair because its unit cost can be significantly less than that of several smaller repairs.

Multiple-Lane Repairs

On multiple-lane highways, deterioration may occur only in one lane or across two or more lanes. If distress exists in only one lane, it is not necessary to repair the other lanes. When two or more adjacent lanes contain distress, generally one lane is repaired at a time so that traffic flow can be maintained.



NOTES
a – Minimum length is 1. 8 m (6 ft).
b – Check distance between patches and nearby joints
c – Replace the entire slab it there are multiple intersecting cracks

Figure 6.2. Example selection of full-depth repair boundaries (ACPA 2006).

Matching joints in adjacent lanes is generally not necessary, as long as a fiberboard has been placed along the longitudinal joint to separate the lanes. However, if the distressed areas in both lanes are similar and both lanes are to be repaired at the same time, it may be desirable to align repair boundaries to avoid small offsets and to maintain continuity. If blowups occur during the repair of one lane, it may be necessary to cut pressure relief joints at intervals of 180 to 370 m (600 to 1,200 ft) or to delay repair work until cooler weather prevails (Snyder, Smith, and Darter 1989).

CRCP

The types of CRCP distresses that can be addressed through full-depth repairs are identified in table 6.2. Again, these recommendations should be evaluated by each agency and modified for use under their local conditions.

Sizing the Repair

As illustrated in figure 6.3, subsurface deterioration accompanying structural distresses of CRCP can be quite extensive. Subbase deterioration is particularly prevalent near punchouts and wherever there is settlement or faulting along the longitudinal lane joint. The results of coring and deflection studies provide information on the extent of deterioration beneath the slab surface, and such studies are recommended on projects of any magnitude.

Guidelines for the determination of repair boundaries for CRCP are given below (TRB 1979; Darter, Barnett, and Morrill 1982; Gagnon, Zollinger, and Tayabji 1998):

- A minimum repair length of 1.8 m (6 ft) is recommended if the reinforcing steel is tied; 1.2 m (4 ft) if the steel is mechanically connected or welded.
- The repair boundaries should not be closer than 460 mm (18 in) to adjacent non-deteriorated cracks; however, if cracks are very closely spaced, it may be necessary to place the repair as close as 150 mm (6 in) to an existing tight transverse crack.
- Full-lane-width repairs are generally recommended, although a half-lane width (1.8 m [6 ft]) may be used when all distress is contained within that width.

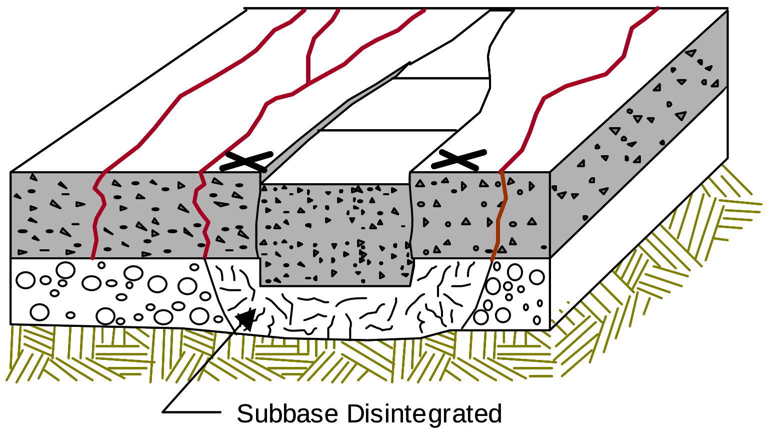


Figure 6.3. Potential deterioration of subbase near CRCP structural distress (punchout).

These criteria are given to provide adequate lap length and cleanout, and to minimize repair rocking, pumping, and breakup. Figure 6.4 illustrates these construction recommendations.

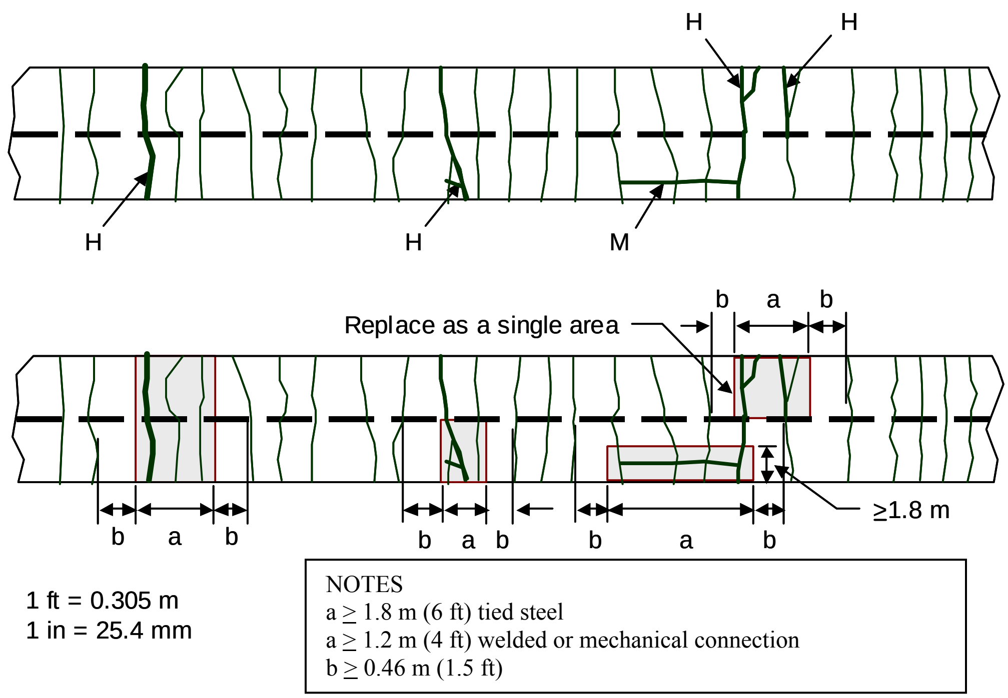


Figure 6.4. Example of repair recommendations for a CRCP.

Multiple-Lane Repair Considerations

If a distress such as a wide crack with ruptured steel occurs across all lanes, special considerations are necessary because of the potential for:

- Blowups in the adjacent lane.
- Crushing of the new repair during the first few hours of curing by the expanding CRCP.
- Cracking of the repair during the first night as the existing CRCP contracts.

In order to minimize these problems, it may be necessary to place the concrete in the afternoon or evening to avoid being crushed by the expanding CRCP slab. In addition, it is recommended that the lane with the lowest truck traffic be repaired first.

Selecting Repair Materials

The repair material should be selected based on the available lane closure time. The current state of the art in concrete pavement repair is such that virtually any opening time requirement can be met (from less than 1 hour to 24 hours or more), using either conventional portland cement concrete (PCC) or a proprietary material. However, faster-setting mixes generally have higher costs and special handling requirements. A good rule of thumb in selecting the material for concrete pavement repair projects is to use the least exotic (i.e., most conventional) material that will meet the opening requirements.

The most widely used repair materials for full-depth repairs are conventional PCC mixtures. Typical full-depth repair operations utilize concrete mixes containing five to seven bags of cement (Type I, and sometimes Type III) per m^3 (360 to 460 kg/m^3 [6.5 to 8.5 bags/yd^3]), and an accelerator to permit opening in 1 to 3 days (ACPA 1994). Type III cement, high cement factors (385 to 530 kg/m^3 [7 to 9.5 bags/yd^3]), and chemical accelerators are required for opening in 4 to 6 hours (Whiting et al. 1994).

Many specialty cements and proprietary materials have also been used successfully in full-depth repairs. Many of the proprietary patching materials are capable of developing the strength required for opening in 1 hour or less, but are very expensive. Because of their high cost, these materials are often considered for use in partial-depth repairs, where the required material quantities are comparatively small and the work must often be completed with little or no disruption to the traffic flow.

Local climatic conditions are an important factor in selecting a repair material. During hot, sunny, summer days, solar radiation can significantly raise the temperature at the slab surface, adding to the temperature gradient. When the ambient temperature is in excess of 32 ºC (90 ºF), it may be very difficult to place some of the very fast-setting materials because they harden so quickly. Although a set retarder can be used with some of these materials to provide longer working times, a better solution may be to use a slower-setting mix.

For high early strength concrete (often referred to as early-opening-to-traffic [EOT] concrete), the early strength gain is typically achieved by reducing the water to cement ratio (w/c), increasing the cement content, and by adding a chemical accelerator. High range water reducers are also typically added to reduce the amount of water required without a loss in workability. Because these early strength mixes typically contain higher cement contents and multiple admixtures, it is not uncommon for them to experience increased shrinkage, altered microstructure, and unexpected interactions (Van Dam et al. 2005). Guidelines are available that summarize the state of practice for EOT concrete repairs, including the identification of material properties impacting EOT concrete performance, the selection of materials and mixture design properties for EOT concrete, and the identification of performance-related tests of fresh and hardened concrete (Van Dam et al. 2005).

Table 6.4 provides examples of high-early-strength mix designs and approximate opening times (ACPA 1994; Jones 1988; Whiting et al. 1994). Laboratory testing of proposed repair materials (using the aggregates that will be used in the project mix) must be conducted to ensure that the opening requirements are met. To ensure adequate durability of hardened concrete, the concrete mix should have between 4.5 and 7.5 percent entrained air, depending on the maximum coarse aggregate size and the climate (ACPA 1995). The slump should be between 50 and 100 mm (2 to 4 in) for overall workability and finishability. Temperature during installation and curing should also be closely monitored as adverse temperature conditions during installation have been linked to premature failures (Yu, Mallela, and Darter 2006).

Table 6.4. Examples of high early-strength mix designs (ACPA 1994; Jones 1988; Whiting et al. 1994).

Mix Component	Type I (GADOT)	Type III (Fast Track I)	Type III (Fast Track II)	RSPC	RSC
Cement, (kg/m^3)	447	381	441	363	386
Flyash, (kg/m^3)	–	43	48	–	–
Course Aggregate, (kg/m^3)	1067	828	776	1011	1070
Fine Aggregate, (kg/m^3)	612	808	774	832	595
w/c Ratio	0.40	0.40 to 0.48	0.40 to 0.48	0.41	0.45
Water Reducer	–	yes	yes	–	–
Air Entraining Agent	As needed to obtain air content of 6 $\pm$ 2 percent.				
$CaCl_2$ % wt. cement	1.0	–	–	–	–
Opening time	4 hr	24-72 hr	12-24 hr	4 hr	4-6 hr

1 kg/m^3 = 1.69 lb/yd^3

Precast panels have been used in some areas where very short work windows are available (ACPA 2006). In some cases, a cracked or damaged slab has been replaced with a precast panel in as little as 4 hours (ACPA 2006). If using precast panels, the dimensions (thickness, width, and length) of the pavement slabs in the repair areas must be clearly defined (ACPA 2006). Because the use of precast panels is a highly specialized technique that is relatively new, it will not be discussed in detail in this document. Several recent papers and reports are available that document the early experience with this technique (Mathis 2001; Merritt and Tyson 2001; Buch, Lane, and Kazmierowski 2006; Hossain, Ozyildirim, and Tate 2006).

Load Transfer Design in Jointed Concrete Pavements

Transverse joint load transfer design is one of the most critical factors influencing the performance of full-depth repairs. Load transfer is the ability to transmit wheel loads (and associated deflections, stresses, and strains) across a joint (or crack) in a concrete pavement. Poor load transfer allows differential movement of the slabs that can cause serious spalling, rocking, pumping, faulting, and even breakup of the adjacent slab or repair itself. In selecting a joint design for a particular full-depth repair project, the performance of various joint designs under similar traffic levels within the agency should be used as a guide.

The use of smooth dowel bars is highly recommended for all full-depth repairs because they provide better performance (less faulting, rocking, and other joint-related distresses) than other means of load transfer. The only exception may be residential streets that carry fewer than 100 trucks or buses per year, for which aggregate interlock joints may be sufficient. Table 6.5 summarizes dowel bar-related design details for different pavement thickness ranges (ACPA 2006).

Some specifications require three, four, or five dowels per wheelpath, whereas others require dowels across the entire lane width (ACPA 2006). Figure 6.5 shows one recommended layout of the dowels or tie bars. At least four to five dowels should be located in each wheelpath to provide effective load transfer. The use of 38-mm (1.5-in) diameter dowel bars is recommended for most interstate pavements because they provide the most effective load transfer (ACPA 1995). For light traffic and for pavements less than 250 mm (10 in) thick, 32-mm (1.25-in) diameter dowel bars may be acceptable (ACPA 1995). Experience has shown that 25-mm (1-in) diameter dowel bars are not adequate to withstand the bearing stresses in repair joints (Snyder et al. 1989; ACPA 1995).

Table 6.5. Dowel size requirements for full-depth repairs in jointed concrete pavements.

Pavement Thickness, mm (in)	Dowel Diameter, mm (in)	Drilled Hole Diameter, mm (in)		Min. Length, mm (in)	Spacing, mm (in)
		Grout	Epoxy		
≤ 150 (≤ 6)	19 (0.75)	24 (0.95)	21 (0.83)	350 (14)	300 (12)
< 200 (6.5 to 8)	25 (1.0)	20 (1.2)	27 (1.08)		
200 to 240 (8 to 9.5)	32 (1.25)	37 (1.45)	34 (1.33)		
250+ (10+)	38 (1.5)	43 (1.7)	40 (1.58)		

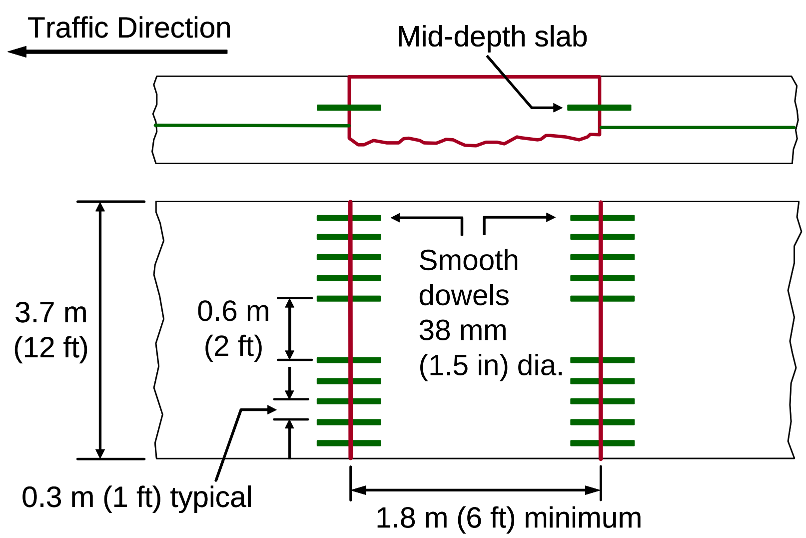


Figure 6.5. Example dowel bar layout.

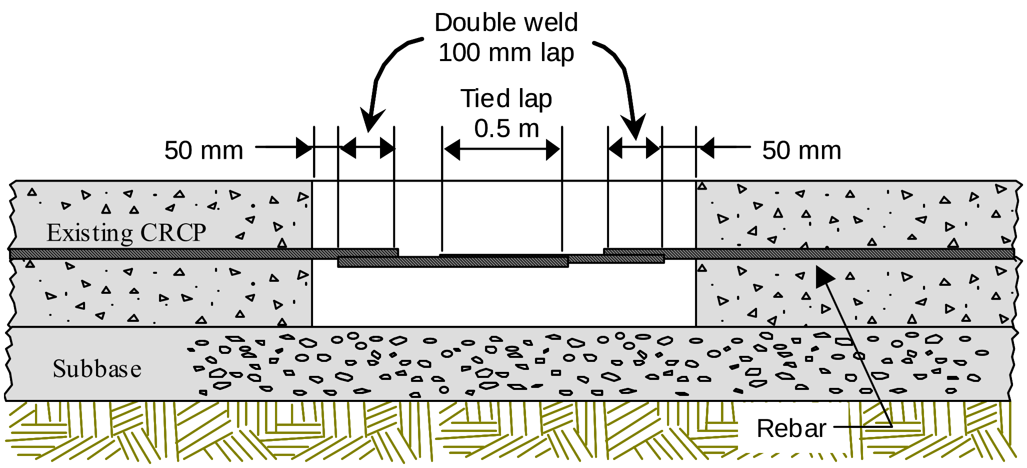
Restoring Reinforcing Steel in CRCP

On CRCP, it is important to maintain the continuity of reinforcement through the full-depth repair. The new reinforcing steel installed in the repair area should match the original in grade, quality, and number. The new bars should be cut so that their ends are at least 50 mm (2 in) from the joint faces, and either tied, mechanically connected, or welded to the existing reinforcement. In placing the bars, chairs or other means of support should be provided to prevent the steel from being permanently bent down during placement of the concrete. Moreover, a minimum of 65-mm (2.5-in) cover should be provided over the reinforcing steel.

Depending on the type of splice used, different overlap lengths are required to allow the splice to develop the full bar strength. For all connection types, a 50-mm (2-in) clearance is required between the end of the lap and the existing pavement. The recommended lap lengths are as follows (FHWA 1985; Gagnon, Zollinger, and Tayabji 1998):

- Tied splice. Tied splices should be lapped 460 mm (18 in) for 16-mm (5/8-in) bars, and 530 mm (21 in) for 19-mm (0.75-in) bars.

- Welded splice. A 6-mm (0.25-in) continuous weld should be made either 100 mm (4 in) long on both sides, or 200 mm (8 in) long on one side. To avoid potential buckling of bars on hot days, the reinforcement must be lapped at the center of the repair as illustrated in figure 6.6. This allows movement of the CRCP ends without damaging the steel. Although this procedure has been used successfully, some problems have resulted from poor workmanship.
- Mechanical connection. These have a minimum lap length of 100 mm (4 in).



1 in = 25.4 mm; 1 ft = 0.3048 m

Figure 6.6. Details of welded or mechanical connection for CRCP repair (FHWA 1985).

Opening to Traffic

There is not a clear consensus on what strength is required for opening fast-track concrete pavements to traffic. Factors such as type of application (full-depth repairs with 1.8-m [6-ft] slabs compared to a localized reconstruction pavement with 4.6-m [15-ft] slabs), expected traffic loadings, and expected edge loading conditions may all affect the required minimum strength. A review of state highway practices suggests a range of 13.8 to 20.7 MPa (2,000 to 3,000 lbf/in^2) compressive strength, and 2.0 to 2.8 MPa (290 to 400 lbf/in^2) flexural strength (third point loading) for the opening of full-depth repairs (Van Dam et al. 2005).

The FHWA (1994) recommends an absolute minimum flexural strength of 2.5 MPa (360 lbf/in^2) (third-point loading) for opening to traffic on any fast-track project. However, an opening flexural strength of 3.1 MPa (450 lbf/in^2) (third-point loading) may be more appropriate if heavy edge loading is anticipated.

In addition to the potential for slab cracking, early trafficking of doweled pavements can result in significant dowel bar bearing stresses, which can lead to “socketing” of the dowel bar and poor load transfer performance (Okamoto et al. 1994). Whiting et al. (1994) recommend the use of the following compressive-strength criteria in addition to typical flexural strength requirements on fast-track projects to avoid crushing of concrete around dowels:

- 13.8 MPa (2,000 lbf/in^2) for concrete pavement slabs containing 38 mm (1.5-in) dowel bars.
- 17.2 MPa (2,500 lbf/in^2) for concrete pavement slabs containing 32 mm (1.25-in) dowel bars.

A recent study on the effects of early-age loading on the concrete surrounding the dowel bar produced a simple and easy-to-use procedure that may be used to establish minimum compressive strength requirements for opening to traffic based on key pavement design inputs, including slab thickness, k-value, and dowel bar diameter (Crovetti and Khazanovich 2005).

A summary of minimum opening strengths for various sizes and thicknesses of full-depth repairs is provided in table 6.6 (ACPA 2006). It is preferable to have a measure of the actual concrete strength before allowing the repair to be opened to traffic, especially if very early opening is required (e.g., 4 hr or less curing time). On such projects, maturity meters or pulse-velocity devices may be used to monitor concrete strength (ACPA 1995).

Table 6.6. Minimum opening strengths for full-depth repairs (ACPA 2006).

Slab Thickness, mm (in)	**Strength for Opening to Traffic, MPa (lbf/in^2)**			
	Repair Length < 3 m (10 ft)		**Slab Replacements**	
	Compressive	**3rd-Point Flexural**	**Compressive**	**3rd-Point Flexural**
150 (6.0)	20.7 (3000)	3.4 (490)	24.8 (3600)	3.7 (540)
175 (7.0)	16.5 (2400)	2.6 (370)	18.6 (2700)	2.8 (410)
200 (8.0)	14.8 (2150)	2.3 (340)	14.8 (2150)	2.3 (340)
225 (9.0)	13.8 (2000)	1.9 (275)	13.8 (2000)	2.1 (300)
250+ (10.0+)	13.8 (2000)	1.7 (250)	13.8 (2000)	2.1 (300)

The HIPERPAV II computer software program (Ruiz et al. 2005a; Ruiz et al. 2005b) may be helpful in identifying the conditions under which special care is needed to avoid random cracking of full-depth repairs. Developed under contract with FHWA, the software takes key environmental, structural design, mix design, and construction inputs, and generates a graph showing the development of concrete strength and stress over the first 72 hours after placement. If the stress exceeds the strength at any time, a high potential for uncontrolled cracking is indicated. For such cases, adjustments can be made to mix properties, curing practices, or the time of concrete placement to reduce the potential for cracking.

6. CONSTRUCTION

The construction and installation of full-depth repairs involves the following steps:

1. Concrete sawing.
2. Concrete removal.
3. Repair area preparation.
4. Restoration of load transfer in JCP or reinforcing steel in CRCP.
5. Concrete placement and finishing.
6. Curing.
7. Diamond grinding (optional).
8. Joint sealing on JCP.

Each of these steps is described for both JCP and CRCP: further guidance can be found in other publications (FHWA 1985; ACPA 1995; ACPA 2006).

Step 1: Concrete Sawing

Jointed Concrete Pavements

Two types of sawed transverse joints have been used for full-depth repairs: rough-faced and smooth-faced (shown in figure 6.7a and 6.7b). The smooth-faced joint, in which saw cuts are full-depth, is recommended. Although smooth-faced joints will not contribute to aggregate interlock load transfer, they are easier to construct and do not contribute to secondary deterioration. Dowels are recommended for all smooth-faced joints.

For JRCP repairs, there is no need to expose the reinforcing steel in the existing pavement because the repairs do not need to be tied into the existing pavement. In fact, for most patches, there is no need to provide reinforcing steel within the repair. Reinforcing steel is only required within repairs that are greater than 4.6 m (15 ft) long, as those long repairs have a tendency to crack. The steel is used in these longer repairs not to prevent the cracks from occurring, but rather to hold the cracks tightly together.

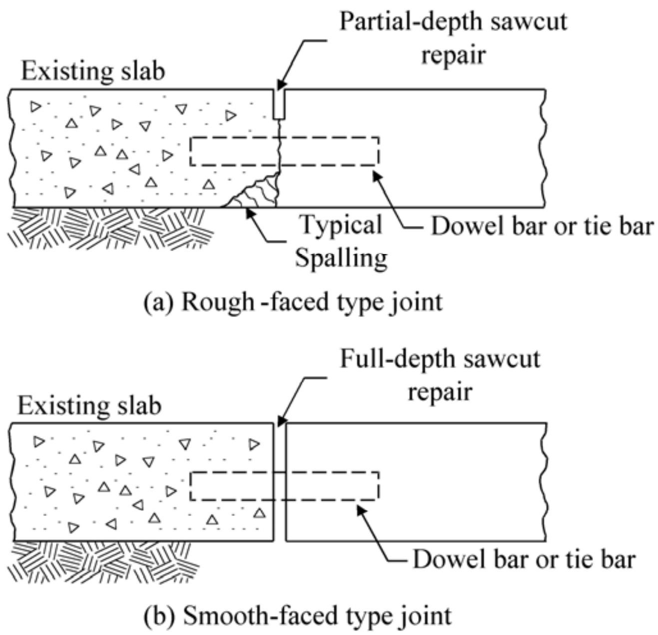


Figure 6.7. Types of sawed transverse joints: (a) rough-faced (b) smooth-faced.

Repair boundaries should be sawed full depth with diamond saw blades. On hot days, it may not be possible to make such cuts without first making a wide, pressure relief cut within the repair boundaries. A carbide-tipped wheel saw may be used for this purpose, but the wheel saw must not intrude on the adjacent lane unless the lane is slated for repair. The wheel sawcuts produce a ragged edge that promotes excessive spalling along the joint. Hence, if wheel sawcuts are made, diamond sawcuts must be made at least 460 mm (18 in) outside the wheel sawcuts. To prevent damage to the subbase, the wheel saw must not be allowed to penetrate more than 13 mm (0.5 in) into the subbase. The longitudinal joint (and concrete shoulder, if it exists) should be cut full depth.

Figure 6.8 illustrates the sawing pattern for JCP. The slanted cut shown in the bottom figure is a pressure relief cut that may be necessary to prevent spalling of the adjacent concrete during concrete removal. This cut should be made when the sawed joint closes up (because of hot weather) before the concrete can be removed. Alternatively, a contractor may elect to saw at night during cooler temperatures (ACPA 1995).

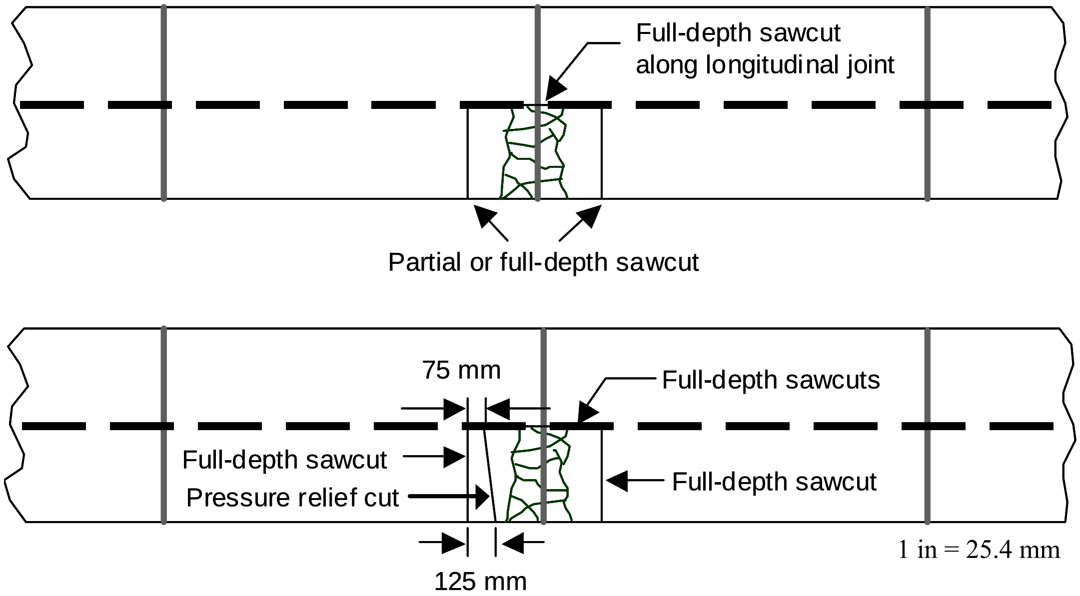


Figure 6.8. Sawcut locations for full-depth repair of JCP.

With full-depth sawcuts, it is very important to limit the traffic loading between the time of sawing and concrete removal to avoid pumping and erosion beneath the slab. It is generally recommended that no more than 2 days of traffic be allowed over the sawed repair areas before removal procedures begin (FHWA 1985).

When an asphalt shoulder is present, it is necessary to remove the shoulder surface approximately 150 mm (6 in) along the repair to provide space for the outside edge form. This also prevents excessive damage to the shoulder when the old concrete is removed. The shoulder should be patched with asphalt concrete after the full-depth repair is placed.

CRCP

For CRCP, two sets of sawcuts are required to provide a rough joint face at repair boundaries. To ensure good repair performance, the joint faces must be rough and vertical, and all underlying deteriorated material must be removed and replaced with concrete. The rough joint faces and continuity of reinforcement (reestablished during repair, keeping the joints tightly closed) provide the load transfer across the repair joints through aggregate interlock.

The rough joint faces are obtained by first making a partial-depth cut around the perimeter of the repair area, to a depth of about one-fourth to one-third of the slab thickness, as shown in figure 6.9 (FHWA 1985). The partial-depth sawcuts should be located at least 460 mm (18 in) from the nearest tight transverse crack. They should not cross an existing crack, and adequate room should be left for the required lap distance and center area. If any of the steel reinforcement is cut, the length of the repair must be increased by the lap length required.

After the partial-depth cuts, two full-depth sawcuts are then made at a specified distance in from the partial-depth cuts as shown in figure 6.9. This distance depends on the method of lapping used to connect reinforcement. The recommended distance is 610 mm (24 in) for tied laps, and 200 mm (8 in) for mechanical connections or welded laps. This distance may be reduced depending on the required lap length.

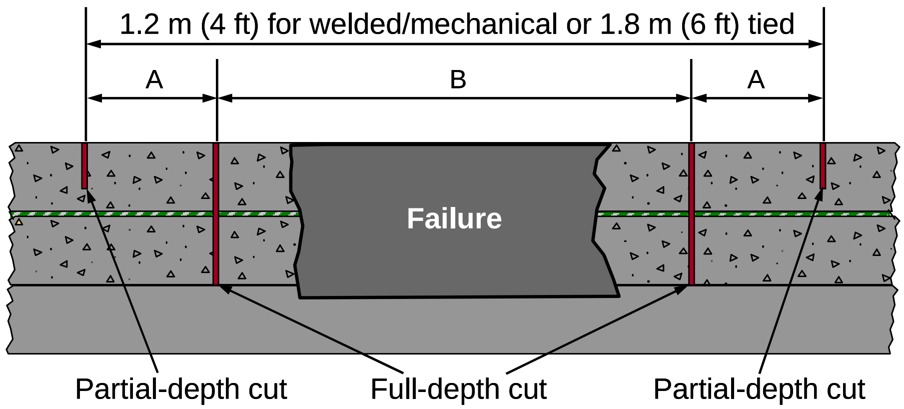


Figure 6.9. Required sawcuts for CRCP (Gagnon, Zollinger, and Tayabji 1998).

In lieu of making two sets of sawcuts, some agencies have experimented with making a single full-depth sawcut in CRCP and not tying into the existing reinforcing steel. Instead, holes are drilled in the faces of the concrete slab and all new rebar are then anchored into the existing slab. Holes for the rebars are drilled to the depth required for a tied lap. This procedure reduces the amount of hand chipping and greatly increases productivity (ACPA 1995).

Step 2: Concrete Removal

<u>Jointed Concrete Pavements</u>

Two methods have been used to remove deteriorated concrete from the repair area:

- <u>Breakup and Cleanout Method</u>. After the boundary cuts have been made, the concrete to be removed is broken up using a jackhammer, drop hammer, or hydraulic ram, and then removed using a backhoe and hand tools. To prevent damage to adjacent concrete, large drop hammers should not be allowed, and large jackhammers must not be allowed near a sawed joint (Darter, Barenberg, and Yrjanson 1985; FHWA 1985; ACPA 1995). Breakup should begin at the center of the repair area and not at the sawcuts.
- <u>Lift-Out Method</u>. After the boundary cuts have been made, lift pins are placed in drilled holes in the distressed slab and hooked with chains to a front-end loader or other equipment capable of vertically lifting the distressed slab. The concrete is then lifted out in one or more pieces (Darter, Barenberg, and Yrjanson 1985; FHWA 1985; ACPA 1995).

Advantages and disadvantages of each removal method are listed in table 6.7. The lift-out method is generally recommended in order to minimize disturbance to the base, which is critical to good performance. This method generally provides the best results and the highest production rates for the same or lower cost, and with the least disturbance to the base (FHWA 1985).

Regardless of the method and equipment used, it is very important to avoid damaging the adjacent concrete slab and existing subbase. In either case, the specifications should state that if the contractor spalls the existing concrete during removal, a new sawcut must be made outside of the sawed area and additional concrete removed at the contractor's expense.

Table 6.7. Advantages and disadvantages of concrete removal methods.

Method	Advantages	Disadvantages
Breakup and Cleanout	Pavement breakers can efficiently break up the concrete, and a backhoe equipped with a bucket with teeth can rapidly remove the broken concrete and load it onto trucks.	This method usually greatly disturbs the subbase/subgrade, requiring either replacement of subbase material or filling with concrete. It also has some potential to damage the adjacent slab.
Liftout	This method generally does not disturb the subbase and does not damage the adjacent slab. It generally permits more rapid removal than the breakup and cleanout method.	Disposal of large pieces of concrete may pose a problem. Large pieces must be lifted out with lifting pins and heavy lifting equipment, or sawn into smaller pieces and lifted out with a front-end loader.

CRCP

The procedure for removing concrete from the center section (between the inner full-depth sawcuts) of the repair area is the same as for JCP. The deteriorated concrete must be carefully removed to avoid damaging the reinforcement and to prevent spalling concrete at the bottom of the joint (beneath the sawcut). This can be accomplished by using jackhammers, prying bars, picks, and other hand tools.

Separating the surrounding concrete from the reinforcing steel must be done without nicking, bending, or damaging the steel in any way. The use of a drop hammer or hydro-hammer should not be allowed in the lap area because this equipment typically damages the reinforcement or causes serious spalling beneath the partial-depth saw joint.

After the concrete has been removed, the reinforcement should be inspected for damage. Any bent bars must be carefully straightened. Bent reinforcement in the repair area will eventually result in spalling of the repair because of the large stresses carried by the reinforcement. If more than 10 percent of the bars are seriously damaged or corroded, or if three or more adjacent bars are broken, the ends of the repair should be extended another lap distance.

Step 3: Repair Area Preparation

All subbase and subgrade materials that have been disturbed or that are loose should be removed and replaced either with similar material or with concrete. If excessive moisture is present in the repair area, it should be dried out before placing new material. Placement of a lateral drain may be necessary where there is standing water. A trench must be cut through the shoulder and a lateral pipe or open-graded crushed stone placed.

It is very difficult to adequately compact granular material in a confined repair area. Hand vibrators generally do not produce adequate compaction to prevent settlement of the repair. Consequently, replacing the damaged portion of a disturbed subbase with concrete is often the best alternative.

When the repair length is less than 4.5 m (15 ft), a bondbreaker board is typically placed along the length of the longitudinal joint to isolate it from the adjacent slab. If the repair is longer than 4.5 m (15 ft), tiebars are typically installed in the face of the longitudinal joint (ACPA 2006).

Step 4: Restoration of Load Transfer in JCP or Reinforcing Steel in CRCP

Restoring Load Transfer in Jointed Concrete Pavements

Smooth, steel dowel bars are recommended for load transfer at both repair joints to allow uninhibited horizontal movement. The dowels are installed by drilling holes on 300-mm (12-in) centers at mid-depth of the exposed face of the existing slab. Tractor-mounted gang drills can be used to drill several holes simultaneously, while maintaining proper horizontal and vertical alignment (ACPA 1995). Single hand-held drills are not recommended because of the likelihood of misalignment (Darter, Barenberg, and Yrjanson 1985).

The dowel holes must be drilled slightly larger than the dowel diameter to allow room for the anchoring material. If a cement grout is used, the hole diameter should be 5 to 6 mm (0.2 to 0.25 in) larger than the dowel diameter (ACPA 2006). A plastic grout mixture provides better support for dowels than a very fluid mixture. If an epoxy mortar is used, the hole diameter should be no more than 2 mm (0.06 in) larger than the dowel diameter, because this type of material can often ooze out through small gaps.

Anchoring the dowels into the existing slab is a critical construction step. Studies have shown that poor dowel embedment procedures often result in poor performance of the repair, because of spalling and faulting caused by movement of the dowels (Snyder et al. 1989). The following procedure is recommended for anchoring dowel bars (Snyder et al. 1989; FHWA 1985; ACPA 1995):

1. Remove debris and dust from the dowel holes by blowing them out with air. If the holes are wet, they should be allowed to dry before installing dowels. Check dowel holes for cleanliness before proceeding.
2. Place quick-setting, non-shrinking cement grout or epoxy resin in the back of the dowel hole. Cement grout is placed by using a flexible tube with a long nose that places the material in the back of the hole. Epoxy-type materials are placed using a cartridge with a long nozzle that dispenses the material to the rear of the hole.
3. Insert the dowel into the hole with a slight twisting motion so that the material in the back of the hole is forced up and around the dowel bar. This ensures a uniform coating of the anchoring material over the dowel bar.
4. Optionally, place a grout retention disk (a thin donut-shaped plastic disk) over the dowel and against the slab face, as illustrated in figure 6.10. This prevents the anchoring material from flowing out of the hole and helps create an effective face at the entrance of the dowel hole (the location of the critical bearing stress).

After placement, the protruding end of the dowels should be lightly greased to facilitate movement. If steel reinforcement is to be provided within the repair (typically in longer repairs), the steel should be placed between concrete lifts with a minimum of 75-mm (3-in) cover and 65-mm (2.5-in) edge clearance.

Restoring Reinforcing Steel in CRCP

As mentioned previously, the continuity of reinforcement must be maintained through full-depth repairs. The splicing of the reinforcement bars should be conducted using the detailed design information presented in the *Design and Materials Considerations* section.

Step 5: Concrete Placement and Finishing

Critical aspects of concrete placement and finishing for full-depth repairs include attaining adequate consolidation and a level finish with the surrounding concrete (Darter, Barenberg, and Yrjanson 1985; Snyder et al. 1989). Special attention should be given to ensure that the concrete is well vibrated around the edges of the repair and that it is not over-finished. Ambient temperatures should be between 4 and 32 °C (40 and 90 °F) for any concrete placement (ACPA 2006).

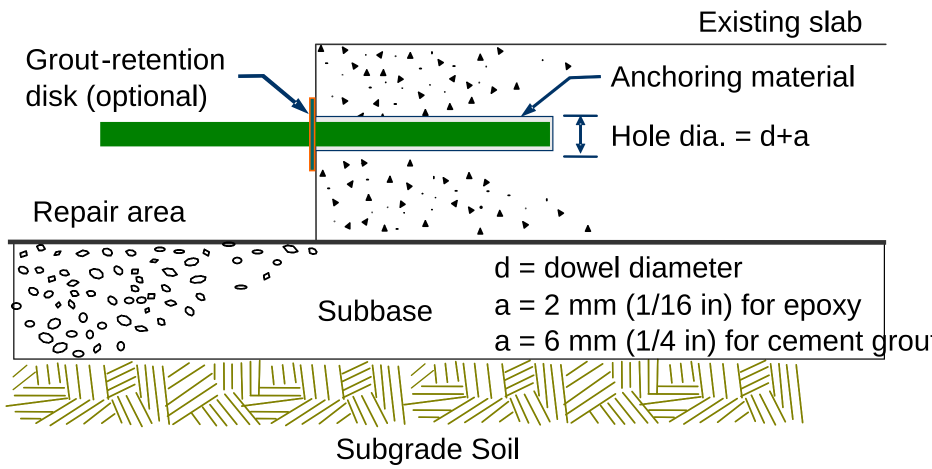


Figure 6.10. Illustration of dowel bar anchoring in slab face.

For repairs less than 3 m (10 ft) in length, the surface of the concrete should be struck off with a screed perpendicular to the centerline of the pavement (ACPA 2006). However, for repairs more than 3 m (10 ft) in length, the surface should be struck off with the screed parallel to the centerline of the pavement (see figure 6.11). The addition of extra water into the concrete truck at the construction site to achieve "greater workability" should be avoided, because this will decrease the strength of the concrete mixture and increase shrinkage. The repair should be struck off two or three times in a transverse direction to ensure that its surface is flush with the adjacent concrete. Following placement, the surface should be textured to match, as much as possible, the texture of the surrounding concrete.

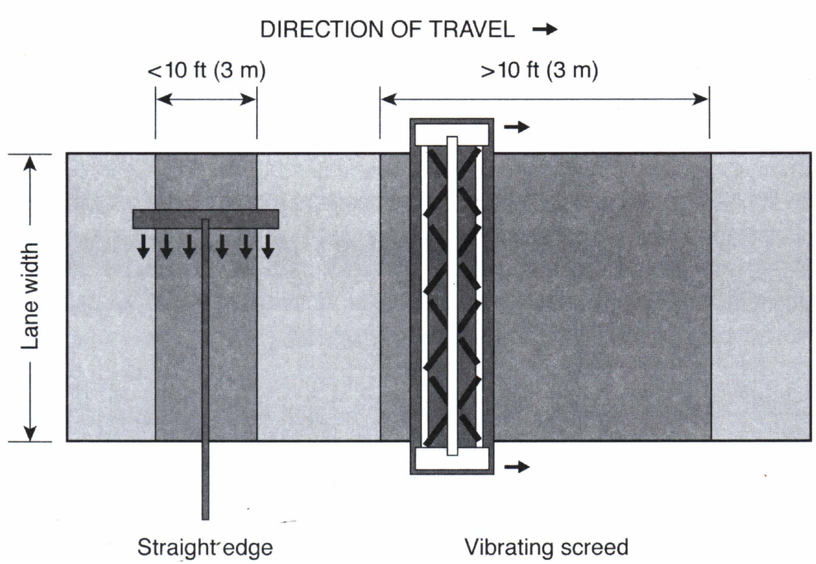


Figure 6.11. Recommended finishing direction depending on size of repair (ACPA 2006).

On longer repairs that require an intermediate joint, the timing of sawing is very important. Sawing too early can cause spalling along the sawcut or dislodging of aggregate particles, whereas sawing too late can lead to random cracking in the patch. In general, the joints should be sawed as soon as possible without damaging the concrete.

On CRCP repairs, it may be necessary to restrict the time of placing concrete to late in the afternoon, depending on the climatic and pavement conditions. On some projects where concrete has been placed in the mornings, expansion of the adjacent slab in the afternoon has resulted in crushing of the repair concrete. This is especially true when the failure extends across all lanes.

Step 6: Curing

Moisture retention and temperature during the curing period are critical to the ultimate strength of the concrete. Proper curing is even more important when using set accelerating admixtures. Therefore, as soon as the bleed water has disappeared from the surface of the concrete (typically within ½ hour of concrete placement), the approved curing procedure should commence to prevent moisture loss from the pavement (ACPA 2006). Typical curing methods include wet burlap, impervious paper, pigmented curing membranes (compounds), and polyethylene sheeting. In general, a normal application of the pigmented curing compound (typically 4.9 m^2/liter [200 ft^2/gal]) gives the best results. A recent FHWA report provides more detailed guidelines on curing (Poole 2005).

On projects with very early opening time requirements (4 to 6 hours), it may be necessary to use insulation blankets to obtain the required strength within the available time. The insulation blankets promote rapid strength gain by keeping the internal temperature of the concrete high, thus accelerating the rate of hydration. In general, insulation blankets are not needed on hot summer days. The use of insulation blankets during cold periods requires special care. The insulation blanket should not be removed when there is a large difference between the concrete and air temperatures, because rapid cooling of the pavement surface following the removal of the insulation blanket can cause cracking of the repair slabs.

Step 7: Diamond Grinding (Optional)

Rehabilitation techniques such as full-depth repairs may result in increased roughness if not finished properly. In particular, differences in elevation between the repair areas and the existing pavement can create an uncomfortable ride. Restoration of a smooth ride may also be an issue when using precast panels. If needed, the best method to blend repairs into a concrete pavement is with diamond grinding. The smooth surface results in improved rideability of the construction project.

Step 8: Joint Sealing on Jointed Concrete Pavements

Experience has shown that both the transverse and longitudinal repair joints must be sawed or formed and then sealed as soon as possible after concrete placement. This will reduce spalling (by lowering the initial point-to-point contact between the existing slab and newly placed repair) and will minimize the infiltration of water. The joint sealant shape factor is the primary factor to consider. Chapter 10 discusses procedures and materials for sealing these joints.

7. QUALITY CONTROL

Quality control/quality assurance practices for full-depth repairs mirror those for the placement of conventional concrete pavement. Paying close attention to the quality of the construction procedures and material handling during construction greatly increases the chances of minimizing premature failures on full-depth repair projects. This section summarizes key portions of a recently developed checklist that has been compiled to facilitate the successful design and construction of good performing partial-depth repairs (FHWA 2005). Although these procedures do not necessarily ensure the long-term performance of a specific repair, the checklist topics are intended to remind both the agency and contractor personnel of those specific design and construction topics that have the potential of influencing the performance of the repair. These checklist items are divided into general categories of preliminary responsibilities, project inspection responsibilities, and clean up responsibilities.

Preliminary Responsibilities

In the initial part of the QC process, agency and contractor personnel should collectively conduct a review of the project documentation, project scope and intended construction procedures, and material usage and associates specifications. Such a collective review is intended to minimize any misunderstandings in the field between agency designers, inspectors, and construction personnel. Specific checklist items for this review are summarized below.

Document Review

As a first step, review the following project-related documents:

- Bid/project specifications and design.
- Applicable special provisions.
- Traffic control plan.
- Manufacturer's specific installation instructions for chosen patch material(s).
- Manufacturer's material safety data sheets (MSDS).

Project Review

In an attempt to maximize the efficiency of the field construction process, the following reviews of the project scope-related items should be conducted:

- Verify that pavement conditions have not significantly changed since the project was designed and that a full-depth repair is still appropriate for the pavement.
- Check that the estimated number of full-depth repairs agrees with the number specified in the contract.
- Agree on quantities to be placed, but allow flexibility if additional deterioration is found below the surface.

Materials Checks

A number of material-related checks are recommended prior to the start of a full-depth repair project. Specifically, agency and contractor personnel should collectively verify that:

- The concrete patch material is being produced by a supplier listed on the agency's Approved/Qualified Supplier List as required by the contract documents.
- The mix design for the material has been sampled and tested prior to installation as required by the contract documents.
- The load transfer units (dowels) meet specifications and that dowels are properly coated with epoxy (or other approved material) and free of any minor surface damage in accordance with contract documents.
- Dowel-hole cementing grout meets specifications.
- Bond-breaking board meets specifications (typically asphalt-impregnated fiberboard).
- Joint sealant material meets specifications.
- Sufficient quantities of materials are on hand for completion of the project.
- All material certifications required by contract documents have been provided to the agency prior to construction.

Equipment Inspections

In this step, all equipment that will be utilized in the construction of full-depth repairs should be reviewed. Ensuring that construction equipment is in good working order will help avoid construction-related problems during the construction process. The following items should be checked or verified as part of the equipment inspection process prior to the start of a full-depth repair project.

Concrete Removal Equipment

- Verify that concrete saws and blades are in good condition and of sufficient diameter and horsepower to adequately cut the required patch boundaries as required by the contract documents.
- Verify that required equipment used for concrete removal is all on-site and in proper working order and of sufficient size, weight, and horsepower to accomplish the removal process (including front-end loader, crane, fork lift, backhoe, skid steer, and jackhammers).

Patch Area Preparation Equipment

- Verify that the plate compactor is working properly and capable of compacting the subbase material.
- Verify that the gang drills are calibrated, aligned, and sufficiently heavy and powerful enough to drill multiple holes for dowel bars.
- Verify that air compressors are equipped with and using properly functioning oil and moisture filters/traps. This can be accomplished by passing the air stream over a board, and then examining for contaminants.

Testing Equipment

- Verify that the concrete testing technician meets the requirements of the contract documents for training/certification.
- Ensure that material test equipment required by the specifications is all available on-site and in proper working condition (equipment typically includes slump cone, pressure-type air meter, cylinder molds and lids, rod, mallet, ruler, and 3-m [10-ft] straightedge).
- Ensure that sufficient storage area on the project site is specifically designated for the storage of concrete cylinders.
- Verify that handheld concrete vibrators are the proper diameter and operating correctly.
- Verify that all floats and screeds are straight, free of defects, and capable of producing the desired finish.
- Verify that sufficient polyethylene sheeting is readily available on-site for immediate deployment as rain protection of freshly placed concrete, should it be required.

Weather Requirements

Immediately prior to the start of the construction project, the following weather-related concerns should be checked:

- Verify that air and surface temperature meets manufacturer and contract requirements (typically 4 °C [40 °F] and above) for concrete placement.
- Patching should not proceed if rain is imminent. Patches that have been completed should be covered with polyethylene sheeting to prevent rain damage.

Traffic Control

The developed traffic control plan should be reviewed by field personnel prior to construction. Specifically, the following pre- and post-construction traffic-related items should be verified:

- Verify that the signs and devices used match the traffic control plan presented in the contract documents.
- Verify that the set-up complies with the Federal or local agency *Manual on Uniform Traffic Control Devices* (MUTCD) or local agency procedures.
- Verify that traffic control personnel are trained/qualified according to contract documents and agency requirements.
- Verify that unsafe conditions, if any, are reported to a supervisor.
- Ensure that traffic is not opened to the repaired pavement until the patch material meets strength requirements presented in the contract documents.
- Verify that signs are removed or covered when they are no longer needed.

Project Inspection Responsibilities

During the construction process, careful project inspection by construction inspectors can greatly increase the chances of obtaining well performing full-depth patches. Specifically, the following checklist items (organized by construction activity) summarize the recommended project inspection items.

Concrete Removal and Clean Up

- Verify that the boundaries of the removal areas are clearly marked on the pavement surface and the cumulative area of the pavement to be removed is consistent with quantities in the contract documents.
- Verify that the patch size is large enough to accommodate a gang-mounted dowel drilling rig, if one is being used. Note: the minimum longitudinal length of patch is usually 1.8 m (6 ft).
- Verify that boundaries are sawed vertically the full thickness of the pavement.
- Verify that concrete is removed by either the break-up or lift-out method and that that disturbance of the base or subbase is minimal. Note: the sawcut and lift method is preferred to jackhammer removal.
- Verify that after concrete removal, disturbed base or subbase is re-compacted, and additional subbase material is added and compacted if necessary.
- Verify that concrete adjoining the patch is not damaged or undercut by the concrete-removal operation.
- Ensure that removed concrete is disposed of in the manner described in the contract documents.

Patch Preparation

- Verify that the dowel holes are drilled perpendicular to the vertical edge of the remaining concrete pavement using a gang-mounted drill rig
- Verify that the holes are thoroughly cleaned using compressed air.
- Verity that approved cement grout or epoxy is placed in dowel holes, from back to front.
- Verify that dowels are inserted with a twisting motion, spreading the grout along the bar inside the hole. A grout-retention disk can be used to keep the grout from seeping out of the hole.

- Verify that the dowels are installed in transverse joints to the proper depth of insertion and at the proper orientation (parallel to the centerline and perpendicular to the vertical face of the sawcut excavation) in accordance with contract specifications. Typical tolerances are 6 mm (1/4 in) misalignment per 300 mm (12 in) of dowel bar length.
- If used, verify that tiebars are installed at the proper location, to the proper depth of insertion, and to the proper orientation in accordance with contract documents. When the length of the repair is 4.5 m (15 ft) or greater, tiebars are typically installed in the face of the longitudinal joint. When the length of the repair is less than 4.5 m (15 ft), a bondbreaker board is placed along the length of the patch to isolate it from the adjacent slab.
- Ensure that tiebars are checked for location, depth of insertion, and orientation (perpendicular to centerline and parallel to slab surface).

Placing, Finishing, and Curing Patch Material

- Concrete is typically placed from ready-mix trucks or mobile mixing vehicles in accordance with contract specifications.
- Verify that the fresh concrete is properly consolidated using several vertical penetrations of the surface with a hand-held vibrator.
- Verify that the surface of the concrete patch is level with the adjacent slab using a straightedge in accordance with contract documents.
- Verify that the surface of the fresh patch material is finished and textured to match the adjacent surface.
- Verify that adequate curing compound is applied to the surface of the fresh concrete immediately following finishing and texturing in accordance with contract documents. Note: best practice suggests that two applications of curing compound be applied to the finished and textured surface, one perpendicular to the other.
- Ensure that insulation blankets are used when ambient temperatures are expected to fall below 4 °C (40 °F). Maintain blanket cover until concrete attains the strength required in the contract documents.

Resealing Joints and Cracks

- Verify that patches have attained adequate strength to support concrete saws, patch perimeters and other unsealed joints are sawed off to specified joint reservoir dimensions.
- Verify that joints are cleaned and resealed according to contract documents.

Clean Up Responsibilities

- Verify that all concrete pieces and loose debris are removed from the pavement surface and disposed of in accordance with contract documents.
- Verify that mixing, placement, and finishing equipment is properly cleaned for the next use.
- Verify that all construction-related signs are removed when opening the pavement to normal traffic.

8. TROUBLESHOOTING

This section summarizes some of the more common problems that a contractor or inspector may encounter in the field during construction (see table 6.8) and performance problems that may be observed later (see table 6.9). Recommended solutions associated with known problems are also provided.

Table 6.8. Potential full-depth repair construction problems and associated solutions (FHWA 2005; ACPA 2006).

Problem	**Typical Solutions**
Undercut spalling (deterioration on bottom of slab) is evident after removal of concrete from patch area.	▪ Saw back into adjacent slab until sound concrete is encountered. ▪ Make double saw cuts, 150 mm (6 in) apart, around patch area to reduce damage to adjacent slabs during concrete removal. ▪ Use a carbide-tipped wheel saw to make pressure-relief cuts 100 mm (4 in) wide inside the area to be removed.
Saw binds when cutting full-depth exterior cuts.	▪ Shut down saw and remove blade from saw. ▪ Wait for slab to cool, then release blade if possible, or make another full-depth angled cut inside the area to be removed to provide a small pie-shaped piece adjacent to the stuck saw blade. ▪ Make transverse saw cuts when the pavement is cool. ▪ Use a carbide-tipped wheel saw to make pressure-relief cuts 100 mm (4 in) wide inside the area to be removed.
Lifting out a patch for a full-depth repair damages adjacent slab.	▪ Adjust lifting cables and re-position lifting device to assure a vertical pull. ▪ Re-saw and remove broken section of adjacent slab. ▪ Use a forklift or crane instead of a front-end loader.
Slab disintegrates when attempts are made to lift it out.	▪ Complete removal of patch area with backhoe or shovels. ▪ Angle the lift pins and position the cables so that fragmented pieces are bound together during liftout. ▪ Keep lift height to an absolute minimum on fragmented slabs.
Patches become filled with rainwater or groundwater seepage, saturating the subbase.	▪ Pump the water from the patch area, or drain it through a trench cut into the shoulder. ▪ Re-compact subbase to a density consistent with contract documents, adding material as necessary. ▪ Permit the use of aggregate dust or fine sand to level small surface irregularities (12 mm [1/2 in] or less) in surface of subbase before patch material is placed.
Grout around dowel bars flows back out of the holes after dowels are inserted.	▪ Pump grout to the back of the hole first. ▪ Use a twisting motion when inserting the dowel. ▪ Add a grout retention disk around the bar to prevent grout from leaking out.
Dowels appear to be misaligned once they are inserted into holes	▪ If misalignment is less than 6 mm (1/4 in) per 300 mm (12 in) of dowel bar length, do nothing. ▪ If misalignment is greater than 6 mm (1/4 in) per 300 mm (12 in) of dowel bar length on more than three bars, re-saw patch boundaries beyond dowels and re-drill holes. ▪ Use a gang-mounted drill rig referenced off the slab surface to drill dowel holes.

Table 6.9. Potential full-depth repair performance problems and prevention techniques.

Problem	Typical Causes	Typical Solutions
Longitudinal cracking in the patch.	▪ Patch not long enough. ▪ Insufficient isolation from adjacent slabs. ▪ Inadequate curing for ambient conditions. ▪ Expansion of adjacent slabs on young PCC pavements.	▪ Verify patch dimensions. ▪ Use proper material to isolate FDR along longitudinal joints. ▪ Avoid patching in extreme climate conditions. ▪ Use appropriate protection against rapid moisture loss (double application of curing compound, curing blankets).
Transverse cracking in the patch.	▪ Patch too long. ▪ Misaligned dowel bars. ▪ Tie bars instead of dowel bars. ▪ Inadequate curing for ambient conditions.	▪ Verify patch dimensions. ▪ Check dowel size and location. ▪ Use tie bars at only one joint. ▪ Use appropriate curing methods.
Surface scaling.	▪ Poor mix design. ▪ Adding water during placement or finishing. ▪ Overfinishing the surface. ▪ Inadequate curing for ambient conditions.	▪ Check mix design and adjust if necessary. ▪ Do not add additional water at site. ▪ Do not overfinish surface. ▪ Use appropriate curing methods.
Spalling in patch at the transverse or longitudinal joint.	▪ "Point" load causing high compressive stress. ▪ Incompressibles in joint. ▪ Locked load transfer device.	▪ Isolate longitudinal joints and ensure transverse joints are clean. ▪ Install all transverse dowels and tiebars in line with the longitudinal joint and perpendicular to the transverse joint.
Deterioration adjacent to the patch.	▪ Inadequate material removal. ▪ Less than full-depth sawcuts. ▪ Poor removal technique.	▪ Identify removal boundaries outside the area of deterioration. ▪ Sawcut removal areas full depth. ▪ Use removal technique that does not damage adjacent pavement.
Settlement of the patch.	▪ Inadequate load transfer. ▪ Poor base preparation. ▪ Lack of sealant. ▪ Subsurface moisture.	▪ Follow guidelines for tiebars and load transfer devices. ▪ Prepare subsurface layers properly. ▪ Remove source of any subsurface water. ▪ Seal joints following construction.

9. SUMMARY

Full-depth repairs of concrete may be necessary wherever deterioration extends beyond the upper third of the slab and is adversely affecting ride or safety. Such repairs, when properly constructed, can last as long as the original pavement, greatly improving long-term performance. Proper full-depth repair procedures must be followed to obtain these benefits however, whether the concrete surface is being covered up or simply patched.

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NOTES

CHAPTER 7. RETROFITTED EDGE DRAINS

1. LEARNING OUTCOMES

This chapter discusses the installation of retrofitted edge drains to improve the drainage of existing concrete pavements. After completion of this chapter, the participant should be able to accomplish the following:

1. List benefits of positive pavement drainage.
2. List components of edge drain systems.
3. Describe recommended installation procedures.
4. Identify typical construction problems and remedies.

2. INTRODUCTION

Many pavement research studies have suggested that proper pavement drainage can extend pavement life from several years to more than twice that of a conventional "undrained" pavement (Cedergren 1987; Forsyth, Wells, and Woodstrom 1987; Christory 1990; Christopher 2000). Although the ideal time to address drainage concerns is during initial construction, many older pavements were initially constructed without adequate drainage. Faced with this problem, a number of state agencies have installed retrofitted edge drains to alleviate moisture-related problems on these older pavements.

The purpose of retrofitted edge drains is to collect water that has infiltrated into the pavement structure. These drains then discharge the water to the ditches through regularly spaced outlet drains. Retrofitted edge drains are most commonly used on concrete pavements that have begun to show early signs of moisture-related distresses (such as pumping and joint faulting). Agencies typically install the drains in an effort to delay or slow the development of those moisture-related distresses.

Although positive drainage is expected to contribute to the performance of pavement structures, several recent studies have suggested that other factors may have a bigger effect on performance than drainage (NCHRP 2002). For example, the presence of a permeable base on a doweled JPCP had minimal contribution to performance, whereas the same permeable base on a nondoweled JPCP significantly improved performance (NCHRP 2002). In that same vein, a recent paper states that many of today's pavements are less vulnerable to the detrimental effects of excessive moisture, largely because of the addition of key design features such as thicker slabs, doweled joints, widened slabs, and stabilized bases (Hall and Crovetti 2007). However, positive drainage may still be required for pavements without those design features that are exposed to excessive moisture throughout the year (Hall and Crovetti 2007).

This chapter presents information regarding the process of retrofitting existing concrete pavements with edge drains. Included are discussions of key definitions, guidance on project selection, limitations and effectiveness of the method, design considerations, typical costs, and construction considerations. Also included are examples of the many successes and documented problems associated with the use of retrofitted edge drains.

3. PURPOSE AND PROJECT SELECTION

Purpose of an Effective Drainage System

The purpose of a pavement drainage system is to remove excess water that infiltrates the pavement structure in an effort to reduce, or eliminate, the development of moisture-related damage. When an existing pavement begins showing signs of moisture-related damage, the agency generally has two options for improving the pavement's drainage: 1) wait and redesign the subdrainage system when reconstruction of the pavement is required, or 2) retrofit the existing pavement with longitudinal edge drains. When a pavement is reconstructed, the designer has the luxury of conducting a complete pavement subsurface drainage analysis in order to optimize the selection of all components of the pavement drainage system. Pavement subsurface drainage analysis and design methods are available in references by Moulton (1980), Cedergren, O'Brien, and Arman (1986), Cedergren (1987), FHWA (1992), and NHI (1999). The *DRIP* (*Drainage Requirements in Pavements*) computer software is also available to perform detailed drainage analyses (Mallela et al. 2002).

In rehabilitation projects where retrofitted edge drains are to be installed, pavement layers are already in place and little can be done to improve their individual ability to drain. As a result, the only practical way to improve subsurface drainage is to shorten the drainage path. Figure 7.1 presents a pavement cross section that shows how the presence of retrofitted longitudinal edge drains can improve the drainability of the pavement.

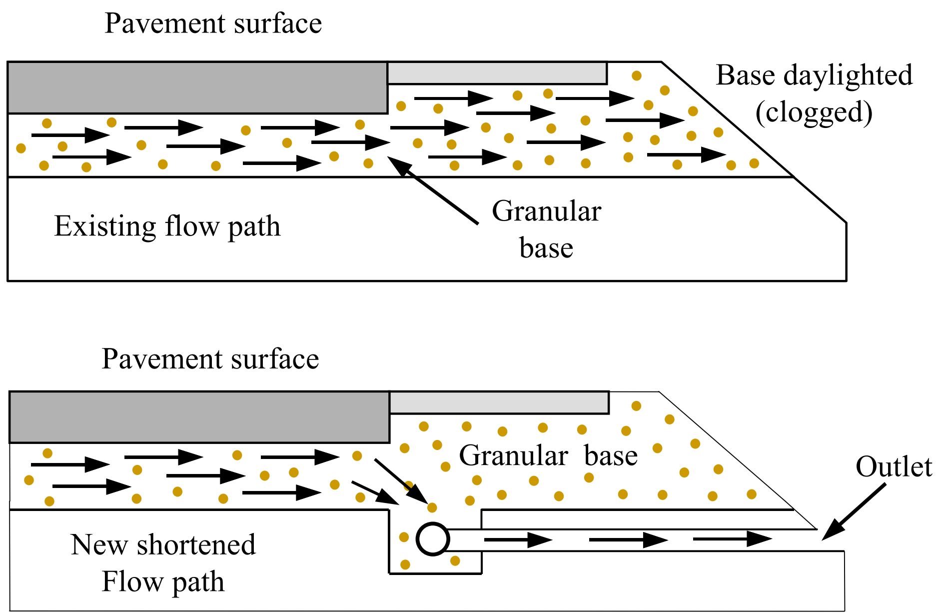


Figure 7.1. Longitudinal drain added to shorten flow path.

Project Selection for Retrofitting Edge Drains

The presence of moisture-related damage is a good indicator of projects with poor drainage; however, it is not always immediately clear if retrofitted edge drains are an appropriate rehabilitation option for a given project. To assist in making this decision, a great deal of project information is needed. As a first step in selecting projects for retrofitted edge drains, a comprehensive survey should be conducted to assess current pavement conditions, identify the sources of water, and assess the erodibility of the base material. The types of moisture-related distresses present provide a good indication of the appropriateness of installing retrofitted edge drains.

A good candidate project for retrofitted edge drains is a pavement that is showing early signs of moisture-damage, is relatively young (i.e., less than 10 years old), and is only exhibiting a minimal amount cracking (less than 5 percent cracked slabs) (Mathis 1990; FHWA 1992). Many studies have concluded that retrofitted edge drains are <u>not</u> effective at prolonging the service life of pavements that have already experienced significant moisture-related deterioration (Wells and Wiley 1987; Young 1990; VDOT 1990).

In general, pavements in which the following condition characteristics are present are <u>not</u> considered good candidates for retrofitted edge drains (Wells 1985; ITD 2007):

- More than 10 percent of the surface exhibits cracking.
- A high number of transverse joints are spalled.
- Where pumping has occurred (unless the voids under the pavement are to be corrected).
- Localized distress exists such as edge punchouts, transverse cracking, longitudinal and diagonal cracking, all of which require extensive patching to return the pavement to an adequate level of service.
- A cement-treated base exists that is no longer intact.
- Pavements where the existing base contains greater than 15 percent fines (material passing the 0.075-mm [No. 200] sieve). Base materials with these characteristics may be too impermeable for an effective retrofitted subdrainage installation (FHWA 1990).

In addition to condition considerations, an ideal candidate for retrofitted edge drains is a project that has acceptable geometrics (longitudinal and transverse slopes) and adequate depth and condition of roadside ditches. It is important that these pavement characteristics be adequate (or improved during edge drain installation) so that water can effectively be removed.

4. LIMITATIONS AND EFFECTIVENESS

The performance of pavements with retrofitted edge drains has been mixed. In many instances, the retrofitted edge drains have been effective in removing water from the pavement structure (especially water entering through the lane-shoulder joint), which reduced the development of moisture-related distresses. However, in other instances, retrofitted edge drains have been found to be ineffective in addressing drainage problems, or in some cases have even contributed to the further deterioration of the pavement structure (Gulden 1983; Wells and Nokes 1993). This inconsistent performance has been mostly attributed to a combination of improper usage (project selection), improper design, damage during installation, lack of post-installation maintenance, or the failure to provide other pavement repairs that are needed at the time of retrofitting edge drains.

Over the years, a number of national and state studies have focused on assessing the limitations and effectiveness of retrofitted edge drain installations. The results of some of these studies, as well as a summary of SHA experience, are presented in the remainder of this section.

Research Study Results

A number of research studies have documented both the successes and problems that agencies have had with retrofitting edge drains in existing concrete pavements. A summary of notable research on retrofitted drainage (presented in chronological order) is provided below.

- One nationwide study of concrete pavement performance showed that edge drains are highly effective in reducing pumping and faulting in jointed concrete pavements (Darter et al. 1985). The study also showed that edge drains are effective in reducing joint deterioration in D-cracked pavements.
- A study by Bradley et al. (1986) examined the performance of concrete pavements with and without longitudinal edge drains in Arkansas, Florida, Louisiana, and New Mexico. The study concluded that edge drains can be effective at extending pavement life.
- A 1986 study by the Permanent International Association of Road Congresses (PIARC) investigated the effectiveness of edge drains in reducing pumping when combined with nonerodible materials (PIARC 1986). That study indicated that care must be taken to ensure that the drains are needed, are adequately designed, and are properly installed in a pavement if the pavement's performance is to be improved. A 1998 study emphasized this latter point by

concluding that many failures of pavements with subsurface drainage can be attributed to poor design and construction practices (Daleiden 1998).

- FHWA Experimental Project 12, *Concrete Pavement Drainage Rehabilitation*, evaluated the performance of edge drains in 10 States (Baumgardner and Mathis 1989). This study concluded that most of the water being removed through retrofitted edge drains is the water that is either infiltrating the lane-shoulder joint or draining through the voids and channels that have developed at the slab–base interface. In addition, the results of this study suggested that by the time retrofitted edge drains are typically added much of the damage is already done, and the improved drainage may be of questionable value.
- Under an NCHRP study, Koerner et al. (1994) investigated the performance of forty-one geocomposite edge drain installations, of which the performance of ten was found to be unacceptable. This contrasted with the performance of other types of edge drains that were deemed "very acceptable." The failure to place the geocomposite edge drain against the base layer was cited as the primary cause of failure, resulting in soil retention and clogging.
- A study by Daleiden (1998) conducted video inspections of in-service edge drains to assess their performance, and revealed that only 30 percent of the in-service edge drains were fully functional. The common causes for poor performance of retrofitted pipe edge drains were discovered to be improper installation, pipe clogging due to fines, and pipe crushing. The common causes of poor performance of geocomposite edge drains were found to be drain damage due to improper installation (crushed or buckled geocomposite panels) and clogging due to caking of fines on the geotextile material.
- A 1998 published report discussed the results of a Wisconsin study that focused on evaluating the use of positive drainage systems in pavement structures (Rutkowski, Shober, and Schmeidlin 1998). As part of this study, three different test sections were used to compare the performance of different types of retrofitted edge drains with control sections without positive drainage systems. In all three cases, the performance of the control sections with edge drains was not found to be significantly better than the performance of the sections with retrofitted edge drains. Also, the researchers concluded that the retrofitting of edge drains was not effective in preventing or reducing the progression of faulting.
- In a study of the performance of diamond ground pavement sections, time-series performance data for pavements with and without retrofitted edge drains were examined (Rao et al. 1999). It was found that the sections with edge drains were faulted about the same as the nondrained ground sections. It was further determined that the edge drains were ineffective due to clogging (Rao et al. 1999).
- The conclusions of a recent NCHRP synthesis study found a general good performance of geocomposite edge drains, and reported that most failures were predictable and related to either a poor drainage design, a misapplication of the treatment, or improper construction techniques (Christopher 2000).
- A national study of pavement drainage showed mixed results in terms of the benefits of retrofitted drainage on pavement performance (NCHRP 2002). In some cases, the addition of retrofitted edge drains reduced the rate of faulting development, whereas in other cases there was no such reduction (NCHRP 2002).

Agency Experience with Retrofitted Edge Drains

Several agencies have significant experience with the installation of retrofitted edge drains. The following sections summarize some of the more notable agency experiences.

- In 1981, the State of California began a research project to determine if edge drains were effective at providing rapid drainage, and therefore, minimizing pumping and faulting on their nondoweled JPCP. The results of this research showed that edge drains were indeed effective at reducing the faulting in JPCP by 88 percent (Wells 1985). An additional 5 to 10 years of additional service life was attributed to the installation of retrofitted edge drains.
- In the mid 1970s, Georgia installed retrofitted pipe edge drains on several heavily trafficked nondoweled JPC pavements that were experiencing pumping and joint faulting. These pavements had granular bases with high fine contents. Although the retrofitted edge drains reduced the visible signs of pumping, the magnitude of joint faulting and number of cracked slabs continued to increase. An investigation into this poor performance found that significant amounts of fines from the base and subgrade were being transported out of the pavement structure via the edge drain system (Gulden 1983).
- Based on an evaluation of the performance of its edge drain installations (both initially installed and retrofitted), Indiana placed a moratorium on the use of geocomposite edge drains in September 1995 (Hassan et al. 1996). Reasons for the moratorium include a concern over the potential clogging of these drains, as well as their susceptibility to damage during installation (Andrewski 1995; Christopher 2000).
- Kentucky has been installing longitudinal edge drains on concrete pavements for over 25 years, mostly on the Interstate and parkway systems (Allen 1990). Since 1984, Kentucky has almost exclusively been using geocomposite edge drains. In direct comparisons of geocomposite and pipe edge drains, they report a number of interesting findings. The geocomposite edge drains were found to start draining much more rapidly than pipe edge drains after a rainfall event—a few minutes compared to 24 to 48 hours. However, in studies done by both excavation and bore scope, it was found that some damage (crushing and buckling of the geocomposite edge drain core) to the geocomposite drains had occurred as a result of excessive compactive forces during backfill operations (Fleckenstein and Allen 2000).

  In 1997, Kentucky completed an in-depth research study of the performance and construction of their highway edge drain systems. After this study was completed, the Kentucky Department of Highways (DOH) began requiring that all new edge drain installations be inspected with video cameras as part of an initial quality control program. As a result of the camera inspections, the number of edge drain outlet failures decreased from 20 percent to approximately 2 percent by the year 2000 (Fleckenstein and Allen 2000).
- A number of State highway agencies have recently documented problems with their use of geocomposite edge drains. Similar to Indiana, Pennsylvania has reported problems including clogging due to siltation and crushing of the drain during installation (Christopher 2000). Illinois discontinued the use of geocomposite edge drains after the results of an extensive evaluation of drainage design policies found numerous examples of improper design, construction, and maintenance (DuBose 1995). Michigan and Wisconsin have also reportedly discontinued the use of geocomposite edge drains due to problems resulting in decreased service life and high initial costs. Although Ohio reported some construction problems, they still found that their geocomposite edge drains were working as a secondary drainage system (Christopher 2000).
- In 1997, the Maine Department of Transportation installed a retrofitted edge drain on a section of highway composite pavement near New Gloucester, Maine. A survey of the experimental sections after being in-service for 5 years found that while the sections with retrofitted edge drains showed significantly less load-related cracking and reflection cracking than the control sections, the same sections exhibited more edge cracking (MDOT 2003) It was believed that the edge cracking was attributed to settlement of the experimental drainage systems (MDOT 2003).

5. MATERIALS AND DESIGN CONSIDERATIONS

Materials Considerations

Types of Edge Drains

Historically, the following three types of edge drains have been used on retrofitted edge drainage projects:

- Pipe edge drains.
- Prefabricated geocomposite edge drains (PGED).
- Aggregate trenches or “French drains.”

Aggregate trench drains—aggregate (permeable material) backfilled subsurface trench constructed along the edge of the pavement—are not generally recommended because they have a relatively low hydraulic capacity and cannot be maintained (FHWA 1992). More detailed descriptions of pipe edge drains and prefabricated geocomposite edge drains are included in the following sections.

Pipe Edge Drains

A pipe edge drain system consists of a perforated longitudinal conduit placed in an aggregate filled-trench running along the length of the roadway. Water is discharged from the pavement through regularly spaced transverse outlet pipes connected to the longitudinal drainage pipe. Perforated corrugated plastic is commonly used for the longitudinal collector pipe, although rigid, smooth-walled plastic pipe is being used more widely. The trench is partially lined with geotextile fabric (in areas where it comes in contact with either the subbase or subgrade materials) to prevent the infiltration of fines, and then filled with stabilized or nonstabilized open-graded material. A typical cross section of a pavement retrofitted with a pipe edge drain system is presented in figure 7.2.

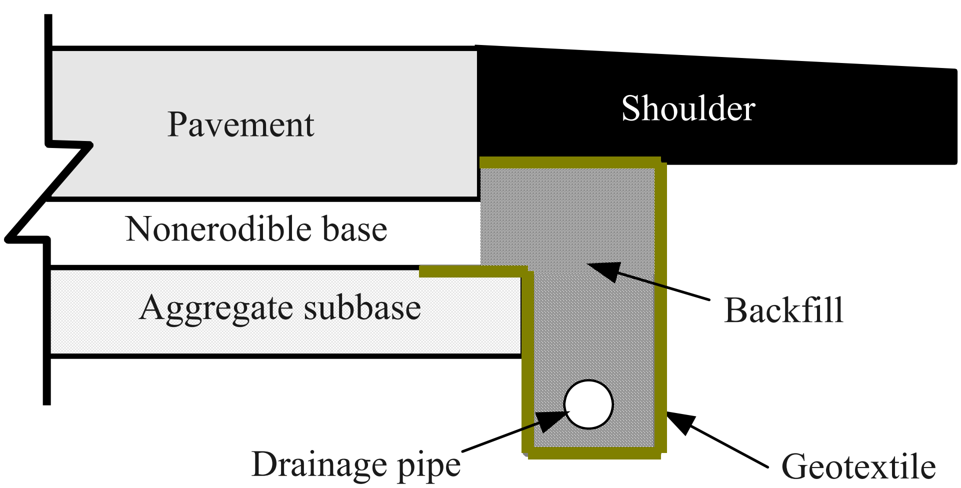
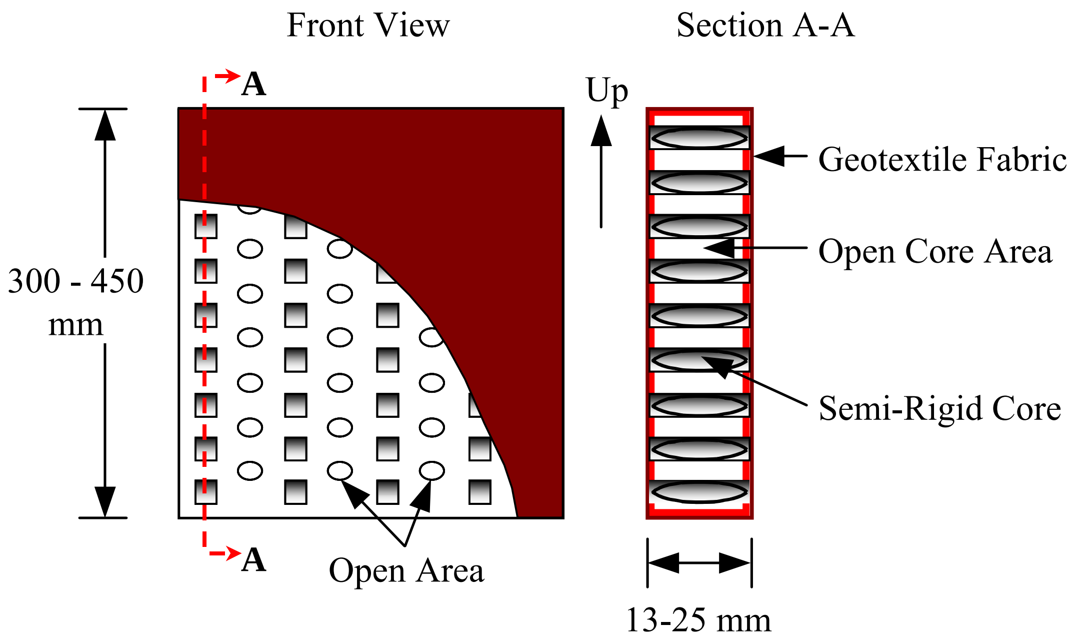


Figure 7.2. Recommended design for retrofitted pipe edge drains (NHI 1999).

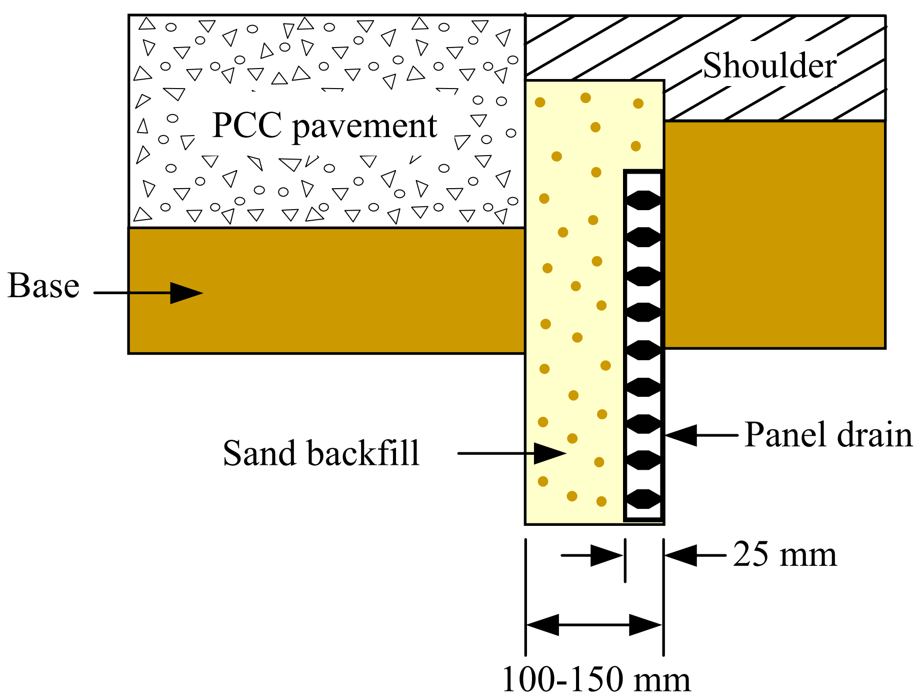
Prefabricated Geocomposite Edge Drains (PGEDs)

PGEDs, also known as “panel” or “fin” drains, consist of an extruded plastic drainage core wrapped with a geotextile filter. Figures 7.3 and 7.4 show details of a typical geocomposite edge drain and a recommended installation detail, respectively.



1 in = 25.4 mm

Figure 7.3. Typical prefabricated geotextile edge drain design (Fleckenstein, Allen, and Harison 1994).



1 in = 25.4 mm

Figure 7.4. Recommended installation detail for geocomposite edge drains (Koerner et al. 1994).

Geocomposite edge drains are typically 13 to 25 mm (0.5 to 1 in) thick and are manufactured in long strips that are coiled into rolls. Their size and the incorporation of a geotextile filter in their design means that they can be placed in narrower trenches (as compared to conventional pipe edge drain installations). In some of the early projects, the trenched soil was also used to backfill the trench. Many incidents of drain clogging (infiltration of fine material) have been attributed to this practice. An evaluation of the field performance of geocomposite edge drains found the infiltration of fines into the drain to be the most common problem with this type of edge drain (Koerner et al. 1994). These problems are believed to be adequately addressed through modifications to the backfill material (i.e., requiring a good quality granular backfill/filter material) and careful placement of the geocomposite edge drain.

Although geocomposite edge drains generally have less drainage capacity than pipe edge drains, this is not typically a problem on most retrofitted drainage projects. The reason for this is that the majority of pavements identified as good candidates for retrofitted edge drains are those that were originally constructed with poorly draining bases and subbases. However, due to these recognized capacity limits, geocomposite edge drains should be used with caution on rehabilitation projects where high water inflows are expected (e.g., HMA overlays on cracked and seated or rubblized concrete pavements). Newer geocomposite material products are now being developed with higher hydraulic capacities for use on these types of projects.

The two main advantages of geocomposite edge drains over traditional pipe edge drains are 1) they are easier to install, and 2) they are substantially cheaper in cost. One disadvantage of geocomposite edge drains is their susceptibility to damage during construction. If proper care is not taken during backfilling operations, crushing, bending, or buckling of the drainage core may occur (Koerner et al. 1994). However, past research studies have indicated that drain clogging can be minimized by using the installation detail shown in figure 7.4, and that damage from backfilling can be avoided by compacting the sand backfill using water puddling (Fleckenstein, Allen, and Harison 1994; Koerner et al. 1994). Overall, many studies have concluded that geocomposite edge drains can function as well as pipe edge drains as long as they are properly installed (Mathis 1990; Koerner et al. 1994).

Backfill Material

The backfill/filler material placed in the trench around the pipe or alongside the geocomposite serves the following functions:

- It acts as a drainage medium to provide a means by which water is moved from the pavement layers to the drainage pipe.
- It acts a filter system that prevents fines from moving into and clogging the drainage system.
- It supports and confines the drain pipe or geocomposite, providing protection both during construction and while in service.
- It provides stabilization to the soil around the drainage trench.

There are specific procedures available for designing the backfill/filler material to ensure that the drainage feature, be it a pipe or geocomposite, does not become clogged with fines. Recommended gradations are included in the *Highway Subdrainage Design* manual (Moulton 1980).

For pipe edge drains, the backfill material for the trench should be at least as permeable as the base material. In a permeable base section, the backfill material will usually be the same as the base material. AASHTO No. 57 gradation should provide sufficient permeability and stability for use as nonstabilized backfill material. Nonstabilized pea gravels are not recommended as the backfill material because they cannot be compacted satisfactorily (Wells 1990). Proper compaction of the backfill material is important to avoid settlement over the edge drain.

Design Considerations

The design of edge drains is a multi-step process that mainly consists of calculating the amount of water that is expected to infiltrate a pavement, and then selecting edge drain details that allow the drainage system to effectively remove the water from the pavement. In addition to sizing the components of the drainage system, it is important to design filters (geotextile or aggregate) that are effective at preventing fines from entering the edge drain (not clogging) over the life of the system (Christopher 2000). The grade of the invert must also be established to maintain flow and the outlets must be spaced and sized appropriately to prevent backup in the edge drain system (Christopher 2000).

Details for designing edge drains for new construction or major reconstruction projects are presented by Moulton (1980) and FHWA (1992). A computer program, DRIP (*Drainage Requirements in Pavements*), is also available for conducting the detailed drainage analyses (Mallela et al. 2002). This section provides an abbreviated explanation of the major considerations associated with designing effective retrofitted edge drains, with more detailed information provided elsewhere (Moulton 1980; FHWA 1992; NHI 1999).

Estimate Design Flow Rate

The first step in the design of retrofitted edge drains is the determination of the net inflow of water. The subdrainage system must be adequately sized to handle the flow of water to which it will be subjected. As previously mentioned, for rehabilitation projects, surface infiltration is of primary concern. Groundwater, meltwater, and subgrade outflow are generally relatively small and often ignored in the analysis.

The infiltration of water through cracks, joints, and voids in the pavement surface is a major source of water that must always be included in estimating the net inflow. The amount of infiltration is a function of not only pavement cracking and surface permeability, but also of the ability of the base course to accept and remove water. Consequently, the actual infiltration will be the lesser of two values: the amount of water that could enter through cracks, joints, and so on, or the amount of water that the base course is able to accept.

The *design flow rate* is an estimate of the amount of infiltrated water that will be required to be discharged through the edge drain system (in units of volume per time). This value is typically estimated by knowing detailed information about the base (e.g., width, thickness, permeability) and encountered slopes (cross-slope and longitudinal edge drain slope). Details of the available methods for computing this design flow rate are described in the NHI Reference Manual on subsurface drainage (NHI 1999).

Edge Drain (Collector) Type

As mentioned previously, two types of longitudinal edge drains are commonly used for retrofitted drainage projects: pipe edge drains and prefabricated geocomposite edge drains. It is important that the selected collector type be compatible with the existing pavement structure, as well as the surrounding materials.

For pipe edge drains, several types of drainage pipe of various lengths and diameters have been used successfully in collector systems. Highway agencies use flexible, corrugated polyethylene (CPE) or smooth rigid polyvinyl chloride (PVC) pipe, adhering to AASHTO M 252 or AASHTO M 278 Class 50, respectively. For geocomposite edge drains, product selection should consider an evaluation based on the test procedures outlined in ASTM D 6244-98, *Test Method for Vertical Compression of Geocomposite Pavement Panel Drains* (Christopher 2000).

Edge Drain (Collector) Sizing

Edge drains must be sized so that their capacity is larger than the expected design flow rate. The diameter of pipe edge drains is often selected as the minimum diameter that facilitates maintenance (cleaning) activities and allows a reasonable distance between outlets (Christopher 2000). Pipe diameters typically range from 38 to 203 mm (1.5 to 8 in), with 102 mm (4 in) being the most common. The larger sizes are commonly preferred because of their ability to be easily cleaned and maintained. However, California uses a 75-mm (3-in) pipe and reports no difficulty in cleaning (Christopher 2000). A typical cross-section for a geocomposite edge drain has a width of 13 to 25 mm (0.5 to 1.0 in) and a height of 300 to 450 mm (12 to 18 in) (see figure 7.3) (Fleckenstein, Allen, and Harison 1994).

The computation of the actual flow capacity (required to determine drain sizes) is fairly complicated and is beyond the scope of this chapter. A detailed explanation of these computation methods is found in the NHI Reference Manual on subsurface drainage (NHI 1999).

Edge Drain Location

The design depth for the collector pipes should consider the down elevation available for outletting the water, the likelihood and depth of frost penetration, and economics. Where significant frost penetration is not likely and no attempt is being made to remove or draw the groundwater, it is recommended that the trench depth be deep enough to allow the top of the pipe to be located 50 mm (2 in) below the subbase/subgrade interface. When significant frost penetration is expected, the trench should be constructed only slightly deeper than the expected depth of frost. In ditch sections, the maximum depth of the collector trench is limited by the depth of the ditch.

The location of the drain within the trench is also a major concern for retrofitted geocomposite edge drains. The recommended approach is to place geocomposite edge drains on the shoulder side of the trench, as illustrated in figure 7.4. Studies have shown that this approach will minimize voids within the trench, alleviate the problem of soil loss through the geotextile filters, and avoid bending and buckling of the geocomposite edge drain (Koerner et al. 1994).

Grade Considerations

In most cases, the collector pipes are placed at a constant depth below the pavement surface. This results in the pipe grade being the same as the pavement grade. However, when the pavement grade is very flat, other means must be employed to ensure water can flow through the pipe. One solution is to increase the grade of the edge drain; previous guidance recommends grades of at least 1 percent for smooth pipes and at least 2 percent for corrugated pipes (Moulton 1980). However, this solution can be impractical for very flat areas. For instance, using a 1 percent grade over a flat section of 200 m (660 ft), the edge drain will have to be 2 m (6.6 ft) deep on the low side. A more practical solution is to use smooth pipe and decrease the outlet spacing where flat grades exists.

Trench Width

The required width of trench is a function of construction requirements, drainage requirements, and the permeability of the trench material. Depending on pipe size, many agencies use a trench width of 200 to 250 mm (8 to 10 in) to allow proper placement of the pipe and compaction of the backfill material around the pipe. A narrower trench of 100 to 150 mm (4 to 6 in) is typical for geocomposite edge drains.

Filter Design

Koerner et al. (1994) indicated that the geotextile materials play a pivotal role in edge drain systems. Acting as a filter layer, the geotextile must simultaneously allow water to pass and prevent fines from passing, and it must perform these functions throughout the life of the drainage system (Koerner et al. 1994). For pipe edge drain systems, geotextiles are used to line the trench wherever the backfill material comes into contact with the subgrade; geotextile drains are, themselves, wrapped with geotextile fabric.

Geotextiles consist of either woven or non-woven mats of polypropylene or nylon fibers. The fabrics are used in place of graded filter material, permitting greater use to be made of locally available gradations without special processing. To be effective, the selected geotextile must have the following three characteristics (Koerner et al. 1994):

- The voids must be sufficiently open to allow water to pass through the geotextile and into the downstream drain without building excessive pore water pressures in the upstream soil.
- The voids must be sufficiently tight to adequately retain the upstream soil materials so that soil loss does not become excessive and clog the downstream drain.
- The geotextiles must perform the previous two conflicting tasks (open voids versus tight voids) over the anticipated lifetime of the drainage system without excessively clogging.

As mentioned previously, untreated aggregate bases with more than 15 to 20 percent fines are not good candidates for retrofitted drains because the geotextile will become clogged with fines (FHWA 1990). Geotextiles should be designed considering both the subbase and subgrade soils using the filter criteria in the FHWA geosynthetics design manual (Holtz, Christopher, and Berg 1998). If geotextile fabrics are not used, the gradation of the aggregate used to fill the trench must be designed to be compatible with the subbase and subgrade soils using standard soil mechanics filter criteria (Christopher 2000).

Outlet Considerations

The outlet pipe should be a 100-mm (4-in) diameter stiff, non-perforated smooth-walled PVC or high-density polyethylene (HDPE) pipe with minimum slope of 0.03 m/m (3 ft in 100 ft) (Christopher 2000). Good compaction control of the backfill below, around, and above the pipe is required to avoid transverse shoulder sags (Christopher 2000).

The outlet end should be placed at least 150 mm (6 in) above the 10-year ditch flow line and protected with a headwall and splash block that is blended into the slope. Figure 7.5 illustrates the recommended outlet pipe design (FHWA 1992).

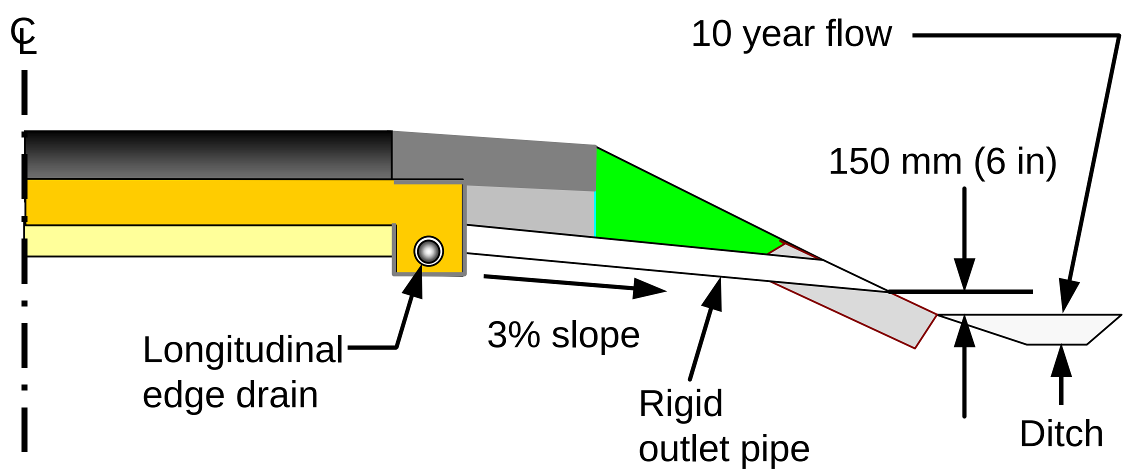
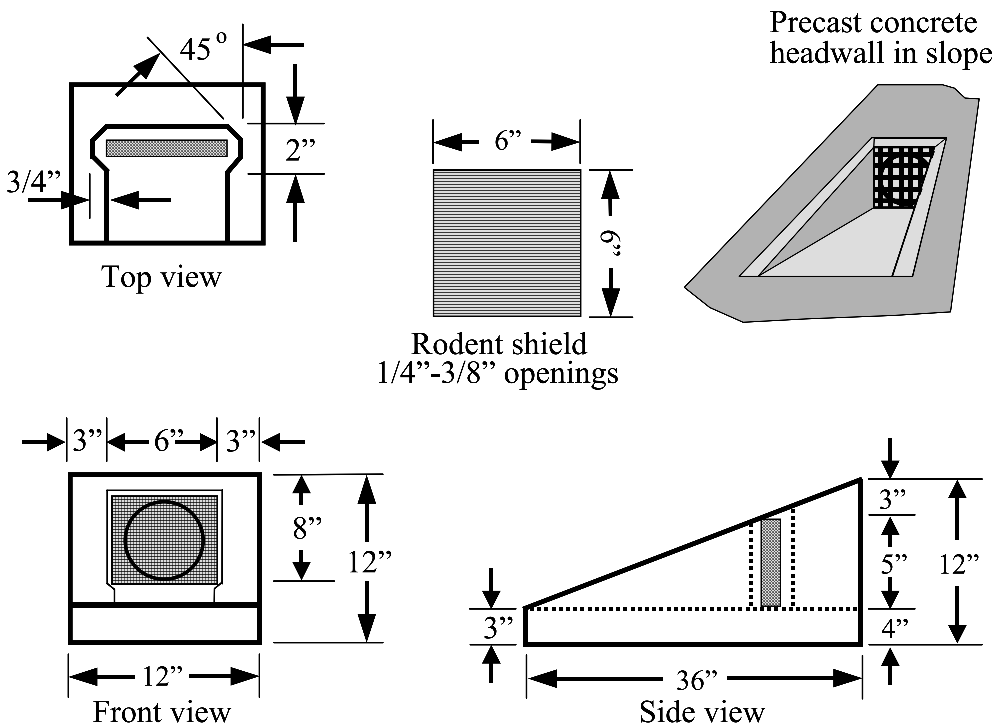


Figure 7.5. Outlet pipe design (FHWA 1992).

The location of outlets is controlled in part by topography and highway geometrics, in that the locations must permit free and unobstructed discharge of the water. In general, the outlet spacing should not exceed 76 to 91 m (250 to 300 ft) in order to permit cleaning (Christopher 2000). On some projects, poor pavement performance has been attributed to excessive outlet spacings.

Headwalls are recommended at outlet locations because they protect the outlet pipe from damage, prevent slope erosion, and facilitate the location of outlet pipes (FHWA 1992). These can be either cast-in-place or precast and should be placed flush with the slope to facilitate mowing operations. To prevent animals from nesting in the pipe, the headwall should be provided with a removable screen or similar device that allows easy access for cleaning. If high ditch flows are expected, flap valves can be used to prevent backflow into the drainage system. A precast headwall with a rodent screen is shown in figure 7.6.



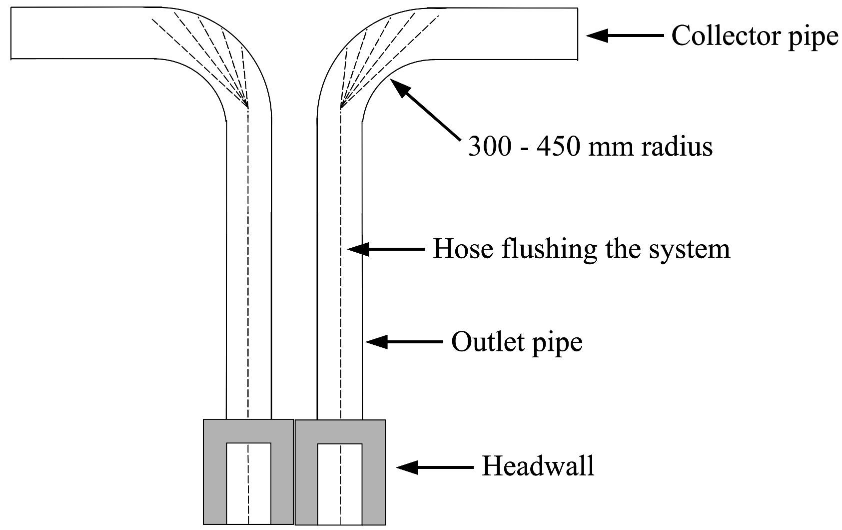
1 in = 25.4 mm

Figure 7.6. Precast headwall with rodent screen (FHWA 1992).

If pipe edge drains are used, the outlet pipes should be connected with the collector pipe through elbows with minimum radii of 305 to 457 mm (12 to 18 in). This alignment facilitates access for cleaning and flushing the pipe. A dual outlet system is also recommended to allow video inspection and maintenance from either end. A recommended outlet system design is shown in figure 7.7.

Other Repair Considerations

It is critical that other necessary repairs to the pavement also be considered when designing a retrofitted edge drain project. If the pavement does not receive the needed repairs prior to (or at the same time as) the installation of the retrofit drains, the effectiveness of the retrofitted edge drains will be limited (NHI 1999). For instance, concrete pavements that exhibit visible pumping and noticeable faulting should, as a minimum, be subsealed prior to the installation of edge drains. Joint resealing, joint load transfer restoration, and full-depth repairs should also be seriously considered. Without these repairs, continued faulting, loss of support, and slab cracking can be expected, even with retrofitted edge drains. Studies in California have shown that if the pavement is severely deteriorated, the addition of retrofitted edge drains will not have a significant effect on prolonging the service life of the pavement, and in some cases may even accelerate the damage (Wells 1985).



1 in = 25.4 mm

Figure 7.7. Recommended outlet detail to facilitate system cleaning and video camera inspections (FHWA 1992).

6. CONSTRUCTION CONSIDERATIONS

Proper construction and maintenance are extremely important to ensure effective edge drains. Inconsistent performance of edge drains, resulting from construction or maintenance problems, has hampered the ability to determine the effectiveness of edge drains in improving pavement performance. The construction steps involved in retrofitting edge drains on an existing pavement differ slightly depending on the type of edge drain being used. The differences in construction considerations for pipe edge drains and geocomposite edge drains are presented separately below.

Pipe Edge Drains

Trenching

It is important to maintain correct line and grade when installing longitudinal underdrains. A mechanical track-driven trencher is often used to create a trench along the edge of the pavement. A large diameter, carbide-tipped wheel saw may also be used. The spoils from the trench must be expelled from the trench and any excess, loose, or foreign material swept away.

As described previously, where significant frost penetration is not likely and no attempt is being made to remove or draw the groundwater, it is recommended that the trench depth be deep enough to allow the top of the drain be located 50 mm (2 in) below the subbase/subgrade interface. When significant frost penetration is expected, the trench should be constructed only slightly deeper than the expected depth of frost to ensure that the system can function during freezing periods. In ditch sections, the maximum depth of the collector trench is limited by the depth of the ditch. Outlets from the system should be located 150 mm (6 in) above the ditch flowline to preclude backflow of water from the ditch. Similarly, if the system is to outlet into a storm drain system, the outlet invert should be at least 150 mm (6 in) above the 10-year expected water level in the storm drain system (see figure 7.5).

Placement of Geotextile

When pipe edge drains are used, the trench should be lined with a geotextile to prevent migration of fines from the surrounding soil into the drainage trench; however the top of the trench adjacent to the permeable base should be left open to allow a direct path for water into the drainage pipe. The geotextile must satisfy the filter requirements described previously in this chapter.

Placement of Drainage Pipes and Backfilling

If a layer of bedding material will be placed prior to placing the drainage pipes, the grooving of the trench bottom has to be done after placing the bedding material. When placing CPE pipes, extra care is also required to prevent overstretching of the pipes during installation. The typical limit for tolerable longitudinal elongation of CPE pipes is 5 percent (NHI 1999).

The backfill material should be placed using chutes or other means to avoid dumping the material onto the pipe from the top of the trench. To prevent displacement of drainage pipes during compaction, the backfill material should not be compacted until the trench is backfilled above the level of the top of the pipes. To avoid damage to the pipes, a minimum of 150 mm (6 in) of cover over the drainage pipe is recommended before compacting (NHI 1999).

Achieving adequate consolidation in a narrow trench can be difficult. Inadequate compaction can lead to settlement, which in turn will result in shoulder distresses. California uses treated permeable materials to backfill drainage trenches to avoid the settlement problem (Wells 1985). A minimum density of 95 percent Standard Proctor (AASHTO T-99) is recommended. A Minnesota study showed that satisfactory compaction can be achieved by running two passes (two lifts, one pass per lift) with a high-energy Vermeer vibratory wheel (Ford and Eliason 1993). Each pass of the vibratory wheel is effective in achieving the target density to a depth of 300 mm (12 in). The Minnesota study also showed that the degree of compaction can be verified easily using a dynamic cone penetrometer (DCP).

Automated equipment has been developed that can be used to install either smooth-walled or corrugated plastic pipes. Figure 7.8 shows one piece of equipment that can install pipe drains at a rate of about 5 km (3.1 mi) per day.

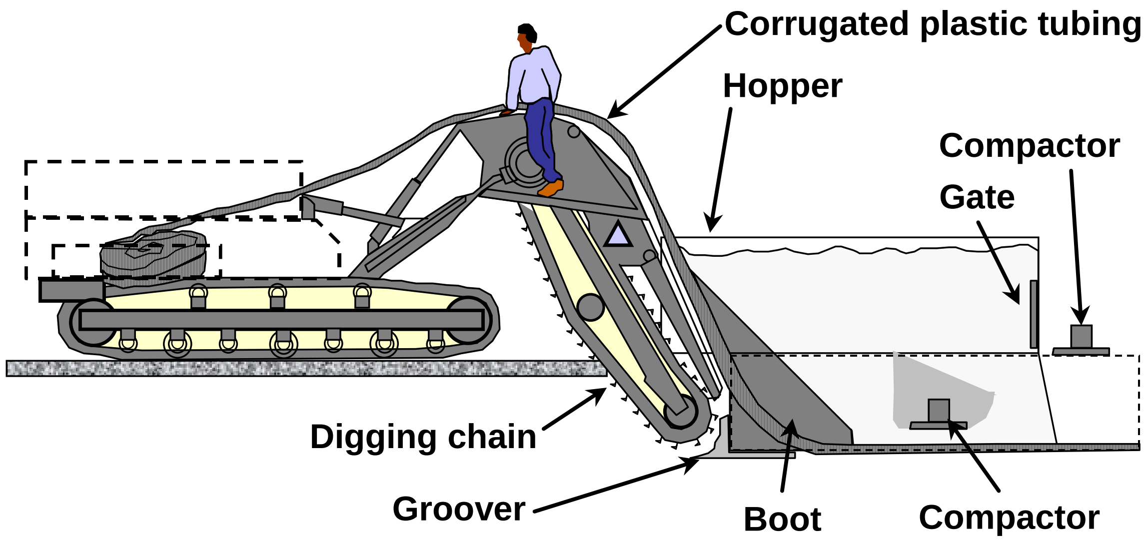


Figure 7.8. Automated equipment for installing pipe edge drains (NHI 1999).

Headwalls and Outlet Pipes

Placing the lateral outlet pipe, constructing the headwalls, and marking the outlet drains with outlet markers are the final steps in the installation of the underdrain pipe. When placing the outlet pipe, it is important to avoid high or low spots in the outlet trench, and to make sure that the exposed end is not turned upward or otherwise elevated. Precast headwalls are recommended to prevent clogging and damage from mowing operations. A rodent screen or wire mesh placed over the ends of the pipe should also be used to keep small animals out.

Geocomposite Edge Drains

Trenching

The trench should be cut 100 to 150 mm (4 to 6 in) wide and deep enough to place the top of the panel drain 50 mm (2 in) above the bottom of the pavement surface layer. Typical dimensions for a geocomposite edge drain consist of an inside cross-sectional thickness of 13 to 25 mm (0.5 to 1 in) and a depth of 300 to 450 mm (12 to 18 in).

Installation of the Geocomposite Edge Drain

The drain should be placed on the shoulder side of the trench, and the trench should be backfilled with coarse sand to ensure intimate contact between the geotextile and the material being drained. Achieving this contact is very important to prevent loss of fines through the geotextile. Maintaining the verticality of the drain panel in the trench during the backfill operation by utilizing a drain panel-positioning wheel or plate is recommended (Elfino, Riley, and Bass 2000).

When required, splices should be made prior to placing the drain in the trench and using the splice kits provided by the manufacturer. The splice should not impede the open flow area of the panel. Vertical and horizontal alignment of the drain should be maintained through the splice, and the splice should not allow infiltration of backfill or any fine material.

Headwalls and Outlets

Prior to any backfilling, the geocomposite edge drains should be connected to drainage outlets. As with pipe edge drains, it is recommended that headwalls be used on the outlets to prevent clogging and damage from mowing operations. Finally, all outlet drains should be clearly marked with outlet markers.

Backfilling

For geocomposite edge drains, excessive compaction can cause problems. Excessive compactive forces can cause crushing and buckling of the geocomposite edge drain panels. The recommended procedure is to backfill using coarse sand and compact by flushing with water (Koerner et al. 1994). The cuttings from the drainage trench are not a suitable backfill material when installing a geocomposite edge drain. If the panel design is not symmetrical about the vertical axis, the panel should be installed with the rigid or semi-rigid back facing the sand backfill (Fleckenstein, Allen, and Harison 1994).

7. TROUBLESHOOTING

Poorly maintained drains can be worse than having no drains at all. It cannot be overemphasized that all subdrainage features, whether installed during initial construction or retrofitted, must be adequately maintained in order to perform properly. Some of the problems that can occur over the life of a drainage system include (Christopher 2000):

- Crushed or punctured outlets.
- Outlet pipes that are clogged with debris, rodent nests, mowing clippings, vegetation, and sediment.
- Edge drains (both pipe drains and fin drains) that are filled with sediment, especially at slopes of less than 1 percent.
- Missing rodent screens at outlets.
- Missing outlet markers.
- Erosion around outlet headwalls.
- Shallow ditches that have inadequate slopes and that are clogged with vegetation.

Adequate maintenance actually begins in the design stage, when a system is constructed so that it can be adequately maintained. This includes the placement of outlet markers, 610 to 914 mm (24 to 36 in) above the ground and suitably marked, to locate transverse outlets, using concrete headwalls with permanent anti-intrusion protection (screens), and specifying proper connectors to allow periodic flushing or jet rodding of the edge drain system. Permanent markers and concrete headwalls also serve as a reminder of the existence of the system and the need for its maintenance.

It is recommended that routine drainage-related maintenance activities be conducted at least twice a year. Examples of some of these maintenance activities include:

- Mowing around drainage outlets.
- Inspection of the drainage outlets and flushing if necessary.
- Removal of vegetation and roadside debris from pipe outlets, daylighted edges, and ditches.
- Replacement of missing rodent screens, outlet markers, and eroded headwalls.
- Inspection of ditches to ensure that adequate slopes and depths are maintained.

Even when all design parameters are properly evaluated and included in the design, the effect of retrofitted subdrainage on pavement performance may not be as expected, and the benefits discussed earlier may not be attainable. An evaluation program that provides feedback data will help the design engineer to determine if there are any aspects of the design that may be detrimental to long-term performance. These programs cannot be short-term evaluations because many moisture-related distresses take time to develop.

8. SUMMARY

Pavement engineers are often faced with older concrete pavements that are displaying moisture-related damage, which may be attributed to a combination of inadequate initial drainage design, subsurface drainage system damage, or inadequate drainage system maintenance practices. To address these drainage-related problems, one rehabilitation option is the retrofitting of the existing pavement with edge drains.

The historical field performance of retrofitted edge drains has been mixed, ranging from reduced pavement deterioration to a detrimental effect on a few projects. The cases of poor performance have generally been attributed to inappropriate use, improper installation, or lack of maintenance; however, even when the edge drains do drain water, the benefits are difficult to assess. For the time being, local experience may offer the best guidance on whether retrofitted edge drains will be effective. Proper construction and maintenance are extremely important to the long-term effectiveness of edge drains.

The installation of retrofitted edge drains should be considered on projects in which all of the following conditions are met:

- The primary source of water affecting pavement performance is surface infiltration.
- The pavement is less than 15 years old.
- The base material is not highly erodible (less than 15 percent material passing 0.075-mm sieve) or dense.
- The pavement is in relatively good condition (i.e., there are no signs of severe moisture-damage and the pavement contains less than 5 percent cracked slabs).

Both pipe and geocomposite edge drains have been used with success on retrofitted drainage projects. The design and construction details differ slightly between these two drain types. Geocomposite edge drains are less expensive to install but are difficult to maintain (i.e., they are nearly impossible to clean if they become clogged). Typically geocomposite drains have lower hydraulic capacities than pipe drains, although newer materials are changing this trend. Pipe edge drains on the other hand have higher hydraulic capacities but are more expensive.

9. REFERENCES

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NOTES

CHAPTER 8. LOAD TRANSFER RESTORATION

1. LEARNING OUTCOMES

This chapter presents information on load transfer restoration (LTR) of joints and cracks in concrete pavements. Upon completion of this chapter, the participants will be able to accomplish the following:

1. List benefits and applications of load transfer restoration.
2. Describe recommended materials and mixtures.
3. Describe recommended construction procedures.
4. Identify typical construction problems and remedies.

2. INTRODUCTION

Load transfer restoration (LTR) is the installation of dowel bars or other mechanical devices at transverse joints or cracks in order to effectively transfer wheel loads across slabs and reduce deflections. As implied by the term "restoration," these devices are retrofitted in existing pavements that either do not have load transfer devices or in which the existing devices are not working. The procedure is also an effective means of providing positive load transfer across random transverse cracks.

Doweled concrete pavements normally exhibit adequate load transfer, but nondoweled jointed plain concrete pavements (JPCP) typically show lower levels of load transfer because they rely on aggregate interlock of the abutting joint faces for load transfer. Aggregate interlock is only effective if the opposing joint faces remain in close contact (in which openings are less than 0.6 mm [0.025 in]) (Kelleher and Larson 1989). Transverse cracks in both JPCP and JRCP also rely on aggregate interlock for good performance and may exhibit poor load transfer if aggregate interlock is not maintained.

Restoration of load transfer is expected to enhance pavement performance by reducing pumping, faulting, and corner breaks, and also by retarding the deterioration of transverse cracks. In most instances, the pumping and faulting mechanism can be corrected by installing joint load transfer devices. Diamond grinding of the pavement surface is often done in conjunction with LTR to restore rideability.

This chapter presents useful information associated with using LTR as an effective pavement preservation technique for concrete pavements. Specifically, this chapter focuses on identifying good candidate projects for LTR, recognizing the limitations and effectiveness of LTR, understanding the many material, design, and construction considerations, and identifying and remedying common construction problems. Another related technique that is discussed briefly in this chapter is cross-stitching. Cross-stitching is a preservation method designed to strengthen nonworking longitudinal cracks that are in relatively good condition (ACPA 2001a).

3. PURPOSE AND PROJECT SELECTION

Load Transfer Efficiency (LTE)

In order to select good candidate projects for LTR, it is first important to understand the concept of load transfer efficiency (LTE) and how to measure it. LTE is a quantitative measurement of the ability of a joint or crack to transfer load. LTE may be defined in terms of either *deflection* load transfer or *stress* load transfer. Deflection LTE is more commonly used since it can be easily measured on existing pavements with a falling weight deflectometer (FWD). The most common mathematical formulation for expressing deflection load transfer efficiency is:

$$\mathrm{LTE} = \frac{\Delta_{\mathrm{UL}}}{\Delta_{\mathrm{L}}} \times 100 \qquad (8.1)$$

where:

LTE = Load transfer efficiency.
Δ_{UL} = Deflection stress on the unloaded side of the joint.
Δ_L = Deflection stress on the loaded site of the joint.

The concept of deflection load transfer is illustrated in figure 8.1. If no load transfer exists, then the unloaded side of the joint experiences no deflection when the wheel is applied on the approach side of the joint, and the LTE computed from equation 8.1 is zero percent. If perfect load transfer exists, both sides of the joint experience the same magnitude of deflection under the wheel loading, and the LTE computed from equation 8.1 is 100 percent.

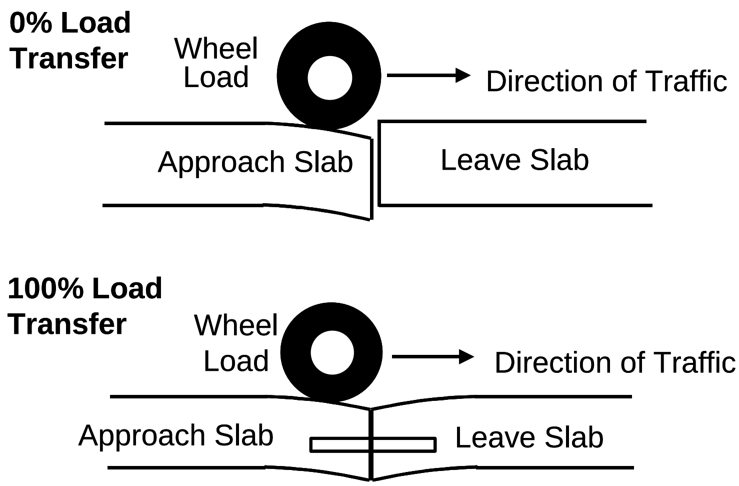

Figure 8.1. Illustration of deflection load transfer concept.

LTE should be measured during cooler temperatures (ambient temperatures less than 21 ºC [70 ºF]) and during the early morning when the joints will not be tightly closed. In addition, LTE must be determined using a device such as the FWD that is capable of applying loads comparable in magnitude and duration to that of a moving truck wheel load. LTE should be measured in the outer wheelpath, which is subject to the heaviest truck traffic wheel loads. Deflection measurements for the determination of LTE should be taken with sensors placed as close to the joint or crack as possible.

The magnitude of the corner deflections should be considered in addition to the LTE. It is possible for slab corners to exhibit very high deflections, yet still maintain a high LTE. In this case, even though the LTE is high, the large corner deflections can lead to pumping of the underlying base course material, faulting, and perhaps corner breaks. A useful parameter to help assess this is the differential deflection (DD), which is the relative displacement between the loaded and unloaded sides of the joint and is computed as follows:

$$DD = \Delta_L - \Delta_{UL} \qquad (8.2)$$

The DD should be computed along with the LTE over a project to gain a more complete understanding of the load transfer characteristics of a joint or crack. A recommended limit on the magnitude of differential deflection is 0.13 mm (5 mils) (Odden, Snyder, and Schultz 2003).

Selecting Candidate Projects for Load-Transfer Restoration

The following are general characteristics associated with good candidate pavements for LTR (FHWA/ACPA 1998):

- Pavements with structurally adequate slab thickness, but exhibiting significant loss of load transfer due to lack of dowels, poor aggregate interlock, or base/subbase/subgrade erosion.
- Relatively young pavements that, because of insufficient slab thickness, excessive joint spacing, inadequate steel reinforcement at transverse cracks, and/or inadequate joint load transfer, are at risk of developing faulting, working cracks, and corner cracks unless load transfer is improved.

In general, the pavement should be in relatively good condition with a limited amount of structural cracking (Bendaña and Yang 1993). Pavements exhibiting significant slab cracking, joint spalling, or materials-related distress such as D-cracking, should not be considered candidates for LTR (Larson, Peterson, and Correa 1998).

One set of recommendations on the condition of a joint or crack suitable for LTR is that it exhibits a deflection load transfer of 60 percent or less, faulting greater than 2.5 mm (0.10 in) but less than 6 mm (0.25 in), and differential deflection of 0.25 mm (0.01 in) (FHWA/ACPA 1998). The recommendation from Washington State is that LTR should be considered on pavements that have an average faulting between 3 mm (0.125 in) and 13 mm (0.5 in) and when the number of panels with multiple cracks is less than or equal to 10 percent (Pierce et al. 2003). Caltrans (2006) has similar requirements as Washington State, and also includes differential deflection (0.25 mm [10 mils] or more) and IRI (levels between 2.3 and 3.2 m/km [150 and 200 in/mi]) as additional consideration factors.

LTR may also be used in other applications, including at transverse cracks (if the cracks are fairly uniform and have not widened or started faulting) and in preparation for an overlay. In the former, LTR helps to maintain structural integrity and improves ride quality, and in the latter, LTR can help reduce the incidence and severity of reflection cracking, spalling, and deterioration of the overlay (and may also result in a thinner overlay thickness).

4. LIMITATIONS AND EFFECTIVENESS

Load transfer restoration is not a new rehabilitation technique. Georgia first explored LTR on a project on I-75 in 1980 (Gulden and Brown 1985), and Puerto Rico's experience goes back to at least 1983 (Larson, Peterson, and Correa 1998). In the past decade, many more highway agencies have tried or began using LTR as a pavement preservation technique.

An example of a successful dowel bar retrofit program can be found in Washington State. Since 1992, Washington State has used LTR to rehabilitate many miles of JPCP with good success (Pierce 1994; Pierce 1997). In 2002, after an assessment of the first 10 years of experience with dowel retrofit projects, Washington State reported that although some isolated distress has appeared on some of the earlier constructed projects, overall the dowel bar retrofit projects are performing very well (Pierce et al. 2003). Puerto Rico has also reported good performance on many miles of retrofitted dowel bars. A review of over 7,000 dowel bars installed 8 years earlier indicated that fewer than 0.5 percent of the repairs had failed (FHWA/ACPA 1998).

While there has been good documented success with this technique, a few states have experienced some problems with their initial LTR trials. For example, in 1999 and 2000, Wisconsin installed retrofitted dowel bars on portions of Interstate 39. In 2001, a review of these projects found that the patch material used to backfill the slots was deteriorating at the joints in many parts of the project (Bischoff and Toepel 2002). In response to these observed material problems, Wisconsin constructed 15 additional test sections and three control sections in 2001 to study patch materials, dowel bar materials, and the effects of sealed and unsealed joints on dowel bar retrofit projects (Bischoff and Toepel 2002). After 1 year of service, the performance of the test sections was reviewed. While two patching materials showed some debonding and microcracking due to shrinkage, the other patch materials were found to be performing well with no distresses (Bischoff and Toepel 2002).

To address the material shrinkage problem, Wisconsin conducted a follow-up study in which they were able to successfully modify their patching materials to reduce unwanted shrinkage (Bischoff and Toepel 2004). Because of the sensitivity of patching materials to loading and environmental conditions, it is extremely important to test and modify (if necessary) patching materials in the laboratory and on test sections, prior to using them on a wider scale in the field.

5. MATERIALS AND DESIGN CONSIDERATIONS

When designing a LTR project, it is important to determine what load transfer device will be used, what repair (filler) material will be used to fill the slots, and where to place the load transfer devices and in what configuration. This section summarizes the materials and dowel configurations recommended by industry and commonly used by many states.

Load Transfer Device Type

Many different types of load transfer devices have been used to restore load transfer across joints and cracks in existing concrete pavements. The most effective method, and the one currently recommended by the FHWA, is the placement of smooth, round dowel bars in small slots cut across transverse joints in the pavement (FHWA/ACPA 1998). This has proven to be an effective method of restoring load transfer in a variety of concrete pavement projects (Darter, Barenberg, and Yrjanson 1985; Gulden and Brown 1985; Gulden and Brown 1987; Pierce 1994; Pierce 1997; Pierce et al. 2003).

The required size of the dowel bars is dependent on the pavement thickness. A minimum dowel bar length of 350 mm (14 in) is recommended to allow for at least 150 mm (6 in) of embedment on each side of the joint or crack, adequate room for an expansion cap on each end of the dowel bar, and reasonable placement tolerances (ACPA 2006). A complete summary of the recommended dowel size requirements for dowel bar retrofit projects is presented in table 8.1.

Table 8.1. Dowel size requirements for dowel bar retrofit projects (ACPA 2006).

Pavement Thickness, mm (in)	Diameter, mm (in)	Minimum Length, mm (in)	Spacing, mm (in)
< 200 (< 8)	25 (1.0)	350 (14)	300 (12)
200 to 240 (8 to 9.5)	32 (1.25)	350 (14)	300 (12)
250 + (10 +)	38 (1.5)	350 (14)	300 (12)

Repair (Filler) Materials

The repair or filler material is the substance used to encase the load transfer device in the existing pavement. Desirable properties of the repair material include little or no shrinkage, thermal compatibility with the surrounding concrete (e.g., similar coefficients of thermal expansion), good bond strength with the existing (wet or dry) concrete, and the ability to rapidly develop sufficient strength to carry the required load so that traffic can be allowed on the pavement in a reasonable period of time. To aid in this process, many agencies maintain a qualified product list of suitable repair materials.

The patch material is the most critical factor in the placement of retrofitted load transfer devices (ACPA 2006). Generally, materials found to work well for partial-depth repairs (as described in Chapter 5) also work well as a repair or backfill material for LTR (FHWA/ACPA 1998). One of the most important factors to control is the water content of the patching material in order to reduce the probability of shrinkage cracks and debonding (Rettner and Snyder 2001). Table 8.2 summarizes recommended tests and material properties for suitable repair materials (Jerzak 1994). It is important that these materials be tested for freeze-thaw durability to ensure long-term performance.

Table 8.2. Recommended properties of repair materials (Jerzak 1994).

Property	Test Procedure	Recommended Value
Neat Material		
Compressive Strength, 3 hr	ASTM C-109	Minimum 21 MPa (3046 lbf/in^2)
Compressive Strength, 24 hr	ASTM C-109	Minimum 34 MPa (4931 lbf/in^2)
Abrasion Loss, 24 hr	California Test 550	Maximum loss 25 g (0.06 lbm)
Final Set Time		Minimum 25 minutes
Shrinkage, 4 days	ASTM C-596	Maximum 0.13 percent
Soluble Chlorides	California Test 422	Maximum 0.05 percent
Soluble Sulfates as SO_4	California Test 417	Maximum 0.25 percent
Maximum Extended Material		
Flexural Strength, 24 hr	California Test 551	Minimum 3.4 MPa (493 lbf/in^2)
Bond to Dry PCC, 24 hr	California Test 551	Minimum 2.8 MPa (406 lbf/in^2)
Bond to SSD PCC, 24 hr	California Test 551	Minimum 2.1 MPa (305 lbf/in^2)
Absorption	California Test 551	Maximum 10 percent

Portland Cement Concrete

Portland cement concrete (PCC) is commonly used as a repair material for LTR. It is cheaper than other materials, is widely available, and presents no thermal compatibility problems with its use. Many mixes use a Type III cement and an accelerator to improve setting times and reduce shrinkage. Sand and an aggregate with 9.5 mm (0.375 in) maximum size are commonly used to extend the yield of the mix.

Rapid-Setting Proprietary Materials

Several proprietary materials are available for use as a repair material for LTR. The main advantage of these types of materials is that they are quick-setting, thereby allowing earlier opening times to traffic. State highway agencies typically maintain a list of approved proprietary products for use in pavement construction. It is strongly recommended that any patch material without an acceptable history of performance under similar conditions of load and environment be tested in the laboratory for specification compliance before being used in the field. Also, it is critical that all manufacturer's instructions be followed when working with these proprietary materials (ACPA 2006).

Epoxy-Resin Adhesives

Epoxy-resin adhesives have been used to improve the bond between the existing concrete and the repair materials. Epoxy-resin adhesives should meet the requirements of AASHTO M 235 and the manufacturer's recommendations should be closely followed.

Dowel Bar Design and Layout

In order for the retrofitted dowel bars to be effective, they must be of sufficient size and placed in a suitable configuration. While the recommended dowel dimensions are discussed in table 8.1, the recommended dowel configuration is presented in this section. Currently it is recommended that three to four dowels (spaced 300 mm [12 in] apart) be used in each wheelpath, with the outermost dowel being 300 mm (12 in) from the lane edge, except where tiebars from adjacent lanes or shoulders are encountered (ACPA 2006). An illustration of this recommended dowel bar configuration is presented in figure 8.2.

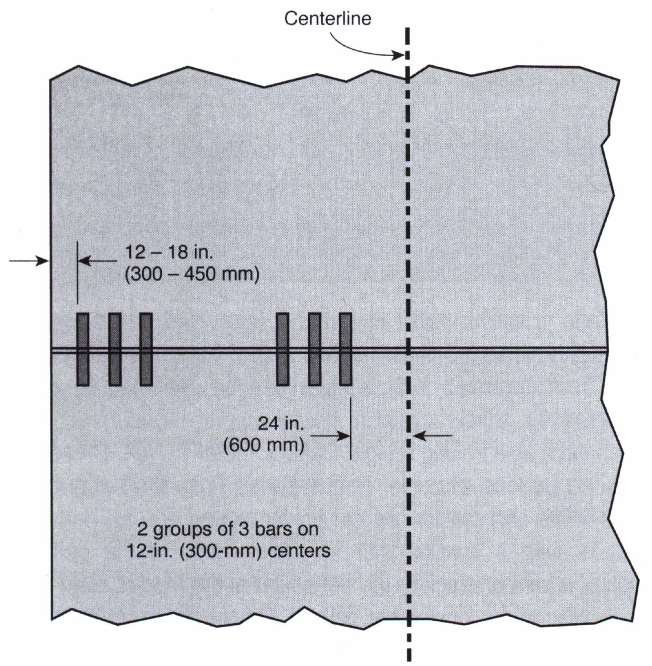

Figure 8.2. Recommended dowel bar configuration (ACPA 2006).

A second design consideration is the dimensions of the slots themselves. The slot must be sufficiently long to enable the dowel to lie flat across the bottom of the slot without hitting the curve of the saw cut; this typically requires the surface length of the saw cut to be 1 m (3 ft) for a 350-mm (14-in) long dowel bar (FHWA/ACPA 1998). The width of the slot is typically 65 mm (2.5 in). The created slot should be deep enough to position the centerline of the dowel at the mid-depth of the slab, allowing a clearance of approximately 13 mm (0.5 in) beneath the dowel bar for placement on chairs. The bottom of the slot should also be flat and uniform across the joint. Figure 8.3 shows an illustration of the slot details.

6. CONSTRUCTION CONSIDERATIONS

The completion of a LTR project involves the following steps:

1. Slot creation.
2. Slot preparation.
3. Dowel bar placement.
4. Repair material placement.
5. Diamond grinding (optional).
6. Re-establishment of joint and joint sealing.

A subset of these construction procedures are illustrated in figure 8.4. Detailed design and construction guidelines are provided by FHWA/ACPA (1998), and Larson, Peterson, and Correa (1998).

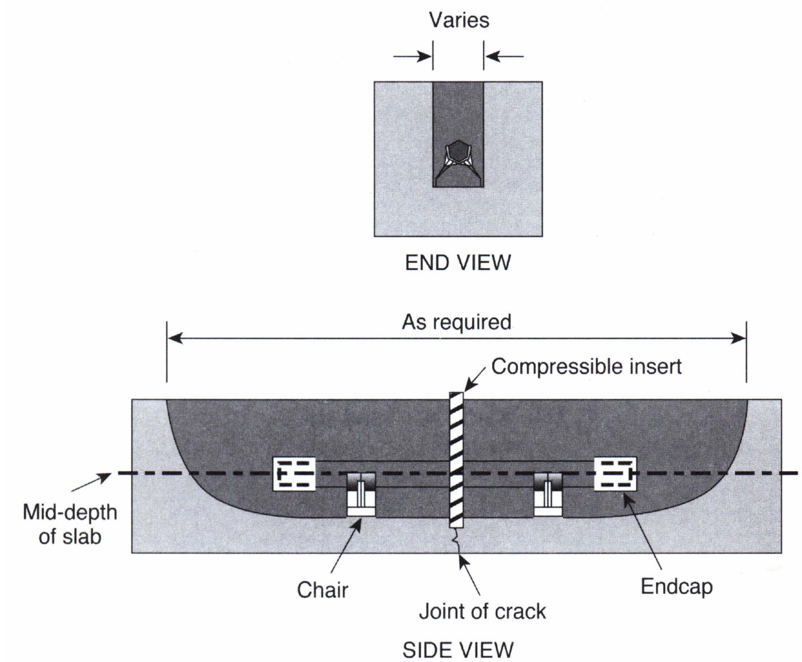

Figure 8.3. Retrofitted dowel installation details (ACPA 2006).

Step 1: Slot Creation

The recommended method of creating slots for dowel bar retrofit projects is with a diamond-bladed slot cutting machine. While modified milling machines have been used in the past to create slots, the International Grooving and Grinding Association (IGGA) and the American Concrete Pavement Association (ACPA) do not currently support the use of this technique for creating slots (ACPA 2001b).

Diamond saw slot cutters make two parallel cuts for each dowel slot; the "fin" area between the cuts is then broken up with a light jackhammer. Diamond saw slot cutters have been developed that can cut either three or six slots (in one or two wheelpaths) at the same time (FHWA/ACPA 1998). Production rates for this method of slot cutting can exceed 2,500 slots per day. It is important that the slots be parallel to the centerline of the pavement and that the resulting slots be cut to the prescribed depths, widths, lengths, and spacings.

Step 2: Slot Preparation

After the saw cuts have been made, lightweight jack hammers (less than 14 kg [30 lb]) or hand tools are used to remove the concrete in each slot. Jackhammers should not be used in a vertical plane (i.e., perpendicular to the pavement surface) due to the increased chance of the jackhammer punching through the bottom of the slot (Pierce et al. 2003). After removing the concrete wedge, the bottom of the slot must be flattened with a small hammerhead mounted on a small jackhammer.

Once the jackhammering operations are completed, the slots are thoroughly sandblasted to remove dust and sawing slurry and to provide a good surface to which the repair material can bond. This is followed by airblasting and a final check for cleanliness before the dowel and patch material are placed. High-pressure water blasting has also been used successfully to clean slots (Pierce et al. 2003).

Prior to the placement of the dowels or patch material, the joint or crack in the slot is caulked with a silicone sealant to prevent intrusion of any patch material that might cause a compression failure. The sealant should not extend 13 mm (0.5 in) beyond the joint because excessive sealant will not allow the repair (filler) material to bond to the sides of the slot.

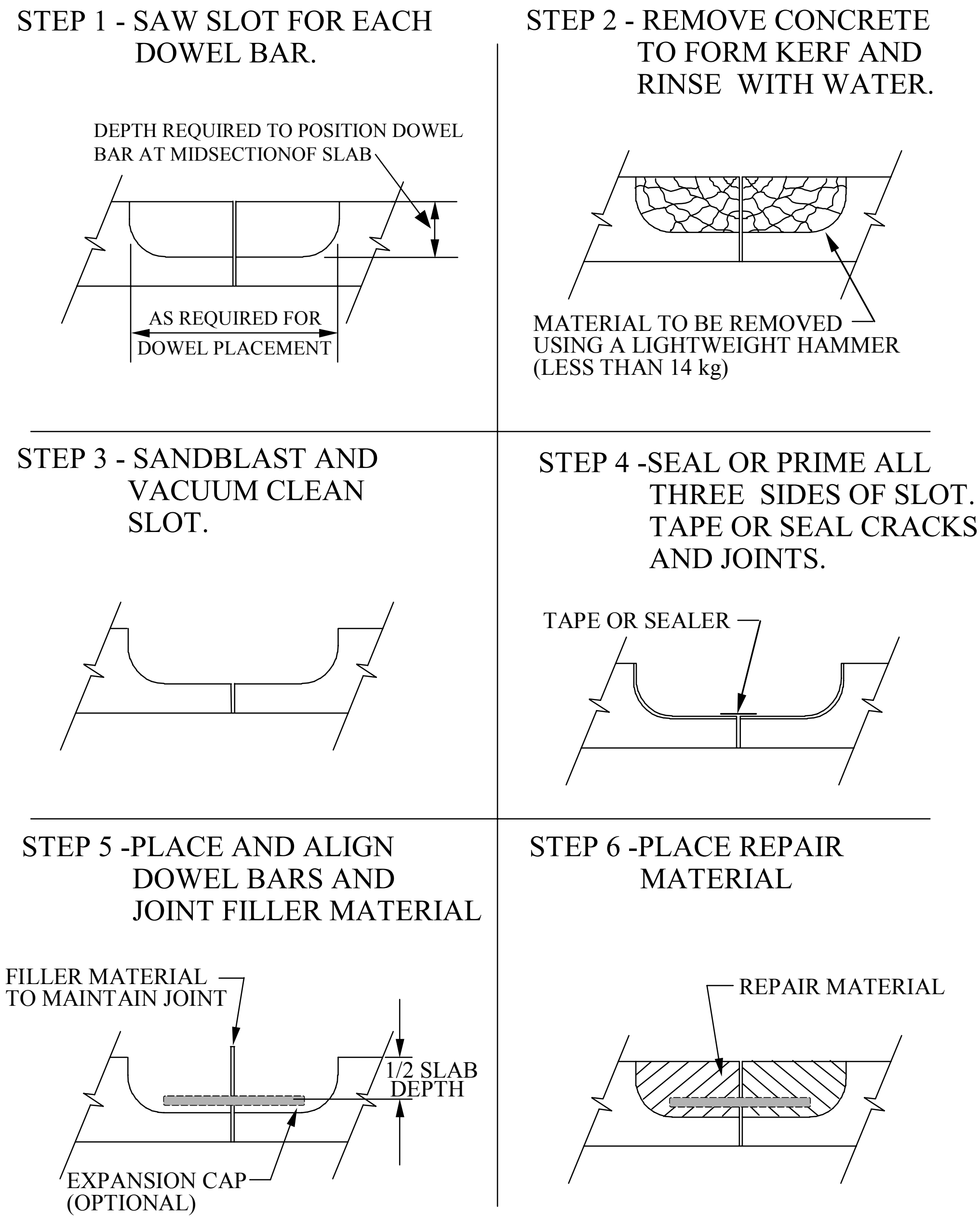

Figure 8.4. Construction procedures for retrofitted dowel bar installation (Larson, Peterson, and Correa 1998).

Step 3: Dowel Bar Placement

The dowel bars should be coated with a bond breaking material (e.g., curing compound or a manufacturer-supplied material) along their full length to facilitate joint movement. Expansion caps can be placed at both ends of the dowel to allow for any joint closure after installation of the dowel. Dowels are typically placed on support chairs (non-metallic or coated to prevent corrosion) and positioned in the slot so that the dowel rests horizontally and parallel to the centerline of the pavement at mid-depth of the slab. The proper alignment of the dowel bar is critical to its effectiveness. A filler board or expanded polystyrene foam material must be placed at the mid-length of the dowel to prevent intrusion of the repair material into the joint or crack (causing point bearing forces), as well as to help form the joint in the slot (ACPA 2006).

Step 4: Repair Material Placement

Once the dowel has been placed and the filler board material is in position, the repair material is then placed in the slot according to the manufacturer's recommendations. The repair material should be placed in a manner that will not cause movement of the dowel bar within the slot (i.e., the repair material should not be dumped onto the slots) (Pierce et al. 2003). Placing the repair material on the surface adjacent to the slot and shoving it towards and into the slot results in minimal dowel movement (Pierce et al. 2003).

A small spud vibrator (i.e., ≤ 25 mm [1.0 in] in diameter) should be used to consolidate the patching material. The patching material in the dowel bar slots should not be overworked, which would otherwise cause migration of the fine material to the surface (Pierce et al. 2003).

After consolidation and finishing, a curing compound should be placed on the patching material to minimize shrinkage. Depending upon the type of repair material, the pavement may be opened to traffic in as little as a few hours. Recent studies have shown that the minimum compressive strength required to open a repair to traffic is about 13.7 MPa (2,000 lbf/in^2) for slabs 200 mm (8 in) or thicker (FHWA/ACPA 1998).

Step 5: Diamond Grinding (Optional)

Rehabilitation techniques such as load-transfer restoration may result in increased roughness if not finished properly. This is typically due to differences in elevation between the finished repair area and the existing pavement, or perhaps due to shrinkage or settlement of the repair material. Consequently, after the installation of retrofitted dowel bars, the entire pavement project is often diamond ground to provide a smooth-riding surface.

Step 6: Re-Establishment of Joint and Joint Sealing

After the material has cured and the surface diamond ground, the transverse joint should be re-established by sawing over the length of the joint and through the filler board. The joint should then be prepared and sealed as described in Chapter 10.

7. QUALITY CONTROL

As with any pavement project, the performance of LTR projects is greatly dependent on the quality of the utilized materials and construction procedures. Paying close attention to this quality during construction greatly increases the chances of minimizing premature failures on LTR projects. An excellent source for QC recommendations is a recently published paper in which a comprehensive set of construction-related recommendations and lessons-learned are summarized (Pierce et al. 2003). These recommendations are based on 10 years of LTR experience in Washington State. The remainder of this section is largely based on the recommendations summarized in the report by Pierce et al. (2003) and in the FHWA's *Dowel-Bar Retrofit for Portland Cement Concrete Pavements Checklist* (FHWA 2005).

Preliminary Responsibilities

Prior to the start of construction procedures, the agency should review pertinent project-related documents, the project's current condition, and materials to be used on the project. The following specific lists of items are provided as a model QC checklist for these preliminary items.

Document Review

All of the following documents should be reviewed prior to the start of any construction activities. Any suspected problems should be identified and reconciled as part of the preliminary review process.

- Bid/project specifications and design.
- Special provisions.
- Agency application requirements.
- Traffic control plan.
- Manufacturer's installation instructions for patch materials.
- Material safety data sheets (MSDS).

Project Review

An updated review of the current project's condition is warranted to ensure that the project is still a viable candidate for LTR. Specifically, the following should be verified or checked as part of the project review process:

- Verify that the pavement conditions have not significantly changed since the project was designed.
- Verify that the pavement is structurally sound. A significant amount of slab cracking and/or corner breaks are indicators of structural deficiencies.
- Check estimated quantities for dowel bar retrofit.

Review of Materials

In preparation for the construction project, the following list summarizes many of the material-related checklist items that should be checked or reviewed:

- Verify that dowel slot cementing grout meets specification requirements.
- Verify that the dowel slot cementing grout is from an approved source or listed on agency qualified products list (QPL) (if required).
- Verify that the component materials for the dowel slot cementing grout have been sampled, tested, and approved prior to installation as required by contact documents.
- Verify that the additional or extender aggregates have been properly produced, with acceptable quality.
- Verify that the material packaging is not damaged (i.e., leaking, torn, or pierced).
- Verify that caulking filler meets specification requirements.
- Verify that dowels, dowel bar chairs, and endcaps meet specification requirements.
- Verify that dowel bars are properly coated with epoxy (or other approved material) and free of any minor surface damage in accordance with contract documents.
- Verify that curing compound meets specification requirements.
- Verify that joint/crack re-former material (compressible insert) meets specification requirements (typically polystyrene foam board, 12 mm [1/2 in] thick).
- Verify that joint sealant material meets specification requirements.
- Verify that all sufficient quantities of materials are on hand for completion of project.
- Ensure that all material certifications required by contract documents have been provided to the agency prior to construction.

Inspection of Equipment

Prior to beginning construction, all construction equipment must be examined. The following are equipment-related items that should be checked:

- Verify that the slot sawing machine is of sufficient weight, horsepower, and configuration to cut the specified number of slots per wheelpath to the depth shown on the plans.
- Verify that jackhammers for removing concrete are limited to a maximum rated weight of 14 kg (30 lb).
- Verify that the sandblasting unit for cleaning slots is adjusted for correct sand rate and has oil and moisture filters/traps.
- Verify that air compressors have sufficient pressure and volume to adequately remove all dust and debris from slots and meet agency requirements.
- For auger-type mixing equipment used to mix repair materials, ensure that auger flights or paddles are kept free of material buildup, which can cause inefficient mixing operations.
- Ensure that volumetric mixing equipment (e.g., mobile mixers) are kept in good condition and calibrated on a regular basis to properly proportion mixes.
- Ensure that material test equipment required by the specifications is all available on site and in proper working condition (e.g., slump cone, pressure-type air meter, cylinder molds and lids, rod, mallet, ruler, 3-m [10-ft] straightedge).
- Verify that vibrators are the size specified in the contract documents (typically 25 mm [1 in] in diameter or less) and are operating correctly.
- Verify that the concrete testing technician meets the requirements of the contract document for training/certification.
- Ensure that sufficient storage area is available on the project site specifically designated for the storage of concrete cylinders.

Weather Requirements

The weather conditions at time of construction can have a large impact on the performance of the LTR technique. Specifically, the following weather-related items should be checked immediately prior to construction:

- Review manufacturer installation instructions for requirements specific to the repair material used.
- Air and surface temperature meets manufacturer and all agency requirements (typically 4 °C [40 °F] and above) for concrete placement.
- Neither dowel bar installation nor patching should proceed if rain is imminent.

Traffic Control

To manage the flow of traffic through the work zone, the following traffic-related items should be checked or verified:

- Verify that the signs and devices used match the traffic control plan presented in the contract documents.
- Verify that the set-up complies with the Federal or local agency *Manual on Uniform Traffic Control Devices* (MUTCD) or local agency procedures.

- Verify that flaggers are trained/qualified according to contract documents and agency requirements.
- Verify that unsafe conditions, if any, are reported to a supervisor.
- Ensure that traffic is not opened to the repaired pavement until the backfill material has attained the specified strength as required by the contract documents.
- Verify that signs are removed or covered when they are no longer needed.

Project Inspection Responsibilities

Cutting Slots

During the slot creation construction step, the inspector should ensure that:

- Slots are cut parallel to each other, and to the centerline of the roadway within the maximum tolerance permitted by the contract documents, typically 6 mm (1/4 in) per 300 mm (12 in) of dowel bar length.
- The number of slots per wheelpath (typically 3 or 4) is in agreement with contract documents.
- Slots are aligned to miss any existing longitudinal cracks.
- The cut slot length extends the proper distance on each side of the joint/crack, as required by the contract documents. This is especially important for skewed joints and cracks.
- Slots are sawed to sufficient depth so that the center of the dowel bar is placed at the mid-depth of the pavement. Slots that are cut too deep will contribute to corner cracks when traffic loads are applied.
- Slot widths should be sized to be the exact width of the dowel bar chairs.

Removing Material from Slots

It should be verified that concrete fin removal is conducted with only lightweight 14 kg (30 lb) jackhammers. During the process of removing material from the slots, the contractor should take extra care to prevent the jackhammer from punching through the bottom of the slot. The bottom of the slots should then be smoothed and leveled using a lightweight bush hammer.

Slot Cleaning and Preparation

The following should be closely inspected when cleaning the slots and the adjacent area and preparing the slots prior to the placement of the dowels:

- After concrete removal, the slots should be prepared by sandblasting. A physical check of the slot's cleanliness (using a tool such as a scraper) should be made to ensure no slurry residue remains on the sides of slots.
- After sandblasting, the slots should be cleaned using air blasting. A second air blasting may be required immediately before placement of dowel slot cementing grout if slots are left open for a duration exceeding that permitted in the contract documents.
- Concrete chunks, dirt, debris, and slurry residue should be cleaned 1.0 to 1.2 meters (3 to 4 feet) away from the slot's perimeter. This practice minimizes the possibility of reintroducing unwanted material into the slot during subsequent operations.
- The existing joint/crack is sealed with an approved caulking filler material along the bottom and sides of the slot to prevent the repair material from entering the joint/crack. Special care must be taken to ensure that the sealant does not extend 13 mm (0.5 in) beyond the joint (i.e., into the slot).

Placement of Dowel Bars

During the placement of dowels into the cut slots, QC inspections should ensure that:

- Plastic endcaps are placed on each end of the dowel bar to account for pavement expansion as required by the contract documents.
- Dowel bars are completely coated with an approved compound prior to placing into chairs. Dowel bars that have a factory-applied coating should be free of dirt and debris, and free of nicks and abrasions. The factory-applied coating should be clearly visible; otherwise, an additional application of an approved material must be applied. Dowel bars should not be coated once they have been placed in the slots as the sides and bottom of the slots will become contaminated.
- Proper clearance is maintained between the supported dowel bar and the sidewalls, ends, and bottom of the cut slot in accordance with contract documents.
- Joint forming material (foam core insert) is placed at mid-point of each bar and in line with the joint/crack, to allow for expansion and to re-form the joint/crack.
- The chairs placed on the dowel bars are strong enough to allow full support of the dowel bar. Chairs should allow at least 13 mm (0.5 in) clearance between the bottom of the dowel and the bottom of the slot.
- End caps allow at least 6 mm (0.25 in) of movement at each end of the bar. End caps placed on each end of the bar reduce the risk of dowel bar lockup at negligible extra cost.
- Dowels are centered across the joint/crack such that at least 175 mm (7 in) of the dowel extends on each side.
- Dowels are placed within the following tolerances:
  - Placed within 25 mm (1 in) of the center of the existing pavement depth.
  - Centered over the transverse joint with a minimum embedment of 175 mm (7 in).
  - Placed parallel to the centerline and within the plane of the roadway surface.
  - Placed to a horizontal tolerance of ± 13 mm (0.5 in), vertical tolerance of ± 13 mm (0.5 in), and skew from parallel (per 450 mm [18 in]) of ± 13 mm (0.5 in). Dowel bars placed outside of the acceptable tolerances can cause joint lock up that leads to cracking.

Mixing, Placing, Finishing, and Curing of Repair Material

To achieve a well-performing LTR project, it is imperative that good methods and procedures be used when mixing, placing, finishing, and curing the chosen repair material. Specifically, the following should be ensured during construction:

- Repair materials are being mixed in accordance with the material manufacturer's instructions.
- Quantities of repair materials being mixed are small, to prevent material from setting prematurely.
- Concrete surfaces, including the bottom of the slot, are dry.
- Material is consolidated using small, hand-held vibrators, which do not touch the dowel bar assembly during consolidation. Inspectors should also ensure that the grout material is not over consolidated. Each slot should only require two to four short, vertical penetrations of a small diameter spud vibrator.
- Repair material is finished flush with surrounding concrete, using an outward motion to prevent pulling material away from patch boundaries. Material is finished slightly "humped" if diamond grinding is to be employed.

- Adequate curing compound is applied immediately following finishing and texturing.

Clean Up

After the LTR construction procedures are complete, all remaining concrete pieces and loose debris on the pavement should be removed. Old concrete should be disposed of in accordance with agency specifications. Mixing, placement, and finishing equipment should be properly cleaned in preparation for their next use.

Diamond Grinding

If diamond grinding is specified for use in combination with a LTR project, the grinding should be completed within 30 days of the placement of the repair material.

Resealing Joints/Cracks

Inspectors should ensure that the joints/cracks are resealed after diamond grinding (if specified) in accordance with agency specification.

8. TROUBLESHOOTING

This section summarizes some of the more common problems that a contractor or inspector may encounter in the field during construction (see table 8.3) and performance problems that may be observed later (see table 8.4). Also included in the list are performance problems that may develop shortly after the project is completed and opened to traffic. Recommended solutions associated with known problems are also provided.

9. CROSS-STITCHING

Introduction

Cross-stitching is a preservation method designed to strengthen nonworking longitudinal cracks that are in relatively good condition (ACPA 1995). The construction process consists of grouting tiebars into holes drilled across the crack at angles of 35º to 45º to the pavement surface (see figure 8.5). This process is effective at preventing vertical and horizontal movement or widening of the crack or joint, thereby keeping the crack tight, maintaining good load transfer, and slowing the rate of deterioration.

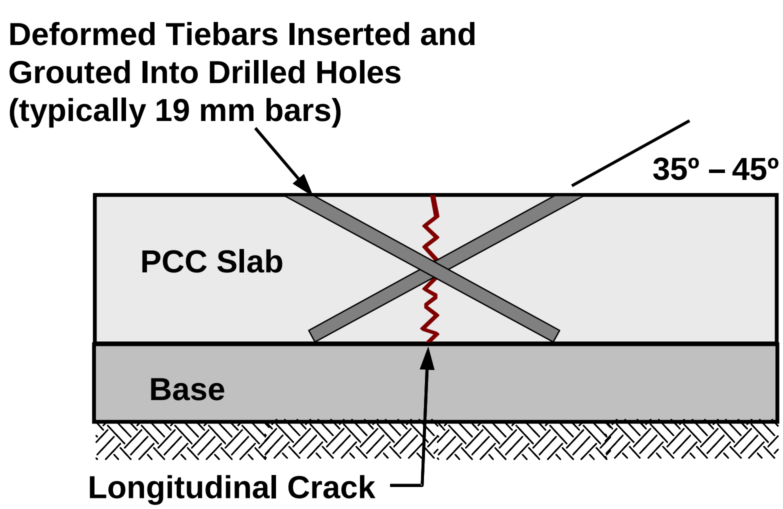

Figure 8.5. Cross-stitching of longitudinal crack (ACPA 1995).

Table 8.3. Potential LTR-related construction problems and associated solutions (FHWA 2005; ACPA 2006).

Problem	Typical Cause(s)	Typical Solution(s)
Slots are not cut parallel to the roadway centerline.	Improper alignment of slot cutting machine.	Misaligned dowels can cause joint/crack lock up that will lead to slab cracking. Fill the original slots with PCC and recut at different locations (note: if the material between the sawcuts has not been removed, fill the sawcuts with an epoxy resin and recut at different locations). The use of a multiple saw slot cutting machine can ensure that slots are parallel to each other.
Dowel bar slots are too shallow.	Improper slot cutting techniques.	If a slot is too shallow, the dowel cannot be placed in its proper place in relation to the center of the slab. The solution is to saw the slots deeper, remove the concrete to the proper depth, and complete as specified.
Dowel bar slots are too deep.	▪ Improper slot cutting techniques. ▪ Improper jackhammer weight. ▪ Improper jackhammering technique.	If dowels are placed in slots that are too deep, corner cracks may develop when traffic loads are applied. The solution is to fill the original slots or sawcuts and recut at different locations). To minimize the probability of creating slots that are too deep: 1. Use a lightweight jackhammer (14 kg [30 lb]). 2. Do not lean on the jackhammer. 3. Do not orient the jackhammer vertically; use a 45° angle and push the tip of the hammer along the bottom of the slot. 4. Stop chipping within 50 mm (2 in) of the bottom of the pavement.
Concrete fin is not easily removed.	Concrete could contain mesh reinforcement.	If mesh reinforcement is observed in the concrete, sever the steel at each end before attempting to remove the fin of concrete.
Jackhammer punching through bottom of slot.	Improper jackhammering technique or extremely deteriorated PCC.	Make a full-depth repair across the entire lane width at the joint/crack.
Areas on dowel where factory-applied dowel coating is missing.	Non-uniform application of the factory-applied dowel coating or mishandling of dowels in field.	Areas of exposed steel can become concentrated points for corrosion that can eventually lead to the lockup of the dowel. If observed, recoat dowel with manufacturer-approved coating substance prior to the placing of the dowel in the slot. Do not coat dowels in the slots as the sides and bottom of the slots may become contaminated.
Dowel cannot be centered over joint/crack because slot does not extend far enough.	Improper slot preparation.	Chip out additional slot length with a jackhammer to facilitate proper placement of the dowel in accordance with contract documents. Typically at least 175 mm (7 in) of each 350 mm (14 in) dowel extend on each side of the joint/crack. Properly sized chairs will fit snugly into the slot.
Joint/crack caulking filler material in the joint does not extend all of the way to the edge of the slot.	Improper caulk installation.	Improperly placed caulking in the joint can allow incompressible repair material to enter the joint; therefore, increasing the probability of a compression failure. Extend the caulking to the edge of the slot prior to the placement of repair material. If repair material does enter the joint adjacent to the slot, it must be removed using a technique agreed upon by the agency and the contractor.
Caulking material in joint or crack extrudes into a slot more than 13 mm (0.5 in).	Improper caulking installation.	Excessive caulking will not allow the repair (filler) material to bond to the sides of the slot. Therefore, remove excess caulking before placing repair material.
Dowels are misaligned after vibration	▪ Vibrator contact with dowel assembly. ▪ Over vibration of material. ▪ Improper width of slots.	1. Do not allow the vibrator to touch the dowel assembly. 2. Check for over vibration; each slot should require only two to four short, vertical penetrations of a small diameter spud vibrator. 3. Ensure that the slots are sized the exact width of the plastic dowel bar chairs.

Table 8.4. Potential LTR-related performance problems and prevention techniques.

Problem	Typical Cause(s)	Typical Solution(s)
Cracking of in-place patch material.	▪ Joint is not well isolated. ▪ Dowels are not all properly aligned. ▪ Patch material too strong. ▪ Patch opened to traffic too soon. ▪ Used material encountered too much shrinkage.	Confirm proper construction practices are followed; patch material used is resistant to cracking.
Pop out of patch material.	▪ Slot is not properly cleaned or prepared. ▪ Improper curing (i.e., unexpected material shrinkage during curing)	Verify proper construction procedures are followed.
Wearing off of patch material.	Some materials are not very durable, or don't perform well if not properly mixed and handled.	Check material specifications, material preparation, and placement conditions to be sure that material is being handled properly.

Cross-stitching was first used on a U.S. highway by the Utah Department of Transportation (UDOT) in 1985 (ACPA 2001a). UDOT engineers used cross-stitching to strengthen uncontrolled cracks on a new 229-mm (9-in) JPCP design on I-70 in central Utah. Considerable reflection cracking from the 102-mm (4-in) lean concrete base occurred soon after construction. The cracks of major concern were the longitudinal cracks in or near the wheelpaths of the driving lanes. In February 2000, after 15 years of service, a review of this project found the pavement to be in generally good condition, with some faulting across nondoweled transverse contraction joints. The performance of cross-stitched cracks was favorable in most areas, except those with the highest degree of deterioration.

Purpose and Application

Cross-stitching is applicable for a number of situations where strengthening cracks or joints is required, including the following (ACPA 2001a):

- Strengthening longitudinal cracks in slabs to prevent slab migration and to maintain aggregate interlock.
- Mitigating the issue of tiebars being omitted from longitudinal contraction joints (due to construction error).
- Tying roadway lanes or shoulders that are separating and causing a maintenance problem. However, do not use cross-stitching to tie "new" lanes.
- Tying centerline longitudinal joints that are starting to fault.

Cross-stitching is not recommended for use on transverse cracks, especially those that are working because cross-stitching does not allow movement. If used on working transverse cracks, a new crack will likely develop near the stitched crack, or the concrete will spall over the reinforcing bars (ACPA 1995). Also, experience demonstrates that stitching is not a substitute for slab replacement if the degree of cracking is too severe, such as when slabs have multiple cracks or are shattered into more than 4 to 5 pieces (ACPA 2006).

In cases where you need to tie drifted slabs together, it is not necessary to attempt to move the drifted slabs together before cross-stitching. The primary concern in this case is preventing the backfill material (either epoxy or grout) from flowing into the space between the slabs (ACPA 2006). For these cases, a sand cement grout is a suitable backfill for this purpose (ACPA 2006).

Construction Considerations

Cross-stitching generally uses a 19-mm (0.75-in) diameter deformed tie bar to hold the crack tightly together and enhance aggregate interlock (ACPA 2001a). The bars are typically spaced at intervals of 500 to 750 mm (20 to 30 in) along the crack, and are alternated to each side of the crack (see figure 8.6). Heavy truck traffic typically requires a 500-mm (20-in) spacing while a 750-mm (30-in) spacing is adequate for light traffic and interior highway lanes (ACPA 1995). A properly drilled hole is one that intersects the crack at mid-depth (ACPA 1995). Recommendations on cross-stitching bar dimensions and angles/locations of holes are presented in table 8.5.

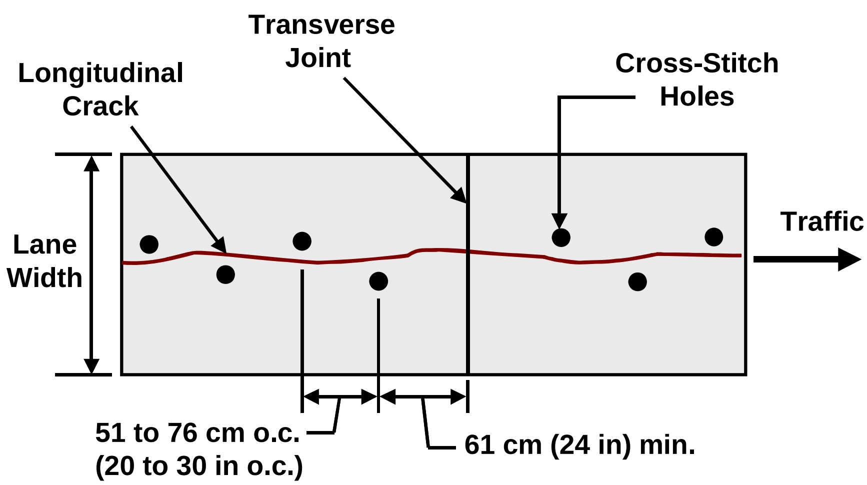

Figure 8.6. Schematic of cross-stitch tiebar installation (ACPA 2001a).

Table 8.5. Cross-stitching bar dimensions and angles/locations or holes (ACPA 2006).

Angle	Slab Thickness, in (mm)							
	7 (175)	8 (200)	9 (225)	10 (250)	11 (275)	12 (300)	13 (325)	14 (350)
	Distance from Crack to Hole, in (mm)							
35°	5.00 (125)	5.75 (145)	6.50 (165)	7.25 (180)	7.75 (195)	8.50 (210)	—	—
40°	—	—	—	—	6.50 (165)	7.25 (180)	7.75 (195)	8.25 (205)
45°	—	—	—	—	—	6.00 (150)	6.50 (165)	7.00 (175)
	Length of Bar, in (mm)							
35°	8.00 (200)	9.50 (240)	11.00 (275)	12.50 (315)	14.50 (365)	16.00 (400)	—	—
40°	—	—	—	—	12.50 (315)	14.00 (350)	16.00 (400)	18.50 (465)
45°	—	—	—	—	—	12.00 (300)	14.00 (350)	16.50 (415)
	Diameter of Bar, in (mm)							
	0.50 (13)	0.75 (19)	0.75 (19)	0.75 (19)	0.75 (19)	0.75 (19)	1.0 (25)	1.0 (25)

The process of cross-stitching requires the completion of the following steps and considerations (ACPA 2001a; ACPA 2006):

- Drill holes at an angle so that they intersect the crack at mid-depth (it is important to start drilling the hole at a consistent distance from the crack, in order to consistently cross the crack at mid-depth). Select a drill that minimizes damage to the concrete surface, such as a hydraulic powered drill, and select a drill diameter no more than 9.5 mm (0.375 in) larger than the tiebar diameter.
- Blow air into the holes to remove dust and debris after drilling.
- Pour epoxy into the hole, leaving some volume for the bar to occupy the hole.
- Insert the tiebar.
- Remove excess epoxy and finish flush with the pavement surface.

The pavement may be reopened to traffic as soon as the epoxy has fully set.

10. SUMMARY

This chapter provides guidance for properly designing and installing retrofitted dowel bars in concrete pavements. These devices are intended to restore load transfer across joints or cracks that exhibit poor load transfer from one side of the joint or crack to the other.

Load transfer restoration is seeing more widespread use as benefits have been found to include a reduction in faulting rates, improvements to overall pavement performance, and extensions to pavement life. Currently, only the use of dowel bars placed in slots are recommended, because they have a good long-term performance record, are reliable, and are effective in reducing faulting.

Pavements most suited to dowel bar retrofitting are those that are in relatively good condition (little or no distress), but are exhibiting poor load transfer. The optimum time for the application of this strategy is when the pavement is just beginning to exhibit signs of distress, such as pumping or the onset of faulting.

While the chapter primarily focuses on the details of the load-transfer restoration technique, the chapter also contains a brief discussion on the pavement cross-stitching technique, which is used primarily to strengthen nonworking longitudinal cracks that are in relatively good condition (ACPA 2001a).

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NOTES

CHAPTER 9. DIAMOND GRINDING AND GROOVING

1. LEARNING OUTCOMES

This chapter describes recommended procedures for surface restoration of concrete pavements. Upon completion of this chapter, the participants will be able to accomplish the following:

1. Differentiate between diamond grinding and diamond grooving, and list the benefits of each.
2. Identify appropriate blade spacing dimensions for grinding and grooving.
3. Describe recommended construction procedures.
4. Identify typical construction problems and remedies.

2. INTRODUCTION

Diamond grinding and diamond grooving are two different surface restoration procedures that are used to correct concrete pavement surface distresses or deficiencies. Each technique addresses a specific pavement shortcoming, and each may be used in conjunction with other pavement preservation techniques as part of a comprehensive pavement preservation program. In some situations, it may be justified to use one of these techniques as the sole preservation technique.

3. PURPOSE AND PROJECT SELECTION

Diamond Grinding

Diamond grinding is the removal of a thin layer of hardened concrete pavement surface using closely spaced, diamond saw blades mounted on a rotating shaft. Diamond grinding is primarily conducted to restore or improve ride quality by eliminating surface irregularities. Restoring ride quality improves pavement load-carrying capacity and adds value to an in-place pavement (ACPA 2000).

Diamond grinding was first used in California in 1965 on a 19-year old section of Interstate 10 to eliminate significant faulting (Neal and Woodstrom 1976). In 1983, concrete pavement restoration (CPR) was conducted on this same pavement section, including the use of additional grinding to restore the rideability and skid resistance of the surface. In addition to diamond grinding, this CPR project included slab replacement, spall repair, and installation of edge drains. In 1997, this pavement was reground for a third time, where it is carrying nearly 2.25 million equivalent single axle load (ESAL) applications per year in the truck lane.

Since its first use in 1965, the use of diamond grinding has grown to become a major element of concrete pavement preservation projects. Diamond grinding has been employed on concrete pavement surfaces for a variety of reasons, including the following:

- Removal of transverse joint and crack faulting.
- Removal of wheelpath "rutting" caused by studded tire wear.
- Removal of permanent slab warping at joints (in very dry climates where significant warping has occurred).
- Texturing of a polished pavement surface exhibiting inadequate macrotexture (improving skid resistance).
- Improvement of transverse slope to improve surface drainage.
- Tire/pavement noise abatement.

General guidelines for considering diamond grinding on a specific project include the following:

- Faulted joints in excess of 3 mm (0.125 in).
- Roughness in excess of 1.0 to 1.4 m/km (63 to 90 in/mi).
- Wheelpath wear up to 10 mm (0.375 in).

However, it is important to recognize that diamond grinding is not appropriate for all cases. When selecting candidate projects for diamond grinding, many pavement-related characteristics such as structural condition, pavement materials, traffic level, and current visible distress (types, severities, and extent) must be taken into consideration. The following guidelines are available to help determine the feasibility of diamond grinding for a particular project:

- Pavements with significant roughness (i.e., values above 3 m/km [190 in/mi]) may be beyond the window of opportunity for cost-effective diamond grinding (Correa and Wong 2001). For cases where roughness is significant, another procedure such as an overlay may be a better alternative for improving smoothness (Correa and Wong 2001).
- If there is evidence that a severe drainage or erosion problem exists, as indicated by significant faulting (greater than 6 mm [0.25 in]) or pumping, actions should be taken to alleviate the problem prior to grinding (Correa and Wong 2001).
- Structural distresses such as pumping, loss of support, corner breaks, working transverse cracks, and shattered slabs will require repairs before grinding (Correa and Wong 2001). If the cause of faulting is not addressed prior to grinding, many agencies have found that the faulting will shortly reappear (Pierce 1994).
- Joints and transverse cracks with a deflection load transfer less than 60 percent should be retrofitted with dowels prior to diamond grinding. An effort should be made to restrict total deflection at the joints to less than 0.4 mm (15 mils) (Correa and Wong 2001).
- The hardness of the aggregate, and its direct impact on the cost of grinding, has often influenced whether or not a project was a feasible grinding candidate. Grinding a pavement with extremely hard aggregate (such as trap rock, river gravel, or quartzite) takes more time and effort than grinding a pavement with a softer aggregate (such as limestone). A 2001 study indicated that typical diamond grinding costs ranged from \$2.00 to \$8.00 per m^2 (\$1.70 to \$6.70 per yd^2), with costs as high as \$12.00 per m^2 (\$10.00 per yd^2) for concrete with very hard river gravel (Correa and Wong 2001).
- Concrete pavements suffering from durability problems, such as D-cracking, reactive aggregate, or freeze-thaw damage indicate that diamond grinding is not a suitable preservation technique, and that a more substantial rehabilitation strategy may be required (Correa and Wong 2001).
- Significant slab replacement and repair may be indicative of continuing progressive structural deterioration that grinding would not remedy.

If a pavement project contains few structural or materials-related problems, the decision on whether to diamond grind or not often comes down to an assessment of smoothness and faulting levels. Correa and Wong (2001) quantitatively define the “window of opportunity” for using diamond grinding by defining “trigger” and “limit” values for smoothness and faulting. Trigger value are those values at which the highway agency should consider diamond grinding, whereas limit values define the point at which the pavement has deteriorated so much that it is no longer cost effective to grind (Correa and Wong 2001). A summary of the recommended trigger and limit values for different pavement types and traffic levels are provided in tables 9.1 and 9.2.

Table 9.1. Trigger values for diamond grinding (Correa and Wong 2001).

	JPCP			JRCP			CRCP		
Traffic Volumes[1]	High	Med	Low	High	Med	Low	High	Med	Low
Faulting, mm avg (in avg)	2.0 (0.08)	2.0 (0.08)	2.0 (0.08)	4.0 (0.16)	4.0 (0.16)	4.0 (0.16)	N/A		
Skid Resistance	Minimum Local Acceptable Levels								
PSR[2]	3.8	3.6	3.4	3.8	3.6	3.4	3.8	3.6	3.4
IRI, m/km (in/mi)	1.0 (63)	1.2 (76)	1.4 (90)	1.0 (63)	1.2 (76)	1.4 (90)	1.0 (63)	1.2 (76)	1.4 (90)

<u>Notes</u>:

1. Volumes: High ADT>10,000; Med 3,000<ADT<10,000; Low ADT<3,000.
2. PSR = Present serviceability rating.

Table 9.2. Limit values for diamond grinding (Correa and Wong 2001).

	JPCP			JRCP			CRCP		
Traffic Volumes[1]	High	Med	Low	High	Med	Low	High	Med	Low
Faulting, mm avg (in avg)	9.0 (0.35)	12.0 (0.50)	15.0 (0.60)	9.0 (0.35)	12.0 (0.50)	15.0 (0.60)	N/A		
Skid Resistance	Minimum Local Acceptable Levels								
PSR[2]	3.0	2.5	2.0	3.0	2.5	2.0	3.0	2.5	2.0
IRI, m/km (in/mi)	2.5 (160)	3.0 (190)	3.5 (222)	2.5 (160)	3.0 (190)	3.5 (222)	2.5 (160)	3.0 (190)	3.5 (222)

<u>Notes</u>:

1. Volumes: High ADT>10,000; Med 3,000<ADT<10,000; Low ADT<3,000.
2. PSR = Present serviceability rating.

Diamond Grooving

Diamond grooving is a process in which parallel grooves are cut into the pavement surface using diamond saw blades with a typical center-to-center blade spacing of 19 mm (0.75 in). The principal objective of grooving is to provide escape channels for surface water, thereby reducing the incidence of hydroplaning that can cause wet weather crashes. It should only be used on pavements that are structurally and functionally adequate.

Grooving on concrete pavements has been performed since the 1950s to reduce the potential for wet weather skidding crashes on highways and airfield runways. Grooving may be performed either transversely or longitudinally. The advantages of transverse grooving are that it provides the most direct channel for the drainage of water from the pavement and it introduces a surface that provides considerable braking traction. Although common on runways and bridge decks, transverse grooving is not commonly used on highway pavements due in part to construction difficulties encountered in maintaining traffic on the adjacent lane and in part to excessive noise.

Longitudinal grooving is more commonly used on highways, and is often done in localized areas where wet weather crashes have been a problem, such as curves, exit ramps, and intersection approaches. Although longitudinal grooving does not improve the drainage characteristics of the pavement surface as well as transverse grooving, it does provide a channel for the water and produces a tracking effect that helps keep vehicles from skidding off the pavement, particularly on horizontal curves.

4. LIMITATIONS AND EFFECTIVENESS

Diamond Grinding

In the past decade, studies of the effectiveness of diamond grinding have indicated excellent long-term performance when grinding is conducted in conjunction with other required CPR activities (Rao, Yu, and Darter 1999; Correa and Wong 2001; Stubstad et al. 2005). One possible explanation for this positive impact on pavement life is the long standing theory that eliminating faulting reduces the dynamic effects of loadings on the pavement.

Field studies of diamond-ground pavement have indicated that diamond grinding can be an effective, long-term treatment. For example, a 1999 study of 76 projects in 9 states showed that the average longevity of diamond ground projects (i.e., the time until second grinding or rehabilitation was needed) was 14 years, while the expected longevity at an 80 percent reliability level was 11 years (i.e., 80 percent of the sections lasted at least 11 years) (Rao, Yu, and Darter 1999; Rao et al. 2000). A 2005 study of diamond-ground projects in California revealed that, on average, diamond ground pavements maintain their smoothness between 16 and 17 years, while the expected longevity at a 90 percent reliability level was 14.5 years (Stubstad et al. 2005). This is shown in figure 9.1.

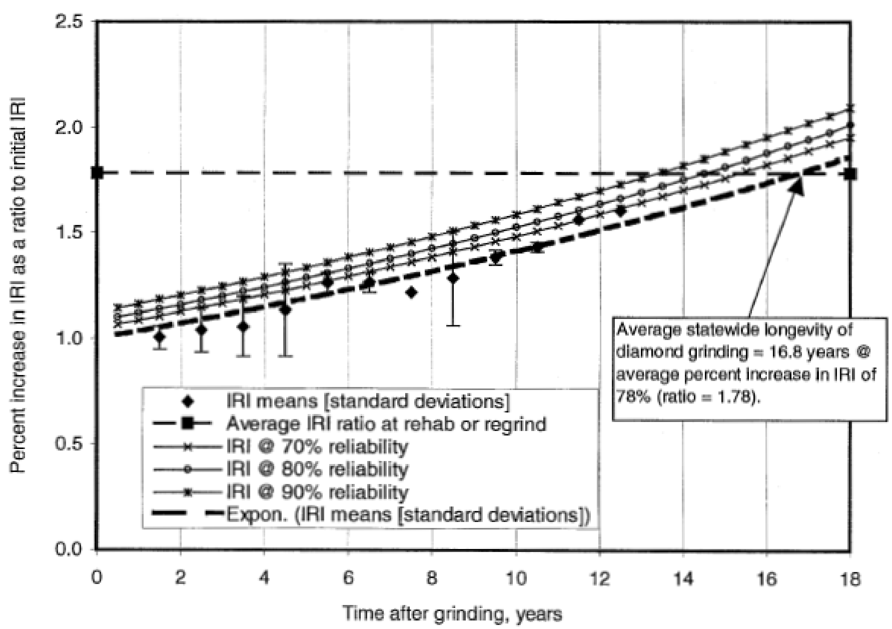


Figure 9.1. Survivability of diamond ground pavements in California (Stubstad et al. 2005).

In addition to addressing pavement roughness, diamond grinding also produces a pavement surface with ample macrotexture that provides good friction resistance. In Arizona, a recent study showed that the increase in friction values associated with different grinding configurations ranged between 15 and 41 percent, with an overall average improvement of 27 percent (Scofield 2003). In Wisconsin, Drakopoulos et al. (1998) found that the overall crash rate for diamond ground surfaces was only 60 percent of the rate for the unground surfaces. The diamond-ground pavements provided significantly reduced crash rates up to 6 years after the grinding, although a major portion of the diamond ground texture wore off within the first 2 years of grinding.

Another documented benefit of diamond grinding is its ability to reduce tire-pavement noise. An unwanted characteristic of pavements with faulted transverse joints or cracks is the thumping or slapping created by the tires as they pass over the joints or cracks. Because diamond grinding removes this faulting, the result is not only a smoother pavement, but a quieter one as well. Measurements on highways in Belgium indicate a reduction of up to 5 dbA in pavement noise levels after diamond grinding (Correa and Wong 2001). In addition, some studies have indicated that diamond grinding can have a positive influence on improving the frequency of the pavement-tire noise. For example, a study by the Michigan DOT found that grinding reduced noise by 5 dbA in the peak frequency of 500 Hz and the first harmonic of 1000 Hz (DeFrain 1989). The ability of diamond grinding to improve noise frequency is particularly true for pavements that were transversely tined with uniformly spaced grooves (Correa and Wong 2001).

As part of a recent noise mitigation study, the Arizona Department of Transportation (ADOT) investigated the pavement-tire interaction noise associated with a number of different surface textures including a longitudinal-tined section, a transverse-tined section, and a number of diamond ground sections with different grinder configurations (Scofield 2003). The results showed that the ground sections were quieter than the tined sections with most ground sections having noise levels less than 98 dBA. In comparison, the uniform longitudinal tined (19-mm [0.75-in]) and uniform transverse tined (19-mm [0.75-in]) sections had measured noise levels of 99.1 and 102.5 dBA, respectively (Scofield 2003).

Although diamond grinding is highly effective in removing faulting and restoring smoothness, the underlying mechanism of the faulting distress must be treated in order to prevent its redevelopment (ACPA 2000). The observation from one study indicates that following diamond grinding, faulting redevelops at a fast rate initially but stabilizes to the rate comparable to that just prior to grinding (Rao, Yu, and Darter 1999). This is illustrated in figure 9.2, which shows time-series faulting data from the 1999 diamond grinding study. Therefore, to stop faulting from rapidly returning in nondoweled JPCP sections after grinding, other CPR work such as dowel bar retrofitting and perhaps slab stabilization must be conducted in conjunction with the grinding operation. Figure 9.3 illustrates an example of the effects of concurrent work on faulting performance of diamond-ground pavements (Snyder et al. 1989). The results emphasize the need to combine grinding with other appropriate preservation techniques to minimize the recurrence of faulting.

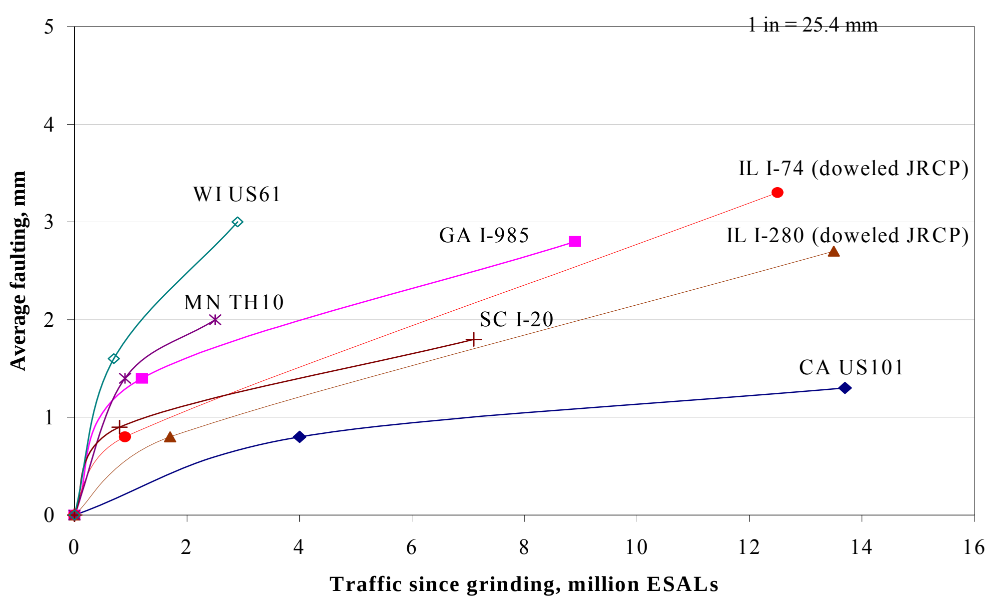


Figure 9.2. Time history faulting data (since diamond grinding) for diamond ground projects (Rao, Yu, and Darter 1999).

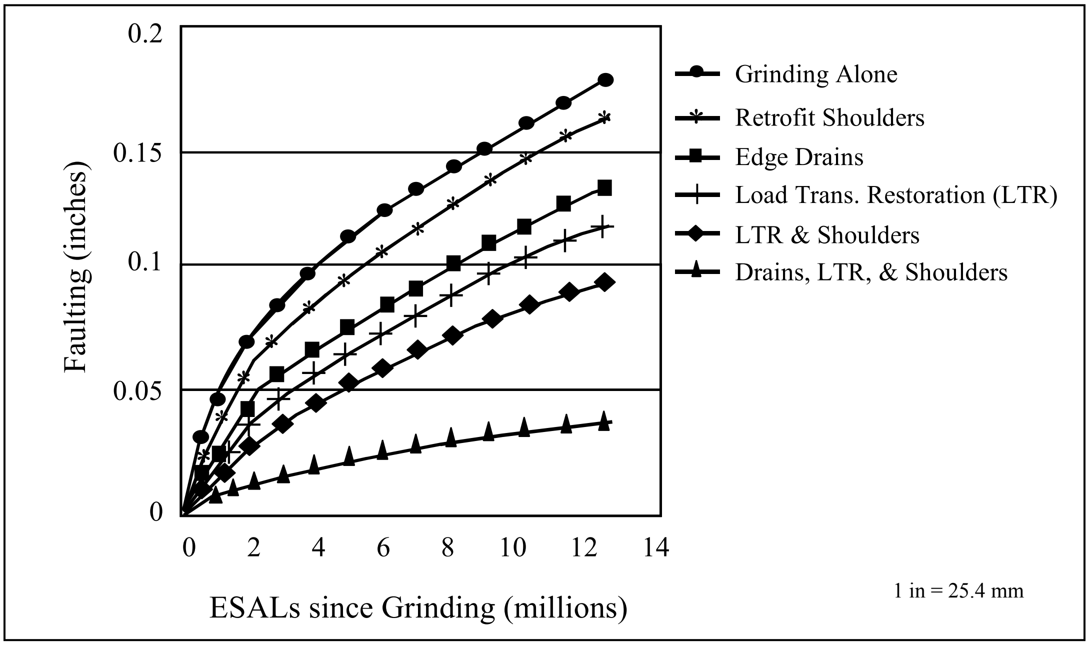


Figure 9.3. Effect of concurrent CPR techniques on pavement roughness over time (Snyder et al. 1989).

Diamond Grooving

As previously described, diamond grooving increases the macrotexture of the pavement and provides channels for the water to escape, thereby decreasing the potential of hydroplaning. Figure 9.4 shows the increase in the number of wet weather crashes over time on a California highway before longitudinal grooving and the large decrease in the number of crashes after grooving (Ames 1981). Nearly an 80 percent reduction was achieved, a value similar to what has been documented by other states such as Pennsylvania (75 percent) and Louisiana (64 percent) (Ames 1981; Walters 1979).

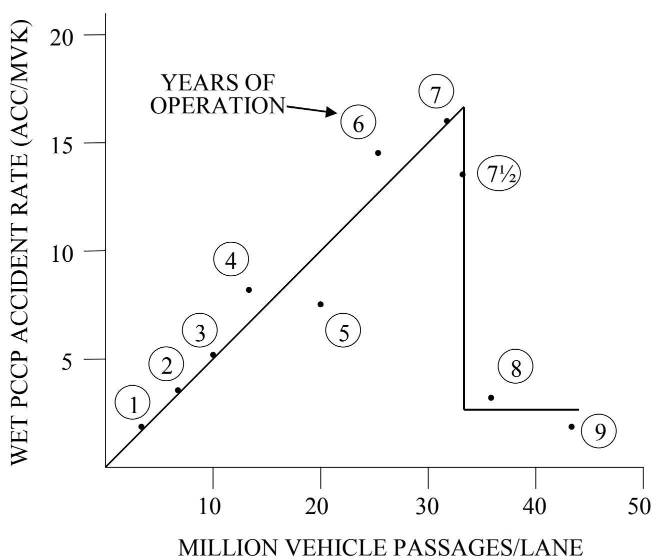


Figure 9.4. Wet weather crashes (crashes/million vehicle kilometers) for a selected California pavement before and after longitudinal grooving (Ames 1981).

Historically, a stated disadvantage of longitudinal grooving has been the perception by motorcyclists, and drivers of small vehicles, that longitudinal grooving impairs their ability to control their vehicle. Although some small lateral movement may be encountered by these vehicles on longitudinally grooved pavements, using 3 mm (0.125 in) wide grooves and groove spacings of 19 mm (0.75 in) have minimized these effects.

In 2007, several of Caltran's original grooved pavements were re-evaluated for noise. It was determined that longitudinal grooving is not an effective treatment for noise mitigation, but is effective in providing lateral stability and improved friction (ACPA 2007). It was also acknowledged that longitudinal grooving is not performed for noise reduction purposes.

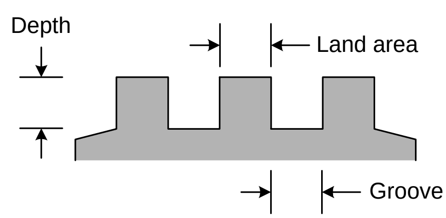
5. DESIGN CONSIDERATIONS

Prior to performing diamond grinding or grooving, pavement information should be obtained and evaluated to determine the feasibility of these rehabilitation techniques on their own or concurrent with other rehabilitation techniques.

Diamond Grinding

When considering a diamond grinding operation, information on the degree of faulting at transverse joints (and cracks if applicable) is needed. Information regarding past efforts to correct faulting should also be noted. Concurrent restoration techniques, such as load transfer restoration, undersealing, and retrofitted edge drains, should be considered to help minimize the recurrence of joint faulting after grinding. Plans and specifications should clearly define areas for diamond grinding and which concurrent restoration activities are required.

The surface characteristics of the pavement after grinding are highly dependent on the blade spacing, which in turn is selected based upon the hardness of the aggregate. The frictional resistance of easily polished aggregate (or softer aggregate such as limestone) can be improved by increasing the blade spacing to increase the "land area" between the sawed grooves. A summary of typical groove widths (blade kerf), land area (spacer width), and depth of diamond ground surfaces is presented in figure 9.5.



	Range	Hard Aggregate	Soft Aggregate
Groove width	2.29 – 3.81 mm (0.090 – 0.150 in)	2.54 – 3.81 mm (0.100 – 0.150 in)	2.29 – 3.56 mm (0.090 – 0.140 in)
Land area	1.52 – 3.30 mm (0.060 – 0.130 in)	2.03 mm (0.080)	2.54 mm (0.100)
Depth	1.52 mm (0.060)	1.52 mm (0.060)	1.52 mm (0.060)
No. of Blades	165 – 200/m (50 – 60/ft)	175 – 200/m (53 – 60/ft)	165 – 180/m (50 – 54/ft)

Figure 9.5. Typical dimensions for diamond grinding operations (ACPA 2006).

Although the friction characteristics for softer aggregates may be improved by increasing the spacing between blades, light vehicles and motorcycles may experience vehicle tracking. Many agencies specify tighter blade spacing primarily to reduce light vehicle tracking (Rao, Yu, and Darter 1999).

Because diamond grinding is removing a portion of the slab thickness, there is a concern about potential reductions in load-carrying cracking. However, studies have indicated that this slight reduction in slab thickness will not significantly compromise the fatigue life of the slab, largely because the long-term strength gain of the concrete offsets any slight reductions in slab thickness (Rao, Yu, and Darter 1999). The study suggests that a typical concrete pavement may be ground up to three times (13 to 18 mm [0.5 to 0.7 in]) without compromising the fatigue life of the pavement.

Diamond Grooving

Grooving operations are intended to reduce hydroplaning and accompanying crashes. Information regarding an area with a high number of crashes, as well as surface friction data for the section, should be reviewed prior to considering grooving operations.

Areas to be grooved should be clearly indicated on project plans. The grooves should have the dimensions shown in figure 9.6, as these have proven to be most effective for highways. The entire lane area should be grooved; however, allowance should be made for small areas that were not grooved because of pavement surface irregularities.

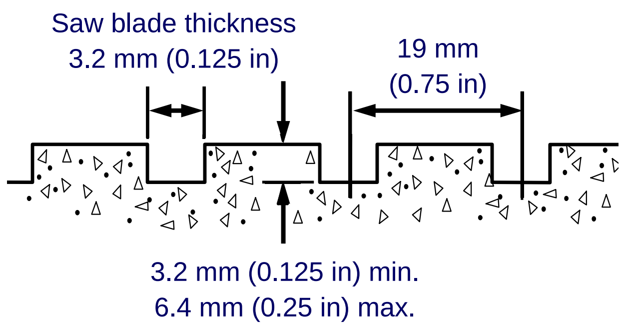


Figure 9.6. Typical dimensions for diamond grooving operations.

6. CONSTRUCTION CONSIDERATIONS

Diamond Grinding

<u>Equipment</u>

Grinding equipment uses diamond blades mounted in series on a cutting head. The front wheels of the equipment will pass over a bump or fault, which is then shaved off by the centrally mounted cutting head. The rear wheels then track in the freshly ground smooth path. The cutting head typically has a width ranging from 1.22 to 1.27 m (48 to 50 in). The desired corduroy texture is produced using a spacing of 164 to 197 blades per meter (50 to 60 blades per ft). New improved grinding machines and grinding blades have greatly increased the capability to provide extremely smooth profiles.

<u>Procedures</u>

Grinding should be performed continuously along a traffic lane for best results. Grinding should always be started and ended perpendicular to the pavement centerline and should also be consistently maintained parallel to the centerline. Grinding has typically been conducted on multi-lane facilities using a mobile single lane closure, allowing traffic to be carried on any adjacent lanes. The traffic control plan must comply with the Federal or local agency Manual of Uniform Traffic Control Devices (MUTCD).

Grinding equipment should have a long reference beam so the existing pavement can be used as a reference. By blending the highs and lows, excellent riding quality can be obtained with a minimum depth of removal. Low spots will likely be encountered, and specifications should recognize this. Generally, it is required that a minimum of 95 percent of the area within any 1 m by 30 m (3 ft by 100 ft) test area be textured by the grinding operation. Isolated low spots of less than 0.2 m^2 (2 ft^2) should not require texturing if lowering the cutting head would be required (ACPA 2000).

Because of the relatively narrow width of the cutting head, more than a single pass of the grinding equipment will be required. It is recommended that the maximum overlap between adjacent passes be 50 mm (2 in). Some projects use multiple grinding machines working together to expedite grinding operations.

Diamond Grooving

Equipment

Equipment used to groove pavements is specifically designed for this task. Because fewer diamond blades are required on the cutting head, the head width can be substantially greater than that used in diamond grinding. Some equipment are available that have grinding head width of 1.8 m (6 ft) or more.

The diamond blades are spaced to increase the "land area" between grooves, as illustrated in figure 9.6. Typically, the blades are spaced 19 mm (0.75 in) apart for longitudinal grooving, and the grooves have a width between 2.5 and 3 mm (0.1 and 0.125 in), and are cut to a depth of 3 to 6 mm (0.125 to 0.25 in). For transverse grooving, random grooves spaced 10 to 40 mm (0.4 to 1.6 in) apart and 3 mm (0.125 in) wide are recommended to reduce tire noise (Hibbs and Larson 1996).

Procedures

As previously indicated, grooving is most commonly performed longitudinally along the pavement. Typically, only localized areas (such as curves or intersection approaches) are grooved, instead of the entire project length. However, surface friction and wet weather crash data can be used to determine the extent of the grooving that may be needed.

Procedures typically follow those described previously for diamond grinding. The traffic control plan must comply with Federal or local agency MUTCD standards to ensure the safety of the construction personnel and traveling public.

Slurry Removal

The grinding and grooving operation produces a slurry consisting of ground concrete and the water used to cool the blades. This slurry is picked up by on-board wet-vacuums, and must be disposed of in accordance with local environmental regulations.

7. QUALITY CONTROL

As with any pavement project, the performance of LTR projects is greatly dependent on the quality of the construction procedures. Paying close attention to the procedures during construction greatly increases the chances of obtaining a surface with desired characteristics at the end of the project. The remainder of this section summarizes the recommended quality control activities for diamond grinding as presented in FHWA's *Diamond Grinding of Portland Cement Concrete Pavements Checklist* (FHWA 2005). Although this list of activities in this checklist is specific to the diamond grinding process, many of the same activities can easily be applied to the diamond grooving process.

Preliminary Responsibilities

Prior to the start of construction procedures, the agency should review of pertinent project-related documents, the project's current condition, and materials to be used on the project. The following specific lists of items are provided as a model QC checklist for these preliminary items.

Document Review

All of the following documents should be reviewed prior to the start of any construction activities. Any suspected problems should be identified and reconciled as part of the preliminary review process.

- Bid/project specifications and design.
- Special provisions.
- Agency application requirements.
- Traffic control plan.
- Equipment specifications.
- Manufacturer's instructions.
- Material safety data sheets (MSDS) (if required for concrete slurry).

Project Review

An updated review of the pavement condition is warranted to ensure that the project is still a viable candidate for diamond grinding. The following should be evaluated as part of the review process:

- Verify that the pavement conditions have not significantly changed since the project was designed.
- Assess the overall condition of the joints and cracks. Joints and transverse cracks exhibiting severe faulting (equal to or greater than 12 mm [0.5 in]) or displaying evidence of pumping (e.g., surface staining or isolated wetness) are potential candidates for load transfer restoration with dowels prior to diamond grinding.
- Verify that structural repairs are completed in the proper sequence (i.e., full-depth repairs, partial-depth repairs, load-transfer restoration, diamond grinding, and joint resealing).

Equipment Inspections

Prior to beginning construction, all construction equipment must be examined. The following are equipment-related items that should be checked:

- Verify that the diamond grinding machine meets requirements of the contract documents for weight, horsepower, and configuration.
- Verify that the blade spacing on the diamond-grinding cutting head meets the requirements of the contract documents.
- Verify that the vacuum assembly is in good working order and capable of removing concrete slurry from the pavement surface.
- Verify that the profilograph or pavement profiler meets requirements of the contract documents.
- Verify that the unit has been calibrated in accordance with manufacturer's recommendations and contract documents.
- Verify that the profilograph operator meets the requirements of the contract documents for training/certification.

Project Inspection Responsibilities

During the construction process, an inspector should verify that:

- Diamond grinding proceeds in a direction parallel with the pavement centerline, beginning and ending lines normal to the pavement centerline.
- Diamond-grinding results in a corduroy texture extending across the full lane width and complying with contract documents.
- Texturing cut into the existing pavement surface is in accordance with texturing requirements presented in the contract documents. Although typical values were presented in figure 9.5, specific dimensions and tolerances contained in the project documents take precedence.
- Each application of the diamond-ground texture overlaps the previous application by no more than the amount designated in the contract documents, typically 50 mm (2 in).
- Each application of the diamond-ground texture does not exceed the depth of the previous application by more than the specified amount (typically 6 mm [0.25 in]).
- The transverse slope of the ground surface is uniform to the extent that no misalignments or depressions that are capable of ponding water exist. Project documents typically have specific measurable criteria for transverse slope that must be met.
- The diamond-ground texture meets smoothness specifications (check on a daily basis).
- The concrete slurry is adequately vacuumed from the pavement surface and is not allowed to flow into adjacent traffic lanes.
- The grinding residue is not discharged into a waterway, a roadway slope within 61 m (200 ft) of a waterway, or any area forbidden by the contract documents or engineer. Concrete slurry from the grinding operation is typically collected and discharged at a disposal area designated in the contract document.

Weather Requirements

The following weather-related items should be checked immediately prior to construction:

- Air and/or surface temperature should meet minimum agency requirements (typically 2 °C [35 °F] and above) for diamond grinding operations in accordance with contract documents.
- Diamond grinding shall not proceed if icy weather conditions are imminent.

Traffic Control

To manage the flow of traffic through the work zone, the following traffic-related items should be checked or verified:

- Verify that the signs and devices used match the traffic control plan presented in the contract documents.
- Verify that the set-up complies with the Federal or local agency *Manual on Uniform Traffic Control Devices* (MUTCD) or local agency procedures.
- Verify that the repaired pavement is not opened to traffic until all equipment and personnel have been removed from the work zone.
- Verify that signs are removed or covered when they are no longer needed.
- Verify that unsafe conditions, if any, are reported to a supervisor (contractor or agency).

8. TROUBLESHOOTING

Potential construction problems associated with diamond grinding and diamond grooving that may be encountered are presented in tables 9.3 and 9.4, respectively. Typical causes and recommended solutions are also provided in these tables.

9. SUMMARY

Diamond grinding and grooving are surface restoration techniques that have been used successfully to correct a variety of surface distresses on concrete pavements. The appropriate application of these techniques can result in a cost-effective extension of pavement life.

Diamond grinding uses closely spaced, diamond saw blades to remove a thin layer of material from a concrete pavement surface. Although it is primarily used to restore or improve ride quality by removing transverse joint faulting and other surface irregularities, other common usages of diamond grinding include improving skid resistance (increasing macrotexture) and reducing tire-pavement interaction noise.

Grooving is the use of diamond saw blades to cut longitudinal or transverse grooves into a pavement surface. The purpose of grooving is to provide channels on the pavement that collect water and drain if from the surface. A reduction in surface water translates into a reduction in the potential for wet weather crashes associated with hydroplaning and splash and spray. Longitudinal grooving is commonly employed along local areas such as curves, where the grooves provide a tracking effect that helps hold vehicles on the road. For areas where increased braking resistance is required, transverse grooving is often used. Grooving is usually done on pavements that show little or no structural distress.

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Table 9.3. Potential diamond grinding construction/performance problems and associated solutions (ACPA 2000; ACPA 2006; FHWA 2005).

Problem	Typical Cause(s)	Typical Solution(s)
"Dogtails" (pavement areas that are not ground due to a lack of horizontal overlap).	These are primarily caused by weaving during the grinding operation.	Maintaining the required horizontal overlap (typically 50 mm [2 in] maximum) between passes and steady steering by the operator will avoid the occurrence of dogtails.
"Holidays" (areas that are not ground).	Isolated low spots in the pavement surface.	Lower the grinding head and complete another pass. Typical specifications require 95 percent coverage for grinding texture and allows for 5 percent unground isolated areas.
Poor vertical match between passes.	Inconsistent downward pressure. This is often obtained when unnecessary adjustments to the down-pressure are made.	A constant down-pressure should be maintained between passes to maintain a similar cut depth. A less than 3 mm per 3 m (0.12 in per 10 ft) vertical overlap requirement is often required.
Too much or too little material removed near joints.	▪ Expansion joints or other wide gaps in the pavement can cause the cutting head to dip if the leading wheels drop into the opening. ▪ Slabs deflecting from the weight of the grinding equipment can cause insufficient material to be removed.	▪ Wide gaps can be temporarily grouted to provide a smooth surface. ▪ If slabs deflect from the weight of the grinding equipment, lowering the grinding head may help, but stabilizing the slab or retrofitting dowel bars may be a better alternative.
The fins that remain after grinding do not quickly break free.	This could be an indication of excessive wear on the grinding head, but most likely it is the result of incorrect blade spacing.	The grinding head should be checked for wear before or after each day of operation. If the cutting blades are not worn, the blade spacing should be reduced.
Large amounts of slurry on the pavement during grinding.	Most likely this indicates a problem with the vacuum unit or skirt surrounding the cutting head.	If large amounts of slurry are left on the pavement, or slurry flows into adjacent traffic lanes or drainage structures, the surface grinding operations should be stopped. Inspect the equipment and make necessary repairs.
Light vehicles and motorcycles experience vehicle tracking	This indicates a problem with the spacing between the blades.	Reduce the spacing between the blades.

Table 9.4. Potential diamond grooving construction problems and associated solutions (ACPA 2000).

Problem	Typical Cause(s)	Typical Solution(s)
Lack of horizontal overlap.	As with grinding operations, this is primarily caused by weaving during the grooving operation.	Lack of horizontal overlap or weaving during grooving operations may cause lighter vehicles and motorcycles to experience increased vehicle tracking. Maintaining the required horizontal overlap between passes and steady steering by the operator will avoid the occurrence of this problem.
Isolated areas with inconsistent groove depth.	Isolated low spots in the pavement surface.	Although the effects of variable depth grooves are less readily apparent to traffic (no dip in the pavement surface is created), a uniform depth is desirable to ensure the intended drainage characteristics. The grooving head may need to be lowered in areas known to contain isolated low spots.
Inconsistent groove depth near joints.	As with diamond grinding: ▪ Expansion joints or other wide gaps in the pavement can cause the cutting head to dip if the leading wheels drop into the opening. ▪ Slabs deflecting from the weight of the grooving equipment can cause insufficient material to be removed.	▪ Wide gaps can be temporarily grouted to provide a smooth surface. ▪ If slabs deflect from the weight of the grooving equipment, lowering the grooving head may help, but stabilizing the slab or retrofitting dowel bars may be a better alternative.
Large amounts of slurry on the pavement during grooving.	As with grinding, this indicates a problem with the vacuum unit or skirt surrounding the cutting head.	If large amounts of slurry are left on the pavement, or slurry flows into adjacent traffic lanes or drainage structures, the surface grooving operations should be stopped. Inspect the equipment and make necessary repairs.

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CHAPTER 10. JOINT RESEALING AND CRACK SEALING

1. LEARNING OUTCOMES

This chapter describes recommended procedures for both joint resealing and crack sealing operations on concrete pavements. Upon successful completion of this chapter, the participants will be able to accomplish the following:

1. List the benefits of joint resealing and crack sealing.
2. List the desirable sealant properties and characteristics.
3. Describe recommended installation procedures.
4. Identify typical construction problems and appropriate remedies.

2. INTRODUCTION

Joint and crack sealing is a commonly performed pavement maintenance activity that serves two primary purposes. One purpose is to reduce the amount of moisture that can infiltrate a pavement structure, thereby reducing moisture-related distresses such as pumping, joint faulting, base and subbase erosion, and corner breaks. The other is to prevent the intrusion of incompressible materials so that pressure-related distresses such as spalling, blowups, buckling, and slab shattering are prevented.

Previous studies have indicated that sealant materials became ineffective anywhere from 1 to 4 years after placement (Peterson 1982; PIARC 1992). However, recent improvements in sealant materials, an increased recognition of the importance of a proper reservoir design, and an emphasis on effective joint preparation procedures have been found to increase the expected life of sealant installations (Smith et al. 1991; Smith et al. 1999). Although joint sealing continues to be a widely used maintenance technique, there remains a persistent controversy over whether joint sealing is needed for new concrete pavement construction (Shober 1986; Shober 1997; McGhee 1995; Hall and Crovetti 2000). Nevertheless, the general recommendation is that if the pavement was sealed originally, then it should continue to be resealed at appropriate intervals.

This chapter presents detailed discussions on the appropriate use and recommended installation procedures for joint resealing and crack sealing operations. It also provides information QC procedures, and troubleshooting. The focus is on "sealing" operations, in which the joint or crack is carefully prepared and a high-quality sealant material is installed.

3. PURPOSE AND PROJECT SELECTION

As previously described, free water entering joints or cracks can accumulate beneath the slab, contributing to distresses such as pumping, loss of support, faulting, and corner breaks. In addition, incompressibles that infiltrate poorly sealed joints or cracks interfere with normal opening and closing movements, causing compressive stresses in the slab and increasing the potential for spalling. If the compressive stresses exceed the compressive strength of the deteriorated pavement, blowups or buckling may occur. Even if blowups do not occur, continual intrusion of incompressibles may cause the pavement to "grow." This growth can force movement of nearby bridge abutments or other pavement structures that may, over time, cause serious damage and necessitate major rehabilitation.

Sealing operations in concrete pavements may be performed at both joints and cracks to minimize water ingress and to prevent the infiltration of incompressibles. Most joint sealing and resealing operations focus on the transverse joints, although the longitudinal joints (lane-shoulder or lane-lane) are generally sealed at the same time.

Application of Joint Resealing

Joint resealing should be performed when the existing sealant material is no longer performing its intended function. This is indicated by missing sealant, sealant that is in place but not bonded to the joint faces, or sealed joints that contain incompressibles. Some agencies specify that joints be resealed when a certain amount of sealant material (typically 25 to 50 percent) has failed, whereas other agencies base their decision on pavement type, pavement and sealant condition, and available funding (Evans, Smith, and Romine 1999).

The optimum time of the year to perform joint resealing is in the spring or the fall when moderate installation temperatures are prevalent. The greatest benefits from resealing are expected when the pavement is not severely deteriorated and when joint resealing is performed in conjunction with other pavement restoration activities, such as full-depth repair, partial-depth repair, and diamond grinding.

Application of Crack Sealing

Crack sealing is a comprehensive operation involving thorough crack preparation and placement of high-quality materials into or over candidate cracks to significantly reduce moisture infiltration and to retard the rate of crack deterioration. Crack sealing is most effective when performed on concrete pavements that exhibit minimal structural deterioration and when the cracks are relatively narrow with minimal spalling. Crack sealing may, however, be used on random transverse and longitudinal cracks of low- or medium-severity where the crack width is 13 mm (0.5 in) or less (ACPA 1995). Full-depth working transverse cracks typically experience the same range of movement as transverse joints; therefore, it is recommended that these cracks be sealed to reduce the potential of water and incompressible infiltration; an alternative to sealing of full-depth working cracks is load transfer restoration (ACPA 1995).

4. MATERIAL SELECTION

When planning a joint resealing project, one of the primary design activities is the selection of an appropriate sealant material. Material selection is dependent on a number of factors, including:

- Climate conditions (at time of installation and during the life of the sealant).
- Traffic level and percent trucks.
- Crack characteristics and density.
- Material availability and cost.
- Contractor experience.
- Safety concerns.

The remainder of this section discusses material selection considerations for a given sealing project. Specifically, this section introduces the different types of sealant materials that are typically used on concrete pavement sealing projects, introduces some of the more critical performance-related material properties, and discusses cost considerations that may impact the selection of the sealant material.

Available Material Types

Joint resealing and crack sealing operations generally employ either hot-applied thermoplastic materials or cold-applied thermosetting sealant materials. Table 10.1 lists some of the hot- and cold-applied materials available for sealing joints and cracks in concrete pavements (ACPA 2006). Details of the different material type categories typically used for joint resealing or crack sealing projects are described below. Note that although preformed sealant types are included in table 10.1, preformed neoprene compression seals are recommended for use only in new pavements (ACPA 2006).

Table 10.1. Common sealant types and related specifications for sealants used on concrete pavements (ACPA 2006).

Sealant Type	Specification(s)	Description
Liquid, Hot-Applied		**Thermoplastic**
Rubberized Asphalt	ASTM D 6690, Type II	Self-leveling
Polymeric	ASTM D 6690, Type I	Self-leveling
Elastomeric	ASTM D 3406	Self-leveling
Elastic	ASTM D 1854	Jet fuel resistant
Elastomeric PVC Coal Tar	ASTM D 3569, 3582	Jet fuel resistant (though PVC is rarely used
Liquid, Cold/Ambient-Applied		**Thermosetting**
Single Component		
Silicone	ASTM D 5893	Non-sag, toolable, low modulus
Silicone	ASTM D 5893	Self-leveling, no tooling, low modulus
Silicone	ASTM D 5893	Self-leveling, no tooling, ultra low modulus
Polysulfide	Fed Spec SS-S-200E	Self-leveling, no tooling, low modulus
Polyurethane	Fed Spec SS-S-200E	Self-leveling, no tooling, low modulus
Two Component		
Elastomeric Polymer	Fed Spec SS-S-200E	Jet fuel resistant
Preformed Compression Seals		
Polychloroprene Elastomeric	ASTM D 2628	Jet fuel resistant
Lubricant	ASTM D 2835	(Used in installation)
Expansion Joint Filler		
Preformed Filler Material	ASTM D 1751 (AASHTO M 213)	Bituminous, non-extruding, resilient
Preformed Filler Material	ASTM D 1752 (AASHTO M 153)	Sponge rubber, cork, and recycled PVC
Preformed Filler Material	ASTM D 994 (AASHTO M 33)	Bituminous
Backer Rod	ASTM D 5249	For hot- or cold-applied sealants

Hot-Applied Thermoplastic Sealant Materials

Thermoplastic sealants are bitumen-based materials that typically soften upon heating and harden upon cooling, usually without a change in chemical composition. These sealants vary in their elastic and thermal properties and are affected by weathering to some degree. Thermoplastic sealants are typically applied in a heated form (i.e., hot-applied) on concrete pavements, although some are diluted such that they can be installed without heat (i.e., cold-applied).

In the past two decades, rubberized asphalt has become the sealing industry standard. This type of sealant is produced by incorporating various types and amounts of polymers and melted rubber into asphalt cement. The resulting sealants possess a large working range with respect to low temperature extensibility and resistance to high temperature softening and tracking. In recent years, softer grades of asphalt cement have been used in rubberized asphalts to further improve low temperature extensibility. These materials, referred to as low modulus rubberized asphalt sealants, are used for sealing operations in many northern states because of their increased extensibility. Most of the high-quality rubberized asphalt materials are governed by ASTM D 6690.

<u>Cold-Applied Thermosetting Sealant Materials</u>

Thermosetting sealants are typically one- or two-component materials that either set by the release of solvents or cure through a chemical reaction. Some of these sealants have shown potential for good performance, but the material costs are also higher than standard rubberized asphalt. However, thermosetting sealants are often placed thinner and may have lower labor and equipment costs.

A variety of thermosetting sealant materials are available, including polysulfides, polyurethanes, and silicones. Of these, silicones have been most widely used and have demonstrated long-term performance capabilities. Silicone sealants are one-part cold-applied materials that exhibit good extensibility and strong resistance to weathering. These sealants have good bonding strength in combination with a low modulus that allow them to be placed thinner than the thermoplastic sealants. The performance of silicone sealants is typically tied to joint cleanliness and tooling effectiveness.

Silicone sealants are available in self-leveling and nonself-leveling forms. The nonself-leveling silicone requires a separate tooling operation to press the sealant against the sidewall and to form a uniform recessed surface. Self-leveling silicone sealants can be placed in one step since they freely flow to fill the joint reservoir without tooling. Silicone sealants are governed by ASTM D 5893.

Sealant Properties

Critical sealant properties that significantly affect the performance of the sealant material include:

- Durability.
- Extensibility.
- Resilience.
- Adhesiveness.
- Cohesiveness.

Durability refers to the ability of the sealant to withstand the effects of traffic, moisture, sunshine, and climatic variation. A sealant that is not durable will blister, harden, and crack in a relatively short time. If overbanded onto the pavement surface, a non-durable sealant may soften under higher temperatures and may wear away under traffic.

The extensibility of a sealant controls the ability of the sealant to deform without rupturing. The more extensible the sealant, the lower the internal stresses that might cause rupture within the sealant or at the sealant-sidewall interface. Sealant extensibility is most important under cold conditions because maximum joint and crack openings occur in colder months. Softer, lower modulus sealants tend to be more extensible, but they may not be stiff enough to resist the intrusion of incompressible materials during warmer temperatures.

Resilience refers to the sealant's ability to fully recover from deformation and to resist stone intrusion. In the case of thermoplastic sealants, however, resilience and resistance to stone intrusion are often sacrificed in order to obtain extensibility. Hence, a compromise is generally warranted, taking into consideration the expected joint or crack movement and the presence of incompressible materials for specific climatic regions.

As sealant material in a joint or crack is elongated, high stress levels can develop such that the sealant material is separated from the sidewall (adhesive failure) or the material internally ruptures (cohesive failure). Sealant adhesiveness is one of the most important properties of a good sealant, and often the cleanliness of the joint or crack sidewalls determines the sealant's bonding ability. Cohesive failures are more common in sealants that have hardened significantly over time.

Cost Considerations

In terms of material costs only, the thermoplastic materials are generally less expensive than the thermosetting materials. However, when making any cost comparisons, the total installation cost and the anticipated life of the sealant material must be considered. Some of the better performing materials have a higher unit cost, but may last sufficiently longer or require less material so that the overall (life-cycle) cost of the materials may actually be lower than less expensive sealants.

5. DESIGN CONSIDERATIONS

After the selection of a suitable sealant material has been made, the design of a joint or crack sealing project requires decisions to be made regarding the selection of the sealant reservoir dimensions (for joint resealing only) and the selection of an appropriate sealant configuration.

Transverse Joints

In new concrete pavement design, the selection of appropriate joint sealant reservoir dimensions is primarily dependent on the expected joint movement due to climatic conditions, moisture conditions, and traffic loads, combined with the specific properties of the selected sealant material. However, in a joint resealing operation, the width of the joint is already determined, and it is generally desirable to limit the amount of widening that is done to minimize material requirements and potential "wheel slap" from excessively wide joints. Consequently, the primary consideration in joint resealing is the selection of an appropriate joint shape factor needed to enhance the performance of the sealant.

Joint Shape Factor

Sealant Stresses

The performance of thermoplastic and thermosetting sealants (such as rubberized asphalt and silicone) depends on the stresses that develop in the sealant. Work by Tons (1959) showed that the stresses that occur in a given sealant material are primarily a function of the shape of the sealant at the time it is poured. Figure 10.1 illustrates the stresses produced in sealants placed to different depths. As each sealant material is elongated (simulating the opening of the joint), the sealant placed to a greater depth experiences much greater stresses than the shallower sealant. These higher stresses result from the "necking down" effect that occurs as the sealant is stretched. The material attempts to maintain a constant volume, but is restrained at the reservoir faces by adhesion to the pavement. With the deeper sealant, the necking down effect and the resultant stresses are greater.

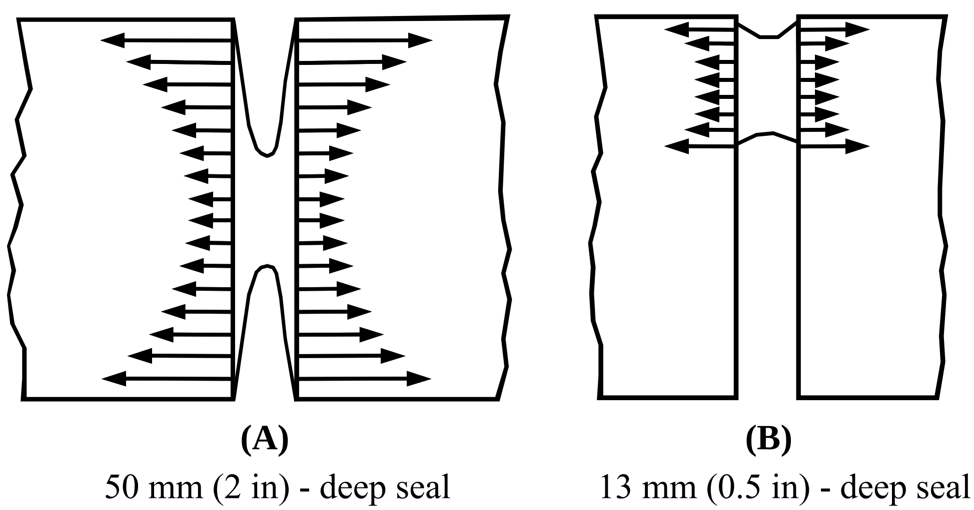

Figure 10.1. Relative effect of shape factor on sealant stresses.

The dimensions of the in-place sealant are described in terms of a "shape factor." The shape factor is defined as the ratio of the sealant width (W) to the sealant depth (D), as illustrated in figure 10.2. A proper shape factor minimizes the stresses that develop within the sealant and along the sealant/pavement interface as the joint opens.

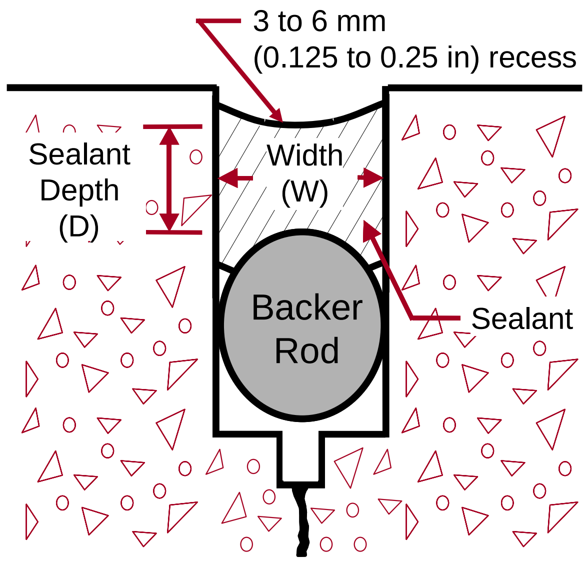

Figure 10.2. Illustration of sealant shape factor.

For good performance, the sealant must also be kept from bonding to the bottom of the reservoir. A backer rod, also shown in figure 10.2, may be installed in the reservoir to help achieve the desired shape factor, to prevent the sealant from bonding to the bottom of the reservoir, and to prevent the uncured sealant from running down into the crack beneath the reservoir. It is important that the backer rod, which is generally a polyethylene material, be compatible with the selected sealant material.

Recommended Shape Factors

The design of a sealant reservoir (i.e., determining how wide to saw the joint and how deep to place the sealant) should take into consideration the amount of strain or deformation from stretching that the sealant will experience. Most hot-poured thermoplastic sealants on the market today are designed to withstand strains of roughly 25 to 35 percent of their original width, whereas silicone sealants are designed to tolerate strains from 50 to 100 percent. As an example, a thermoplastic material placed in a 13-mm (0.5-in) wide joint can withstand an opening of 3 mm (0.125 in) (13 mm x 25 percent) before exceeding a strain of 25 percent. A silicone material placed in a 13-mm (0.5-in) wide joint can withstand an opening of 6.5 mm (0.25 in) (13 mm x 50 percent) before exceeding a strain of 50 percent.

Shape factors recommended for different sealant types are summarized in table 10.2 (Evans, Smith, and Romine 1999). It is also generally recommended that the sealant be recessed between 3 and 6 mm (0.125 and 0.25 in) below the surface of the pavement. These recommendations assume that the joints are opened to a uniform width.

Longitudinal Joints

Because of the limited amount of movement, concrete to concrete longitudinal joints rarely have a designed reservoir. These joints are typically very narrow (around 6 mm [0.25 in] wide) and are commonly sealed with a thermoplastic material. A backer rod may or may not be used.

Table 10.2. Typical recommended shape factors (Evans, Smith, and Romine 1999).

Sealant Material Type	Typical Shape Factor (W:D)
Rubberized Asphalt	1:1
Silicone	2:1
Polysulfide and Polyurethane	1:1

For longitudinal joints between a mainline concrete pavement and a hot-mix asphalt (HMA) shoulder, vertical movements are the primary concern. This joint, which one study indicated is the entry point for up to 80 percent of the water entering a pavement structure from the surface, is a particularly difficult joint to seal because of the differential vertical movement that occurs between the two materials (Barksdale and Hicks 1979). The differential vertical movements are due to the differences in the thermal properties of the materials and to the structural difference of their cross sections. Settlements or heaving of the shoulder are quite common along these joints, and they often will require a wider reservoir to withstand that vertical movement. A recent study found that an effective lane-shoulder joint seal reduced the total amount of water entering the pavement system by as much as 85 percent for a given rain event (Olson and Roberson 2003). A reservoir configuration of either 19 mm by 19 mm (0.75 in by 0.75 in) or 25 mm by 25 mm (1 in by 1 in) is commonly used for the lane-shoulder joint in order to accommodate the anticipated movements.

Sealant Configurations

Joints in concrete pavements are typically sealed in the recessed configuration shown in figure 10.3. However, some manufacturers of hot-poured thermoplastic materials recommend that the recess be eliminated and that the joint be filled flush with the surface with sealant. The purported benefits of this modification are the tendency for these sealants to remain more ductile when subjected to the kneading action of passing tires and the elimination of the reservoir area where sand and stones can collect. The use of an overband configuration is also occasionally advocated, with a perceived benefit being provided from the additional bonding area.

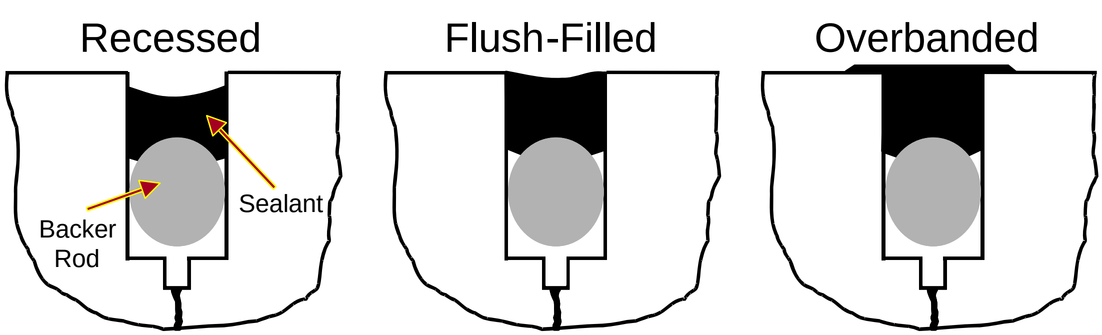

Figure 10.3. Joint sealant configurations.

Although the overbanded configuration has demonstrated good performance in some applications (Evans et al. 1999; Evans, Smith, and Romine 1999), they are not universally appropriate. For example, three disadvantages of the practice of overbanding are (Evans, Smith, and Romine 1999):

- On high-trafficked pavements, overbanded sealant material is typically worn away by traffic within 1 to 3 years. After the sealant is worn, traffic tires can pull the sealant from the joint edge, leading to adhesion failure.
- Snow plow blades used on highways in cold regions tend to damage overbanded sealants by pulling them up form the pavement surface.
- The overband can negatively impact ride quality and create an aesthetically unpleasant surface.

It should be noted that silicone sealants should never be overbanded or placed flush with the pavement surface. Manufacturers of silicone sealants recommend a minimum of 6 to 9 mm (0.25 to 0.38 in) recess below the surface (Smith et al. 1999).

6. CONSTRUCTION CONSIDERATIONS

After the sealant material has been selected, careful attention must be paid to the installation procedure to ensure the sealant provides the desired design life. Many sealing projects have performed poorly because of improper or inadequate installation procedures and practices. Successful sealing projects require close attention to detail.

Transverse Joint Resealing

The resealing of transverse joints in concrete pavements consists of the following steps, each of which is described in a subsequent section:

1. Old sealant removal.
2. Joint refacing.
3. Joint reservoir cleaning.
4. Backer rod installation.
5. New sealant installation.

Step 1: Old Sealant Removal

The first step of the joint resealing process is to remove the old sealant from the joint. Initial removal can be done by any procedure that does not damage the joint itself, such as using a rectangular joint plow or removal with a diamond-bladed saw. Another method that has been used is high pressure waterblasting.

Diamond-bladed sawing as a means of sealant removal has gained acceptance because it combines the sealant removal and refacing steps in a single process. It is most effective at removing existing silicone sealants and existing thermoplastic sealants when they have hardened and will not melt and “gum-up” the saw blade or joint face.

Complete removal of the old sealant is not required for the entire depth of the joint if the required reservoir depth is less than the existing sealant that is present. However, if there are incompressibles present, the old sealant should be completely removed to ensure free-moving, clean joints.

Step 2: Joint Refacing

The purpose of the refacing operation is to provide a clean surface for bonding with the new sealant and to establish a reservoir of the proper size to produce the desired shape factor. If a diamond-bladed saw has been used for sealant removal, refacing can be performed at the same time. If a joint plow or some other means has been used to remove the old sealant material, then a separate joint refacing operation must be performed.

Refacing is generally done using a water-cooled saw with diamond blades. A single full-width blade is useful for maintaining joint width; however, the edges wear quickly, reducing the effectiveness of the sawing. Two blades separated by a spacer to the desired width can be used on the same arbor. The core diameter of these blades should be at least 4.8 mm (0.19 in) to keep the blades from toeing into the joint. Blade overheating and warping can result from using thin blades. Typically, a joint is widened by 3 mm (0.125 in); 1.5 mm (0.0625 in) on each face. Care should be exercised when refacing; joint reservoirs that are widened excessively will increase the probability of “wheel slap” creating unwanted noise.

Routers have also been used to reface joint reservoirs, but their production is much slower than diamond-bladed saws. In addition, they can leave irregular or spalled joint walls and may smear the existing sealant on the sidewalls. Therefore, the use of routers is not recommended for joint refacing operations.

Step 3: Joint Reservoir Cleaning

The importance of effective cleaning of the joint sidewalls cannot be over-emphasized. Dirty or poorly cleaned joint or crack sidewalls can reduce the performance of even the best sealant and the most reliable sealant reservoir design. Several common materials that may contaminate the joint sidewalls include:

- Old sealant left on the joint or crack sidewalls.
- Water-borne dust (laitance) from the sawing operation.
- Oil or water introduced by the compressed air stream.
- Dust and dirt not removed during the cleaning operation.
- Debris entering the joint after cleaning and prior to sealing.
- Other contaminants that may inhibit bonding, such as moisture condensation.

Immediately after joint refacing, the joint should be cleaned with high-pressure air or water followed by sandblasting. Sandblasting effectively removes laitance (wet-sawing dust) and any other residue on the joint faces, and should be conducted in two passes so that each joint face is cleaned. Air compressors used with the sandblasters must be equipped with working water and oil traps to prevent contamination of the joint bonding faces. Compressors should be tested prior to sandblasting operations using a clean white cloth to ensure oil/water free operations. The use of hot-air lances to dry joint reservoirs should be used with caution, as overheating can damage the concrete (ACPA 2004).

Following sandblasting, the entire length of each joint face should be visibly clean with exposed concrete. Very close attention must be paid to the sandblasting operation to ensure consistent, thorough cleaning. During the sandblasting operation, a proper helmet and breathing apparatus and any other appropriate safety equipment should be used to protect the operator.

Immediately prior to backer rod and sealant installation, the joints should be blown again with high pressure (> 621 kPa [90 lbf/in^2]) clean, dry air to remove sand, dust, and other incompressibles that remain in the joint. A backpack blower typically cannot generate sufficient pressure to clean joints thoroughly and should not be used for final cleaning. Joints and surrounding surfaces should be airblown in one direction away from prevailing winds, taking care not to contaminate previously cleaned joints. Care must also be taken not to blow debris into traffic in adjacent lanes. Power-driven wire brushes should never be used to remove old sealant or to clean a joint in a concrete pavement. This procedure is essentially ineffective and can smear the old sealant across the concrete sidewall, creating a surface to which the new sealant cannot bond.

Step 4: Backer Rod Installation

Typical backer rod materials include polychloroprene, polystyrene, polyurethane, and polyethylene closed-cell materials; paper, rope, or cord should not be used (ACPA 2006). The backer rod should be installed as soon as possible after the joints are airblasted. The backer rod should be approved by the sealant manufacturer, and be about 25 percent larger in diameter than the joint width. The backer rod must be a flexible, nonabsorptive material that is compatible with the sealant material in use. The melting temperature of the backer material should be at least 14 °C (25 °F) higher than the sealant application temperature to prevent damage during sealant placement (ACPA 2006).

Wide joints or segments of joints in which the backer rod does not provide a tight seal should be filled with larger diameter backer rod. The backer rod should be installed to the proper depth and no gaps should exist at the intersections of backer rod strips. The rod should be stretched as little as possible to reduce the likelihood of shrinkage and the resultant formation of gaps.

Step 5: New Sealant Installation

As soon as possible after backer rod placement, the sealant material should be installed. This helps to avoid problems that occur when the backer rod is left in place too long before the sealant is placed, such as condensation on the backer rod and debris collecting in the reservoir. An additional check to verify that the reservoirs are clean and dry helps to ensure good long-term performance.

Hot-Poured Thermoplastic Sealant Materials

Hot-poured thermoplastic sealant materials should be placed only when the air temperature is at least 4 ºC (40 ºF) and rising (FHWA 2002). The sealant material should be installed in a uniform manner, filling the reservoir from the bottom up to avoid trapping any air bubbles. The joint reservoir must not be overfilled during the sealing operation. It is generally recommended that the surface of the sealant be recessed at least 3 to 6 mm (0.125 to 0.25 in) below the surface of the pavement to allow room for sealant expansion during the summer when the joint closes, without extruding the sealant to the point where traffic can pull it from the joint. However, as mentioned previously, some manufacturers recommend that the joint be filled to the surface with sealant. In any case, to avoid “tracking” of the sealant, traffic should not be allowed on the newly sealed joints for about 30 minutes to 1 hour after sealant placement. The sealant manufacturer should be consulted for recommendations on when the sealant can be exposed to traffic.

It is also important to follow the manufacturer’s recommendations with regard to the maximum sealant temperature, the recommended placement temperature, and any prolonged heating limitations. Many of the polymer- and rubber-modified sealants break down when subjected to temperatures above the recommended safe heating temperature. Prolonged heating can cause some sealant materials to gel in the heating tank, while others experience significant changes in their elastic properties. Sealant material that has been overheated tends to burn onto the hot surfaces of the inside of the melter/applicator. This burnt material, if remixed into the new sealant, can reduce sealant performance. Using an additional thermometer to monitor sealant temperatures can help eliminate damage due to sealant overheating.

Silicone Sealants

Silicone sealants should not be placed at temperatures below 4 ºC (40 ºF). As with the thermoplastic materials, silicone sealants should be installed in a uniform manner, from the bottom to the top of the joint, to ensure that no air is entrapped. Low-modulus silicone sealants have properties that allow them to be placed with shape factors of 2. It is not recommended that they be placed any thinner than half the width of the joint, with a minimum thickness of 6 mm (0.25 in). Traffic should not be allowed on the pavement for about 1 hour after sealant placement. Again, the sealant manufacturer should be consulted for recommendations on when the sealant can be exposed to traffic.

As mentioned previously, silicone materials come in two varieties: self-leveling and nonself-leveling. The nonself-leveling silicone sealants must be tooled to force the sealant around the backer rod and against the joint sidewalls. This tooling should also form a concave sealant surface with the lowest point being about 6 mm (0.25 in) below the pavement surface. Successful tooling has been accomplished using such devices as a rubber hose on the end of a fiberglass rod or pieces of a large diameter backer rod.

Self-leveling silicone sealants do not require this tooling operation. Extra care, however, must be taken with placing backer rod for self-leveling silicone sealants, as the sealant can easily flow around loose backer rod prior to curing. Sealant can also flow out at the joint ends if not properly blocked. Even though these sealants do not require tooling, some agencies have mandated tooling in order to enhance the bond between the pavement and the sealant.

When installing both silicone and thermoplastic sealants, such as in a project with silicone sealant in the transverse joints and hot-poured thermoplastic materials in the longitudinal joint, the silicone should be installed first to reduce the potential for contamination of the transverse joint during the longitudinal joint sealing operations.

Other Thermosetting Sealants

Other thermosetting sealants, such as polysulfides and polyurethanes, require a curing period to gain their strength and resiliency. Most polymeric thermosetting sealants consist of two components that are carefully mixed as the material is being placed in the joint. These sealants require a special application nozzle and careful control of the application equipment. Quality control should include testing the sealant for adequate cure, and traffic should not be allowed on these sealants until the surface has skinned over and the possibility for stone intrusion is minimized.

Longitudinal Joint Resealing

As previously described, two types of longitudinal joints in concrete pavements may also be addressed as part of a resealing operation: longitudinal joints between adjacent concrete pavement slabs, and the longitudinal joint between the mainline concrete pavement and an HMA shoulder. While the procedures are essentially the same as transverse joint resealing, some additional considerations should be noted.

Concrete to Concrete Longitudinal Joints

Longitudinal joints between adjacent concrete slabs are found between adjacent traffic lanes or between a concrete mainline pavement and a concrete shoulder. This joint is generally tied together with deformed tiebars so that movements are not excessive and conventional joint sealing operations can be followed.

Because of the limited amount of movement that occurs at these joints, they are generally sealed with a hot-poured thermoplastic material. In the resealing operation, typically no reservoir is formed or needed. If the transverse joints are to be sealed with silicone, it is important that the longitudinal joints be sealed last to prevent contamination of the transverse joints with hot-poured thermoplastic material.

Concrete Mainline/HMA Shoulder Joint

The longitudinal joint between a concrete mainline pavement and a HMA shoulder can be a very difficult joint to seal. The differences in the thermal properties of each material and the differences in the structural cross section often result in large differential vertical movements. Additionally, significant horizontal movement, or separation, often accompanies the vertical movement. Because water easily infiltrates the pavement structure at this type of joint, it should be sealed to minimize water infiltration.

Again, the steps required for the sealing of lane-shoulder joint are the same as transverse joint sealing operations. However, it is important that a sufficiently wide reservoir be cut in the existing HMA shoulder to allow for the anticipated vertical movements. Common reservoir dimensions range from 19 mm by 19 mm (0.75 in by 0.75 in) to 25 mm by 25 mm (1 in by 1 in). The reservoir can be created using either a router or a diamond-bladed saw.

The reservoir should be cleaned prior to the placement of the sealant material. A backer rod is generally not needed if proper depth control during the creation of the reservoir has been maintained. Many agencies use hot-poured thermoplastic materials to seal this joint, although there are some silicone materials that have been specifically developed for this type of application.

Crack Sealing

With the exception of a sealant removal step, the sealing of cracks in concrete pavements essentially follows the same basic steps as the sealing of joints: refacing, cleaning, backer rod installation, and sealant installation (ACPA 1995). The first step is to reface the crack to the desired width. However, the random orientation of most concrete pavement cracks makes it difficult to create a uniform sealant reservoir directly over the crack. The formation of a reservoir should be accomplished with a small diameter diamond-bladed saw (ACPA 2006). Note that while crack routers have been used in the past to form sealant reservoirs, their use is not recommended due to the chipping and micro-cracking damage this equipment causes to the concrete (ACPA 2004). The cutting blades for the crack saws are typically 175 to 200 mm (7 to 8 in) in diameter and 6 to 13 mm (0.25 to 0.5 in) wide. The width of the saw cut generally provides an appropriate shape factor to accommodate the expected crack movement. Smaller blade diameters, in addition to lightweight two- or three-wheel unit designs, allow crack saws to pivot and follow irregular crack profiles.

Once the reservoir is created, the crack should be cleaned following those steps prescribed for joint resealing. Sandblasting is particularly recommended to remove laitance from the sawing operation. After cleaning, the crack is blown with compressed air and the backer rod (if specified) and sealant material are installed. The same precautions that apply to the installation of sealant materials into joints also apply here (ACPA 1995).

Construction Equipment

A brief description of the equipment used in joint and crack sealing operations is included in this section.

<u>Equipment for Sealant Removal and Joint/Crack Refacing</u>

Joint Plow

A joint plow is a rectangular blade mounted on the hydraulic mount of a tractor or the bucket of a skid loader. The plow blade is inserted into the joint and pulled along each joint edge, scraping the sealant from the sidewalls. The blade must be rectangular and fit freely into the joint. A V-shaped blade should not be used because these blades can spall the joint. The rectangular tool must be mounted such that it is free to move vertically and horizontally in the joint without binding. Blades of several widths should be on hand, as joint widths are seldom uniform over an entire project.

Diamond-Bladed Saw

Diamond-bladed saws are typically 26 to 46 kW (35 to 65 hp), water-cooled devices equipped with diamond-edged blades. A single, full-width blade is useful for maintaining joint width; however, the edges wear quickly, reducing the effectiveness of the sawing. Two blades separated by a spacer to the desired width can be used on the same arbor.

Equipment for Joint Cleaning

Sandblasting Equipment

Sandblasting equipment consists of a compressed air unit, a sandblasting machine, hoses, and a wand with a venturi-type nozzle. The compressed air supply is the most critical part of the sandblasting operation. At least 620 kPa (90 lbf/in^2) of pressure and 4.3 m^3/min (150 ft^3/min) of oil- and moisture-free air should be provided. Additionally, the use of a jig is recommended to reduce operator fatigue and ensure that the sandblast nozzle is properly positioned to direct sand against the sidewalls to provide more efficient cleaning (Evans, Smith, and Romine 1999).

Airblasting Equipment

Airblasting equipment consists of high-pressure air compressors with hoses and wands. High-pressure air compressors are effective at removing dust and debris from a joint, but are not as effective as sandblasting at removing laitance. As a minimum, compressed air units should have a blast pressure of 690 kPa (100 lbf/in^2) and a blast volume of 4.3 m^3/min (150 ft^3/min).

Equipment for Joint Sealant Placement

Melters

Hot-poured thermoplastic materials are heated and mixed in an indirect-heat, agitator-type melter. These machines burn either propane or diesel fuel, and the resulting heat is applied to a transfer oil that surrounds a double-jacketed melting vat containing the sealant material. This indirect method of heating is safer and provides a more controlled and uniform heat.

Silicone Pumps

One-component silicone materials are typically pumped from storage containers using compressed air powered pumping equipment. A feed rate of at least 1.5 L/min (0.4 gal/min) is recommended and the wand should be equipped with a nozzle that allows filling from the bottom up.

Applicators

Most sealant applicators are pressure-wand systems, normally equipped on sealant melters. The applicator consists of a pump, hoses, and an applicator wand. Sealant material is pumped directly from the melter-vat through the system and into the joint. Some low-productivity, filling operations use a cornucopia pour pot, which is a hand-held, conical-shaped pot used to apply unheated or partially-heated emulsions into joints.

7. QUALITY CONTROL

Proper sealant application is a process that relies heavily upon the care and conscientiousness of the contractor. Paying close attention to this quality during construction greatly increases the chances of minimizing premature failures on joint resealing and crack sealing projects. The remainder of this section summarizes the QC recommendations summarized in a recent FHWA checklist (FHWA 2002).

Preliminary Responsibilities

Prior to the commencing of construction procedures, the agency should conduct a review of pertinent project-related documents, the project's current condition, and materials to be used on the project. The following specific lists of items are provided as a model QC checklist for these preliminary items.

Document Review

All of the following documents should be reviewed prior to the start of any construction activities (FHWA 2002):

- Bid/project specifications and design.
- Special provisions.
- Traffic control plan.
- Manufacturer's sealant installation instructions.
- Material safety data sheets (MSDS).
- Agency application requirements.

Any suspected problems should be identified and reconciled as part of the preliminary review process.

Project Review

An updated review of the current project's condition is warranted to ensure that the project is still a viable candidate for joint resealing or crack sealing. Specifically, it should be verified that conditions have not significantly changed since the project was designed, and that the prevailing distresses are still in the acceptable ranges used for project selection. Also, the selected methods for sealant removal, refacing, and cleaning should be reviewed. Finally, for joint resealing projects, the selected joint design and sealant type should be reviewed to make sure they are still appropriate for the expected project climate and conditions.

Review of Materials

In preparation for the construction project, the following list summarizes many of the material-related items that should be checked or reviewed prior to construction (FHWA 2002):

- Sealant meets specification requirements.
- Sealant material is from an approved source or listed on agency qualified products list (QPL) (if required).
- Sealant material has been sampled and tested prior to installation (if required).
- Sealant material packaging is not damaged (i.e., leaking, torn, or pierced).
- Backer rod is of the proper size and type for the selected sealant material.
- Chemically curing sealants (if used) are within shelf life.
- Sufficient quantities of all materials are available for completion of the project.

Inspection of Equipment

Prior to beginning construction, all construction equipment must be examined. The following sections describe equipment-related items (specific to the different available sealant types) that should be checked prior to construction (FHWA 2002).

Hot-Applied Sealant Melters

For hot-applied sealant melters, an indirectly heated double boiler type melter with effective agitation is typically used. Prior to construction, these melters should be inspected to ensure that they are in good working order with all internal mechanisms (such as heating, agitation, pumping systems, valves, and thermostats) functioning properly. Also, the contractor should verify that the proper size wand tips are available.

Cold-Applied Sealant Pumps (Single- and Two-Component Materials)

For cold-applied sealant materials, the contractor should make sure that the pump is in working order, the follower plates are in good shape and lubricated, and that the hoses are not plugged. For two-component pumps, the contractor should verify that the pump contains a mixing head that meets manufacturer's requirements, and that the pump is delivering material at the correct ratio.

Joint Cleaning Equipment

For the joint cleaning equipment, the following items should be verified (FHWA 2002):

- Abrasive cleaning unit is adjusted for correct abrasive feed rate and has oil and moisture trap.
- Abrasive cleaning uses environmentally acceptable abrasive media.
- Abrasive cleaning operators use air purification systems as required.
- Air compressors have sufficient pressure and volume to adequately clean joints and meet agency requirements.
- Air compressors are equipped with oil and moisture filters/traps that are properly functioning.
- Joint plows (if used) are of correct size and configuration to remove required amount of old sealant without spalling joint edges.
- Concrete saws/blades are of sufficient size to adequately cut the required joint width and depth, and the saw is in good working order.
- Waterblasting equipment can supply the water and pressure required by specifications.

Weather Requirements

The weather conditions at time of construction can have a large impact on the performance of the installed sealant. Specifically, the following weather-related items should be checked prior to construction (FHWA 2002):

- Review manufacturer installation instructions for requirements specific to the sealant material used.
- Air and/or surface temperature meets manufacturer and all agency requirements (typically 4 °C [40 °F] and rising) for sawing and sealing.
- Sealing should not proceed if rain is imminent.
- Application does not begin if there is any sign of moisture on the surface or in the joint or crack.

Traffic Control

Immediately prior to construction it should be verified that the on-site traffic control signs and devices match those defined in the traffic control plan. Also, it should be verified that the set-up complies with the Federal or local agency *Manual on Uniform Traffic Control Devices* (MUTCD).

After the sealing activities have been completed, traffic should not be allowed back on the pavement until the sealant has adequately cooled or cured so that it is not tracked by vehicle tires.

Construction Inspection Responsibilities

Joint Preparation

During the joint preparation steps, the inspector should ensure the following (FHWA 2002):

- All safety mechanisms and guards on equipment are in place and functioning properly, and operators are using required personal protective equipment.
- Old sealant is removed from the joint.
- Joint is refaced to produce a joint reservoir that coincides with the selected sealant material.
- After refacing, joints are flushed with high pressure water to remove all saw slurry and debris.
- Joint surfaces are cleaned using abrasive cleaning or waterblasting.
- Abrasive cleaning is accomplished with the nozzle 25 to 50 mm (1 to 2 in) above the joint using two passes (each directed at one of the joint faces).
- Joint is blown clean with clean dry air. A propane torch or hot-air lance should <u>not</u> be used for drying.
- Joint is inspected prior to sealing by rubbing finger along the joint walls to insure that no contaminants (dust, dried saw residue, dirt, moisture, or oil) are on the joint walls. If dust or other contaminants are present, reclean the joint to a satisfactory condition.

Backer Rod Installation

During the backer rod installation process, the inspector should check that the backer rod is being installed uniformly to the required depth. Also, the inspector should check that the backer rod fits snugly in the joint (no gaps along the side), and is not being stretched or damaged during installation.

Sealant Installation

Hot-Applied Sealants

As previously discussed, many of the newer sealant materials are sensitive to heating and application temperatures. The use of supplementary temperature monitoring devices is recommended so that the sealant temperature can be closely observed. Underheating the material results in poor bonding, while overheating the material destroys its ductile properties and increases its aging.

More specifically, as part of a comprehensive QC plan, a project inspector should check or verify the following:

- Melter heat transfer medium is heated to the correct temperature range.
- Sealant is being heated into the manufacturer's recommended pouring or application temperature range. In addition, the inspector should check that the heating temperature does not exceed the material's safe heating temperature.
- Sealant is continuously agitated to assure uniformity, except when adding additional material.
- Operator wears required personal protective equipment.
- If melter is equipped with a heated hose system, the hose is heated prior to beginning sealant application.
- If melter does not have a heated hose, verify that the hose is unplugged and clear prior to beginning sealant application.
- If melter does not have a heated hose, the sealant should be recirculated through the hose to warm the hose prior to application. During idle periods, or if it is noted that sealant is cooling in the hose, sealant shall be recirculated through the hose back to the material vat to keep the hose at an acceptable temperature.
- Melting vat should be kept at least one-third full to help maintain temperature uniformity.

- Joint is filled from the bottom up to the specified level to produce a uniform surface with no voids in the sealant.
- Detackifier or other blotter is applied to reduce tack prior to opening to traffic (if needed).
- Traffic is not allowed on the project until sealant is tack-free or cooled.
- Verify adequate adhesion at several random sections of cooled sealant. A simple knife test can be used to determine how well the sealant has adhered to the side walls (ACPA 1995). Such a test consists of using a dull knife blade or thin metal strip to probe between the sealant and the side wall. A loose, effortless penetration indicates adhesion loss, while good adhesion provides resistance (ACPA 1995).

Cold-Applied Sealants (Single- and Two-Component)

During the installation of a single- or two-component sealant, as a minimum, the project inspector should check the following:

- Joint is filled from the bottom up to the specified level to produce a uniform surface with no voids in the sealant.
- Nonsag sealants are properly tooled to force the material against the sidewalls and to form a smooth surface at the specified recess from the surface.
- Sealant is permitted to cure to a tack-free condition prior to opening to traffic.
- Verify adequate adhesion at several random sections of cured sealant. As with the hot-applied sealants, a simple knife test can be used to test for adhesion.

Clean Up

After the joint resealing or crack sealing construction process is complete, any excess sealant application or spills must be removed from the surface. Melters and other application equipment should be properly cleaned in preparation for their next use.

8. TROUBLESHOOTING

As indicated in the previous section, there are a number of factors to consider to help ensure the proper application of joint or crack sealant. When problems occur during the sealing process, it is often because one or more of the construction QC steps was ignored. Table 10.3 summarizes some of the more common construction and performance problems associated with joint resealing or crack sealing and suggested remedies.

9. SUMMARY

This chapter presents information on joint and crack sealing in concrete pavements. The need for sealing operations is discussed, including guidelines for identifying candidate projects. Various available sealant materials are presented, along with their properties, applicable specifications, and design considerations.

Procedures for the sealing of transverse joints, longitudinal joints, and cracks in concrete pavements are described. In almost every project, a successful sealing operation includes the following steps: removing the old material (joint resealing only), refacing the existing joint/crack reservoir, cleaning, installing backer rod, and installing the new sealant material. As the quality of the construction practices is extremely important to the long-term performance of the sealant installation, recommended QC and troubleshooting procedures are also presented.

Table 10.3. Potential joint resealing and crack sealing construction problems and associated solutions (FHWA 2002; ACPA 2006).

Problem	**Typical Solutions**
Punctured or stretched backer rod.	A punctured or stretched backer rod can result in an improper shape factor or adherence of sealant to bottom of reservoir. Both of these conditions have detrimental effects on the long-term performance of the sealant. If observed, remove the existing backer rod and install a new backer rod using the recommended procedures.
Burrs along the sawed joints.	Burrs along the sawed joint can make it difficult to install the sealant. To remedy, drag a blunt pointed tool along the sawed joint, or use a mechanized wire brush, to remove sharp edges (ACPA 1993). Note: the joint or crack will have to be recleaned prior to sealing.
Raveling, spalling, or other irregularities of the joint walls prior to sealant application.	This is most likely caused by improper care in sealant removal or joint cleaning steps. Note: A V-shaped joint plow blade can spall joint sidewalls. Irregularities on joint walls can reduce the sealant's lateral pressure, therefore, allowing the sealant to extrude or pop from the joint (ACPA 1995). If irregularities are observed, the agency and contractor should agree on an appropriate method for repairing potential problem areas.
Sealant not adhering to joint/crack.	▪ Reclean joint or crack. ▪ Allow sidewalls to dry before sealing. ▪ Heat to correct temperature or verify temperature gauges. ▪ Wait for higher ambient temperature before sealing. ▪ Use correct recess for joint width (especially important for cold applied sealants).
Sealant gelling in melting chamber (melter).	▪ Check melter temperature gauges. ▪ Use fresh sealant. ▪ Use sealant with longer pot life, or conform to manufacturer's recommended pot life.
Bumps or irregularities in surface of tooled sealant application.	▪ Check tooling utensil or squeegee and ensure it is leaving the correct finish. Repair or replace as necessary. ▪ Ensure that tooling is being conducted within the time after application recommended by the manufacturer. ▪ Decrease the viscosity of the sealant (if applicable).
Cold-applied sealants not setting up.	▪ Use fresh sealant. ▪ Use correct mix ratios and mixing systems.
Sealant picks up or pulls out when opened to traffic.	▪ Close to traffic and delay opening. ▪ Seal during cooler temperatures. ▪ Apply sealant flush with surface or with specified recess. ▪ Use stiffer sealant if too soft for climate. ▪ Use a detackifier or blotter to reduce initial tack. ▪ Install at correct temperature and continuously verify the temperature gauges on the melter. ▪ Repeat preparation routine and then reseal joints that were contaminated with solvent or heat transfer oil. ▪ Reclean joint sidewalls to remove offending material and then reseal.
Voids or bubbles in cured sealant.	▪ Seal during cooler periods and then allow concrete to further dry or use non-sag type sealant to resist void formation. ▪ Backer may be melting with hot-applied sealants; use heat-resistant backer material and check for proper sealant temperature. ▪ Install backer rod carefully to avoid damage (i.e., puncturing). ▪ Apply sealant from the bottom up. ▪ Tighten all connections and bleed off entrapped air. ▪ Replace backer material if moisture is present. ▪ Cure primer according to manufacturer's recommendations.
Sink holes in sealant	▪ Use larger backer material, reapply (top off) sealant to correct level or use non-sag sealant. ▪ Use heat-resistant backer material.

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NOTES

CHAPTER 11. STRATEGY SELECTION

1. LEARNING OUTCOMES

This manual has discussed in detail a variety of concrete pavement preservation and restoration techniques. These range from relatively simple and straightforward treatments, such as joint resealing, to more involved techniques, such as full-depth repairs and retrofitted load transfer devices. So far, however, no guidance has been provided on determining which treatment (or which combination of treatments) is appropriate for a given concrete pavement project.

The selection of an appropriate preservation or rehabilitation treatment for a given concrete pavement project requires a systematic, step-by-step approach that considers all relevant factors. This chapter outlines a recommended step-by-step procedure that can be used to select the most appropriate treatment types or strategies. Upon successful completion of this chapter, the participants will be able to accomplish the following:

1. Describe the treatment selection process.
2. List the components of a life-cycle cost analysis (LCCA).
3. List other factors that might enter into the selection process.

2. INTRODUCTION

Across the country, the maintenance and rehabilitation of the existing highway network has become a central focus. The need to maintain the nation's already-constructed network is essential to the economical operation of the overall transportation system. Because of increasing financial constraints, accomplishing this task has become more and more difficult over the years. Therefore, both the traveling public and highway agencies alike are seeking better solutions to their mutual concerns about the operating conditions on the nation's roads. The incorporation of pavement preservation is viewed to be essential to this process as these techniques have been shown to be effective at delaying more costly and invasive rehabilitation procedures, thereby providing longer service lives, minimizing traffic disruptions, reducing the work zone risks to both workers and highway users, and minimizing life-cycle costs.

Determining the right treatment for the right pavement at the right time can be a complex procedure that requires simultaneously evaluating a number of different influencing factors. This chapter provides information about the types of factors that should be considered when selecting an appropriate preservation strategy for a given pavement. Included among these factors is a life-cycle cost analysis, which is introduced as one way of evaluating the overall cost-effectiveness of competing strategies.

3. TREATMENT SELECTION PROCESS

Overview of the Selection Process

Whenever an evaluation of an individual project is conducted, the immediate goal of that evaluation is to identify the deficiencies in the pavement, and then ultimately to determine how to best address those deficiencies. Typically, the first decision is to determine how extensive the needs are for the pavement. For example, if the pavement is only exhibiting functional deficiencies or localized structural problems, the observed deficiencies can most likely be addressed with one or more concrete pavement preservation activities. If more global structural or material problems exist, then the pavement section is more likely suited for an asphalt or concrete overlay, or perhaps even complete reconstruction in the most severe case. Because discussions of overlays or reconstruction are outside the scope of this course, this chapter focuses on the selection of the most appropriate concrete pavement preservation treatments.

At the project level, the process of determining the most appropriate pavement preservation activities for concrete pavements is a fairly straight forward process. Based on a collective review of a number of recent published documents, the following step-by-step process can be used to determine the most appropriate treatment (or combination of treatments) for a concrete pavement (Hall et al. 2001; Anderson, Ullman, and Blaschke 2002; NCHRP 2004):

1. Conduct a thorough pavement evaluation.
2. Determine causes of distresses and deficiencies.
3. Identify treatments that address deficiencies.
4. Identify constraints that could influence treatment selection.
5. Develop feasible treatment strategies.
6. Assess the life-cycle costs associated with treatment strategies.
7. Select preferred strategy.

Each of these different steps is discussed separately below.

Step 1. Conduct a Thorough Pavement Evaluation

As discussed in chapter 3, conducting a pavement evaluation is the first step in assessing the current deficiencies of the pavement. Overall, the pavement evaluation procedures focus on determining both the structural and functional adequacy of the current pavement. As described in chapter 3, the structural condition refers to the ability of the pavement to carry current and future traffic loading, whereas the functional condition refers to the ability of the pavement to provide a smooth and safe riding surface to the users. The structural condition of the pavement is determined from the results of the condition and drainage surveys, deflection testing, and any material sampling and testing. The functional condition is primarily determined by reviewing the results of any roughness and friction testing. Table 11.1 presents a summary of the different pavement characteristics included in an evaluation, and the different testing methods used to assess them.

Table 11.1. Areas of overall condition assessment and corresponding data sources (adapted from NCHRP 2004).

Attribute	Distress Survey	Drainage Survey	Deflection Testing	Roughness Testing	Friction Testing	Field Sampling and Testing
Structural Adequacy	✓	✓	✓			✓
Functional Adequacy	✓			✓	✓	
Drainage Adequacy	✓	✓	✓			✓
Materials Durability	✓	✓	✓			✓
Maintenance Applications	✓					
Shoulders Adequacy	✓		✓			✓
Variability Along Project	✓	✓	✓			✓

Step 2. Determine Causes of Distresses and Deficiencies

One of the most important steps of the treatment selection process is to collectively review all of the data from the pavement evaluation to determine the causes of any observed distresses and identified deficiencies. A summary of typical concrete pavement distresses and their causes is provided in table 11.2. By knowing the underlying causes of the distresses that are observed, appropriate preservation treatments can be identified.

Step 3. Identify Treatments That Address Deficiencies

The main objective of the third step is to the pavement preservation treatment (or series of preservation treatments) that would be potentially useful at addressing one or more of the identified pavement deficiencies. It is important to remember that the scope of this course is limited to the following concrete pavement preservation treatments:

- Slab stabilization.
- Partial-depth repairs.
- Full-depth repairs.
- Retrofitted edge drains.
- Load transfer restoration.
- Diamond grinding and grooving.
- Joint resealing.

While more specific details on the appropriate uses of each of these treatments are contained in chapters 4 through 10, respectively, a summary of their general uses is presented in table 11.3. For completeness, table 11.3 also shows some of the more common *rehabilitation* activities and the different distresses that they address.

In general, the following sequence of checks can be used to help identify those treatments that may be appropriate for a given project:

1. <u>Assess slab support conditions</u>—When assessing the support conditions of concrete slabs, it is important to test for voids at slab corners, as well as test the load transfer efficiency at transverse joints. One good indication that there is a slab support problem is the presence of *pumping* (i.e., the presence of fine material on the pavement surface at the transverse joints). Concrete slabs that currently do not have structural problems (i.e., corner breaks or linear cracking), but are found to have voids or poor load transfer are good candidates for slab stabilization or load transfer restoration.
2. <u>Correct localized distress that is contained in the upper 1/3 of the slab</u>—In concrete pavements, it is not uncommon to have localized areas of distress that are contained in the upper 1/3 of the slab thickness. At joints, common distresses in this category include joint spalling, or map cracking, crazing, or scaling. If any of these distresses are present in an amount or severity that requires attention, a partial-depth repair is typically the best treatment to correct the distress.
3. <u>Correct localized distress not contained to the upper 1/3 of the slab</u>—When a pavement evaluation locates distress that is not contained to the upper 1/3 of the slab (e.g., corner breaks, transverse cracking, or material-related distress), a full-depth repair is typically required to correct the observed distress.

Table 11.2. Concrete pavement distress types and causes (adapted from Hall et al. 2001).

Distress	Causes	Notes
Linear cracking (transverse, longitudinal, or diagonal)	Fatigue damage, often in combination with slab curling and/or warping; drying shrinkage; improper transverse or longitudinal joint construction; or foundation movement.	Low-severity shrinkage cracks in JRCP and CRCP are not considered structural distress; medium- and high-severity deteriorated shrinkage cracks are. All severities of linear cracking are considered structural distress in JPCP.
Corner breaks	Fatigue damage, often in combination with slab curling and/or warping and/or erosion of support at slab corners.	—
D-cracking	Freeze-thaw damage in coarse aggregates.	—
Alkali-aggregate distress	Compressive stress building up in slab, due to swelling of gel produced from reaction of certain siliceous and carbonate aggregates with alkalies in cement.	Alkali-aggregate reaction includes alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR).
Map cracking and crazing	Alkali-aggregate reaction or overfinishing.	—
Scaling	Overfinishing, inadequate air entrainment, or reinforcing steel too close to the surface.	—
Joint seal damage	Inappropriate sealant type, improper sealant reservoir dimensions for the sealant type, improper joint sealant installation, and/or aging.	Loss of adhesion of sealant to joint walls, extrusion of sealant from joint, infiltration of incompressibles, oxidation of sealant, and cohesive failure (splitting) of the sealant are all considered joint seal damage.
Joint spalling, also called joint deterioration	Compressive stress buildup in the slab (due incompressibles or alkali aggregate reaction); D cracking; misaligned or corroded dowels; poorly consolidated concrete in vicinity of joint; or damage caused by joint sawing, joint cleaning, cold milling, or grinding.	—
Blowups	Compressive stress buildup in the slab (due to infiltration of incompressibles, or alkali aggregate reaction).	A blowup may occur as a shattering of the concrete for several feet on both sides of the joint, or an upward buckling of the slabs.
Pumping	Excess moisture in the pavement structure, erodible base or subgrade materials, and high volumes of high-speed, heavy wheel loads.	—
Faulting	Pumping of water and fines back and forth under slab corners, erosion of support under the leave corner, buildup of fines under the approach corner.	—
Curling/warping roughness	Moisture gradients through the slab thickness, daily and seasonal cycling of temperature gradients through the slab thickness, and/or permanent deformation caused by a temperature gradient in the slab during initial hardening.	—
Bumps, heaves, and settlements	Foundation movement (frost heave, swelling soil) or localized consolidation, such as may occur at culverts and bridge approaches.	Detract from riding comfort; at high severity may pose a safety hazard.
Polishing	Abrasion by tires.	Polished wheelpaths may pose a wet-weather safety hazard.
Popouts	Freezing in coarse aggregates near the concrete surface.	A cosmetic problem rarely warranting repair.

Table 11.3. Concrete pavement restoration treatments best suited for concrete pavement distresses (adapted from Hall et al. 2001).

	Concrete Pavement Preventive and Rehabilitation Treatments													
Distress	Slab stabilization	Partial-depth repair	Full-depth repair	Retrofitted edge drains	Load transfer restoration	Diamond grinding	Grooving	Joint resealing	Pressure relief joints	Asphalt overlay	AC overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay	Reconstruction
Corner breaks			✓								✓		✓	✓
Linear cracking			✓								✓		✓	✓
Punchouts			✓								✓		✓	✓
D-cracking			✓							✓	✓		✓	✓
Alkali-aggregate reaction			✓						✓	✓	✓		✓	✓
Map cracking, crazing, scaling		✓								✓				
Joint seal damage								✓						
Joint spalling		✓	✓								✓		✓	✓
Blowup			✓								✓		✓	✓
Pumping	✓			✓	✓									
Faulting					✓	✓				✓	✓	✓	✓	
Bumps, settlements, heaves			✓			✓				✓	✓	✓	✓	✓
Polishing						✓	✓			✓		✓		

4. Correct functional distresses—Many otherwise sound concrete pavements may be exhibiting functional deficiencies, such as poor friction or excessive roughness. Diamond grinding is typically used to correct roughness problems, but it also has a positive impact on a pavement's friction characteristics. If the only functional problem is found to be a localized area of poor friction (such as at curves or intersections), diamond grooving is often an effective treatment option.

5. Assess joint sealant condition—One final step in the strategy selection process is to assess the performance of the joint sealant. In general, if the original concrete pavement was sealed at the time of initial construction, then every effort should be made to maintain an effectively sealed joint over the life of the pavement. Therefore, if there are any signs of joint sealant damage, or if any other treatment alternatives have caused the effectiveness of the joint sealant to be compromised, joint resealing should be considered. When conducted with other treatments, joint resealing should always be the final activity performed on a pavement before it is opened to traffic.

Step 4. Identify Constraints and Key Selection Factors

After compiling a list of possible effective treatments under step 3, before proceeding further in the treatment selection process it is important to check those possible effective treatments against a list of any project-specific constraints or other key selection factors that may come into play. Some of the potential factors that an agency will need to consider when determining whether or not a possible treatment is feasible for a specific project are the following (AASHTO 1993; Hall et al. 2001):

- Available funding.
- Future maintenance requirements.
- Geometric restrictions.
- Lane closure time.
- Environmental impact (e.g., contamination generated during construction work).
- Conservation of natural resources.
- Agency's experience with the use of the treatment.
- Traffic safety during construction.
- Worker safety during construction.
- Contractors' experience with the treatment.
- Availability of needed equipment and materials.
- Competition amount providers of materials.
- Stimulation of local industry.
- Agency policies.
- Political concerns.

Because the treatments included in the scope of this course are more preventive in nature, it is envisioned that most of these potential constraints will not be an issue when selecting treatments. However, it is important that all outside constraining factors be identified at this point of the selection process to avoid conducting unnecessary work in the upcoming steps.

Step 5. Develop Feasible Treatment Strategies

A treatment strategy is a plan that defines what treatments to apply and when to apply them, over a selected time period. For example, a strategy using only one treatment could be to conduct diamond grinding every 8 to 10 years for the next 25 years. Another strategy could be to conduct dowel bar retrofitting activities, followed by diamond grinding during the same construction project. It is not uncommon to concurrently conduct more than one of the concrete pavement preservation activities in a single project because the various concrete pavement preservation activities complement each other. Therefore, the purpose of this step is to:

1. Determine all of the different activities that need to be conducted to best address the pavement's needs.
2. Determine if it is best to conduct the activities concurrently, or to apply the individual activities at different times in the future.

Each individual treatment combination or treatment timing scenario can be considered a separate treatment strategy for the pavement. While there is usually an obvious choice for the most appropriate strategy, competing strategies can be objectively compared by considering the overall the life-cycle cost associated with each.

Step 6. Assess the Life-Cycle Costs Associated With Treatment Strategies

Because the concrete pavement preservation treatments address different pavement deficiencies, life-cycle costing techniques are not typically needed to help select appropriate strategies. However, where LCCA results do become important is when the concrete pavement preservation treatments and strategies are being considered along with more extensive rehabilitation techniques (i.e., overlays) or reconstruction. An LCCA provides an objective method of comparing the costs associated with different treatments applied at different times over the life of a pavement. These results are of particular interest to those agencies that are trying to document the benefits of using more inexpensive preventive treatment strategies that delay more expensive rehabilitation activities. This section is intended to only introduce the general concepts of a life-cycle cost analysis, with more detailed information available elsewhere (Walls and Smith 1998; Hall et al. 2001; ACPA 2002).

General Concepts of an LCCA

Initial construction costs are often the factor given the greatest consideration in the treatment selection process. However, expected future costs that occur at different times over the life of the treatment must also be considered, but in order to do so they must first be converted to a common basis for comparison purposes. The techniques used to perform the conversion are based on the assumption that the value of money changes with time, due to factors such as inflation.

There are a number of different techniques that are used to equate the value of costs incurred at various points in time. Most commonly, these costs are expressed in terms of either a present worth (PW) cost or an equivalent uniform annual cost (EUAC). Using the PW method, all future costs are adjusted to a PW cost using a selected discount rate. The costs incurred at any time in the future can be combined with the initial construction costs to give a total PW cost over the analysis period. More detailed descriptions of some of the major required LCCA-related inputs are the following:

- Analysis period—The analysis period refers to the time over which the economic analysis is to be conducted, which is not necessarily the same as the “life” of the treatment. Suggested analysis periods for new pavement design are 20 years to 50 years for high volume roadways, and 15 years to 25 years for low volume roadways (AASHTO 1993). The FHWA recommends an analysis period of at least 35 years for all pavement projects, including new or rehabilitation (Walls and Smith 1998). As a general rule of thumb, it is suggested that the analysis period should be long enough to incorporate at least one rehabilitation activity. However, for some rehabilitation work, the analysis period will sometimes be shorter (say 10 to 20 years) depending on the future use of the facility, the need for geometric improvements, and other factors. In any event, it is important that the analysis period be the same for all rehabilitation alternatives being considered.
- Timing and costs of individual CPR treatments and maintenance activities—The construction of a detailed expenditure stream diagram is useful to illustrate the timing and costs associated with the application of different treatments over the analysis period, as shown in figure 11.1. This example reflects the initial costs associated with a pavement rehabilitation project, the annual costs for routine maintenance, and additional periodic costs for activities such as seal coats or other preservation actions. If a salvage value is considered at the end of the project, it is reflected as an income that can be expected from the project at the end of the analysis period. Each of these individual cost types is discussed in more detail in the next section.
- Discount rate—The discount rate is the interest rate used in calculating the present value of future costs. This value that represents the time value of money is often approximated as the difference between the commercial interest rate and inflation rate as given by the consumer price index. Historical discount rates have been in the 3 to 5 percent range (Walls and Smith 1998).

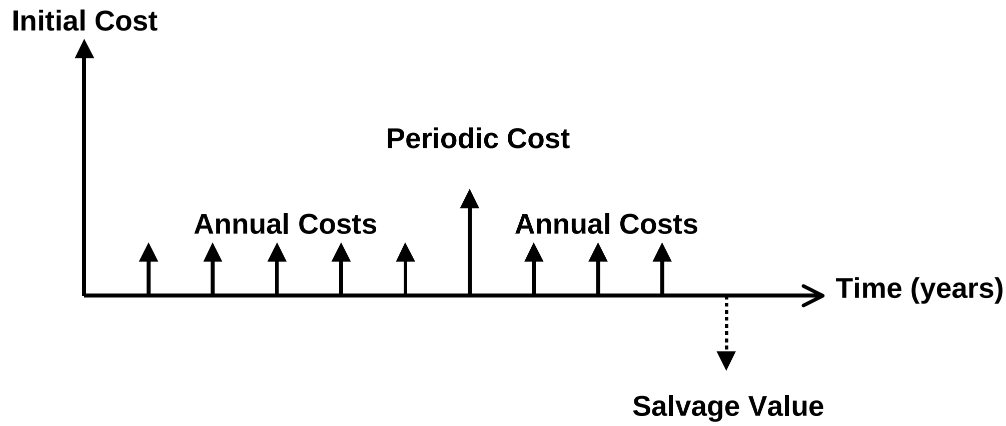

Figure 11.1. Example of an expenditure stream diagram.

Included Cost Types

When conducting an LCCA of different treatment strategies, the engineer has the option of including both agency costs (costs recognized by the highway agency) and user costs (costs recognized by the user). More detailed explanations of the different cost types that are typically included within each of these categories are described below.

Agency Costs

Agency costs are the actual costs incurred by the agency over the analysis life of the project. Three of the most commonly included agency costs are the following:

- Treatment construction costs—Treatment construction costs are the total costs to construct each treatment included in a treatment strategy. These costs typically include design or engineering costs, as well as all construction costs.
- Costs of future maintenance—Future maintenance costs are typically the annual costs associated with maintaining the pavement in a serviceable condition throughout the analysis period. While these are important real costs to include in the analysis, it is recognized that these future maintenance costs are influenced by many factors, including the condition of the pavement at the time of rehabilitation, the quality of construction, future traffic loadings, environmental considerations, and so on.
- Future salvage value—A salvage value reflects any remaining worth of a pavement rehabilitation alternative at the end of the analysis period. Salvage value may be either positive or negative: a positive value represents useful, salvageable material, whereas a negative value represents a cost to remove and dispose of the material that exceeds any possible positive salvage value.

User Costs

User costs are the costs incurred by the user over the life of the project. User costs may be incurred in several ways, but are commonly considered to include the following (Walls and Smith 1998):

- Traffic delay costs—Costs and inconvenience of traffic rerouting, traffic control, fuel consumption, extended trips, and other delay costs.
- Vehicle operating costs—A vehicle traveling on a rough road suffers more wear and consumes more fuel. Other vehicle operating costs include stopping/speed change costs and idling costs.
- Crash costs—Construction zones and rough roads increase the potential for accidents.
- Damage to freight—due to a rough road.

The inclusion of user costs as part of a LCC analysis is a controversial issue. While there is general agreement that traffic delays and rough roads do contribute to increased costs to the user, the actual costs are difficult to quantify, particularly for road roughness. Another significant issue is that user costs are not borne by the agency, and agencies have difficult giving them the same weight in a decision process as their own actual costs. Thus traffic delay and user costs are sometimes considered as one of the criteria in the evaluation of different alternatives (as described later in this chapter), rather than being included in the cost analysis for the comparison of alternatives.

The FHWA's *Interim Technical Bulletin* on life-cycle costing provides an excellent summary of user costs (Walls and Smith 1998). It describes the estimation of each component of user costs (VOC, user delay costs, and crash costs) based on practices current when the report was prepared. Where there are alternate approaches available for calculating a user cost component, they are all presented. That report also devotes an entire chapter to a rational approach for the calculation of work zone user costs.

Analysis Procedure

Traditionally, an LCCA is conducted by treating each of the input variables as discrete, fixed variables. This approach is known as a deterministic approach because all input values are assumed to be fixed and a single LCC result is determined. In practice, however, there is a great deal of uncertainty involved in all parts of the analysis. For example, rarely does the actual performance period of initial designs or of rehabilitation designs match exactly that which is assumed in the analysis. Furthermore, the values of unit costs can vary considerably from that which was assumed, further adding some uncertainty to the results.

In recognition of the shortcomings of a deterministic approach, a probabilistic approach may be conducted that allows an agency to incorporate risk and uncertainty (Walls and Smith 1998). The analyst inputs certain variables in probabilistic terms (including expected values and standard deviations or ranges) and conducts a computer simulation that randomly samples from probabilistic descriptions of the uncertain input variables. After hundreds or thousands of iterations, the result of the analysis is a distribution showing the range of possible outcomes along with the probability of occurrence. The resulting distribution can then be analyzed statistically in order to assess acceptable levels of risks or to identify those critical factors or input variables that are driving the exposure to risk.

The FHWA has produced a software program called *RealCost* that completely automates the LCC methodology as it applies to pavements (FHWA 2004). The software calculates life-cycle values for both agency and user costs associated with both new construction and rehabilitation activities, and can perform both deterministic and probabilistic modeling of pavement cost analysis problems. While *RealCost* compares the agency and user life-cycle costs of alternatives, its analysis outputs alone do not identify which alternative is the best choice for implementing a project. The lowest life-cycle cost option may not be implemented when other considerations such as risk, available budgets, and political and environmental concerns are taken into account.

Step 7. Select Preferred Strategy

A detailed LCCA can be one part of the decision-making process, but by itself does not necessarily identify the most optimal alternative. The lowest life-cycle cost option may not be practical when other considerations, such as available budges, network priorities, or environmental factors are taken into account. In many cases, some of the selection factors and constraints identified in step 4 may over-ride the results of the LCCA. Ultimately, the goal is to select the preferred alternative that best addresses the needs of the pavement while meeting all functional and monetary constraints that exist.

As mentioned previously, it is not uncommon for different treatments to be used concurrently in a single project. However, if used concurrently, it is important to conduct these activities in a logical construction order that maximizes the effectiveness of each individual treatment while protecting any previously performed repairs (ACPA 2006). For example, full- and partial-depth repairs, dowel bar retrofitting, and slab stabilization activities should always be conducted prior to diamond grinding. Delaying diamond grinding until after these other activities have been conducted maximizes the resulting smoothness associated with diamond grinding. A summary of the logical order of conducting pavement restoration techniques is displayed in figure 11.2 (ACPA 2006). Obviously not every project will require every step, but it is recommended that the sequence of these steps be maintained.

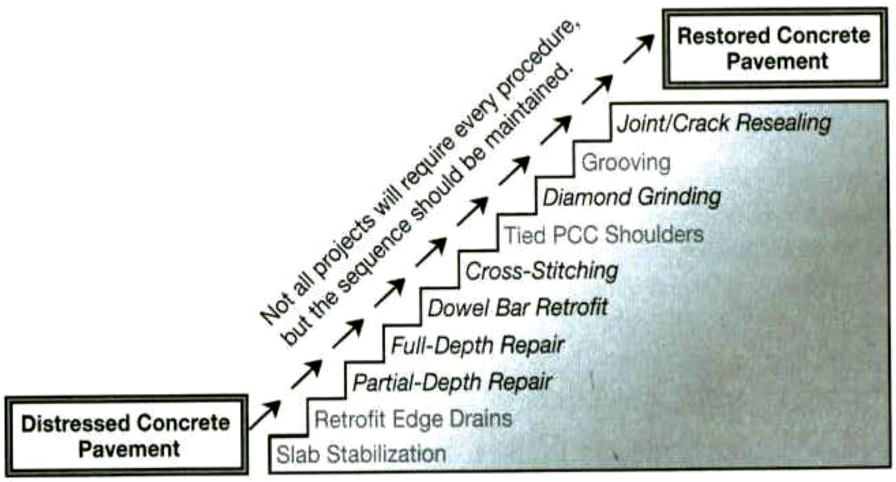

Figure 11.2. Recommended sequence of restoration activities (ACPA 2006).

4. SUMMARY

This chapter describes several basic steps that can be used to determine the most appropriate treatment strategy for a given concrete pavement project. The process begins with conducting a pavement thorough pavement evaluation and determining the causes of any observed distress. Next, treatments that address identified deficiencies are selected, and filtered using any outside constraints that have been identified. After applying any outside constraints, feasible treatment strategies (i.e., combinations of treatments) are determined and associated costs are objectively compared by conducting a LCCA (if necessary). Finally, the appropriate strategy is selected based on LCCA and other factors and the overall logical sequence of treatments is outlined in order to maximize the effectiveness of all treatments.

5. REFERENCES

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NOTES

February 2008

Iowa State University's Center for Transportation Research and Education (CTRE) is the umbrella organization for the following centers and programs:

Aurora
Bridge Engineering Center
Construction Management & Technology
Iowa Local Technical Assistance Program
Iowa Traffic Safety Data Service
Midwest Transportation Consortium
National Concrete Pavement Technology Center
Partnership for Geotechnical Advancement
Roadway Infrastructure Management Systems
Statewide Urban Designs and Specifications
Sustainable Transportation Systems Program

National Concrete Pavement Technology Center
CP Tech Center

ctre
Center for Transportation Research and Education

Iowa State University

FHWA PPETG
New Orleans, LA
May 14-15, 2009

Subcommittee Report:
Support Pavement Preservation Centers and Regional and State Organizations

SHORT-TERM GOALS

1. **Develop and circulate a web-based survey to LTAP centers** – purpose is to identify the types of training, and network of speakers, available on pavement preservation.
   Status: Survey is presently being formatted for an on-line survey tool.

2. **Locate web sites at TRB for "Research in Progress"** – have these web sites linked with other PP websites (NCPP and FP2) for visibility:
   Status: Completed. Advised NCPP of six presentations on Pavement Preservation give an TRB 2008
   - Session 603 - "Performance of Pavement Crack Sealants" – Lynch
   - Session 646 - "Pavement Preservation: Economic Assessment of Alternatives"- Gregory
   - Session 696 - "Pavement Preservation: Performance and Prediction Models" – Mueller
   - Session 726 - "Pavement Preservation: Performance of Surface Treatments and Routine Maintenance" – McKemie
   - Session 756 - "Performance and Maintenance of Pavement Seals" – Moulthrop
   - Session 777 - "Advances in Pavement Chip Seals, Part 1" – Galehouse
   - Session 791 - "Advances in Pavement Chip Seals, Part 2" – Bahia

3. **Review and comment on websites:**
   - American Public Works Association (APWA) – **Completed** June 18, 2008.
   - California Pavement Preservation Center (CPPC) – **Completed** June 18, 2008.
   - National Association of County Engineers (NACE) – Schedule once NACE advises the website update is completed (Jon Rice will advise).
   - Transportation System Preservation Technical Services Program (TSP2) – **Completed** September 15, 2008.
   - FHWA Preservation – Schedule and evaluate if PPETG is being promoted.

   Added after Newport Meeting:
   - Iowa State University, Concrete Pavement Technology Program – Schedule evaluation (`www.cptechcenter.org`) – Schedule and evaluate.

4. **Identify representation from American Public Works Association (APWA)**
   Status: Completed. APWA President Noel Thompson recommended FHWA consider the appointment of Mr. Craig Olson, P.E. to the PPETG. Craig Olson is the Public Works Director/City Engineer for Clyde Hill, Washington and also a member of the APWA Transportation Committee. Craig is very interested in pavement preservation and has a background at both the City and State level.

5. **Advertise services provided by the Texas PP Center, including how they promote virtual online presentations and training.**
   Status: Agenda item for this PPETG meeting – presentation by Dr. Yetkin Yildrim.

6. **Work with Iowa State University to integrate preservation in the Concrete Pavement Technology Program, and identify the appropriate contact person.**
   Status: Transferred. This goal will be addressed by the subcommittee on "Portland Cement Concrete".

7. **Contact the Iowa PP Center and advise them of the services provided by this PPETG.**
   Status: Transferred. This goal will be addressed by the subcommittee on "Portland Cement Concrete".

Additional accomplishment:
Jon Rice gave a presentation at the 2009 NACE Annual Conference on April 22 on "Managing our County Road Pavement Preservation Program."

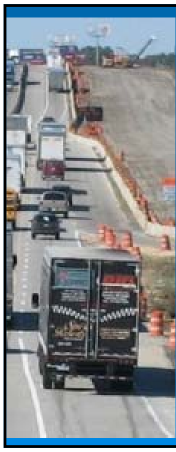
LONG-TERM GOALS

1. **Obtain pictures/videos of construction activities showing PP techniques for flexible and rigid pavements; forward for posting on NCPP as public domain.**
   Status: Ongoing. Solicit help from ETG members with future stimulus PP projects.
   - Mike Voth provided NCPP with information on PME study from their Utah preservation project. This information was also provided to the California Pavement Preservation Center for their review.
   - Provided CPCC, FP2 and NCPP with information on the Utah Cold In-place Recycling (CIR) Project on I-15.

2. **Provide FP2 and NCPP with links to web site for upcoming conferences and workshops – solicit assistance from APWA and NACE.**
   Status: Completed.
   - NACE and APWA conferences are now posted on NCPP's event calendar.
   - NACE website, under Resources, has links to the ETG, FP2 and NCPP, along with articles on pavement preservation.
   - Working with APWA to post articles on pavement preservation.
   - Provided NACE with Better Roads article, "Pavement Preservation – It's All About the Timing" to publish in the NACE Newsletter.
   - Provided FP2 and NCPP with Better Roads article, "Pavement Preservation – It's All About the Timing" to consider for inclusion on websites.

3. **Identify the way to integrate PP programs to be linked, if not integrated, with asset management systems.**
   Status: Transferred. This goal will be addressed by the subcommittee on "Pavement Preservation Acceptance & Implementation".

4. **Provide FP2 with links to PP award programs given by industry and non-profit associations (e.g.; AEME, ARRA, ISSA, CPPC, CCSA, etc.)**
   Status: Incomplete.



Brief Summary of

National Conference on Preservation, Repair, and Rehabilitation of Concrete Pavements

Expert Task Group on Pavement Preservation
May 14, 2009
New Orleans, Louisiana

Kurt Smith, Applied Pavement Technology, Inc.

Overview

Specifics

- Dates: April 22-24, 2009
- Location: Hyatt Regency, St. Louis
- Fourth in a series of conferences hosted by the FHWA under its Concrete Pavement Technology Program (CPTP)
  - Whitetopping
  - Long Life
  - Materials and Accelerated Construction
  - Preservation, Repair, and Rehabilitation (PRR)

Participants

Total Attendees = 147

- Eight Countries
  - Australia, Belgium, Canada, Colombia, South Africa, South Korea, Spain, USA
- Thirty States and DC
  - Arizona, California, Colorado, Connecticut, Delaware, Florida, Georgia, Illinois, Indiana, Iowa, Kansas, Maine, Maryland, Michigan, Minnesota, Missouri, Nebraska, Nevada, New York, North Carolina, North Dakota, Ohio, Oklahoma, Oregon, Pennsylvania, South Carolina, South Dakota, Texas, Virginia, Washington, & District of Columbia

Sponsors

- Federal Highway Administration
- AASHTO
- TRB
- ACPA
- IGGA
- Cement Association of Canada
- PCA
- Missouri DOT
- ISCP
- MO/KS ACPA
- National Concrete Pavement Technology Center

Conference Program

- Pre-conference workshops
- Seven topical sessions
- Three discussion forums

Pre-Conference Workshops

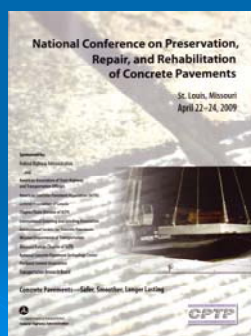
- Concrete Pavement Preservation
  - Pavement Preservation Overview
  - Treatment Design/Construction
  - Strategy Selection
- Life-Cycle Cost Analysis Forum
  - RealCost Applications
  - CA4PRS Demonstration
  - State LCCA Presentations

Topical Sessions

- Plenary Session
- Pavement Condition Evaluation
- Concrete Pavement Preservation, Repair, and Rehabilitation (I)
- Concrete Pavement Surface Texture
- Concrete Pavement Preservation, Repair, and Rehabilitation (II)
- Concrete Repair Techniques
- Emerging Technologies

Discussion Forums

- PRR Decision Making Process
- PRR Practices
- Alternative Delivery Methods for PRR



Conference Highlights

What's New

- Movement to less intrusive treatments
- Improved guidelines for preservation, repair, and rehabilitation (PRR) treatments
- More widespread use of cross-stitching
- Precast repairs becoming more mainstream
- Continued work in surface texturing
  - Diamond grinding for noise reductions
  - Next generation concrete surface
- Innovative use of thin concrete overlays
- Finding "buried treasure" by removing HMA overlays and re-exposing underlying PCC

What's The Same

- Confusion on preservation/repair/rehabilitation terminology and applications
- Demonstrated and effective use of "workhorse" rehabilitation treatments
  - Full-depth repairs
  - Diamond grinding for ride and safety
  - Dowel bar retrofit
- Continued mixed performance from:
  - Partial-depth repairs
  - Retrofitted edge drains ("*Don't drain if you won't maintain*")

Topical Highlights

- Demonstrated PRR Success
- Full-Depth Repair
- Cross Stitching
- Texturing
- Thin PCC Overlays
- Finding "Buried Treasure"

PRR Success

Demonstrated Success of PRR Procedures

- Missouri DOT
- Caltrans
- Highway 407, Ontario (private sector concession)
- Washington State Decision Criteria

PRR Success

Missouri PRR Experience

- Traditional treatments: FDR and HMA OL
- Early 2000s: adoption of less intrusive preservation treatments
  - Cross-stitching
  - Partial-depth repairs
  - Dowel bar retrofit
  - Diamond grinding
  - Undersealing
  - Slab jacking (bridge approaches)
- Overall good performance

PRR Success

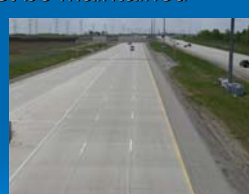
Caltrans

- Maintenance Technical Advisory Guide (MTAG) for Rigid Pavements (2nd *Ed. 03/08*)
  - Strategy selection
  - Joint resealing/crack sealing
  - Diamond grinding/grooving
  - Dowel bar retrofit
  - Partial-depth repair
  - Full-depth repair
- Particular emphasis on grinding, DBR, and FDR
- Ongoing investigations on optimal timing and key threshold values

PRR Success

Ontario (Highway 407 ETR)

- Private concession responsible for 1000 lane-km of highway (600 lane-km PCC)
- High level of service must be maintained
- Prefer techniques with minimal disruption
  - Crack stitching
  - Load transfer retrofit
  - Texturization
  - Undersealing
  - Full-depth concrete slab replacement
  - Thin surface restoration techniques



PRR Success

Highway 407 ETR Costs and Performance

Treatment	Expected benefit, years	Typical Unit Cost	
		Unit	Cost (US $)
Crack stitching	> 10	Each	$ 70
Dowel bar retrofit	8 – 10	Lane	$ 1,300
Exp. joint retrofit	15 – 20	Lane	$ 4,200
Texturization	4 – 6	m^2	$ 2 to 8
Subsealing/jacking	>10	kg	$ 15
Slab replacement	> 20	m^2	$ 375
Micro-surfacing	5 – 7	m^2	$ 4
Thin HMA overlay	8 – 10	m^2	$ 7

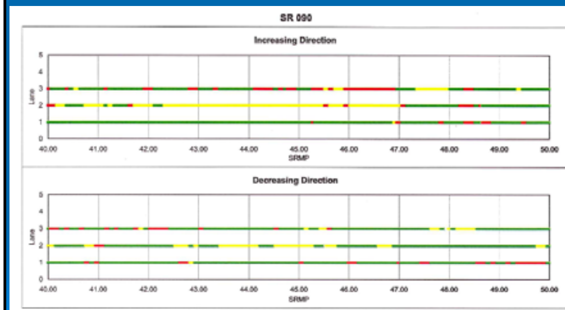
PRR Success

PRR Decision Criteria (Washington State DOT)

- Faulting < 1/8 inch & panels with multiple cracks < 10%: **Do Nothing**
- Faulting 1/8 to 1/2 inch & panels with multiple cracks < 10%: **Dowel Bar Retrofit**
- Faulting > 1/2 inch & panels with multiple cracks < 10%, and ADT > 50,000: **Reconstruction**
- Panels with multiple cracks > 10%: **Reconstruction**

PRR Success

WSDOT Rehab Criteria



Red Reconstruct
Yellow DBR
Green No Rehab Needed

Greater consideration of "surgical" rehabilitation in selected areas

Full-Depth Repair

Full-Depth Repair

- Continued mainstay of PRR
- Some concerns about the longevity and durability of "fast-track" repairs
- Increased use of precast slabs for FDR
  - Michigan
  - New Jersey
  - Virginia
  - Ontario

Full-Depth Repair

Why Precast Concrete?

- Need for rapid repair
- Higher quality concrete
- Better curing conditions (at fabrication plant)
- Few weather restrictions on placement
- No on-site curing of concrete required
- Intermittent (repairs) or continuous applications (local reconstruction)

Full-Depth Repair

Precast Slabs for Repair

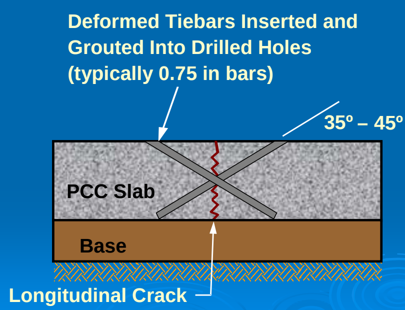
Cross Stitching

Cross Stitching

- Accepted treatment for
  - Early longitudinal cracks in new construction (late or shallow sawcutting)
  - Existing longitudinal cracks in older pavements
  - Misplacement of tie bars at construction
- Advantages:
  - Quick and easy to install
  - Less intrusive
- Good performance (MO, ONT, Europe)

Cross Stitching

Cross Stitching Schematic



Cross Stitching

Cross Stitching Process

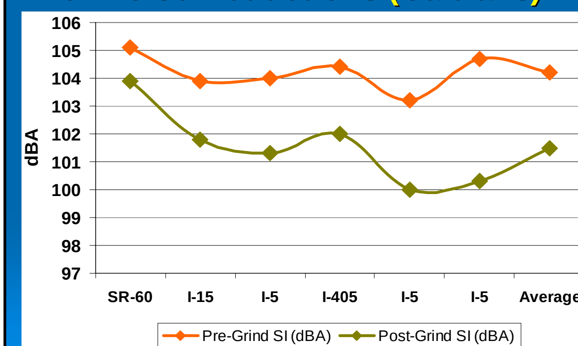
Texturing

Texturing

- Diamond grinding historically for restoration of pavement rideability and surface friction
  - Caltrans 17 years
  - Missouri 8+ years
  - Georgia 10+ years
- More recent benefit of reducing noise emissions
- Research and development of next generation concrete surface (NGCS)

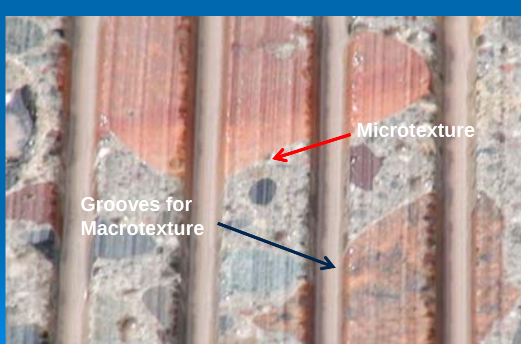
Texturing

Diamond Grinding for Noise Reductions (Caltrans)



Texturing

Next Generation Concrete Surface



Texturing

NGCS vs. Diamond Grinding

Thin PCC Overlays

Thin PCC Overlays

- Three recent unique applications:
  - KS: Bonded concrete inlay
  - MO: Unbonded concrete overlay with a geotextile interlayer
  - MI: Unbonded overlay of a composite pavement

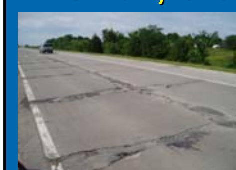
Thin PCC Overlays

KS: I-35 Bonded Overlay (2008)

- 9-in JRCP/CTB (30 ft slabs)
- Distress – extensive spalling
- Bonded overlay
  - Mill 2 inches
  - Shotblast & apply slurry
  - Place 2-inch bonded inlay

Thin PCC Overlays

MO: Route D UBOL, Greater KC Area (2008)



- 8 in JPCP w/ D-cracking
- Fabric interlayer
- 5-inch UBOL
- 6 ft x 6 ft panels

Thin PCC Overlays

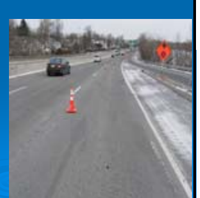
MI: Thin UBOL Gratiot Ave, Detroit (2005)

- 1.5-inch milling
- 1-inch HMA separator
- 4 inch JPCP
- 6 ft x 6 ft panels

Buried Treasure

Finding "Buried Treasure"

- Re-exposing PCC pavement that had been overlaid with HMA
- New Jersey Highway 21
  - 9 in JRCP built in 1931 & 1958 with numerous HMA overlays
  - Coring and subsurface investigations
    - ➢PCC structurally sound and in relatively good condition

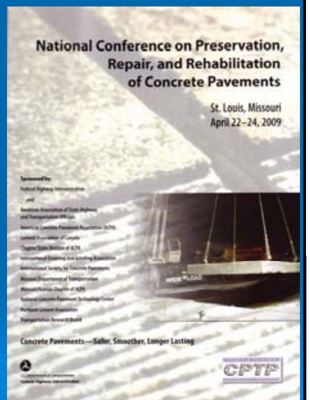


Buried Treasure

Work Activities

- HMA milling
- Slab stabilization
- Catch basin repair
- Precast full-depth
- Partial-depth repairs
- Diamond grinding
- Joint Resealing

National Conference on Preservation, Repair, and Rehabilitation of Concrete Pavements


National Conference on Preservation, Repair, and Rehabilitation of Concrete Pavements

St. Louis, Missouri
April 22–24, 2009

Concrete Pavements—Safer, Smoother, Longer Lasting

CPTP

Concrete Pavement Solutions for Lower Volume Roads

Jeffery Roesler, P.E.
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Department of Civil and Environmental Engineering
University of Illinois Urbana-Champaign
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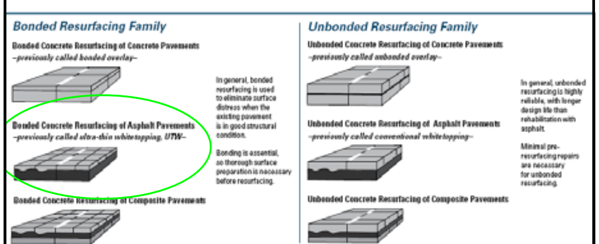
February 24, 2009
Traffic and Highway Engineering Conference
Urbana, IL

Outline

- Ultra-Thin Whitetopping (UTW)
  - UTW Design/Materials
  - UTW Construction
- Concrete Slabs w/ Optimized Slab Geometry
  - Test Sections
  - Preliminary Results

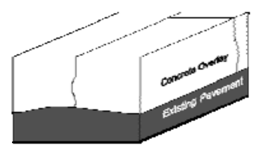
Concrete Overlays or Inlays



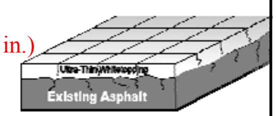
CPTech Center, 2008

UTW vs. "Whitetopping"

- Whitetopping (h> 6 in.)
  - Well-established
  - Proven pavement rehabilitation technique
  - Doesn't consider bond



- Ultra-Thin Whitetopping (h≤6 in.)
  - Recent technology
  - 10+ years experience
  - PCC/AC bond is essential



Applications for UTW

- Streets
- Intersections
- Bus pads
- County Roads
- Parking lots

Local UTW Projects

- UIUC Parking Lots
  - Talbot (1998)
  - E-15 Law School (2006)
  - McKinley Health Center (2007)
- U.S. 36 Decatur (1998) & Tuscola (1999)
- Monticello Road (2000)

UIUC Parking Lot E-15 (2006)

- UIUC campus FRC UTW Project
  - 2.5 inch AC
  - 3.5” Thick slab
  - 4’ x 4’ Joint spacing
  - 3 lb/yd^3 or 0.2% Fibers
  - Fly ash

Material	Amount (lb/yd^3)
Coarse Agg	1903
Fine Agg	1214
Cement	428
Water	219
Fly Ash	133
Strux Fibers	3
Admixture	Daracem 19

Asphalt Before

During Paving

Sawcutting

2007 UIUC Parking Lot Project

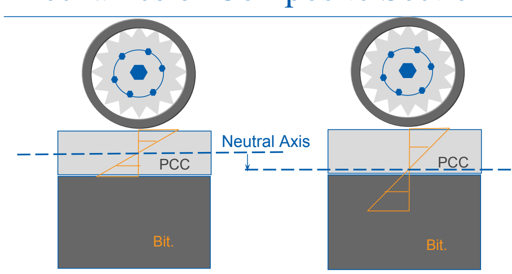
Piatt County: Highway 4 (2000)

Milled AC Surface and 5.5” PCC

UTW Design Questions?

- What condition of the existing HMA is acceptable?
- How do I ensure bond?
- What concrete material constituents and proportions do I use?
- What slab size do I specify?
- Aren’t fibers a waste of money?

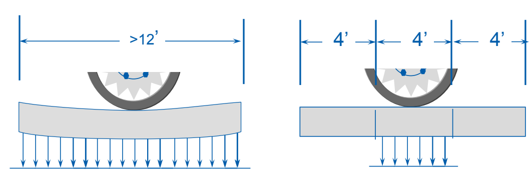
Mechanics of Composite Section



Unbonded **Bonded**

Riley

Short Joints Reduce Stresses



Univ. of IL UTW Project Overview

- UTW Projects in Illinois
- Evaluation of Overlays
- Concrete Material Testing
- Construction Recommendations
- Design Program and Material Guidelines

UTW Project ICT R27-3A
Acknowledgements

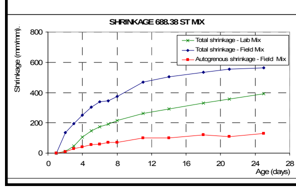
- Illinois Department of Transportation
  - *Illinois Center for Transportation*
  - *www.ict.uiuc.edu*
- Amanda Bordelon
  Graduate Research Assistant, UIUC
- Randall Riley
  IL-ACPA
- Anastasios Ioannides, Ph.D., P.E.
  Professor, University of Cincinnati
- Matthew Beyer & Dong Wang
  Graduate Research Assistant, UIUC

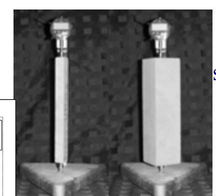
Optimizing Concrete Materials for UTW

- Cement content, aggregate type, and FIBERS
- Composite Beam Test
- Beam Fracture Tests
- Strength and Shrinkage Tests

Concrete Shrinkage

- Minimize autogenous and drying shrinkage
  - Pre-mature cracking
  - Bond failure

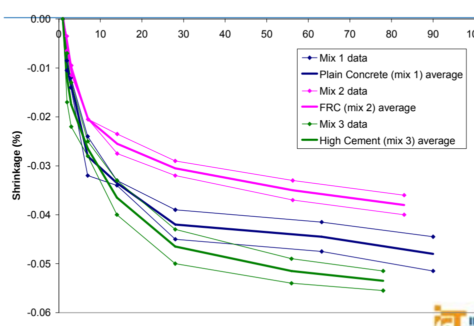




Concrete shrinkage prism

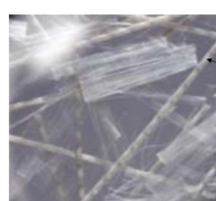
ASTM C157

Shrinkage Results

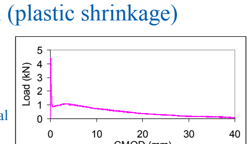


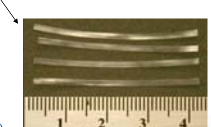
Fibers

- Structural vs. non-structural (plastic shrinkage)



Structural Fibers





Micro-Fibers (non-structural)

Mixture Designs for UTW Evaluation

- FRC - Schanck Ave (near Chicago, IL)
  - Studied with 0, 4, and 6 lb/cy fibers
  - Gravel versus limestone coarse aggregate
- High cement – Anna and Brazil
- Base design – JPCP – Dan Ryan (90/94 express lanes)

		Schanck Ave				Anna	Dan Ryan	Brazil 2
		Plain	4 lb	6 lb	Gravel			
Cement	lb/yd^3	517	518	522	493	774	447	748
Fly Ash	lb/yd^3	140	141	142	134	0	0	0
Slag	lb/yd^3	0	0	0	0	0	113	0
Silica Fume	lb/yd^3	0	0	0	0	0	0	75
Water	lb/yd^3	268	268	271	255	280	236	288
Coarse Aggregate	lb/yd^3	1978	1982	2000	1886	1851	1939	1926
Fine Aggregate	lb/yd^3	1004	1006	1015	957	1034	1264	940
Fibers	lb/yd^3	0	4	6	0	0	0	0
Air Entrainer	ml/yd^3	306	77	77	73	114	66	169
Water-Reducer	ml/yd^3	458	459	0	0	687	397	0
Super Plasticizer	ml/yd^3	0	0	463	0	0	0	917
w/cm ratio		0.41	0.41	0.41	0.41	0.36	0.42	0.35

Concrete Material Testing **Summary**

- *FRC vs. Plain*
  - Increase fracture response
  - Increases slab load capacity before cracking (slab tests)*
  - Increase load capacity after cracking
    - Composite beam and slab tests
- Many mixtures can be used for UTW but……
  - Limit cement content
    - Early shrinkage effects can lead to de-bonding
  - Need toughness
    - Large deformations
    - Post-crack performance

* FRC slabs tested by *Roesler et al., 2004*

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How to specify FRC?

- 7 fiber types studied
- Comparison of Flexure Strength Tests
  - ASTM 1018 ◄withdrawn in 2006
  - ASTM 1609-07
  - JCI-SF4
- Residual Strength and Strength Ratio determined

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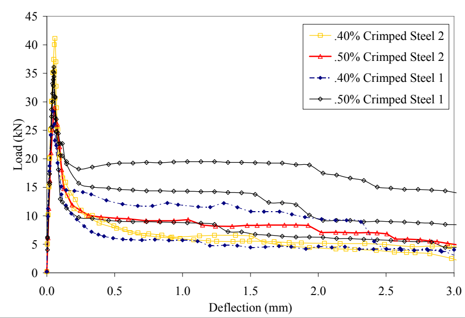
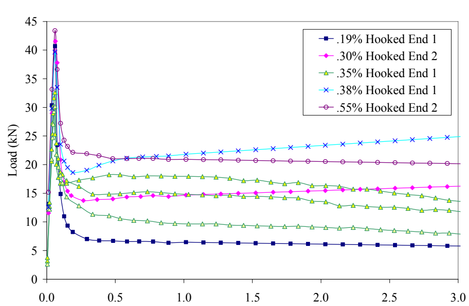
4-Point Bending Flexure Results
Crimped Steel



CENTER FOR ORTATION

4-Point Bending Flexure Results
Hooked End Steel



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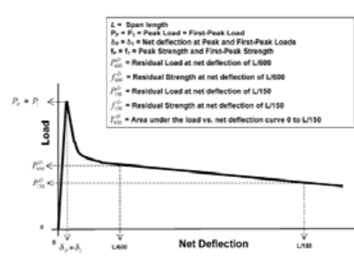
Flexure Test Method ASTM C1609-07



Beams: 6x6x21 in

Span: 18 in

L/150 = 0.12 in

$$MOR = \frac{P_1 L}{bd^2}$$

$$f_{150}^{150} = \frac{P_{150}^{150} L}{bd^2}$$

$$R_{150}^{150} = \frac{f_{150}^{150}}{MOR} * 100$$

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Test Results

Typical R_{150}^{150} values

Fiber Type	Volume Fraction	Dosage Used	Age	Mix	Peak Load	Flexural Strength (MOR)	JCI-SF4 Standard		ASTM C1609 Standard		
							$f_{e,3}$	$R_{e,3}$	f_{150}^{150}	T_{150}^{150}	R_{150}^{150}
	%	(lb/yd^3)	(days)	-	(lb)	(psi)	(psi)	(%)	(psi)	(lb-in)	(%)
Straight Synthetic	0.19	3.0	14	4	6,623	556	8	1.4	24	88	4.3
	0.26	4.0	14	4	5,472	456	92	20.2	83	130	18.3
	0.29	4.5	14	1	9,276	773	125	16.2	95	180	12.3
	0.33	5.0	56	2	8,138	680	148	21.8	126	193	18.5
	0.50	7.7	56	2	8,088	699	276	39.5	224	348	32.1
	0.58	8.9	14	1	8,939	745	347	46.6	382	500	51.3
Twisted Synthetic	0.30	4.6	14	2	8,101	675	84	12.5	68	120	10.1
	0.50	7.7	14	2	6,487	541	143	26.4	135	203	24.9
Crimped Synthetic	0.40	6.1	14	3	8,160	673	131	19.8	129	190	19.5
Crimped Steel 1	0.40	53.0	14	2	6,112	513	20	3.9	67	204	13.0
	0.50	66.0	56	2	9,052	766	269	35.1	185	347	24.1
Crimped Steel 2	0.40	52.8	14	3	8,828	710	117	16.5	64	175	8.9
	0.50	66.0	7	2	6,511	543	160	29.5	88	227	16.3
Hooked End 1	0.19	25.0	14	1	9,145	762	132	17.3	108	190	14.2
	0.35	46.2	56	2	8,278	678	291	42.8	234	385	34.5
	0.38	50.2	14	1	8,911	743	424	57.0	467	610	63.0
Hooked End 2	0.30	39.6	14	1	9,795	816	292	35.7	305	420	37.4
	0.55	72.6	14	1	9,754	813	396	48.7	377	570	46.4

How many fibers?

- Every fiber is different
  - Don't specify volume or weight!!
  - *Depends on concrete mixture*
- R_{150}^{150} = 20%
  - Distressed HMA and/or $h_{pcc} \leq 4$

Existing ACPA Design Guide

- Failure:
  - % cracked slabs and % reliability
- Material Inputs:
  - PCC thickness, MOR, Econc, joint spacing
  - FRC type and amount
  - AC thickness, Eac, surface preparation
  - Soil k-value
- Climate:
  - Temperature profile information (ΔT, CTE, %time at ΔT)
- Traffic:
  - load spectrum and design life

UIUC Modifications for IDOT

- ESALs instead of Load Spectra
- Simplify climate effect
- Use of R_{150}^{150} to determine fiber-reinforcement influence
- Removal of Asphalt Fatigue equations
- Characterize existing condition
- Simplified design spreadsheet!

UTW Design Equations

$$\log(\sigma_{18}) = 5.025 - 0.465\log(k) + 0.686\log(L/l_e) - 1.291\log(l_e)$$

$$\sigma_T = 28.037 - 3.496(CTE*\Delta T) - 18.382(L/l_e)$$

$$l_e = \left(\frac{I_e}{(1-0.15^2)*k}\right)^{0.25}$$

$$NA = \frac{\frac{(E_c t_{PCC}^{\;2})}{2} + E_b t_{BIT}\left(t_{PCC} + {t_{BIT}}/{2}\right)}{E_c t_{PCC} + E_b t_{BIT}}$$

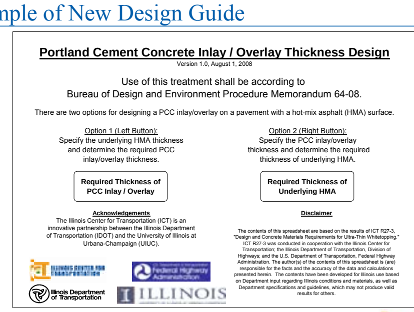
$$R^* = 1 - \frac{(1-R)*P_{cr}}{0.5}$$

$$\log N_{PCC} = \left[-\frac{SR_{total}^{\;-10.24}\log(R^*)}{0.0112}\right]^{0.217}$$

$$I_e = (Eh\wedge3/12)e = \frac{(E_c t_{PCC}^{\;3})}{12} + E_c t_{PCC}(NA - \frac{t_{PCC}}{2})^2 + \frac{(E_b t_{BIT}^{\;3})}{12} + E_b t_{BIT}(t_{PCC} - NA + \frac{t_{BIT}}{2})^2$$

Spreadsheet

Example of New Design Guide



Portland Cement Concrete Inlay / Overlay Thickness Design

Version 1.0, August 1, 2008

Use of this treatment shall be according to
Bureau of Design and Environment Procedure Memorandum 64-08.

There are two options for designing a PCC inlay/overlay on a pavement with a hot-mix asphalt (HMA) surface.

Option 1 (Left Button):
Specify the underlying HMA thickness and determine the required PCC inlay/overlay thickness.

Required Thickness of PCC Inlay / Overlay

Option 2 (Right Button):
Specify the PCC inlay/overlay thickness and determine the required thickness of underlying HMA.

Required Thickness of Underlying HMA

Acknowledgements

The Illinois Center for Transportation (ICT) is an innovative partnership between the Illinois Department of Transportation (IDOT) and the University of Illinois at Urbana-Champaign (UIUC).

Disclaimer

The contents of this spreadsheet are based on the results of ICT R27-3, "Design and Concrete Materials Requirements for Ultra-Thin Whitetopping." ICT R27-3 was conducted in cooperation with the Illinois Center for Transportation; the Illinois Department of Transportation, Division of Highways; and the U.S. Department of Transportation, Federal Highway Administration. The author(s) of the contents of this spreadsheet is (are) responsible for the facts and the accuracy of the data and calculations presented herein. The contents have been developed for Illinois use based on Department input regarding Illinois conditions and materials, as well as Department specifications and guidelines, which may not produce valid results for others.

Illinois Department of Transportation

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http://www.dot.state.il.us/desenv/PCC_Inlay-OverlayDesignSprdsht.xls

PCC Thickness Calculation

PCC Inlay / Overlay Design Sheet, Required Thickness of PCC

Help

Input	Variable	Value	Unit
Design Traffic Factor (BDE Manual, Figure 54-4C)	TF	2.50	
Modulus of Rupture (3-point bending, 14-day average)	MOR	750	psi
FRC Residual Strength Ratio	R_{150}^{150}	20%	
Remaining Thickness of Asphalt	h_{ac}	3.0	in.
Joint Spacing	L	72	in.
Elastic Modulus of Concrete	E_c	3,600,000	psi
Coefficient of Thermal Expansion	CTE	5.50E-06	in./in./°F
Elastic Modulus of Asphalt	E_{AC}	350,000	psi
Modulus of Subgrade Reaction	k	100	pci
Thickness of Concrete	h_c	**5.48**	in.

Solved

Compute Concrete Thickness

Default Inputs

Variable	*Default Value*
MOR	750 psi (Note 1)
R_{150}^{150}	0% (w/o fiber reinforcement) 20% (w/ fiber reinforcement)
L	48 in. or 72 in.
E_c	3,600,000 psi
CTE	5.50 x 10^{-6} in./in./°F
E_{AC}	100,000 psi (poor) 350,000 psi (moderate) 600,000 psi (good)
k	100 pci

Note 1: The design MOR is the mean design strength, not the minimum 550 psi flexural strength (center-point loading) specified for opening to traffic. Also note that as MOR increases the risk of debonding increases and the effectiveness of synthetic fibers decreases.

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Slab Thickness Table - Example

With Synthetic Fibers ($R_{150,3}$ = 20%)

Thickness of HMA Layer, h_{ac} = 2.5 in.

Design Parameters		
Traffic Factor for 48" Panel, *L*	**Traffic Factor for 72" Panel, *L***	**Inlay/Overlay Thickness, h_c** (in.)
---	---	3
≤ 0.065	---	3.5
≤ 0.7	---	4
≤ 5	≤ 0.05	4.5
≤ 5	≤ 0.27	5
≤ 5	≤ 1.2	5.5
≤ 5	≤ 4.5	6

Without Synthetic Fibers ($R_{150,3}$ = 0%)

Thickness of HMA Layer, h_{ac} = 2.5 in.

Design Parameters		
Traffic Factor for 48" Panel, *L*	**Traffic Factor for 72" Panel, *L***	**Inlay/Overlay Thickness, h_c** (in.)
≤ 0.042	---	4.5
≤ 0.15	---	5
≤ 0.45	≤ 0.014	5.5
≤ 1	≤ 0.033	6

Design Inputs: E_{AC} = 350,000 psi; E_C = 3,600,000 psi; MOR = 750 psi; k = 100 pci; CTE = 5.5 x 10^{-6} in./in./°F; P_{cr} = 20%; R = 85%; ΔT = -1.4 °F/in.; % *Time* = 58%

Where: E_{AC} = Elastic Modulus of HMA Layer
E_C = Elastic Modulus of PCC Overlay or Inlay
MOR = Modulus of Rupture
k = Modulus of Subgrade
CTE = Coefficient of Thermal Expansion
P_{cr} = Percent of Panels with Cracking
R = Reliability Factor
ΔT = Temperature Gradient
% *Time* = Occurrence of Temperature Gradient

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UTW Applicability

- Concrete inlay/overlay thickness
  - 3 to 6 inches
- Minimum HMA thickness
  - 2.5 inches
- Synthetic fibers
  - Required when $h_{PCC} \leq 4$ inches
  - Optional when $h_{PCC} > 4$ inches
- Maximum ESALs
  - 5 million
  - Traffic Factor = 5

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Concrete Strength

- Design Strength Input = 750psi @ 14 days
- IDOT Minimum Flexural Strength = 550 psi or compressive = 3,500 psi
  - 14 days

Note: 14-day center point = 90-day third point flexural strength

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Other Material Specifications

- Water-cementitious ratio
  - 0.42
  - Targets shrinkage and maximum strength
- Total Cementitious Content
  - 575 lb/cy or 605 lb/cy
- Aggregate size
  - Maximum coarse aggregate size = 1.0 inch

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Construction Recommendations

- Geometry constraints
  - Slab size 4 to 6 feet typical
- Bonding
  - Mill and clean best
- Saw-cut timing
  - Early-entry saws or soft-cut
  - Cure properly
  - Dependent on climate, geometry, and materials

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Bonding Issues

FIGURE 7 Milled HMA surface before overlay.

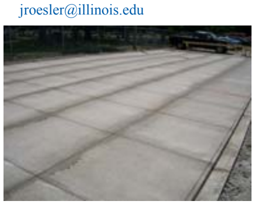
Rasmussen and Rozycki 2004

Keys to UTW Performance

- Adequate (HMA and soil) support layers
- PCC-HMA bond
- Slab size / joint spacing
- Concrete material selection
- Design Inputs
  - Traffic, layer thickness, climate, etc.

Design and Concrete Material Requirements for Ultra-Thin Whitetopping

Contact Info:
Jeff Roesler
jroesler@illinois.edu



Report: http://www.ict.uiuc.edu/Publications/
Spreadsheet: http://www.dot.state.il.us/desenv/PCC_Inlay-OverlayDesignSprdsht.xls

FHWA PPETG
TELECONFERENCES

This is a quick reference on how to schedule teleconferences so your Subcommittee can interact and address short/long-term goals between PPETG meetings.

1. Identify who on your Subcommittee has the ability to connect members via telephone – either by conference call or a toll-free number (with access code). This member will serve as your teleconference coordinator.

2. Teleconferences should be limited to a specific length of time, such as one hour, to make it practical for members to schedule on their calendar.

   The Subcommittee Chair emails the subcommittee members to identify a "one-hour" period that typically will work for each of them. Request the members to respond by a specific date.

   Identify a one-hour time period the members are able to participate. Pay attention to the time the members provide; your members are most likely located in various parts of the country. The following are equivalent time zones for the same one-hour time period:
   - 8:00-9:00 am Pacific time
   - 9:00-10:00 am Mountain time
   - 10:00-11:00 am Central time
   - 11:00-12:00 pm Eastern time

3. The Subcommittee Chair then emails its members requesting future date availability; for example, the next two or three months. Request members to respond by a specific date.

4. The Subcommittee Chair then identifies dates the Subcommittee Chair and the teleconference coordinator are both available. However, the teleconference coordinator is not mandatory if a toll-free number (with access code) is being provided for the members to call and be connected to the teleconference.

   All members may not be available for each teleconference. Try to pick dates that provides you the most availability, or at least the members you need present for the topic to be discussed.

5. The Subcommittee Chair sends an email to the members identifying what dates the teleconferences will occur and request they place them on their calendar. It is a good idea to say what topic will be discussed at the upcoming teleconference(s).

6. The week before each teleconference, remind all members of the upcoming teleconference. If a toll-free number is not being used, remind the members to email the teleconference coordinator with the number they can be reached so they can be connected to the teleconference.

ROUGH ROADS AHEAD

FIX THEM NOW OR PAY FOR IT LATER

AASHTO
THE VOICE OF TRANSPORTATION

TRIP
a national transportation research group

What's Wrong with Our Roads?

Killer potholes. In a flash they can dislodge a hubcap, shred a tire, or even worse, cause a driver to lose control of a car. But they can also be a symptom of a much deeper problem —deteriorating pavement that takes much more to repair than a simple patch.

As fundamental as our transportation system is to our daily lives, our highways and bridges are aging, under-funded, and inadequate to meet the demands we place upon them today, much less in the future. And across America motorists are paying the price.

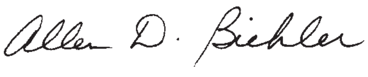
For state departments of transportation, preserving the condition and performance of the transportation system we have built is the top priority.

In Pennsylvania, for example, work will begin later this year on more than 240 projects to repair and improve 608 miles of highway and 399 bridges. The projects will be financed with $1 billion in federal economic-stimulus money combined with about $2 billion in federal and state funds. This represents the most the Pennsylvania Transportation Department has ever committed to construction in a single year.

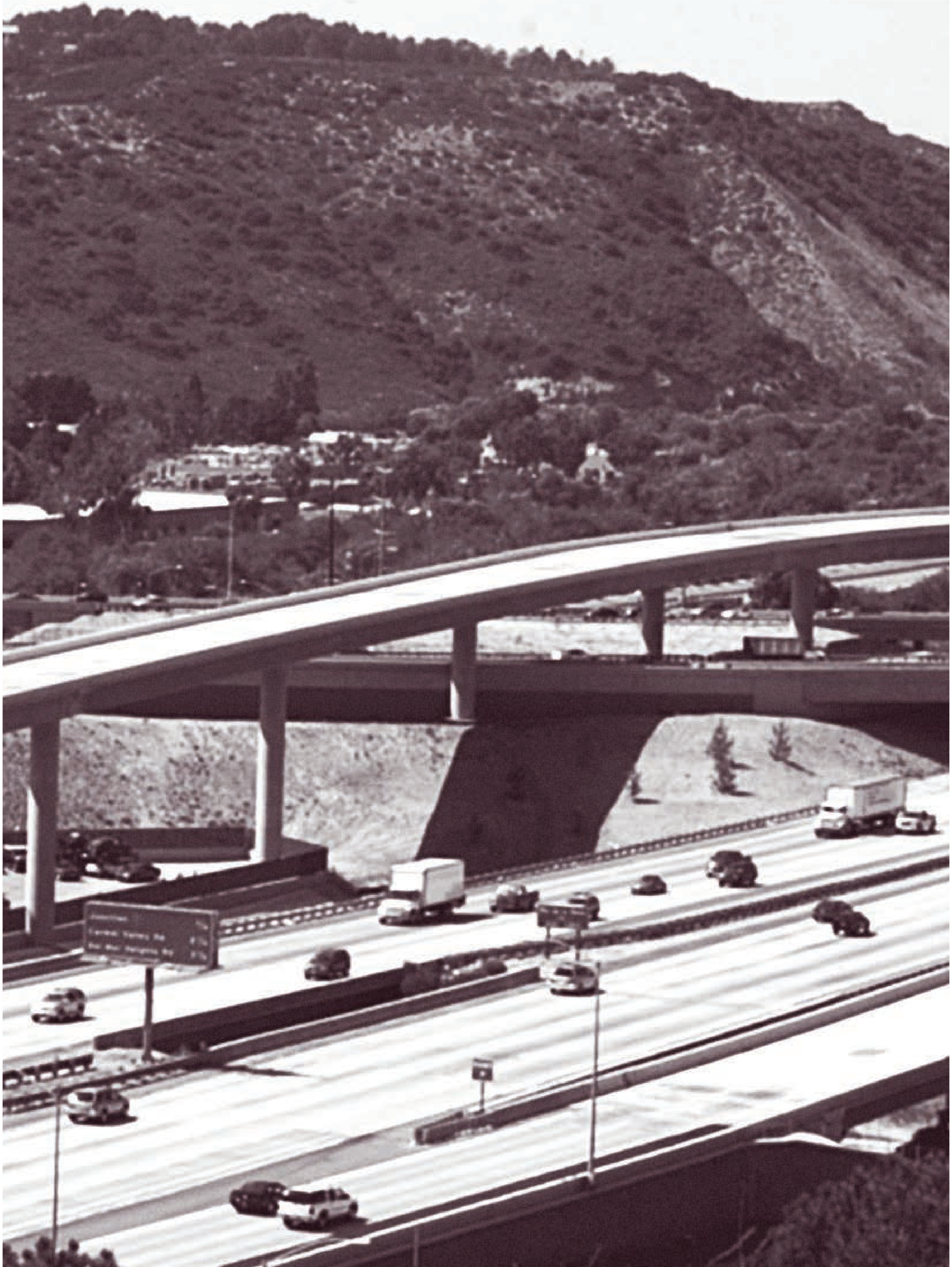
New technology, materials, and procedures are helping extend the life of our highways and bridges. States are also spending "smart" by making the investments needed to keep a road in good repair, rather than paying more later to address greater deterioration.

But the needs are enormous and poor-quality pavement is reflected in the increased operating costs that motorists must pay.

This report, developed by AASHTO in conjunction with TRIP, a national transportation research group, documents the preservation needs of the nation's highways and the solutions that can be applied. As we look to the next authorization of federal-aid surface transportation programs, rebuilding and improving our nation's core transportation infrastructure must be a fundamental goal.



Allen D. Biehler
Secretary, Pennsylvania Department of Transportation
President, AASHTO



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Rough Roads Ahead, Fix Them Now or Pay for It Later, is a joint product of AASHTO, the American Association of State Highway and Transportation Officials, and TRIP.

AASHTO is the "Voice of Transportation" representing State Departments of Transportation in all 50 states, the District of Columbia, and Puerto Rico. A nonprofit, nonpartisan association, AASHTO serves as a catalyst for excellence in transportation. TRIP is a national highway research group based in Washington, DC.

These associations gratefully acknowledge the work of Christine Becker of Christine Becker Associates, and Frank Moretti, Director of Policy and Research, TRIP, for their efforts in developing this report. Thanks, too, to the Federal Highway Administration and the many state DOTs who contributed their information and photographs for this report.

Cover photo: Valerie Sinco

Executive Summary

ROUGH ROADS AHEAD: SAVING AMERICA'S HIGHWAYS

America's $1.75 trillion public highway system is in jeopardy. Years of wear and tear, unrelenting traffic, an explosion of heavy trucks, deferred maintenance, harsh weather conditions, and soaring construction costs have taken their toll on America's roads.

While the American Reinvestment and Recovery Act of 2009 will provide $27 billion for highway projects, that money will barely make a dent in highway maintenance, preservation, and reconstruction needs. The recent AASHTO *Bottom Line* report documented the need for all levels of government to invest $166 billion each year in highways and bridges. More than half of that amount would be needed for system preservation.

Saving America's highways demands more than short-term stimulus funds and quick fixes based on available funding. It will require a greater and smarter investment of transportation dollars to ensure a new and better transportation program.

A Snapshot of Rough Roads

- Only half of the nation's major roads are in good condition.
- One in four urban roads is in poor condition.
- Major urban centers have the roughest roads—some with more than 60 percent of roads in poor condition.
- Rural roads are in better condition than urban roads. In 2007, 60 percent of rural roads were in good condition.
- Overall, 72 percent of the Interstate Highway System is in good condition. But age, weather conditions, and burgeoning traffic—particularly multi-axle trucks—are eroding ride quality. In eight states, 20 percent of the Interstate highways were rated as mediocre or poor.

ROUGH ROADS LEAD TO HIGHER COSTS

Only half of the nation's major roads are in good condition, based on an analysis of recent Federal Highway Administration data. The situation is worse in high traffic, urban areas where one in four roads is in poor condition. In some major urban centers, more than 60 percent of roads are in poor condition.

The American public pays for poor road conditions twice—first through additional vehicle operating costs and then in higher repair and reconstruction costs. For the average driver, rough roads add $335 annually to typical vehicle operating costs. In urban areas with high concentrations of rough roads, extra vehicle operating costs can be as high as $746 annually.

Sustaining deteriorating roads costs significantly more over time than regularly maintaining a road in good condition. Costs per lane mile for reconstruction after 25 years can be more than three times the costs of preservation treatments over the same 25-year period.

CHALLENGES FACING AMERICA'S HIGHWAYS

Unrelenting traffic is tough on roads. Traffic growth has far outpaced highway construction, particularly in major metropolitan areas. The number of miles driven in this country jumped more than 41 percent from 1990 to 2007—from 2.1 trillion miles in 1990 to 3 trillion in 2007. Nearly 66 percent of that driving passed over urban roads, which are showing the most wear and tear. In some parts of the county, dramatic population growth

Courtesy of Missouri Department of Transportation.

has occurred without much of an increase in road capacity, placing enormous pressure on roads that, in many cases, were built 50 years ago.

Soaring construction costs during the past five years are straining state and local budgets. By the summer of 2008, asphalt prices were up 70 percent, concrete 36 percent, and steel 105 percent. Diesel fuel, used to operate heavy construction equipment, soared 305 percent, including a 63 percent jump in one year. Over time, these higher costs have eroded states' purchasing power on construction projects. In the past few months, however, the economic recession appears to have moderated some of these costs. In fact, many bids for stimulus projects are coming in below engineers' estimates.

The explosion of freight truck traffic is punishing aging highways. The Interstate system is bearing the brunt of truck traffic and showing the impact. Today, on average, every mile of Interstate highway sees 10,500 trucks a day. More than 80 percent of freight tonnage moving across the United States is carried by trucks driving on the 50-year-old Interstate system.

> "Managing a highway system is like playing chess. You have to look at the whole board, the whole system, not just the next move. Sure we do reactive things, but our best strategy is when we look down the road eight years or more, look at every section of road, and budget to keep those roads in good condition."

—Gary Ridley, Director, Oklahoma Department of Transportation

Investment has not kept up with maintenance and preservation needs. Delayed and deferred maintenance leads to higher repair and reconstruction costs—pay me now or pay me more, lots more, later. Michigan DOT Director Kirk L. Steudle said, “It is important to slow the rate of decline in the good road so that it stays in good shape rather than slipping into fair or poor condition.” Spending $1 to keep a road in good condition prevents spending $7 to reconstruct it once it has fallen into poor condition, he added. But soaring construction costs, tight budgets, and increasing needs make it hard for states to sustain preservation programs. That is why most states are using their stimulus funds to make up for lost time from deferred maintenance and preservation.

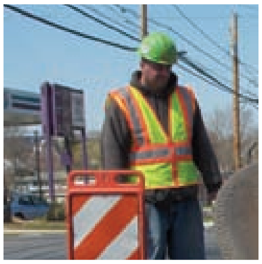
HIGHWAY MAINTENANCE NEEDS EXCEED AVAILABLE FUNDS

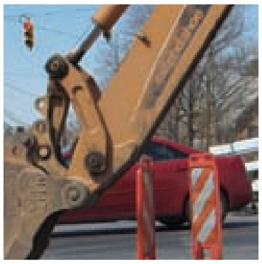
Keeping good roads in good condition is the most cost-effective way to save America’s highways. But the needs are high and the available funding limited. For example:

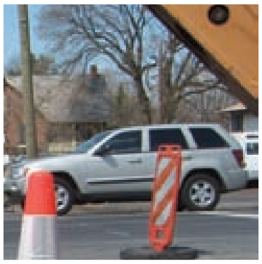
- **Oregon** needs $200 million annually over the next 10 years to maintain roads at the current levels. It has $130 million available annually.
- **Texas** needs $73 billion during the next 22 years to maintain current conditions. The Department is spending $900 million per year and losing ground.
- **Rhode Island** needs $640 million annually to preserve its highway system and has only $354 million available each year.

Stimulus funds will fill in some of the gaps.

- **Oregon** will use half of its $224 million of stimulus funds for pavement resurfacing and preservation projects.
- **Texas** is spending $800 million in stimulus funds to stabilize pavement and bridge conditions for the next few years.
- **Rhode Island** will use its $137 million primarily for preservation and maintenance projects. The extra funds provide about 5 percent of the projected shortfall in preservation funds over the next 10 years.
- **South Dakota’s** stimulus allocation will provide about one year’s worth of preservation funding to help with the backlog of needs.







Courtesy of Pennsylvania DOT.

STRATEGIES FOR SAVING AMERICA'S HIGHWAYS

Use the best materials throughout the life of a road. From filling a pothole to reconstructing a major highway, using materials designed to meet specific climate and traffic conditions will extend the service life of a road and reduce costs over the long run. Research into new materials, constant monitoring of pavement conditions, and matching materials to traffic and weather conditions all contribute to long-term durability of a road.

Keep good roads good. Maintaining a road in good condition is easier and less expensive than repairing one in poor condition. Achieving that goal involves a carefully planned and consistently funded pavement preservation program that makes proactive improvements in good roads to keep them good. "You can spend too much time and money chasing after potholes while watching the system fall farther and farther behind," said Pennsylvania DOT Secretary Allen Biehler.

Create a multi-modal freight strategy. Ensuring that roads can handle the projected growth in freight-bearing trucks involves more than building sturdier roads. It will require a commitment to a multi-modal freight strategy that may include (1) building a network of dedicated truck lanes; (2) expanding rail capacity to sustain its share of freight movement; (3) fixing bottlenecks and reducing congestion in metropolitan areas; (4) improving conditions from ports and distribution centers to the Interstate and rail systems; and (5) a funding model that includes freight-related user fees to implement the strategy.

View highways as public assets to be managed rather than projects to be fixed. Asset management is a comprehensive approach to ensuring the most cost-effective return on investments for operating, maintaining, upgrading, and expanding transportation systems. It starts from the assumption that the nearly 4 million miles of public roads are a valuable national asset, essential to the vitality of the American economy.

Invest to save America's highways. When the Interstate system was first designed in the 1940s, lines were put on a map to describe the vision for a country connected by a network of limited access highways. "Planners said this is what we want it to look like. Now let's figure out how to pay for it," said Oklahoma DOT Director Ridley. "Now we work in the reverse. We say here's how much money we have, and let's decide what we want to do with that. That approach doesn't produce the best decisions." Rebuilding for the future requires a national commitment to significant and sustained investment in transportation infrastructure based on a vision of what we want our transportation system to look like in the 21st century and beyond.

It is time for a greater and smarter investment of transportation dollars to ensure a new and better transportation program.

Are we there yet? No—but we can be.

> "We as stewards of the transportation system have no choice but to drive home the message that maintaining an acceptable condition for our highways—preserving the system—is vital to our country's future."
>
> —Allen D. Biehler, AASHTO President;
> Secretary, Pennsylvania Department of Transportation

Highways to Everywhere

A well-connected highway system, maintained in good condition, is critical to the nation's economy. With a current value of $1.75 trillion, preserving the system of roads and highways so they last for generations and meet changing needs should be a top priority for all levels of government. Even with continued growth in public transit, enhanced rail services, and a national commitment to reduce greenhouse gas emissions from vehicles, roads remain a vital component of the system that moves people and goods throughout the country.

Roads are essential to everyday life.

- Nearly 24 million children—55 percent of the country's kindergarten through high school population—ride 450,000 school buses 180 days per year.
- Every year, 50,000 ambulances make 60 million trips—that is an average of 164,000 trips per day.
- A fire department responds in one or more vehicles to a fire alarm in the United States every 20 seconds.
- Trucks in the United States carry 32 million tons of goods valued at $25 billion every day.
- The country's 240 million registered vehicles travel more than 2.9 trillion miles annually.

Those vehicles, and the people who drive and ride in them, rely on the nation's nearly 4 million miles of public roads—from Interstate highways to neighborhood streets—to get somewhere to do something.

Highways are a backbone of American life, connecting people, goods, and services. But many roads, particularly in metropolitan areas and population growth centers, are in poor condition. Years of wear and tear, unrelenting traffic, an explosion of heavy trucks, weather conditions, and delayed maintenance because of tight budgets and soaring construction costs have taken their toll on America's roads.

Despite the recent downturn in travel in 2008, the number of miles driven on the nation's roadways has increased 41 percent from 1990 to 2007. Large commercial truck traffic, which places significant stress on pavements, has increased 50 percent during the same time frame.

In some parts of the country, dramatic population growth with minimal capacity expansion has placed enormous pressure on highways. For example, in Utah, between 1990 and 2007, population grew by 47 percent and miles driven by 71 percent—but highway capacity grew by only 4 percent.[1]

Transportation officials across the country are focusing on how to preserve and protect their part of this national asset by building smarter, investing in systematic maintenance programs, and using new technologies to produce longer-lasting roads.

This report examines the condition of America's roads and what it will take to save them.

THE NATION'S HIGHWAYS BY THE NUMBERS

Total miles of public roads—3,967,159

Total miles of roads by ownership
- Federal—128,378 miles (3.2 percent)
- State—783,643 miles (19.8 percent)
- Local—3,055,138 miles (77 percent)

Total miles of rural and urban roads
- Rural - 2,939,042 (74 percent)
- Urban - 1,028,107 (26 percent)

Total Interstate Highway miles—47,000

Annual miles driven in cars and trucks—2.9 trillion

Percent of miles driven on urban roads—65.6 percent

Tons of freight moved on America's highways annually—15 billion

Early History of United States Road Building

1625 Earliest known paved American road—Pemaquid, Maine

1795 First engineered American road—Philadelphia to Lancaster toll turnpike

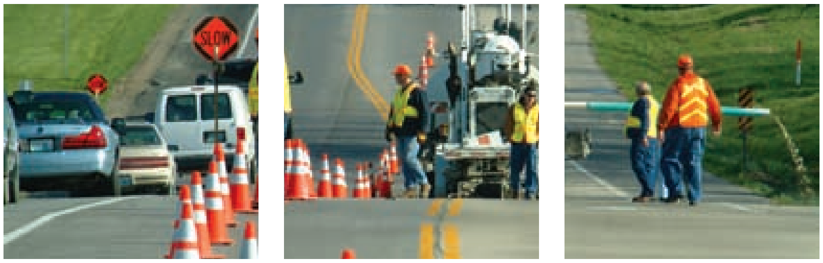
1823 First macadam road constructed in America—Maryland

1872 First asphalt paved roads in North America—Pennsylvania Avenue in Washington, DC, and Fifth Avenue in New York, NY

1893 First rural brick road—Ohio

1906 First bituminous macadam road—Rhode Island

Hammond Surface Streets, Hammond, Indiana



Courtesy of Missouri DOT.

PennDOT workers power wash a bridge structure. Keeping the expansion areas and joints of bridges free of debris and salt accumulation from winter services is a critical maintenance function.

Courtesy of Pennsylvania DOT.

Chapter 1

Rough Roads—Facing the Facts

Potholes are the poster child for rough roads. They are a nuisance, a source of wear and tear on vehicle suspensions and tires, and a safety risk. They can also be indicators of serious road deterioration.

The traveling public values smooth roads. In addition to ride quality, smooth roads improve fuel efficiency, reduce vehicle wear and tear, improve driver safety, and last longer. But how smooth a road needs to be to keep the public happy can vary widely.

The Missouri Department of Transportation (MoDOT) relied on public opinion to shape its Smooth Roads Initiative. Key elements of the Missouri Smooth Roads Initiative were:

- SMOOTHER—pavements were resurfaced, where needed.
- SAFER—striping and delineation improvements were made at all sites in the program.
- SOONER—the entire program for improving 2,300 miles of roadway was completed in only two years.

MoDOT Director Pete K. Rahn said citizen input helped set priorities for transportation investments. “For example, we thought mowing all rights of ways regularly was very important. The citizens told us it wasn’t a high priority for them,” Rahn said.

Nearly 900 citizens participated in a series of road rallies to help the state determine how rough was too rough. Citizens rode in vans with a moderator who tracked their comments as they assessed ride quality along the way.

“What we thought was a rough ride sometimes wasn’t,” Rahn said. “We plan to use a second round of van assessments for our continuing smoothness program.”

The program was launched after voters passed an initiative by a 4 to 1 margin to fund improvements in the state’s highway system. Phase I improved 2,300 highway miles that account for 60 percent of all traffic on the state system, producing an 18 percent increase in Interstate smoothness over a two-year period. Phase II—Better Roads, Brighter Future—is addressing the remainder of the state’s 5,600-mile major highway system. The goal is to bring 85 percent of Missouri’s major highway system up to good condition.

> “A pothole is like a tooth cavity. Left untreated it gets more decayed, more painful, takes more time and money to care for, and sometimes you end up having to urgently call in a specialist. But like cavities, potholes can be prevented.”
>
> “The Fine Art of Pothology: Preventing and Repairing Potholes”
>
> *Better Roads*, March 2009

RATING YOUR RIDE

States generally use the International Roughness Index (IRI) to rate road conditions. Those ratings are used to monitor pavement performance and schedule maintenance and rehabilitation plans. Roads with low IRI ratings are the smoothest. Roads with higher IRI ratings are likely to have cracked or broken pavements and may show significant distress in their underlying foundations.

To get a national perspective on road conditions, the Federal Highway Administration (FHWA) collects data from states annually and summarizes ride conditions using four categories—good, fair, mediocre, and poor. The categories are based partly on a study that measured driver reactions to various road conditions.[2]

Here's what the most recent data shows:

- Only half of the nation's major roads—Interstates, freeways, and other major routes—are in good condition. Unfortunately, 13 percent are in poor condition.

- Rural roads are smoother and in better condition than urban roads. In 2007, 61 percent of rural roadways were in good condition.

- Overall, 72 percent of the Interstate Highway System is rated in good condition. But, age, weather conditions, and burgeoning traffic are eroding ride quality in many states. In eight states, more than 20 percent of the Interstate highways were rated as mediocre or poor.

- One in four urban roads—which carry the brunt of national traffic—are in poor condition.

- Road conditions in urban areas actually improved between 2002 and 2006, but declined in 2007, when 26 percent were reported in poor condition. Factors that may have contributed to a higher percentage of rough roads include aging of urban roads, unrelenting traffic, heavier trucks carrying freight loads, and deferred or delayed maintenance because of tight budgets and soaring construction costs.[3]

- Major urban centers have the roughest roads—more than 60 percent of the roads in the Los Angeles, San Jose, San Francisco-Oakland, and Honolulu areas provide a poor-quality ride.[4]

Pavement Conditions of Urban and Rural Arterial Highways in 2007

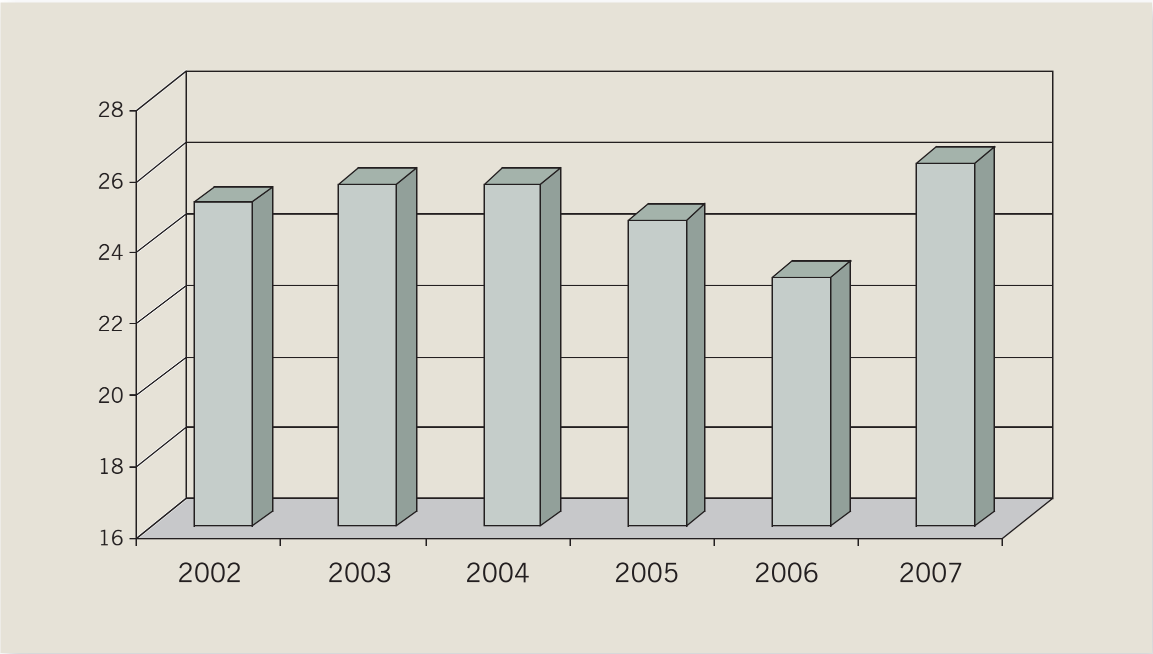
	Rural	Urban	All Major Roads
Poor	4%	26%	13%
Mediocre	15%	27%	20%
Fair	20%	11%	16%
Good	61%	36%	51%

Source: TRIP analysis of FHWA data.

URBAN ROADS MOST TRAVELED

The condition of the nation's major urban roadways is of particular concern to the nation's motorists because these roads and highways are the most heavily traveled in the nation. In 2007, 66 percent of the nation's vehicle travel was carried by its urban roads and highways.[5]

Percentage of Major Urban Roads with Pavements in Poor Condition, 2002 to 2007



Source: TRIP analysis of Federal Highway Administration data.

Although road deterioration is often accelerated by freeze-thaw cycles found most often in the nation's northern states, the urban areas with the highest share of poor pavement conditions in the nation actually include urban areas from a variety of regions.

Urban areas (population 500,000 or more) with highest share of roads in poor condition, 2007

Includes state, city, and county arterial networks in cities and surrounding suburbs

Urban Area	Pct. Poor
Los Angeles	64
San Jose	61
San Francisco - Oakland	61
Honolulu	61
Concord, CA	54
New York - Newark	54
San Diego	53
New Orleans	49
Tulsa	47
Palm Springs - Indio, CA	47
Riverside - San Bernardino, CA	44
Baltimore	44
Sacramento	44
Omaha	41
Oklahoma City	41
San Antonio	38
Mission Viejo, CA	37
Albuquerque	36
Philadelphia	36
Detroit	36

Source: TRIP analysis of Federal Highway Administration data.

Road conditions for urban areas with populations of 500,000 or greater can be found in appendix A. Road condition data for urban areas with populations from 250,000 to 499,000 can be found in appendix B.

Courtesy of National Concrete Pavement Technology Center.

THE COST OF ROUGH ROADS

The American public pays for poor pavement conditions twice—first through additional vehicle operating costs, and then in higher costs to restore pavement to good condition.

Driving on rough roads accelerates vehicle depreciation, reduces fuel efficiency, and damages tires and suspension. TRIP estimates that for the average driver, rough roads add $335 annually to typical vehicle operating costs. In urban areas with high concentrations of rough roads, extra vehicle operating costs are as high as $746.[6] Generally, larger vehicles have a greater increase in operating costs due to rough roads.

This cost estimate is developed using a model that factors in average number of miles driven annually and AAA's 2008 vehicle operating cost data.[7] Research on the impact of road conditions on fuel consumption by the Texas Transportation Institute (TTI) is also factored into the methodology.[8]

Urban Areas with Highest Additional Vehicle Operating Costs Due to Rough Roads, 2007

Includes cities and surrounding suburbs with populations of 500,000 or more

Urban Area	Additional Costs
Los Angeles	$746
San Jose	$732
San Francisco - Oakland	$705
Tulsa	$703
Honolulu	$688
San Diego	$664
Concord, CA	$656
New York - Newark	$638
Riverside - San Bernardino, CA	$632
Oklahoma City	$631
Sacramento	$622
New Orleans	$622
Palm Springs - Indio, CA	$608
Omaha	$592
Baltimore	$589
Albuquerque	$576
Mission Viejo, CA	$571
San Antonio	$529
Detroit	$525
Philadelphia	$525

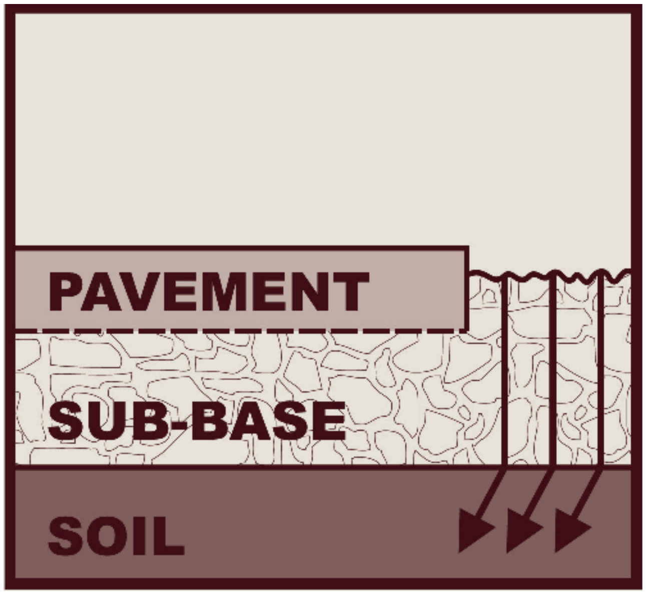
Source: TRIP analysis based on Federal Highway Administration data.

A STITCH IN TIME

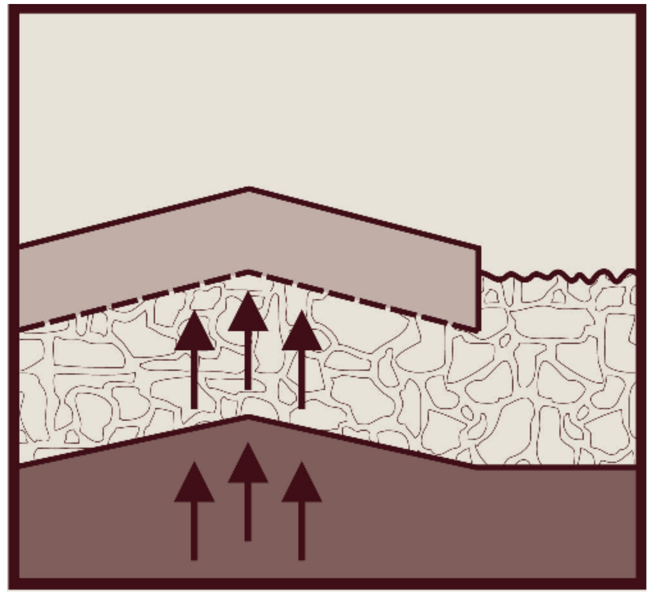
Age, weather, moisture, traffic, heavy trucks, and delayed maintenance cause roads to deteriorate. Old roads eventually wear out—particularly ones that were built 50 or more years ago with less sophisticated construction materials and lower traffic expectations. Moisture, freezing, thawing, and poor drainage also contribute to cracks, ruts, potholes, and foundation deterioration.

Potholes form when moisture from rain or snow works its way into road surfaces and the foundation bed, creating openings and cracks in the pavement that gradually grow larger as traffic passes over the surface. Road surfaces at intersections are especially vulnerable, since slow-moving, stopping, or starting traffic—particularly heavier vehicles—causes higher levels of pavement stress.

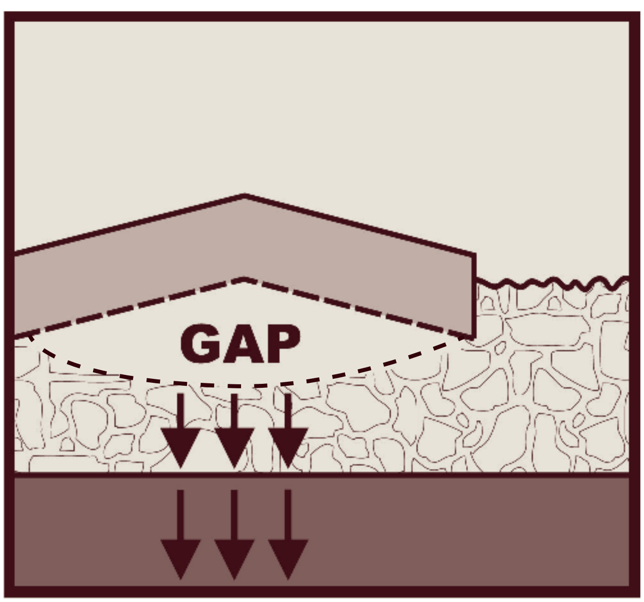
BIRTH OF A POTHOLE



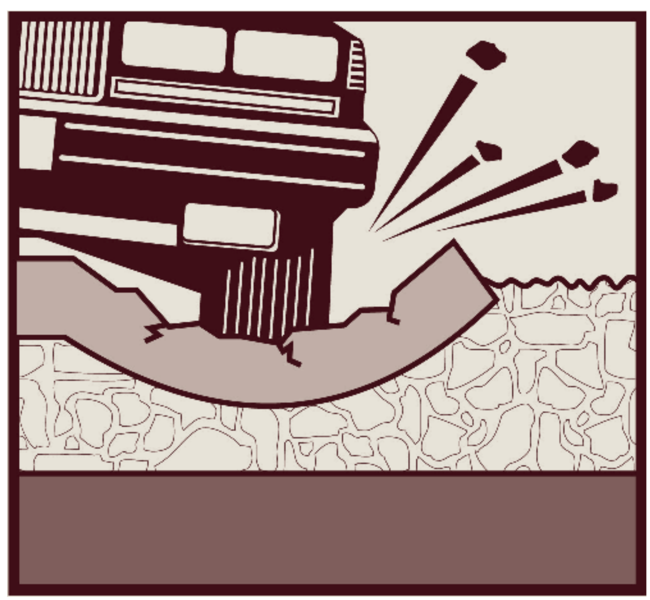
Potholes begin after rain or snow seeps into cracks and down into the soil below the road surface. The soil turns into mud and with no support, a hole can form under the pavement.



Repeated freezing and thawing or heavy traffic causes the ground to expand, pushing the pavement up.



As temperatures rise, the ground returns to a normal level but the pavement often remains raised. This creates a gap, or hollow space between the pavement and the ground below it.



When vehicles drive over this cavity, the pavement surface cracks and falls into the hollow space, leading to the birth of another pothole.

Courtesy of Michigan Department of Transportation.

Roads have five life cycle stages from initial design to disintegration and failure. Actions taken at each stage can affect the long-term durability of the road as well as maintenance and preservation costs. Higher quality investments earlier in the life of the road will save money over the long run because maintaining a road in good condition is less expensive than repairing or rebuilding one in poor condition.[9]

Reconstructing a road that has reached Stage 5 costs significantly more than preserving a road at Stage 3.

Life Cycle of a Road

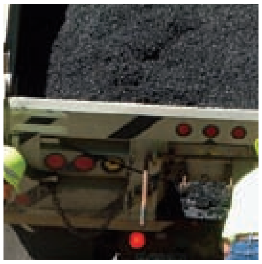
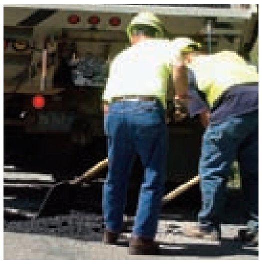
1 **Design**—This stage deals with dimensions, type of materials, thickness of base and top surfaces, and the drainage system. Investments made at the design stage affect the long-term durability of the pavement surface. If, however, sufficient funding is not available to upgrade the design, the road starts out and stays mediocre.

2 **Construction**—A high-quality construction process produces a longer-lasting pavement surface.

3 **Initial Deterioration**—During the first few years of use, the road surface starts to experience some initial deterioration caused by traffic volume, rain, snow, solar radiation, and temperature changes. At this stage, the road appears in good condition, providing a smooth ride. Preservation strategies during Stage 3 will sustain the smooth ride, preserve the foundation, extend the life, and reduce the need for costly reconstruction later on.

4 **Visible Deterioration**—At Stage 4, visible signs of distress such as potholes and cracking occur. Repairs made at this stage using overlays and milling to eliminate ruts will restore a smooth ride and extend the life of the road.

5 **Disintegration and Failure**—Roads not maintained at Stage 3 and repaired at Stage 4, eventually will fail and need costly reconstruction. Once a road's foundation disintegrates, surface repairs have an increasingly short life.

Courtesy of Pennsylvania DOT

HOW ARE PAVEMENT CONDITIONS RATED?

Every year the Federal Highway Administration (FHWA) gathers data on the condition of the nation's major roads, including those maintained by federal, state, or local governments. This report presents the conditions on all arterial routes, including Interstates and limited-access freeways, as well as other major streets and routes within and between urban areas. Most of these routes have at least four lanes, although some key two-lane urban and rural roads, classified as "arterial routes" are included.

RATING YOUR RIDE

States use the International Roughness Index (IRI) to rate road conditions, although some also rate by the Present Serviceability Rating (PSR). The FHWA compiles these data to create an assessment of pavement conditions, rating the roads as poor, mediocre, fair, or good.

The FHWA findings are based partly on a study that measured driver reactions to various road conditions to determine what level of road roughness was unacceptable to most drivers.(10)

Drivers on roads rated as poor are likely to notice that they are driving on a rougher surface, which puts more stress on their vehicles. Roads rated as poor may have cracked or broken pavements. These roads often show significant signs of pavement wear and deterioration and may also have significant distress in their underlying foundation. Road or highway surfaces rated poor provide an unacceptable ride quality and are in need of resurfacing and some need to be reconstructed to correct problems in the underlying surface.

Roads rated as being in either mediocre or fair condition may also show some signs of deterioration and may be noticeably inferior to those of new pavements, but can still be improved to good condition with cost-effective resurfacing or other surface treatments, which will extend the roads' service life.

The FHWA has found that a road surface with an IRI rating below 95 provides a good ride quality and is in good condition; a road surface with an IRI from 95 to 119 provides an acceptable ride quality and is in fair condition; a road surface with an IRI from 120 to 170 provides an acceptable ride quality and is in mediocre condition; and a road with an IRI above 170 provides an unacceptable ride quality and is in poor condition.(11)

> "There is a point in the life of a road where you spend more money for less result. It is like a homeowner who knows he needs a new roof, but keeps patching it to save money. You end up spending way more money patching than it would take to install a new roof—or build a new highway."
>
> —Pete K. Rahn, Director, Missouri Department of Transportation

Pavement Conditions by State, 2007

Includes all Arterial Routes, including Interstates, freeways, and major urban routes

State	Percentage			
	Poor	Mediocre	Fair	Good
Alabama	4	12	11	73
Alaska	18	28	26	28
Arizona	7	14	12	68
Arkansas	9	23	30	38
California	35	31	16	18
Colorado	8	24	24	44
Connecticut	14	33	18	34
Delaware	10	17	29	44
Florida	2	11	10	76
Georgia	0	4	3	92
Hawaii	27	44	19	10
Idaho	11	14	18	57
Illinois	14	20	20	46
Indiana	11	18	15	56
Iowa	18	23	18	41
Kansas	10	5	9	75
Kentucky	3	16	26	55
Louisiana	22	22	17	38
Maine	10	19	17	54
Maryland	26	18	14	42
Massachusetts	18	23	12	47
Michigan	18	19	12	51
Minnesota	10	22	22	47
Mississippi	17	23	18	42
Missouri	16	18	27	39
Montana	3	8	13	76
Nebraska	7	17	14	62
Nevada	5	8	6	81
New Hampshire	13	14	13	60
New Jersey	46	32	13	10
New Mexico	10	12	15	64
New York	22	24	18	35
North Carolina	9	18	24	49
North Dakota	5	20	18	57
Ohio	8	17	16	59
Oklahoma	21	19	20	40
Oregon	4	14	20	62
Pennsylvania	15	29	23	33
Rhode Island	32	36	15	18
South Carolina	7	21	21	51
South Dakota	15	19	15	51
Tennessee	6	11	12	71
Texas	11	21	27	41
Utah	4	25	20	51
Vermont	15	25	15	45
Virginia	6	17	31	46
Washington	11	22	14	53
West Virginia	8	29	21	42
Wisconsin	9	21	17	53
Wyoming	4	14	27	55
U.S. Average	**13%**	**20%**	**16%**	**51%**

Source: TRIP analysis based on Federal Highway Administration data

Additional Vehicle Operating Costs Due to Rough Roads, by State, 2007

State	Additional Costs
Alabama	$162
Alaska	$324
Arizona	$207
Arkansas	$302
California	$590
Colorado	$292
Connecticut	$313
Delaware	$282
Florida	$126
Georgia	$44
Hawaii	$503
Idaho	$318
Illinois	$297
Indiana	$242
Iowa	$383
Kansas	$318
Kentucky	$187
Louisiana	$388
Maine	$250
Maryland	$425
Massachusetts	$301
Michigan	$370
Minnesota	$347
Mississippi	$394
Missouri	$410
Montana	$195
Nebraska	$278
Nevada	$227
New Hampshire	$250
New Jersey	$596
New Mexico	$279
New York	$405
North Carolina	$251
North Dakota	$238
Ohio	$209
Oklahoma	$457
Oregon	$166
Pennsylvania	$346
Rhode Island	$473
South Carolina	$262
South Dakota	$319
Tennessee	$180
Texas	$336
Utah	$176
Vermont	$308
Virginia	$249
Washington	$266
West Virginia	$280
Wisconsin	$281
Wyoming	$230
United States	$335

Source: TRIP

Courtesy of Mississippi DOT.

PennDOT workers engage in crack sealing to keep moisture from penetrating beneath the road surface. In Pennsylvania, which has a vigorous freeze-thaw cycle each winter, keeping moisture out of the area beneath road surfaces is a critical maintenance step.

Courtesy of Pennsylvania DOT.

Chapter 2

Investing to Save America's Highways

Building for the future requires a national commitment to significant and sustained investment in transportation infrastructure.

"In the end, everything ties back to money, and we need to invest enough to preserve this important asset," said Oklahoma DOT Director Gary Ridley.

But the needs are high:

- The **Oregon DOT** needs $200 million per year to maintain current performance levels over the next 10 years compared with a current investment level of $130 million.

- The **Texas DOT** estimates that $73 billion will be required during the next 22 years to maintain current conditions. Today, the department is spending $900 million per year and losing ground. Officials say each one percent drop in good or better pavement condition is another 1,900 lane miles to fix and an additional $760 million in needs.

- The **Rhode Island DOT** needs $639.5 million annually to preserve its highway system. The state has only $354 million available each year to meet the need—leaving an annual funding gap of $285 million.

- **Alabama** needs an immediate investment of $1.4 billion to bring about 4,000 miles of deficient roadways to an adequate performance level. For Interstates, 70 miles must be resurfaced each year to maintain current levels at a cost of $140 million per year. The FY 2009 Interstate maintenance appropriation is $120 million.

- The **Pennsylvania DOT** pegs its need at $2.19 billion per year to maintain the entire state highway system at desired preservation cycles. That estimate does not include the current backlog of substandard pavements.

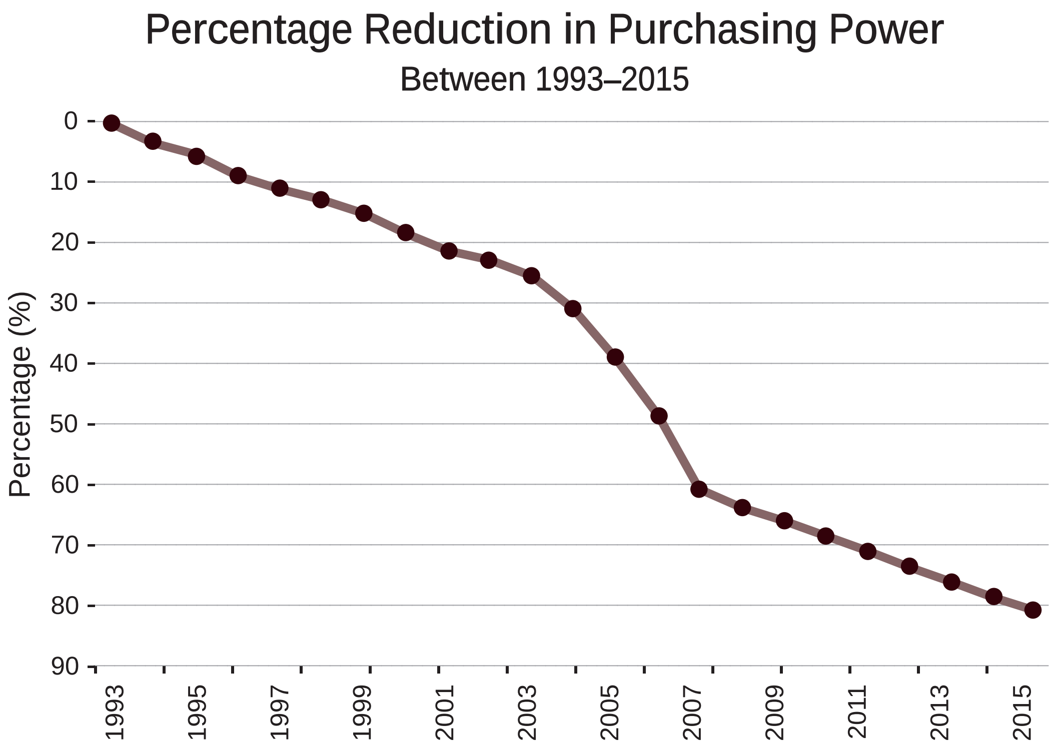
> "Envision a future with more transportation choices and efficiency than ever before. The stranglehold of congestion will be loosened by driving shorter distances, riding transit, and better utilizing our highways. Strategic investment in new lanes, new corridors, and new capacity for all modes will remove bottlenecks and connect America and the world."

Transportation: Invest in Our Future

American Association of State Highway and Transportation Officials, 2007

The **Nebraska Department of Roads** estimates it will need $270 million annually to preserve its highway system. Faced with declining revenue and growing needs, NDOR decided to make asset preservation its top priority to keep roads and bridges at current performance levels. No funding will be allocated to capital improvements until all preservation needs have been met.

Soaring construction costs during the past five years are further straining highway investment budgets. Asphalt prices are up 70 percent; concrete 36 percent; steel 105 percent; and diesel fuel, which is used to operate heavy construction equipment, soared by 305 percent including a 63 percent jump in one year.[12] While price trends have leveled as a result of the economic downturn, overall the purchasing power of a transportation dollar will have declined by 80 percent from 1993 to 2015.



THE BOTTOM LINE FOR INVESTMENT

Research conducted for the American Association of State Highway and Transportation Officials (AASHTO) concludes that the average requirement for ***all*** capital investments for highways and bridges is $166 billion ***annually*** through 2015. Other recent national studies commissioned by Congress project annual investment needs of similar magnitude, ranging from $130 billion to $240 billion though 2020. These levels are significantly higher than the $78 billion invested in highway capital improvements by all levels of government in 2006. According to the 2006 *Conditions and Performance Report* by the U.S. Department of Transportation, some 52 percent (or $36.4 billion) of transportation capital spending by all levels of government in 2004 was dedicated to system rehabilitation.

STIMULUS PROVIDES SHORT-TERM RELIEF

Inadequate levels of transportation funding have resulted in an immense backlog of "ready-to-go" but unfunded projects in the states. A December 2008 AASHTO survey identified more than 5,000 projects valued at $64 billion that states could have underway within 180 days.

In February 2009, President Barack Obama signed the American Reinvestment and Recovery Act of 2009 that provided $48 billion for transportation infrastructure as a means of stimulating the nation's severe economic decline. Of that amount, $27.5 billion was made available for highway projects.

"Because of the need to push money into the economy through job creation, states have applied a good share of their funding for the backlog of preservation needs," said AASHTO Executive Director John Horsley. "Resurfacing projects, for example, extend the life of highways, and can be implemented very quickly to benefit many areas of a state," he explained.

The **South Dakota DOT** said the stimulus money will provide about one year's worth of preservation funding to help with its backlog of needs. "Although this helps in the short-term, it is not a long-term solution," said South Dakota DOT Director of Planning and Engineering Joel M. Jundt.

Virtually all of the **Rhode Island DOT's** $137 million in economic recovery funding is devoted to preservation and maintenance projects—resurfacing, bridge rehabilitation, striping, guardrail, and traffic projects. The extra funds represent about 50 percent of the state's funding shortfall for 2009—or about five percent of the shortfall for the next 10 years.

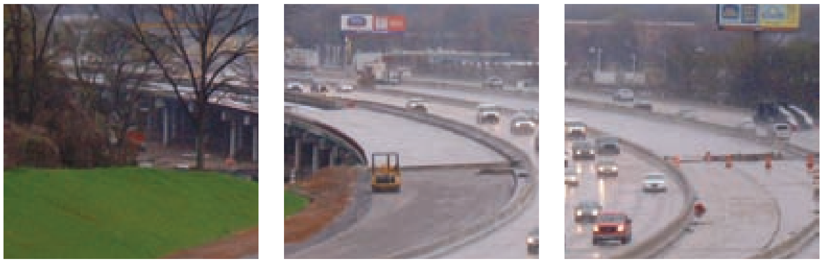
The **Idaho DOT** is using its stimulus allocation to pay for projects that would not be possible without extra federal funding. The projects include major highway widening, bridge replacement/relocation/realignment, and pavement restoration.

The $431 million that the **Maryland DOT** received will help offset some of the $1.3 billion cut from the state's highway capital program. The funds will be used primarily to keep roads in the best shape possible until the economy and federal and state revenues recover.

The **Alabama DOT** will spend $225 million on system preservation projects on non-Interstate routes, $70 million for an Interstate reconstruction project, and $8 million for bridge replacement and widening.

The **Texas DOT** is using a significant part of its stimulus funds to get its pavement preservation program back on track after three years of losing ground. Overall, pavement conditions in Texas were improving when the state spent $1.7 billion per year for rehabilitation and maintenance. Today, Texas spends about $900 million per year and has not been able to keep up with needed investments. Eight hundred million dollars in stimulus funds will help Texas stabilize pavement and bridge conditions for the next few years.

> "Resurfacing projects extend the life of highways, and can be implemented very quickly to benefit many areas of a state."
>
> —John Horsley, Executive Director, AASHTO



Courtesy of Alabama DOT.

Quick action by the Florida DOT and FHWA enabled replacement of the I-10 Escambia Bay Bridge on an accelerated schedule after it was destroyed in a 2004 hurricane.

Courtesy of Florida DOT.

Chapter 3

The Interstate System—An Aging Economic Engine

The Interstate Highway System has made a dramatic difference in how people and goods move across the country. The 47,000-mile system saves time, money, and lives, and has played a critical role in improving business productivity.

Construction of the Interstate system created jobs and produced new roads that expanded mobility for Americans. More importantly, the Interstate system helped create and continues to sustain the economy that has grown during the last 50 years.

"The initial investment in jobs during construction of the Interstate is far overshadowed by the economy that grew over the past 50 years as a direct result of that construction," said Gary Ridley, Director, Oklahoma Department of Transportation. "That's why preserving this asset is essential to our economic future."

TRAFFIC AND TRUCKS CAUSE WEAR AND TEAR

Although most Interstate highways today provide a good quality ride, the system is showing its age largely because of dramatic growth in car and truck traffic.

The 47,000 miles of Interstate highway represent only one percent of total highway mileage in the United States, but carry 24 percent of all traffic. Traffic growth during the past 50 plus years has far outpaced any growth projections made during the initial planning stages.

Much of the increase is due to truck traffic. On average, every mile of the Interstate system sees 10,500 trucks a day. By 2035, that number is expected to double, increasing to 22,700 trucks a day for each mile of Interstate highway.(13)

The surge in truck traffic on Interstate highways and its impact on traffic and road conditions are major factors in assessing the future of the Interstate Highway System. When construction began in the 1950s, the U.S. econ-

> "Our unity as a nation is sustained by free communication of thought and by easy transportation of people and goods. The ceaseless flow of information throughout the Republic is matched by individual and commercial movement over a vast system of interconnected highways crisscrossing the country and joining at our national borders with friendly neighbors to the north and south."
>
> —President Dwight D. Eisenhower, February 1955

Interstates Save Time, Money, and Lives

Interstates:

- Reduce total U.S. motor fuel consumption by 9.7 billion gallons annually
- Save Americans more than $320 billion annually and more than $1,100 per person in time and fuel
- Reduce the cost of transporting goods, which saves about $380 billion annually and $1,300 per person in consumer costs
- Save the average person 70 hours of time annually
- Are twice as safe as travel on other roadways because of safety features that include a minimum of four lanes, gentler curves, paved shoulders, median barriers, and rumble strips

Source: *The Interstate Highway System Saving Lives, Time, and Money* *TRIP, June 2006*

omy was largely self-contained. That has changed dramatically. The percentage of GDP represented by foreign trade increased from 13 percent in 1990 to 26 percent in 2000, and is expected to hit 35 percent in 2020. More than 80 percent of freight tonnage is generally carried by trucks driving on the Interstate Highway System.

Traffic growth during the past 50 years has been so great that most of the expansion capacity planned when the Interstate system was built has been used up. As a result, what was once wide open roadway is now increasingly congested.

Bottlenecks caused by stretched-to-the-limits Interstate interchanges delay commerce, cost consumers time and money, and further erode the Interstate network. In some parts of the country, the leaps in productivity and mobility that were hallmarks of the Interstate for much of its 50-year life are disappearing.

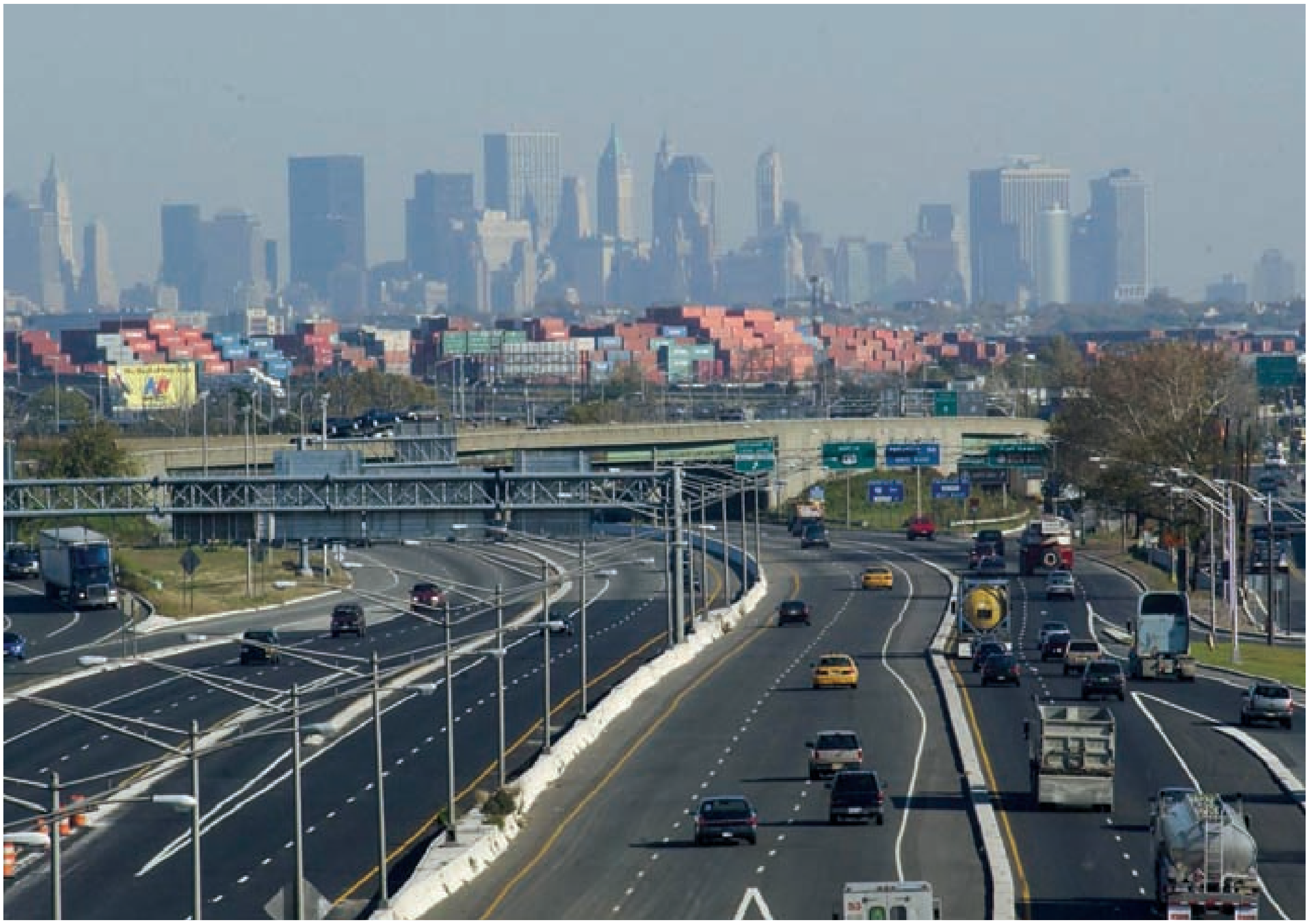
Interstate interchanges in metropolitan areas show the strain of traffic loads most dramatically. For example, the Marquette Interchange in Milwaukee, Wisconsin, was built in 1968 for $33 million to carry 155,000 vehicles per day. It was carrying 300,000 vehicles per day before construction began on a new interchange at a projected cost of $810 million.

Completed three months early in August 2008, the project is expected to be $10 million under budget. The Wisconsin DOT rebuilt the Marquette Interchange to include bridges with a life-span of 75 years. The project illustrates not only the cost of a major interchange reconstruction, but also the need to both preserve and renew such structures to meet traffic needs today and into the future.

Yet another example of a major Interstate replacement project is the Woodrow Wilson Bridge on Interstate 95 just south of Washington, DC. By the year 2000, the 45-year-old bridge had become a notorious bottleneck, carrying more than 200,000 vehicles a day, when it was built to accommodate only 75,000 vehicles a day. The new Woodrow Wilson Bridge was completed in 2008 at a cost of $2.5 billion, and was delivered on time and on budget. The new structure expands the bridge from 6 lanes to 12, two of which will be reserved for use by transit. Its new capacity of 300,000 vehicles a day is expected to accommodate traffic growth for many years to come.

INVESTING IN THE INTERSTATE'S FUTURE

States manage the Interstate Highway System, and they invest significant resources and research into preserving and restoring these critical highways. But they can't do it alone. In 1956, the idea of a federally defined,



Truck lanes on the New Jersey Turnpike just outside of New York City. *Courtesy of New Jersey DOT.*

built, and owned system was rejected in favor of the federal–state partnership that evolved and strengthened over 50 years.

Continuing to invest in restoring, rebuilding, and expanding the Interstate system is an important component of a **comprehensive highway preservation strategy** for the 21st Century.

- **Bridges**: The Interstate system has more than 55,000 bridges, many of which are reaching 40 to 50 years of age. Bridges and other structures of this age usually require substantial rehabilitation, and in another 20 to 30 years, require replacement.

- **Pavement**: The Interstates have approximately 210,000 lane-miles of pavement. As these pavement structures reach 40 and 50 years of life, major portions will need to have their foundations completely reconstructed.

- **Interchanges**: The Interstate system has almost 15,000 interchanges, and many do not meet current operational standards, creating bottlenecks or safety problems. Some of the most significant congestion on the system occurs at major interchanges not designed to carry the volumes of traffic that currently use them. Future traffic will only exacerbate these problems.

Lane expansion as part of rehabilitation is needed to improve freight logistics, reduce urban congestion, catch up to population growth centers, and connect growing metropolitan regions.

Absent significant expansion in the Interstate system, increased traffic, particularly in metropolitan areas, and dramatic growth in freight volumes will lead to more congestion and more wear and tear. Consistent pavement preservation strategies, carefully monitored performance measurements, and technological advancements can only do so much on roadways that are stretched far beyond capacity.

To obtain a clearer picture of coming investment requirements**, AASHTO has recommended** that the U.S. DOT and state DOTs jointly undertake two comprehensive needs assessments of the Interstate Highway System:

- To identify the costs of rebuilding or replacing the existing bridges, pavement foundations, and interchanges; and

- To identify long-term, system-wide expansion needs.

STATES FOCUS ON INTERSTATES

Preserving and reconstructing Interstate highway mileage is a top priority in every state.

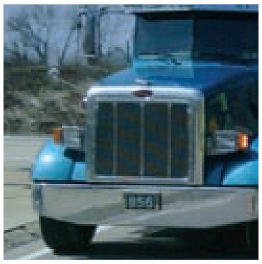
Missouri DOT Director Pete K. Rahn said there is a “huge need” to reconstruct much of his state’s Interstate system. In Missouri, reconstruction of Interstate 70, a major cross-country route, is projected to cost $3.4 billion, and Interstate 44, another national corridor, will cost $4 billion to rebuild. “We’re holding them together with bailing wire and bubble gum through overlays and other repairs,” Rahn said. “But we get less and less life out of rehabilitation treatments because the foundation needs to be rebuilt. An initial overlay might produce seven years of smooth rides, but after a while, potholes, cracks, and rutting will appear within three years.”

The **Pennsylvania DOT** has 128 miles, or 10 percent, of its Interstate system in need of major rehabilitation and reconstruction. Funding is in place to complete work on 77 of those miles.

The **Iowa DOT** has several major Interstate rehabilitation and reconstruction programs underway including:

- A $45 million project to grade, pave, and construct 10 new bridges along with roadway improvements on the Interstate 35-80-235 system interchange near Des Moines. The three-year project, which is nearing completion, will improve overall interchange traffic operations and meet existing and expected short-term traffic growth. The state went with a less costly short-term solution because funds were not available for total reconstruction.

- Addition of one lane in each direction to a 7.3-mile segment of Interstate 80 along with replacement of the entire original 46-year-old pavement at a cost of $96.5 million.



Courtesy of Pennsylvania DOT.

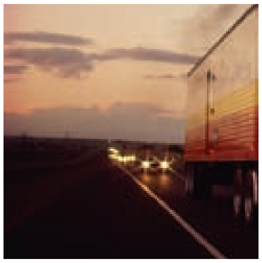
The **Oregon DOT** is rehabilitating nine miles of pavement on a segment of Interstate 84 in the eastern part of the state. The section was originally built in the 1960s and has been resurfaced three times to address damage from increased traffic and environmental conditions.

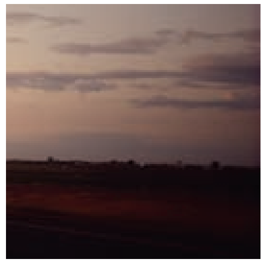
Because truck traffic generally uses the slow lane in this rural part of Oregon, the $27 million project will reconstruct that lane—which is in poor condition—with new concrete pavement, and resurface the existing fast lanes with asphalt pavement. This "black and white" pavement type has been successfully used in three other locations in Oregon.

The **Nebraska Department of Roads** is working on a six-lane reconstruction of Interstate 80 between Omaha and Grand Island, the state's two largest cities, which serves thousands of travelers daily. Upgrading from four to six lanes will improve safety and ease congestion in the state's fastest growing corridor. The $37 million project will be completed in mid-2011. This project is one component in a needed—but unfunded—reconstruction of the entire length of I-80 in Nebraska at projected cost of $100 million per year.

The Woodrow Wilson Bridge has successfully eased traffic on a major East Coast bottleneck. *Courtesy of Eye Construction, Inc.*

> "The Interstate System will never be finished because America will never be finished."
>
> —Francis C. "Frank" Turner, Federal Highway Administrator, 1969–1972
> *Richmond Times-Dispatch*, August 19, 1996





A truck on the warm-mix test track at the National Center for Asphalt Technology.

Courtesy of National Asphalt Pavement Association/ Asphalt Pavement Alliance.

Chapter 4

Trucks and Highways—Working Together to Move Freight

Trucking is the backbone of the nation's freight transportation system—transporting virtually everything we eat, drink, or buy. And trucks drive on highways, streets, and roads. Nearly 80 percent of the 15 billion tons in goods transported through the nation's freight system in 2005 was carried on trucks. Freight tonnage moved in the United States is projected to nearly double over the next 30 years with trucks taking 84 percent of the growth.[(14)]

With this expected growth, creating a low-cost, efficient, and reliable freight system becomes increasingly critical to the country's economic health. And preserving the highway network is a vital piece of the long-term freight strategy for the nation.

But major challenges lie ahead:

- Increasing traffic congestion is costing the freight transportation network nearly $8 billion per year. Higher transportation costs mean higher consumer prices.

- Increased truck traffic contributes to wear and tear on highways. Pavement damage is related to a truck's axle loads rather than the total truck weight. A truck with more axles will have less weight per axle and, therefore, create less pavement damage.

Highways and trucks need to coexist successfully for the good of America's economy. To achieve that goal, a comprehensive action agenda to meet the country's freight needs is essential, including:

- Fixing freight bottlenecks;

- Maintaining durable highway surfaces; and

- Improving access to ports, airports, and distribution centers.

"From any perspective the freight transportation challenge is formidable. Meeting it will require resolve and resources. Not meeting it will be a major national failure."

—Larry L. "Butch" Brown, Executive Director, Mississippi Department of Transportation

FREIGHT BOTTLENECKS COST CONSUMERS

Bottlenecks occur when traffic routinely backs up because volumes exceed capacity of the roadway. The worst bottlenecks are at or near freeway-to-freeway interchanges.

Freight bottlenecks are found on highways that serve major international gateways such as the Ports of Los Angeles and Long Beach, California, at major domestic freight hubs such as Chicago, and in major urban areas where transcontinental freight lanes intersect congested urban freight routes.

Traffic congestion means increased travel times, increased costs, and less reliable pick up and delivery times for truck operators. Freight bottlenecks cause nearly 250 million truck hours of delay annually, costing direct users about $7.8 billion.[15] To make up for traffic delays, shippers add more trucks, which, in turn, creates more congestion. Eventually these increased costs of doing business are passed on to consumers.

BUILDING MORE DURABLE PAVEMENTS TO SUPPORT TRUCK TRAFFIC

Research into developing pavement materials and construction practices to provide more durable road surfaces that can tolerate increased traffic loads—including trucks—is part of the solution. Examples of advanced research include the use of geosynthetic reinforced soil, warm mix asphalt, polymer-based asphalt binders, and admixtures to improve the strength and workability of Portland cement concrete.

Terry Button has been driving the roads in his trucks for more than 29 years. An independent trucker based in Rushville, NY, Button drives up and down the East Coast delivering hay to dealers and suppliers. Button, who serves on the Board of Directors for the Owner Operator Independent Drivers Association, said the repercussions of rough roads are devastating for truckers.

Smooth pavement not only affects his bottom line, it also means a safer ride. "Smooth rides are critical for truckers. It's easier on the equipment, easier on your health. Because with all the bumps, things wear out faster, air ride suspension hangers come off, ball joints wear out. Some night you might be going around a curve and something snaps, and your safety is at risk."

Button said he sees rough roads in every state. "Road smoothness varies greatly—sometimes county to county. We have to make this a priority for this country. If we don't have good transportation, we can't get food to market, and there's nothing more important than that."

DEDICATED TRUCK LANES

Many states are looking at adding truck-only lanes to their Interstates to reduce congestion, improve safety, and move goods faster. Separating trucks from regular automobile traffic can improve highways and reduce truck-caused wear and tear on other roadways. The only completely separated truck lanes that currently exist are a 30-mile segment of the New Jersey Turnpike. California and Texas also have short segments of truck-only lanes.

The biggest obstacle to broad use of truck-only lanes is cost. For example, one state study estimated that constructing a new truck-only lane alongside an existing rural Interstate highway would cost approximately $2.5 million per lane-mile, plus land and acquisition costs.[16] The FHWA estimates that the cost of new highway lane miles ranges from $1.6 million to $3.1 million in rural areas and $2.4 million to $6.9 million in urban areas. The truck-only price tag raises red flags when states look at long lists of reconstruction and expansion needs at a time when highway construction funds are limited. As a result, higher fuel taxes, user fees, and tolls are options that states have considered to pay for dedicated truck lanes.

Courtesy of California DOT.

COMMERCE CORRIDORS FOR EFFICIENT FREIGHT MOVEMENT

Exclusive truck lanes at the state level are a subset of a bigger strategy needed to move freight more efficiently and preserve the nation's highways. Other elements being recommended by many groups, including AASHTO are: fix highway truck bottlenecks, improve intermodal access to ports and distribution centers, fund international gateways, and add capacity to priority trade corridors including a national network of truck-only lanes.

The program would be funded by freight-related user fees outside the Highway Trust Fund, with the federal government providing coordination and the states and Metropolitan Planning Organizations (MPOs) overseeing the planning.

Courtesy of Pennsylvania DOT.

PennDOT workers lay replacement drainage pipe as part of a road maintenance project. Proper drainage for streams that cross beneath roads is a critical maintenance step.

Courtesy of Pennsylvania DOT.

Chapter 5

Managing Highways as an Investment

With an estimated value of $1.75 trillion, highways, streets, and roads are an asset to be managed and preserved rather than a project to be built or fixed. Managing this valuable asset depends on:

- An investment in pavement preservation;
- An organizational commitment to asset management;
- Advancements in materials, maintenance techniques, and technology; and
- Sustained financial investment.

PAY ME NOW OR PAY ME LOTS MORE LATER

Good roads cost less. That is why pavement preservation is such an important part of asset management. The goal is to extend the service life of roads ***before*** they need major rehabilitation or replacement.

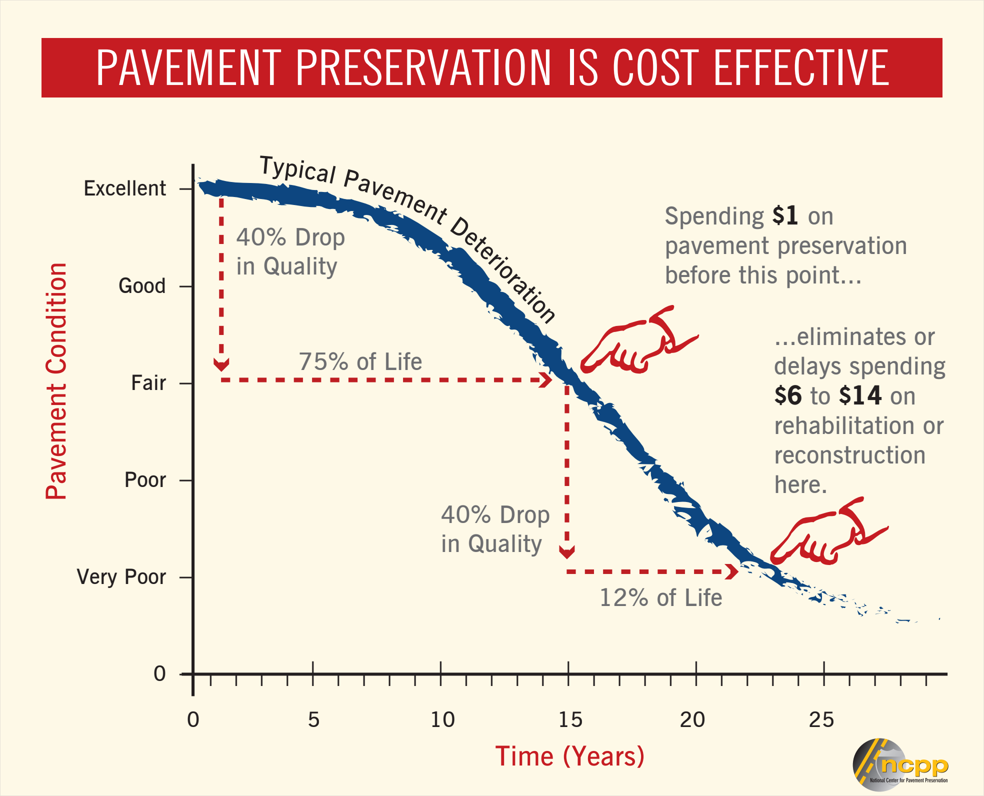
Maintaining a road in good condition is easier and less expensive than repairing one in poor condition. Costs per lane mile for reconstruction after 25 years can be more than three times the cost of preservation treatments over the same 25 years and can extend the expected service life of the road for another 18 years.

Timing is everything in pavement preservation. If rehabilitation is done too early, pavement life is wasted. If it is done too late, the road may require additional costly repair work.

Pennsylvania DOT Secretary Allen D. Biehler said the decision to use a large portion of highway funds for system preservation is one of the biggest challenges facing transportation leaders today.

“We as transportation stewards of the system have no choice but to drive home the message that maintaining an acceptable condition for our highways—preserving the system—is vital to our country’s future,” Biehler said.

> “There is no more fundamental transportation capital investment than system preservation—keeping existing infrastructure in good condition. If preservation investment is deferred, costs increase dramatically, leading to the saying ‘pay me now or pay me more—lots more—later’.”
>
> —Washington Department of Transportation 2007–2026 Highway System Plan



Source: National Center for Pavement Preservation.

"Our focus is fix it first—paying attention to basic day-to-day practices that help us be more successful. Other-wise, you can spend too much time and money chasing after potholes while watching the system fall farther and farther behind."

In Pennsylvania, less than 10 percent of the transportation budget is currently dedicated to expansion—com-pared with more than 20 percent in recent years.

The **Oregon DOT** has a chip-seal preservation program to treat about 780 lane miles of highway at a cost of $7 million per year. The program complements the department's $125 million preservation program, which re-surfaces about 1,000 lane miles during the same period. The chip-seal program lowers the annual cost to main-tain good pavement by increasing the time between higher-cost resurfacing treatments. Over the long-term, the preservation strategy will save $16 million per year in resurfacing costs.

The **Nebraska Department of Roads** recently implemented a Pavement Optimization Program (POP) to manage its highway network and allocate funds to keep the system at its current performance level. POP uses current pavement conditions, pavement deterioration rates, and cost/benefit ratios to develop budget scenarios to ensure effective allocation of funds. The department uses two recently purchased pathway profilers to collect data about the severity and extent of pavement distress to assist in making investment decisions.

The **Michigan DOT** uses a network pavement strategy that provides a "mix of fixes" to extend the life of the road. The three types of fixes are: reconstruction and rehabilitation; capital preventive maintenance; and reac-tive maintenance. Decisions about which fix to use are based on an assessment of the current pavement con-ditions and a projection of the number of years before it will require reconstruction or rehabilitation using a measure known as *remaining service life* (RSL).

Tight budgets force creative strategies for sustaining pavement preservation plans. In **Washington State**, the DOT has identified the need for $1.7 billion in concrete pavement restoration—but less than $20 million per year has been budgeted. To compensate for reduced funding, WSDOT uses a triage strategy—investing first in pavements whose life can be greatly extended if treated immediately, and deferring work on pavements that need complete replacement. The strategy improves and extends the life of the greatest number of lane miles with available funds. Despite budget constraints and soaring construction costs, pavement conditions have continued to improve over the years.

Shifting from Worst-First to Best-First Investments

How do you sell the idea that spending money on a road that looks to be in good condition is a better idea than spending it on one that is bumpy, rutted, rough, and obviously in need of repair? Very carefully, says Michigan DOT Director Kirk L. Steudle, who believes the shift from worst-first to best-first is a good strategy for long-term asset management.

"It is important to slow the rate of decline in the good road so that it stays good rather than slipping into fair or poor condition." Steudle added that spending $1 to keep a road in good condition prevents spending $7 to reconstruct it once it has fallen into poor condition.

Michigan always works on a five-year horizon in its pavement preservation program so, he said, it is important to show where plans to fix that poor road fit into the schedule.

"It is easy to fall into the worst-first strategy, particularly when money is tight," he said. "But that's when staying focused on keeping good roads good and minimizing the amount of deterioration is even more important."

MANAGING TRANSPORTATION ASSETS

Asset management is a strategic approach to managing infrastructure. It focuses on maintaining the condition and performance of ***public assets*** using business and engineering practices to allocate resources based on reliable information and well-defined objectives.

> "The highway system is owned by the public. Our daily focus is on preserving this asset that the public has entrusted to us. In many cases, we're not doing as good a job as we could."
>
> —Gary Ridley, Director, Oklahoma Department of Transportation

"Asset management is a very broad concept that focuses on getting the best return on the investment we put into our transportation system," said Kirk T. Steudle, Director of the Michigan Department of Transportation and Chair of the AASHTO Subcommittee on Asset Management. "It isn't a computer software program, finance and accounting practices, or a pavement preservation program. Asset management includes all that and more.

"We need to focus on operating, maintaining, upgrading, and expanding the entire asset with which we are entrusted in the most cost-effective and efficient way possible," Steudle said.

The **Michigan DOT** Asset Management program encompasses all the physical transportation assets in the state, including more than 9,700 miles of road, 5,679 bridges, 450,000 signs, 4,025 traffic lights, 8 million linear feet of guardrails, 83 rest areas, 13 travel information centers, 85 roadside parks, 27 scenic turnouts, and more. The program is built around five major functions: policy goals and objectives, information and data collection, planning and programming; program delivery, and monitoring and reporting.

Steudle said the program begins with setting a broad policy about the current condition of the asset and then setting a goal for where you want that asset to be within a specific time frame. For Michigan, the goal was to increase the condition of all its roads and highways, moving from 65 percent of state roads in good condition in 1997 to 90 percent in 2007. Pavement preservation was the primary tool for achieving that goal.

The department met its 90 percent goal and improved to 92 percent in 2008. A similar goal-driven asset management process is now underway for the state's bridges.

Michigan has a statewide Transportation Asset Management Council, which brings together all the agencies in the state that have jurisdiction over roads. Its purpose is to broaden the use of transportation asset management throughout the state and ensure that groups are working together, sharing methodology, collecting the same data, and speaking the same language.

Other state DOTs are developing asset management programs as well.

The **Washington State DOT** relies on data collection, analysis, and innovative reporting methods to manage its transportation assets, which include 20,000 lane miles of state roads and 3,000 bridges. The department uses data not only to assess project costs and benefits, but also to analyze tradeoffs in allocating limited funds between preservation and improvement programs and between highway construction and highway maintenance.

The department's *Measures, Markers, and Milestones* report is a critical part of the system, linking performance measures to overall strategic objectives. The state's efforts to communicate its performance led to public support for two funding increases—a five-cent gas tax increase in 2003 and a nine-cent gas tax increase in 2005.

The **Utah DOT,** which manages 6,000 miles of highway, uses dTIMS CT software to support its asset management, bridge management, and pavement management systems. These systems help the department identify the most efficient use of funding based on the current condition of the asset and available funding for preserving it. Because of recent funding limitations, however, the asset management model recommends work that has to be done instead of the work that should be done.

Courtesy of the National Asphalt Pavement Association/Asphalt Pavement Alliance.

TOOLS FOR SUCCESSFUL PAVEMENT PRESERVATION

Successful pavement preservation requires reliable tools for monitoring pavement conditions and the best materials to get the longest life from the roads. A number of different types of sealers and rejuvenators are available based on the existing pavement type and the problem being solved. The most common treatments include chip seals, slurry seals, fog seals, micro-surfacing, thin hot mix asphalt overlays, crack sealing, and joint sealing—all designed to maintain or improve pavement condition and extend its life.

The **Washington State DOT's** Materials Lab identified these tools for pavement preservation:

- **Dowel-bar retrofits** installed in aging concrete to improve smoothness and longevity and help traffic flow smoothly from one concrete slab to the next. State officials believe the technique could add 10–15 years to 30-year-old concrete highways.

- **Pavement recycling** using reclaimed asphalt from older, failed pavements and blending it into a new asphalt mix.

- **Warm-mix asphalt** using chemical additives that allow construction at lower temperatures resulting in lower emissions and improved construction.

- **Bonded concrete overlays** on an existing asphalt pavement to add structure and provide a longer-lasting surface. **Ultra-thin white topping** using a two-to-four inch thick layer of concrete over an existing asphalt road can be installed fairly quickly with minimal traffic disruption.

Reliable equipment to assess and monitor the condition of pavements is as important as the materials used.

The **Michigan DOT** has used ground-penetrating radar to assess conditions that could affect pavement life, such as locating sink holes, and mapping technology to help assess remaining service life on pavements. A laptop computer along with a GPS receiver are used to track road locations on a region map and quickly gather data about the previous service life rating, historic data on the road segment, and previous fix types.

The **Maryland DOT** uses an automatic road analyzer to collect information on roughness, rutting, and cracking as well as a skid truck to collect friction data. The data is fed into the pavement management system to identify targets for both pavement preservation and rehabilitation fixes.

Last summer, the **Oregon DOT** began assessing pavement conditions on a portion of the network using a vehicle equipped with a profiler to measure roughness and scanning lasers to measure rutting. All of the data was collected in a single pass of one vehicle at normal speeds.

Rhode Island uses an automated distress survey to assess pavement conditions and calculate crack density that helps define the appropriate preventive maintenance treatment. In addition, the RIDOT pavement management team selects 100-foot-long monitoring sections representing all of the different treatments, stress levels, and traffic volumes to visually assess effectiveness of the preservation strategy.

The **Minnesota DOT** evaluates its 14,000 miles of highway annually using a van equipped with lasers to measure the smoothness of pavement and cameras to help engineers evaluate the quality of the pavement. The state uses three indicators to report and quantify pavement conditions—*ride quality index,* which measures pavement roughness; *surface rating,* which measures pavement distress; and a *pavement quality index*.

Pothole Killer Streamlines Repairs

Dealing with potholes is part of a pavement preservation strategy. Generally quick fixes to deal with urgent needs—like a really big pothole on a major commuter route—may be needed. But quick fixes rarely last.

Pavement that is maintained in good condition and is designed for the traffic that uses it will usually remain pothole free—even during the toughest freeze and thaw cycles. Like any pavement repair processes, good materials installed properly will produce the best results.

One quick-fix approach that does produce longer-term results is the "Pothole Killer," an all-in-one vehicle that can repair up to 100 potholes a day with only one driver. A traditional four-person pothole crew can patch about 10–15 potholes a day. The Pothole Killer uses a three-step process—it blows the pothole clean of all debris, sprays a special fast dry asphalt emulsion into the hole, and then applies an asphalt aggregate mix on top. The entire process takes about six minutes.

Some cities and states lease rather than purchase the equipment to reduce the capital cost.

Courtesy of Patch Management Pothole Killers.

BUILDING FASTER, CHEAPER, SAFER

Construction strategies that speed up building projects without compromising quality can reduce traffic disruption, control labor costs, and minimize costs to commercial traffic.

Research shows that the traveling public is demanding increased mobility while showing less tolerance for construction delays and construction-related congestion.

Action strategies to build faster, cheaper, and more safely include:

- Innovative traffic management systems including full road closures to expand available work time;
- Accelerated construction management techniques to minimize construction time while enhancing quality and safety for major multi-phase projects; and
- Use of materials that reduce project schedules.

The **Indiana DOT**'s Hyperfix Project in 2003 provides an example of a successful fast-track Interstate renovation. The project involved reconstruction of two heavily traveled Interstates in Indianapolis. The highways carried 175,000 vehicles daily—compared with a design capacity of 61,000. Because of the magnitude of the reconstruction and expected traffic delays, the project team decided to close the highway completely and use a fast-track, round-the-clock construction plan.

The project was completed between two major races at the Indianapolis Speedway, which regularly draws 250,000 participants who use these highways. Work was completed in 55 days—30 days ahead of schedule, saving taxpayers an estimated $1 million in lost wages and lost productivity for each day that traditional construction would have added. Special commuter buses and parking lots were used to keep traffic moving without turning alternative routes into parking lots.

Keys to success included early planning, collaboration among local, state, and federal agencies, and community support. A series of community meetings were held well before construction began to ensure that everyone understood the plans and alternate commuter options. As a result, the public was prepared for traffic impacts long before blasting, drilling, milling, and paving began.

The team wrapped the public face of the entire project around a catchy brand name: Hyperfix. The name so captured the imagination of stakeholders that it became part of local language and lore with advertising billboards and radio talk shows proclaiming the need to "Hypermow" the lawn or "Hyperfix" one's thermostat. One citizen actually was inspired to write a song that celebrated the project's advances in words and music.[17]

The **Missouri DOT** challenges project engineers to use non-traditional project design methods to develop efficient solutions for today's needs. DOT officials say practical design is rooted in the principle that building a series of good, not great, projects will result in a great system. It maximizes the value of a project by ensuring that it is the correct solution for its surroundings.[18]

Before practical design, most projects followed strict guidelines based on road classification type and traffic volume. Now designers look at projects on a case-by-case basis with a goal of building to meet basic needs, rather than the highest standards. State officials estimate the new approach to design has saved taxpayers $400 million in its first two years.

GET IN, GET OUT, STAY OUT

Routine maintenance alone cannot sustain highways that have been in service for nearly 50 years. In many cases, pavement foundations need to be rebuilt to deal with the impacts of age and to modernize roads to meet current conditions.

Longer-lasting materials can make a big difference in the life of a road.

For example:

- Asphalt perpetual pavements can be designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction. Longer-lasting asphalt pavement mixes combine smoothness and safety advantages of traditional asphalt with an advanced, multi-layer paving design that extends the life of a roadway with routine maintenance.[19]

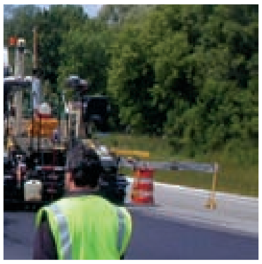
- Superpave gives highway engineers and contractors tools to design and construct asphalt pavements that meet specific climate and traffic conditions. Although it has been in use since the 1990s, current research focuses on measuring resistance to ruts and cracks to come up with even longer-lasting mixes.

- Stone matrix asphalt, which is also called Gap-Grade Superpave, is a new mix that can be used to reduce splash and spray and may have some value in noise reduction. Its main advantage is its durability, providing a long-lasting pavement surface.

- Fast-track concrete pavement produces the strength benefits of traditional concrete with a much shorter preparation time—making it possible to be ready for opening in 12 hours or less after laying. Generally fast-track concrete provides good durability because it has a relatively low water content, which improves strength and decreases salt permeability which, in turn, contributes to deterioration.

- Roller-compacted concrete, another drier mix, can be installed using asphalt paving equipment and compacted with rollers. It has the strength to withstand heavy loads and can resist freeze-thaw cycles.[20]





Courtesy of National Pavement Association/Asphalt Pavement Alliance.

Chapter 6

Rebuilding for the Future

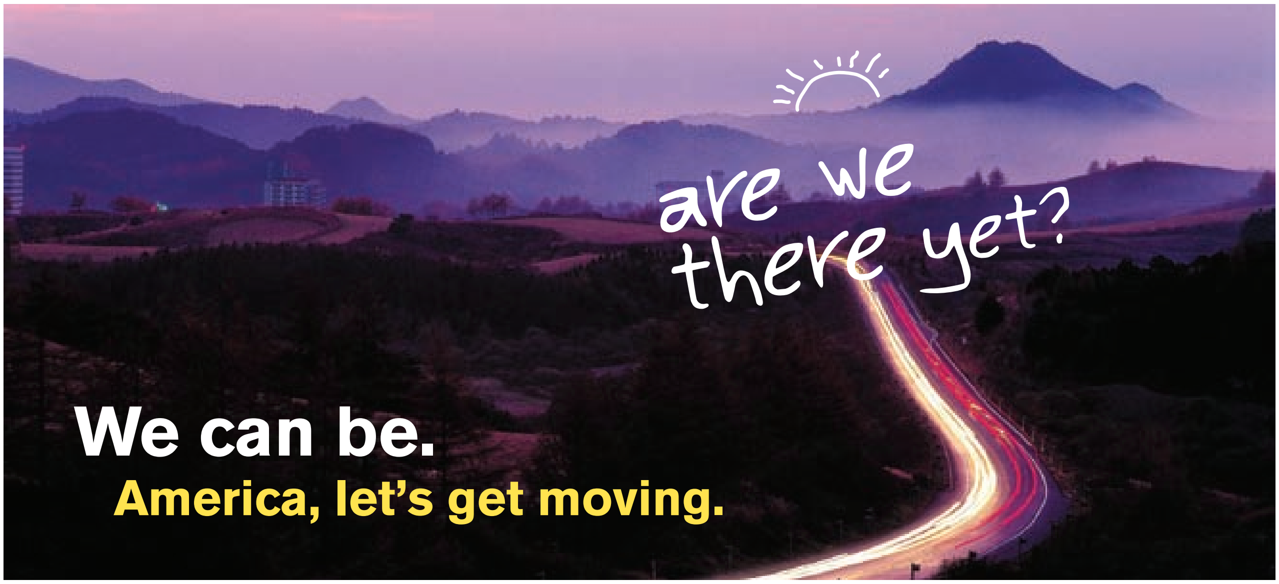
ARE WE THERE YET?

No—but we can be.

Improved management strategies, a focus on preserving essential public assets, better, longer-lasting materials, new approaches to building highways faster, cheaper, and sooner all will help get us there. But it does come down to money. It is time for a greater and smarter investment of transportation dollars to ensure a new and better transportation program.

Pennsylvania DOT Secretary and AASHTO President Allen D. Biehler said getting there also involves thinking differently about highways, land use, and our way of life.

"We need to maintain and preserve our highway system first and then begin to think about other influences at work—global warming, greenhouse gas emissions, where we live and work—that affect traffic congestion and our quality of life," Biehler said.



"As fundamental as it is to our future, our current transportation system is aging, underfunded, and inadequate to meet the demands of tomorrow. States stand ready to meet the challenges with projects that create jobs and bring hope to communities—projects that not only preserve what we already have but expand our horizons..."

—Allen D. Biehler, AASHTO President;
Secretary, Pennsylvania Department of Transportation

Appendices

Appendix A—Pavement Conditions on State, City, and County Arterial Networks, 2007

Urban areas with populations of 500,000 and above, including cities and surrounding suburbs

Urban Area	Poor	Mediocre	Fair	Good
Akron	12%	18%	22%	48%
Albany	14%	34%	19%	33%
Albuquerque	36%	27%	13%	24%
Allentown-Bethlehem, PA	15%	35%	24%	26%
Atlanta	1%	9%	5%	85%
Austin	20%	17%	24%	39%
Bakersfield	5%	38%	33%	23%
Baltimore	44%	26%	11%	19%
Birmingham	17%	30%	10%	43%
Boston	22%	20%	8%	50%
Bridgeport-Stamford, CT	15%	27%	17%	41%
Buffalo	12%	19%	22%	47%
Charlotte	10%	17%	27%	46%
Chicago	18%	28%	15%	39%
Cincinnati	11%	26%	16%	46%
Cleveland	15%	25%	12%	48%
Colorado Springs	12%	29%	25%	34%
Columbus, OH	4%	17%	19%	60%
Concord	54%	19%	17%	9%
Dallas-Fort Worth	29%	39%	17%	15%
Dayton	8%	12%	18%	62%
Denver-Aurora	18%	27%	17%	38%
Detroit	36%	33%	7%	24%
El Paso	19%	30%	30%	21%
Fresno	28%	35%	14%	23%
Grand Rapids	23%	27%	14%	35%
Hartford	19%	33%	17%	30%
Honolulu	61%	27%	6%	6%
Houston	29%	26%	17%	28%
Indianapolis	26%	21%	7%	46%
Jacksonville, FL	2%	16%	13%	69%
Kansas City	31%	17%	14%	38%
Lancaster-Palmdale, CA	13%	40%	24%	23%
Las Vegas	10%	26%	19%	46%
Los Angeles	64%	28%	5%	3%
Louisville	14%	37%	21%	29%
Memphis	27%	23%	15%	34%
Miami	5%	19%	15%	62%
Milwaukee	25%	29%	18%	28%
Minneapolis-St. Paul	22%	30%	18%	30%
Mission Viejo, CA	37%	47%	5%	11%
Nashville	6%	18%	13%	62%
New Haven-Meridian, CT	12%	31%	14%	44%
New Orleans	49%	19%	16%	16%
New York-Newark	54%	28%	10%	8%
Oklahoma City	41%	24%	12%	23%
Omaha	41%	36%	12%	11%
Orlando	7%	13%	13%	68%
Palm Springs-Indio, CA	47%	28%	10%	15%
Philadelphia	36%	36%	17%	12%
Phoenix	10%	16%	14%	60%
Pittsburgh	26%	32%	23%	20%
Portland	9%	17%	17%	58%
Poughkeepsie-Newburgh, NY	9%	39%	36%	16%
Providence	28%	30%	13%	28%
Raleigh	19%	26%	23%	32%
Richmond	14%	35%	28%	24%
Riverside-San Bernardino, CA	44%	44%	7%	4%
Rochester	19%	15%	35%	32%
Sacramento	44%	44%	4%	8%
Salt Lake City	5%	20%	17%	58%
San Antonio	38%	19%	15%	28%
San Diego	53%	31%	6%	10%
San Francisco-Oakland	61%	22%	4%	13%
San Jose	61%	29%	8%	2%
Sarasota-Bradenton, FL	1%	22%	18%	59%
Seattle	21%	21%	13%	45%
Springfield, MA	14%	46%	9%	31%
St. Louis	10%	22%	20%	48%
Tampa-St. Petersburg	3%	16%	15%	67%
Toledo	17%	15%	13%	55%
Tucson	23%	47%	15%	14%
Tulsa	47%	29%	8%	16%
Virginia Beach	23%	28%	21%	28%
Washington, DC, MD, and VA Suburbs	31%	30%	13%	27%

Source: TRIP analysis of Federal Highway Administration data.

Appendix B—Pavement Conditions on State, City, and County Arterial Networks, 2007

Urban areas with populations of 250,000–499,000, including cities and surrounding suburbs

Urban Area	Poor	Mediocre	Fair	Good
Anchorage	14%	37%	14%	35%
Ann Arbor	20%	28%	12%	40%
Antioch, CA	58%	13%	9%	21%
Asheville, NC	22%	21%	27%	30%
Augusta, GA	2%	14%	14%	70%
Barnstable Town, MA	7%	20%	14%	59%
Baton Rouge	37%	23%	21%	18%
Boise	44%	28%	8%	20%
Canton, OH	13%	17%	21%	49%
Cape Coral, FL	2%	34%	9%	55%
Charleston-North Charleston	11%	31%	19%	39%
Chattanooga	6%	25%	14%	55%
Columbia, SC	26%	21%	19%	34%
Corpus Christi, TX	36%	19%	16%	29%
Davenport, IA	36%	19%	18%	28%
Daytona Beach	4%	21%	8%	66%
Denton-Lewisville, TX	17%	45%	22%	16%
Des Moines	39%	18%	18%	25%
Durham, NC	20%	33%	11%	36%
Eugene, OR	5%	12%	13%	70%
Fayetteville, NC	3%	23%	21%	52%
Flint	27%	22%	13%	37%
Fort Wayne	34%	9%	9%	48%
Greensboro, NC	22%	15%	17%	46%
Greenville, SC	20%	32%	19%	29%
Harrisburg	11%	32%	25%	32%
Hemet, CA	44%	53%	1%	2%
Hickory, NC	18%	20%	21%	41%
Jackson, MS	34%	41%	14%	12%
Kissimmee, FL	0%	9%	9%	82%
Knoxville	9%	8%	22%	62%
Lancaster, PA	20%	33%	26%	21%
Lansing	16%	22%	14%	49%
Lexington, KY	7%	45%	9%	39%
Little Rock	26%	34%	17%	23%
Lorain-Elyria, OH	7%	14%	28%	50%
Madison, WI	31%	29%	19%	20%
McAllen, TX	6%	18%	23%	54%
Mobile	15%	14%	17%	55%
Modesto, CA	34%	39%	17%	10%
Naples, FL	0%	31%	7%	63%
Ogden-Layton, UT	4%	14%	17%	65%
Oxnard-Ventura, CA	36%	45%	11%	8%
Palm Bay-Melbourne, FL	11%	14%	7%	67%
Pensacola, FL	1%	15%	26%	58%
Port St. Lucie, FL	2%	27%	10%	61%
Provo-Orem, UT	1%	40%	5%	55%
Reading, PA	18%	44%	24%	14%
Reno	40%	17%	7%	36%
Santa Rosa, CA	52%	39%	8%	1%
Scranton-Wilkes-Barre, PA	26%	40%	20%	13%
Shreveport	35%	40%	12%	13%
South Bend, IN	25%	29%	11%	34%
Spokane	31%	16%	9%	43%
Stockton	42%	34%	8%	16%
Syracuse	16%	14%	20%	50%
Temecula-Murrieta, CA	35%	53%	7%	5%
Trenton, NJ	49%	27%	15%	9%
Victorville-Hesperia, CA	37%	36%	15%	11%
Wichita	42%	21%	5%	32%
Winston-Salem	8%	30%	38%	24%
Worcester, MA	31%	32%	11%	26%
Youngstown, OH	9%	23%	26%	41%

Source: TRIP analysis of Federal Highway Administration.

Appendix C—Additional Vehicle Operating Costs Due to Rough Roads, 2007 *

Urban areas with populations of 500,000 and above, including cities and surrounding suburbs

Urban Area	Cost in Dollars
Akron	$249
Albany	$315
Albuquerque	$576
Allentown-Bethlehem, PA	$340
Atlanta	$68
Austin	$346
Bakersfield	$280
Baltimore	$589
Birmingham	$344
Boston	$320
Bridgeport-Stamford, CT	$290
Buffalo	$248
Charlotte	$247
Chicago	$333
Cincinnati	$261
Cleveland	$290
Colorado Springs	$300
Columbus, OH	$156
Concord	$656
Dallas-Fort Worth	$512
Dayton	$182
Denver-Aurora	$339
Detroit	$525
El Paso	$401
Fresno	$461
Grand Rapids	$394
Hartford	$352
Honolulu	$688
Houston	$463
Indianapolis	$400
Jacksonville, FL	$123
Kansas City	$457
Lancaster-Palmdale, CA	$350
Las Vegas	$246
Los Angeles	$746
Louisville	$355
Memphis	$436
Miami	$165
Milwaukee	$425
Minneapolis-St. Paul	$431
Mission Viejo, CA	$571
Nashville	$185
New Haven-Meridian, CT	$263
New Orleans	$622
New York-Newark	$638
Oklahoma City	$631
Omaha	$592
Orlando	$162
Palm Springs-Indio, CA	$608
Philadelphia	$525
Phoenix	$217
Pittsburgh	$430
Portland	$199
Poughkeepsie-Newburgh, NY	$307
Providence	$418
Raleigh	$372
Richmond	$354
Riverside-San Bernardino, CA	$632
Rochester	$318
Sacramento	$622
Salt Lake City	$187
San Antonio	$529
San Diego	$664
San Francisco-Oakland	$705
San Jose	$732
Sarasota-Bradenton, FL	$146
Seattle	$326
Springfield, MA	$339
St. Louis	$258
Tampa-St. Petersburg	$137
Toledo	$275
Tucson	$473
Tulsa	$703
Virginia Beach	$417
Washington, DC, MD, and VA Suburbs	$458

Source: TRIP.

* AAA reports that the average cost for a motorist traveling 15,000 miles per year is $8,100, although costs vary depending on the vehicle and location.

Appendix D—Additional Vehicle Operating Costs Due to Rough Roads, 2007 *

Urban areas with populations of 250,000–499,000, including cities and surrounding suburbs

Urban Area	Cost in Dollars
Anchorage	$304
Ann Arbor	$359
Antioch, CA	$652
Asheville, NC	$390
Augusta, GA	$124
Barnstable Town, MA	$178
Baton Rouge	$534
Boise	$597
Canton, OH	$256
Cape Coral, FL	$183
Charleston-North Charleston	$301
Chattanooga	$214
Columbia, SC	$424
Corpus Christi, TX	$509
Davenport, IA	$495
Daytona Beach	$156
Denton-Lewisville, TX	$424
Des Moines	$524
Durham, NC	$392
Eugene, OR	$130
Fayetteville, NC	$186
Flint	$413
Fort Wayne	$445
Greensboro, NC	$347
Greenville, SC	$401
Harrisburg	$288
Hemet, CA	$650
Hickory, NC	$340
Jackson	$638
Kissimmee, FL	$61
Knoxville	$182
Lancaster, PA	$384
Lansing	$298
Lexington, KY	$294
Little Rock	$462
Lorain-Elyria, OH	$200
Madison	$486
McAllen, TX	$196
Mobile	$272
Modesto, CA	$538
Naples, FL	$147
Ogden-Layton, UT	$150
Oxnard-Ventura, CA	$560
Palm Bay-Melbourne, FL	$205
Pensacola, FL	$134
Port St. Lucie, FL	$162
Provo-Orem, UT	$196
Reading, PA	$399
Reno	$497
Santa Rosa, CA	$684
Scranton-Wilkes-Barre, PA	$458
Shreveport	$552
South Bend, IN	$431
Spokane	$396
Stockton	$580
Syracuse	$260
Temecula-Murrieta, CA	$571
Trenton, NJ	$620
Victorville-Hesperia, CA	$552
Wichita	$540
Winston-Salem	$300
Worcester, MA	$450
Youngstown, OH	$253

Source: TRIP.

* AAA reports that the average cost for a motorist traveling 15,000 miles per year is $8,100, although costs vary depending on the vehicle and location.

Endnotes

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Status of Quiet Pavement Research

Presented to:
FHWA Pavement Preservation ETG
New Orleans, La
May 15, 2009

Larry Scofield, PE

The Sounds of Silence

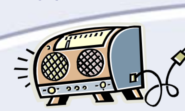
- Quality of Life
- Investment Decisions
- The Future is Different

Highway Noise Control

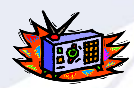
Considered

Appreciated

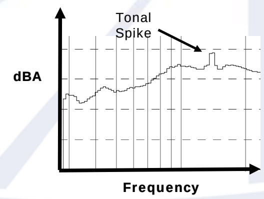
Two Types of Annoyance



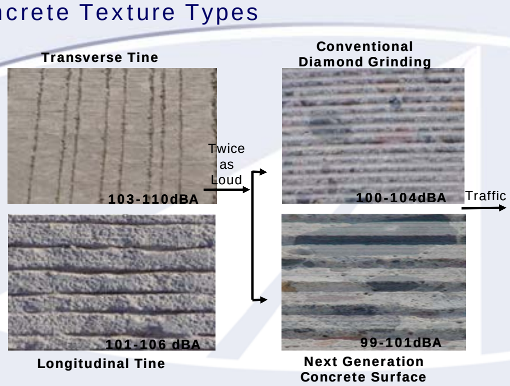
Volume (Too Loud)



Frequency (Off Station)



Concrete Texture Types



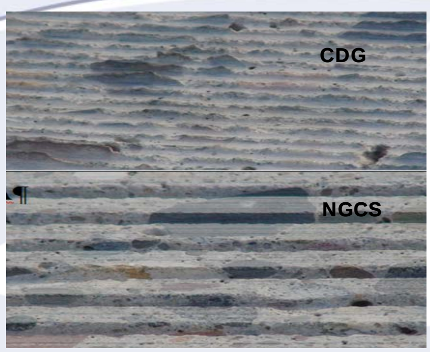
What's New?

- Rehabilitation
- New Construction

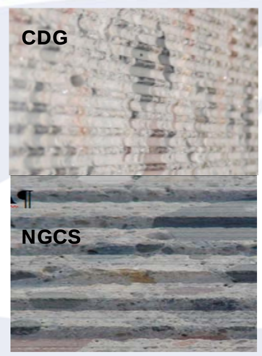
Purdue Research-- Tire Pavement Test Apparatus (TPTA)

Rehabilitation: Next Generation Concrete Surface

NGCS Compared to Pre Traffic CDG



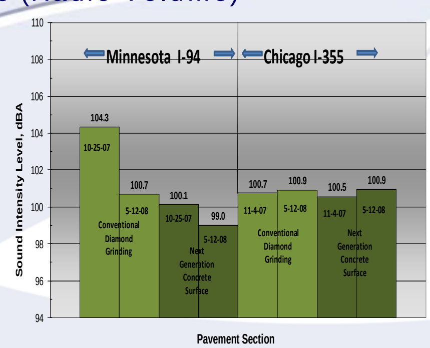
NGCS Compared to Trafficked CDG



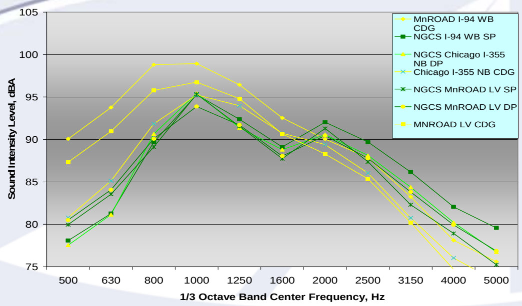
NGCS LITE

} Modify Land

NGCS Results Noise Results to Date (Radio Volume)



OBSI Frequency Spectrums (Radio Station)



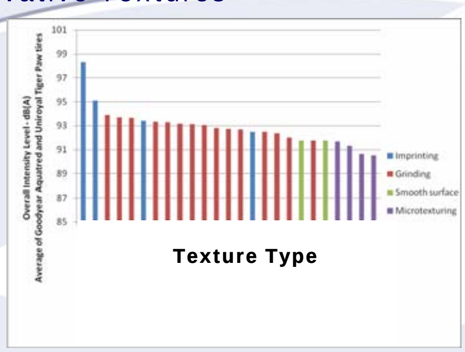
New Construction: TPTA Innovative Textures Work

- Establish the Lower Limit Possible with Concrete
- Evaluate Unique Geometric Patterns
  - Circles
  - Diamond
  - ARFC Texture
  - Waffle Pattern
  - Random Waffle
  - Spherical

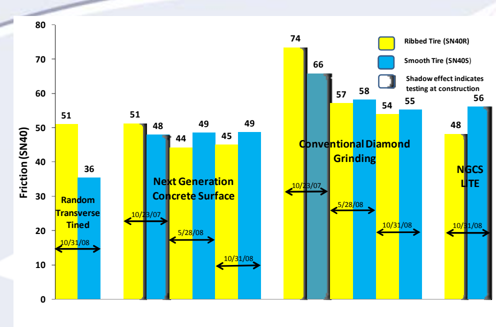
Imprinting Roller for Diamond Pattern Texture

Cast Diamond Pattern

Comparison of Grinding to Innovative Textures

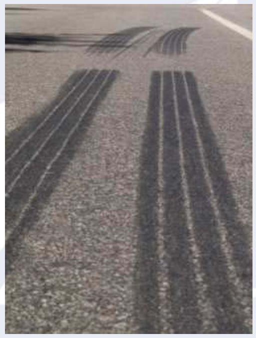


Friction (NS40) as a Function of Time

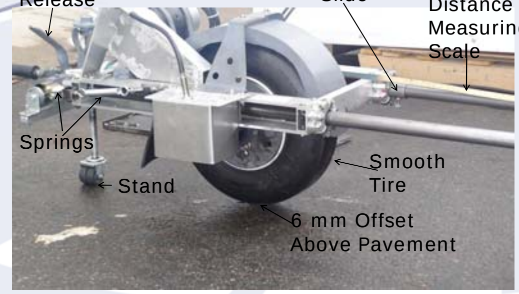


The Sounds of Safety

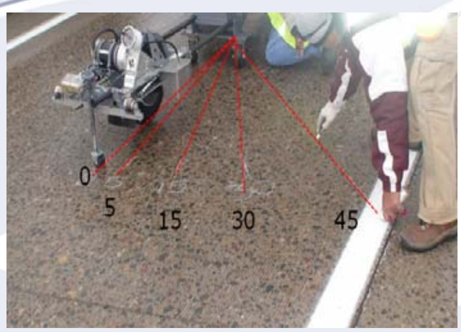
- Friction Durability
- Isotropic vs An-Isotropic
  - Wet Weather Accidents



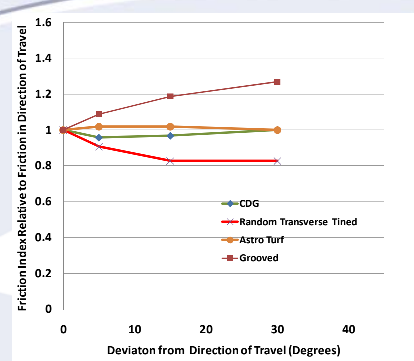
CA-342 Equipment Operation



CT-342 Friction Testing at Angles



Anisotropic Friction Behavior



Questions?

ACPA
www.pavement.com

Pavements 4life
www.pavements4life.com

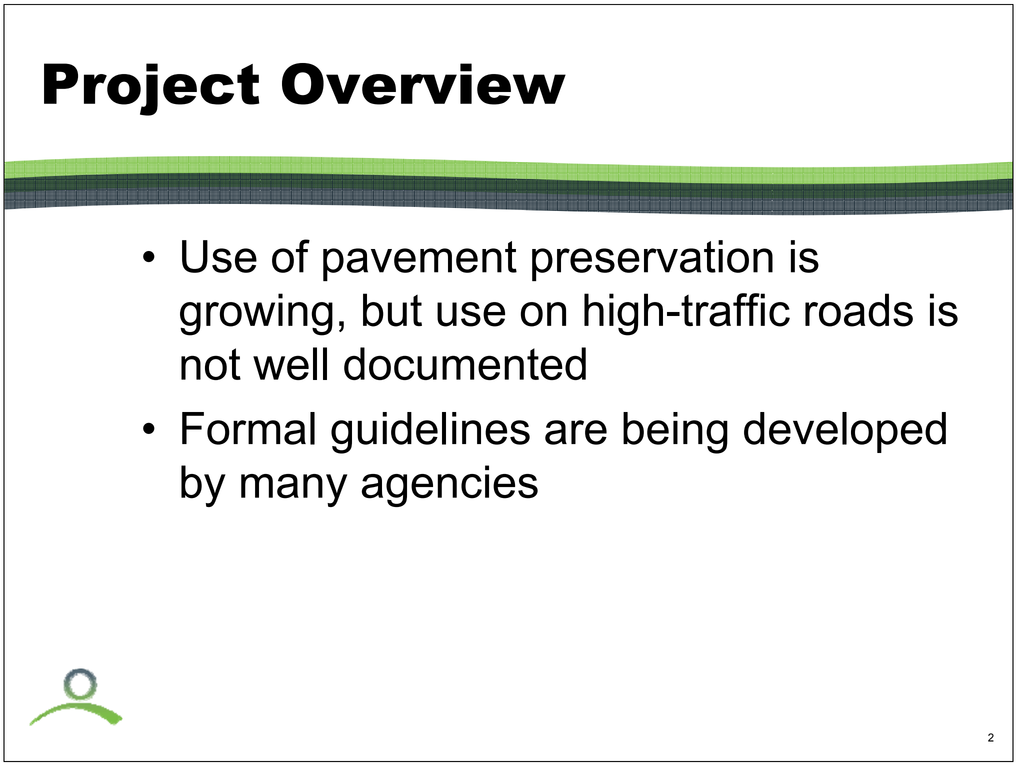
Strategic Highway Research Program 2 (SHRP 2)

Project R26

Preservation Approaches for High-Traffic Volume Roadways

applied pavement TECHNOLOGY

providing engineering solutions to improve pavement performance



Project Overview

The practice of pavement preservation in general, and preventive maintenance in particular, is a growing trend among transportation agencies around the United States. Over the past decade a number of state highway agencies (SHA) have created or formalized preservation programs, including:

Rhode Island, Arizona, California, Nebraska, Missouri, North Carolina, Louisiana, Minnesota, South Carolina, and Nevada.

At the same time, other agencies that might have been practicing preservation for a longer time (Texas and Washington State, for example) have extended their programs to cover a greater proportion of their pavement network. Still other agencies (such as Illinois and Hawaii) are in the process of creating formal preservation programs.

The significance of this trend in public agencies (which is by no means limited to SHAs) is reinforced in several ways:

Some agencies (such as North Carolina, Louisiana, California, and Minnesota) have created a departmental position for a pavement preservation engineer.

Many of agencies have developed or are developing formal guidelines for preservation, such as Caltrans' Maintenance Technical Advisory Guides for both flexible and rigid pavements.

Project Objectives

- Develop preventive maintenance guidelines for high-traffic volume roads
- Identify promising preventive maintenance strategies for high-traffic volume roads
- Recommend further research opportunities

Project Objectives

The objective of this project is to develop guidelines on pavement preservation strategies for high-traffic volume roadways that can be used and implemented by public agencies.

A secondary objective is to identify promising pavement preservation strategies for application on high-traffic volume roadways that might not commonly be used, and make recommendations for further research opportunities.

Project Team

- Principal Investigator: David Peshkin, APTech, Inc.
- Angie Wolters, APTech, Inc.
- Jim Moulthrop, Fugro Consultants, Inc.
- Gerry Eller (formerly of FHWA/FP2)
- Gary Hicks, California Pavement Preservation Center
- Dean Testa, (formerly of Kansas DOT)

Dr. James Bryant, P.E., SHRP2 Program Director

Applied Pavement Technology, Inc. (APTech) has assembled a team that has the necessary background, experience, and contacts to be successful in this research.

The project team is led by APTech's David Peshkin, who has been involved in a broad range of successful pavement preservation research and implementation initiatives around the United States over the past decade. These include development and presentation of training programs for SHAs and the Federal Highway Administration (FHWA), participation in the Pavement Preservation Expert Task Group (PPETG), research (including NCHRP, SHA, and local efforts), and involvement in a number of pavement preservation activities with various industry groups (AEMA, IGGA, and ISSA, for example). At APTech, he is assisted by a highly qualified team of researchers and practitioners with experience in pavement performance and managing pavements.

Other team members include Jim Moulthrop and his colleagues at Fugro Consultants, Inc., and three consultants:

Gerry Eller, currently Executive Director of the Foundation for Pavement Preservation;

Gary Hicks, currently Technical Director of the California Pavement Preservation Center at California State University, Chico; and

Dean Testa, formerly Chief of the Bureau of Construction and Maintenance for the Kansas DOT.

Research Approach

- Phase I
  - Task 1: Research, survey state of practice
  - Task 2: Develop criteria to identify best practices
  - Task 3: Submit Interim Report
- Phase II
  - Task 4: Develop guidelines
  - Task 5: Submit Draft Final Report

Phase I

Task 1. Identify the current state of practice for preservation approaches on high-traffic volume roadways through a national and international literature search and survey. This review should include approaches that are currently successfully implemented and other preservation approaches that have the potential to be successful but have not been regularly deployed.

The project team will undertake a comprehensive literature review of all sources, focusing on work reported in the past 5 years. The literature search will access the Transportation Research Information Service (TRIS) database and be augmented by searching other databases including the National Transportation Information Service, the Engineering Index (EI Compendex), and TRB's Research in Progress (RIP) database. A search of foreign databases will also be conducted to help substantiate international experience with preventive maintenance treatments. These foreign databases could include such sources as the World Road Association (formerly PIARC), the Australian Road Research Board (ARRB), and the French Public Works Research Laboratory (LCPC), among others.

However, it is the research team's experience that the literature search will turn up limited information on the topics of interest in this research. Thus, a large part of the data collection process must rely on conducting an effective survey of practices.

The survey will address treatments for both PCC and HMA-surfaced pavements. It will include a list of common treatments, and call for respondents to link up these and other preservation treatments to different types of roadways, differentiated by traffic volume, rural versus urban, and time available for closure.

The survey will also delve into possible explanations for why an agency has not applied pavement preservation practices to high-traffic volume roadways. These might include lack of experience or a deeply entrenched bias, previous failures, operational or other barriers (traffic control, alternate routes, available closures, safety concerns, and so on). Where certain preservation techniques are being used, the survey must also identify what obstacles, if any, were encountered and how they were successfully addressed.

Task 2. Develop criteria to identify best practices for preservation approaches for high-traffic volume roadways, and apply selection criteria to information obtained from Task 1.

The criteria to consider in determining best practices should ultimately help users improve their ability to identify suitable projects and select appropriate treatments, thereby obtaining improved pavement performance in a cost-effective manner. Best practices are likely to be identified to cover the following topics:

• **Project selection.** Criteria should consider different traffic volumes, environmental conditions, and pavement conditions under which pavement preservation can be effective. Criteria will address the significance of the factors to be considered, and where meaningful discriminating boundaries might be established.

• **Treatment selection.** Criteria should address what treatments are likely to be successful under what conditions. In particular, for high-traffic volume roadways these criteria will include distress types (severity and extent), as well as the combination of

Phase I Status

- Completed
- Summarized state of practice through use of survey
- Developed criteria for preservation best practices
- Interim report submitted

As of November, all but 10 US DOTs had responded to the questionnaire. In addition to DOT responses, 7 Canadian provinces, 2 US cities, 1 turnpike organization, 2 industry representatives, and one federal representative (Central Federal Lands) have responded. Results from questionnaire responses were summarized for use in determining selection criteria to identify best practices (Task 2).

Results of the literature search and survey results have been used to begin work on developing criteria to identify the best practices for preservation of high-traffic volume roadways. Summaries of the literature review and survey response details have also been worked into the draft interim report.

Scope of Present Findings

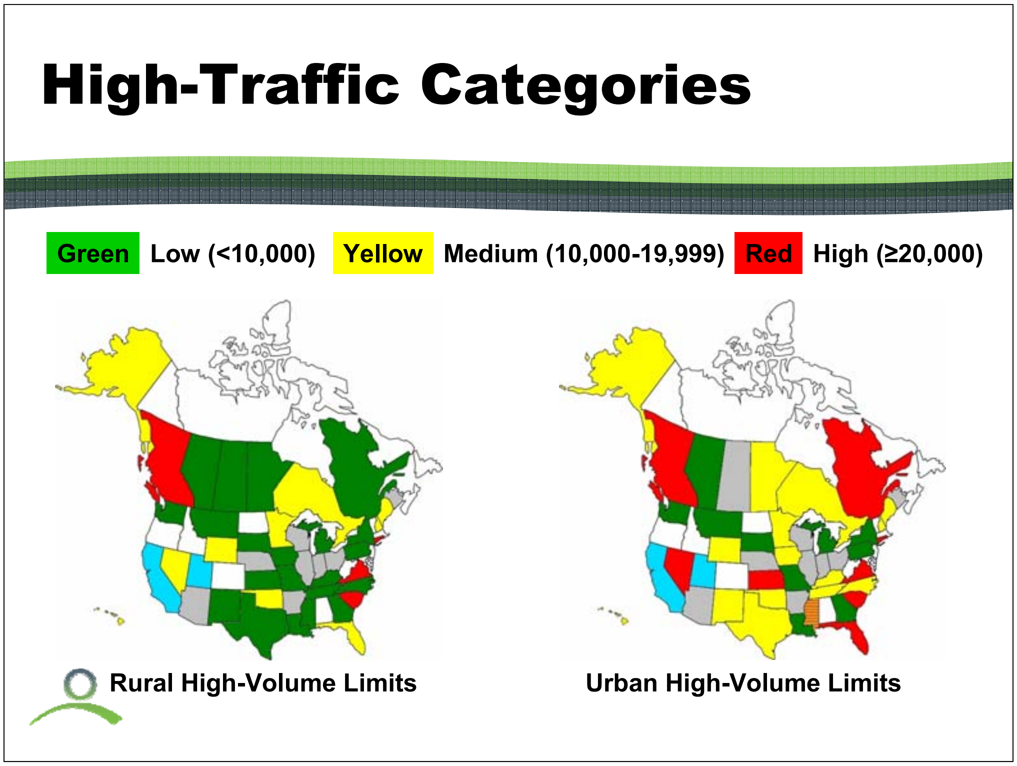
- US and Canadian DOTs
- High-Traffic Rural and Urban Roads:
  – “Low” high-traffic: <10,000 ADT
  – “Medium” high-traffic: 10,000-19,999 ADT
  – “High” high-traffic: ≥20,000 ADT

Preventive Maintenance Treatment Use on HMA-surfaced Rural/Urban Roadways

APTech examined some of the HMA maintenance treatment use trends for rural and urban roadways based upon dividing the HIGH traffic volume practices indicated in Question 1 of the survey into the following subset based upon ADT.

Low: ADT < 10,000
Medium: 10,000 ≤ ADT < 20,000
High: ADT ≥ 20,000

It should be noted that the results are based on survey responses provided by US DOTs and Canadian Provinces only.



Figures—Map of High-Traffic Volume Categories (Low, Medium, High)

Maps of rural and urban high-traffic volume categories per DOT respondents.

TURQUOISE TI ≤ 18 rural / TI ≤ 15 urban, as reported by Caltrans. 2,500 non-interstate/25,000 interstate, as reported by Utah Region 4.

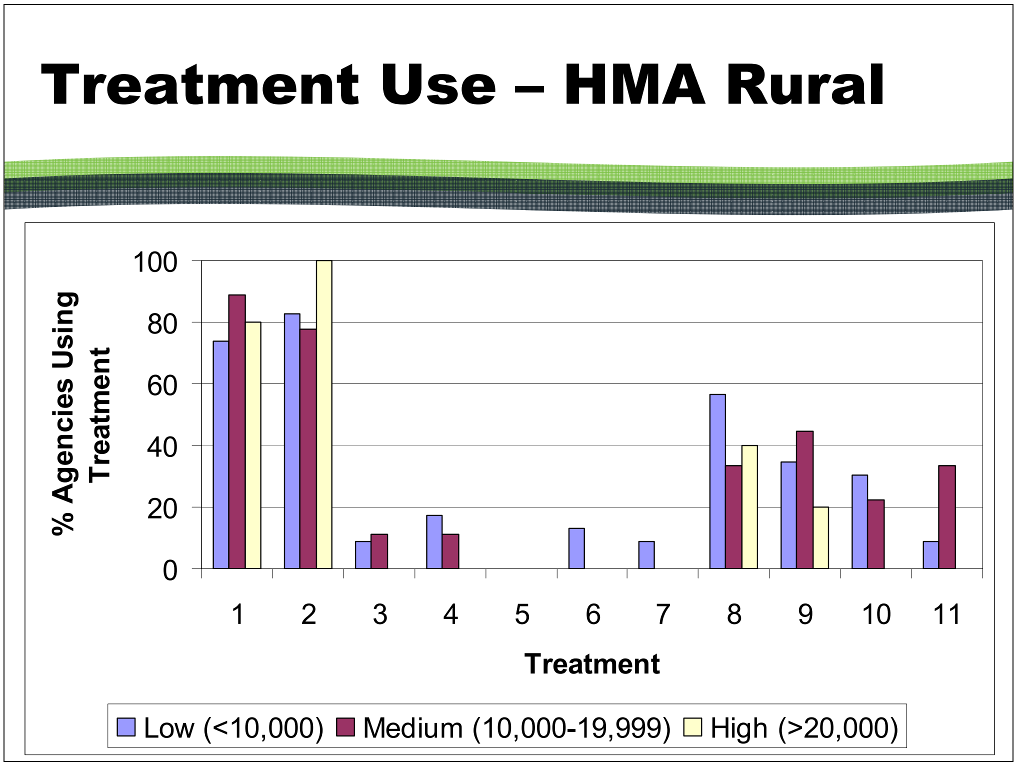
GRAY Did not answer.

WHITE No response to survey at this time.

HMA Treatments

1	Crack Fill
2	Crack Seal
3	Cape Seal
4	Fog Seal
5	Scrub Seal
6	Slurry Seal
7	Rejuvenators
8	Single Course Microsurfacing
9	Multi. Course Microsurfacing
10	Single Course Chip Seal
11	Multi. Course Chip Seal

12	Chip Seal w/ Modified Binder
13	Thin Bonded Wearing Course
14	Thin HMA Overlay
15	Cold Milling and HMA Overlay
16	Ultrathin HMA Overlay
17	Hot In-Place Recycling
18	Cold In-Place Recycling
19	Profile Milling
20	Ultrathin Whitetopping
21	Drainage Preservation
22	Other



Figures—Preventive Maintenance Treatment Use on HMA-surfaced Roads

Rural Fig. Percentage of treatment use per rural high-volume traffic category
(e.g., percentage of DOTs and Provinces with "low" high-traffic (<10,000) using crack seal).

Urban Fig. Percentage of treatment use per urban high-volume traffic category
(e.g., percentage of DOTs and Provinces with "low" high-traffic (<10,000) using crack seal).

<u>Some comments based on the Figures:</u>

Those agencies with "high" (≥ 20,000) high-traffic volume designations have not reported using the following preventive maintenance treatments: cape seal, scrub seal, single and multiple course chip seals, cold in-place HMA recycling, or ultrathin whitetopping; nor have they reported using any "other" treatments specifically.

No agencies have reported using scrub seal.

Two DOTs with "high" high-traffic volume designations, Nevada and Region 4 of Utah, report using fog seal.

For all agencies reporting high-traffic volume designations, crack fill and crack seal are used by at least 60 percent of reporting agencies. Additionally on rural roads, thin HMA overlays and drainage prevention are used by at least 60 percent of agencies; while on urban roads, drainage preservation is used by at least 60 percent. (Cold-milled HMA overlays (<1.5-in.) are used on urban roads by at least 40 percent reporting at this time.)

In general cape seal and rejuvenator are not used by many reporting agencies. On urban roads, cape seal, fog seal, rejuvenator, multiple course chip seal, and cold in-place recycling (<4.0-in.) are not used by many agencies.

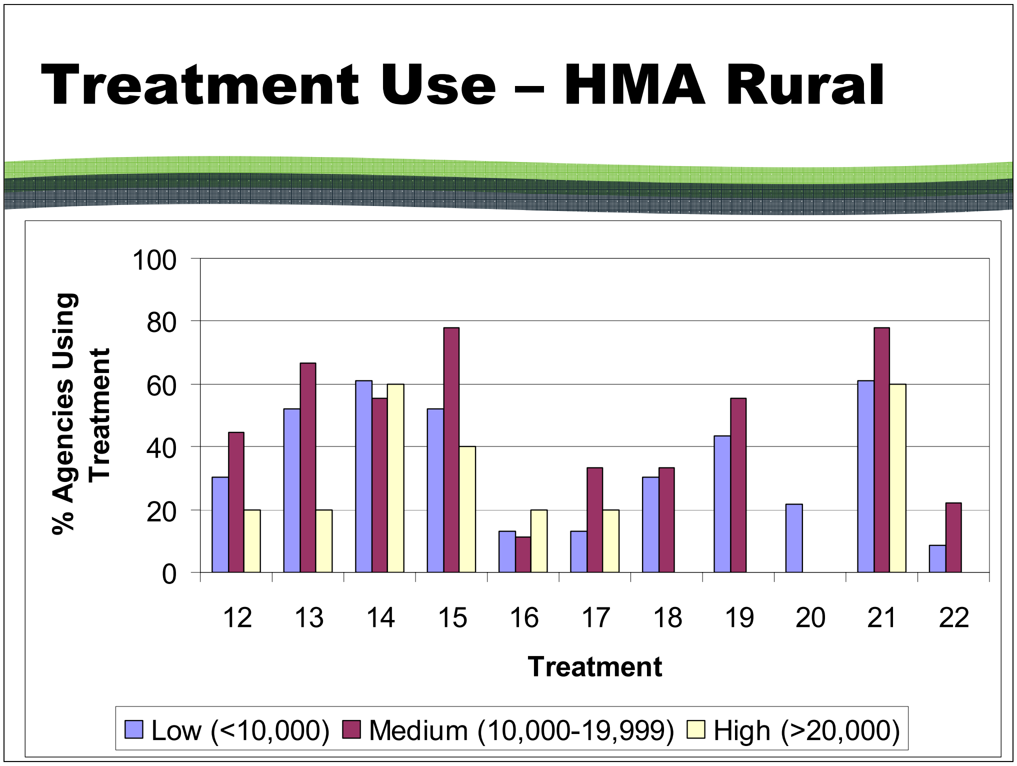
Few agencies—only Hawaii, Minnesota, Montana, and Alberta, Canada—note using "other" preventive maintenance treatments.

Hawaii reports only doing 1.5-in. HMA mill and fill as preventive maintenance on rural and urban high-traffic roadways.

Minnesota requires all chip seal applications receive fog seal.

Montana applies thin HMA overlays (< 2-3/8 in.) on rural high-traffic roadways.

Alberta uses a combination of profile milling and thin overlay on rural high-traffic roadways.



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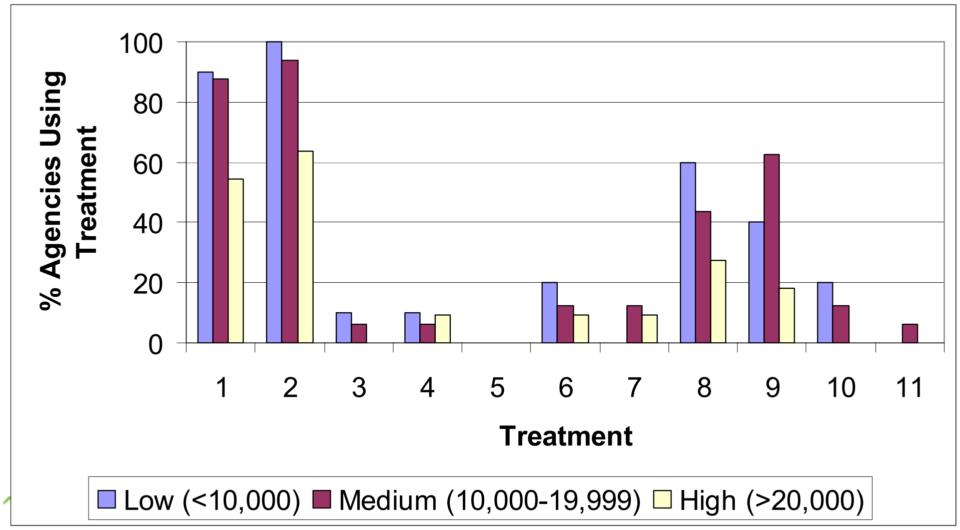
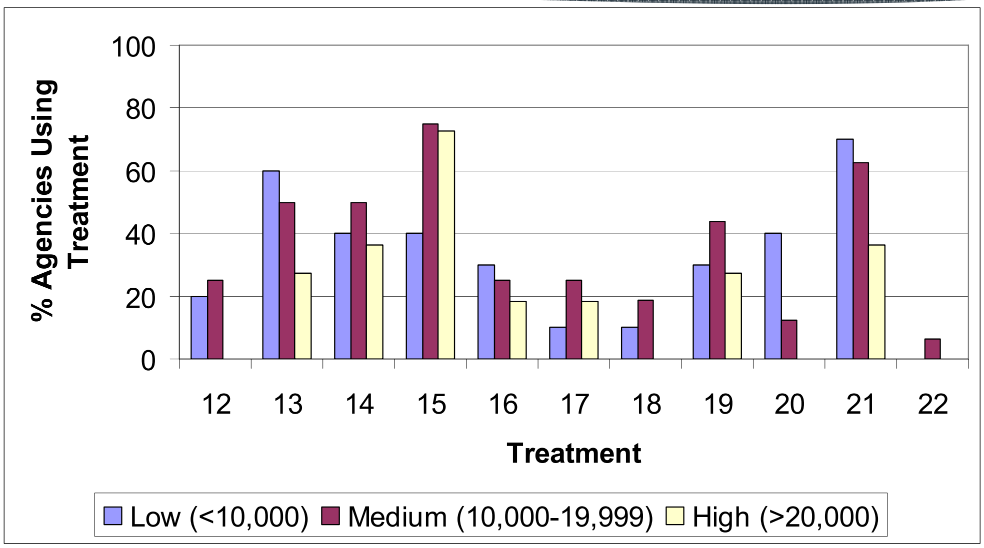
Alberta uses a combination of profile milling and thin overlay on rural high-traffic roadways.

Treatment Use – HMA Urban



Treatment Use – HMA Urban



“High” Traffic (≥ 20,000 ADT)

Popular HMA treatments

- Crack Seal
- Crack Fill
- Drainage Preservation

HMA treatments with limited use

- Cape seal, scrub seal, chip seals, CIPR, ultra-thin whitetopping

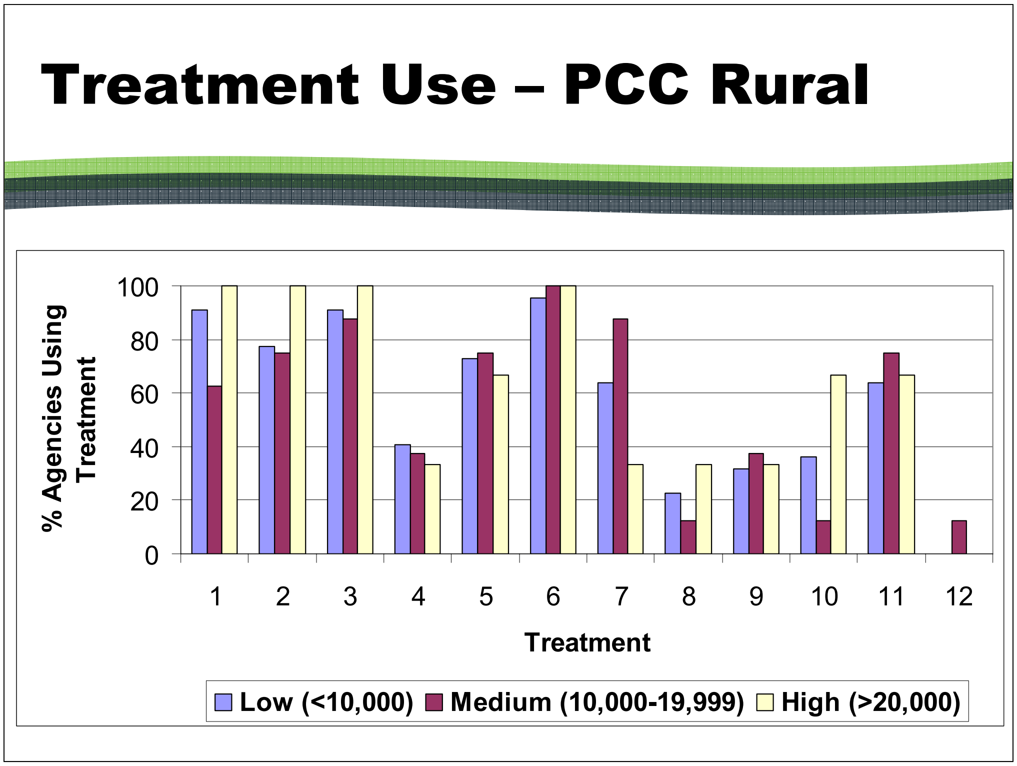
Those agencies with “high” (≥ 20,000) high-traffic volume designations have not reported
using the following preventive maintenance treatments:

cape seal, scrub seal, single and multiple course chip seals, cold in-place HMA recycling,
or ultrathin whitetopping; nor have they reported using any “other” treatments specifically.

PCC Treatments

1	Joint Seal
2	Crack Seal
3	Diamond Grinding
4	Diamond Grooving
5	Partial-Depth Patching
6	Full-Depth Patching

7	Dowel Bar Retrofit
8	Thin PCC Overlay
9	Thin Bonded Wearing Course
10	Thin HMA Overlay
11	Drainage Preservation
12	Other



Figures—Preventive Maintenance Treatment Use on PCC Roads

Rural Fig. Percentage of treatment use per rural high-volume traffic category (e.g., percentage of DOTs and Provinces with "low" high-traffic (<10,000) using joint seal).

Urban Fig. Percentage of treatment use per urban high-volume traffic category (e.g., percentage of DOTs and Provinces with "low" high-traffic (<10,000) using joint seal).

<u>Some comments based on the Figures:</u>

Those agencies with "high" (≥ 20,000) high-traffic volume designations have not reported using thin PCC overlays on urban roads; nor have they reported using any "other" treatments specific to them on either rural or urban roads.

For all high-traffic volume designations (low, medium, and high), joint seal, diamond grinding, and full-depth patching are used by at least 70 percent of reporting agencies.

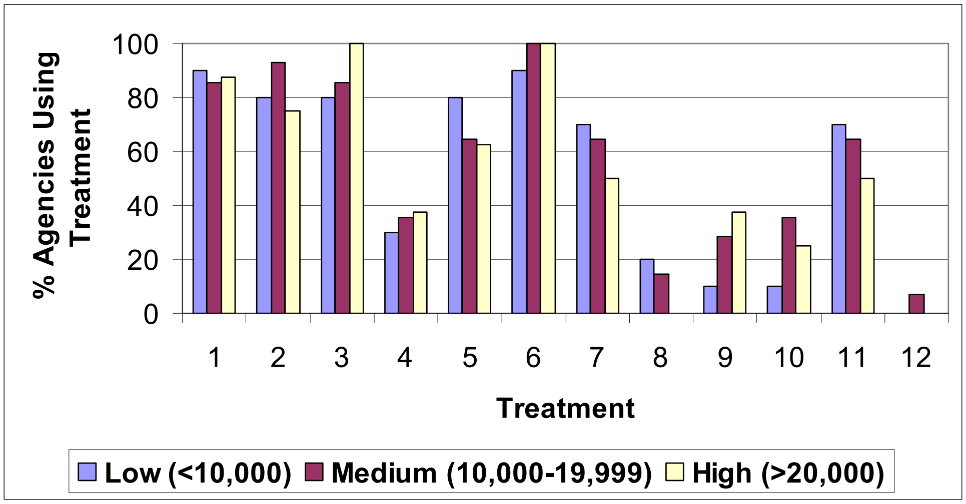
For all agencies reporting "high" high-traffic volume designations, joint seal, diamond grinding, and full-depth patching are used by at least 80 percent of reporting agencies; additionally, crack seal is used on rural roads by 100 percent of reporting agencies.

On both rural and urban roads, less than 45 percent of reporting agencies use diamond grooving and thin bonded wearing course. Additionally, less than 30 percent of reporting agencies use thin HMA overlays (<1.5-in.) on "high" high-traffic urban roads.

Dowel bar retrofitting (load transfer restoration) and drainage preservation is used on urban roads by at least 50 percent of reporting agencies.

The only DOT to report using an "other" treatment on PCC was Maine, which applied a thin bonded wearing course to a PCC section. This section has since been rubbilized and paved with HMA.

Treatment Use – PCC Urban



“High” Traffic (≥ 20,000 ADT)

Popular PCC treatments

- Joint Seal
- Diamond Grinding
- Full-Depth Patching

PCC treatments with limited use

- Diamond grooving, thin bonded wearing course, thin HMA or PCC overlay

Those agencies with “high” (≥ 20,000) high-traffic volume designations have not reported
using thin PCC overlays on urban roads; nor have they reported using any “other” treatments specific to them on either rural or urban roads.

For all agencies reporting “high” high-traffic volume designations, joint seal, diamond
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On both rural and urban roads, less than 45 percent of reporting agencies use diamond
grooving and thin bonded wearing course. Additionally, less than 30 percent of reporting
agencies use thin HMA overlays (<1.5-in.) on “high” high-traffic urban roads.

Criteria to Identify Best Practices

- Project selection criteria
- Treatment selection criteria
- Operational issues criteria

Project selection criteria

- high-traffic volume
- pavement condition
- environment condition

Treatment selection criteria

- evaluation of treatments in practice
- treatment performance
- cost-effective service life
- special considerations

Operational issues criteria

-closure times
- traffic control issues

Project Deliverables

- Final Report
  - Background and Research Approach
  - Findings and Applications
  - Conclusions and Suggested Research
  - Annotated Bibliography
  - Survey Results Summary
  - Guidelines

Following completion of task 4, APTech will prepare a draft final report documenting the entire research effort. The draft final report will draw upon much of the Interim Report, supplemented with a summary of the work conducted under Task 4 (with the guidelines presented in the appendix as a stand-alone document). In addition, the draft final report will also include recommendations for further research, such as related to the promising uses of treatments that are not currently commonly used on high-traffic volume roadways.

The Guidelines Will…

- Identify candidate roadways
  - Define in situ conditions conducive to pavement preservation activities
- Guide treatment selection
  - Delineate applicability and limitations
- Provide graphical supplements
  - e.g., decision trees and treatment matrices

The guidelines will be a useful tool for highway agencies and pavement management practitioners to not only identify suitable candidate high-volume roadways by defining characteristics of existing pavements that lend themselves to pavement preservation activities, but also to select appropriate pavement preservation treatments, delineating the applicability and limitations of various pavement preservation treatments.

In addition to the guidelines and supporting explanatory material, one of the best ways to summarize this type of information is graphically. Decision trees and treatment matrices are two graphical presentation methods that the project team has used successfully in the past and that will be used again in this project. These graphics will be a part of the guidelines and the final report.

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U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: **INFORMATION:** American Recovery and Reinvestment Act Funding and Workforce Development

Date: April 9, 2009

From: King W. Gee
Associate Administrator for Infrastructure

In Reply Refer To:
HIAM-20

To: Directors of Field Services
Director of Technical Services
Division Administrators

The challenges facing transportation departments are significant. Funding shortfalls in the face of considerable infrastructure needs force agencies to make difficult decisions in allocating limited resources. It is important to remember that one of the most critical resources is staffing. Whatever funds are available and whatever activities are undertaken, a trained and knowledgeable workforce is essential to ensure the desired outcome of highway design, construction, maintenance, and operations.

Workforce development has always been a key program within transportation departments, and one strongly supported by the FHWA. Federal regulations require that projects constructed with Federal-aid funds are staffed with qualified materials technicians and construction inspectors (23 CFR 637). The SAFETEA-LU currently provides States with the flexibility to use whatever funds are necessary to conduct a robust training program and give their workers the skills needed to ensure a quality product.

The passage of the American Recovery and Reinvestment Act of 2009 (Recovery Act) and the Omnibus Appropriations Act of 2009 has provided a much-needed infusion of capital into transportation departments' budgets. Many roadway construction and rehabilitation projects, delayed for lack of funds, will be able to proceed. However, the influx of funding and sudden increase in program size will increase our risk by exacerbating the shortage of trained maintenance and construction oversight staff already experienced by many agencies, consultants, and contractors. Many people will be put to work by the projects advanced. These individuals will need to quickly assume positions influencing construction quality and long-term infrastructure performance. It is our mission to support transportation departments in their efforts to train and develop this workforce. If you have immediate technical training needs in your State DOT's, one option is the National Highway Institute (NHI) with over 250 technical courses including over 25 online training courses and six Web conference sessions.

The Recovery Act funding has been allocated to transportation departments with the same provisions already provided under SAFETEA-LU. Specifically, Section 5204(e) of SAFETEA-LU (copy attached) allows funds from five core programs to be used for workforce development activities including employee education and training, and for programs to develop the future transportation workforce through career outreach and preparation. The five core programs are the Surface Transportation Program, National Highway System, Interstate Maintenance, Bridge Program, and Congestion Mitigation/Air Quality. Program funds used for training, education and workforce development activities receive 100 percent Federal funding (i.e., no State matching funds are required). To date, 31 States have employed this approach to support their training and development efforts. As departments move rapidly to apply the Recovery Act, its funds can be easily and quickly applied, with great impact, to training efforts through the provisions of SAFETEA-LU, Section 5204(e). Questions and answers are attached for your reference.

In addition, as the Recovery Act eases the strain on research and implementation budgets, transportation departments will have renewed ability to participate in beneficial pooled-fund efforts. The Transportation Curriculum Coordination Council (TCCC) has existed since 2000 to help State and local transportation departments share resources and address common construction and maintenance workforce challenges. A pooled-fund (solicitation 1205, http://www.pooledfund.org/projectdetails.asp?id=1205&status=1) (copy attached) is currently used by TCCC to fund technical training development for transportation departments nationwide and to avoid costly duplication of effort. The TCCC Core Curriculum, free Web-based training (on the Web site of NHI at www.nhi.fhwa.dot.gov), and numerous instructor-led courses were all made possible through State pooled-fund contribution. The continued success of TCCC and its ability to serve customer needs depends upon renewed contribution from transportation departments.

President Obama's objective for the Recovery Act is to reverse job losses and to strengthen America's infrastructure, achieving not just short-term expenditures but long-term gains. The FHWA division offices should encourage transportation departments to use the Federal funding available to support training and development and provide for the ongoing strength of the Nation's infrastructure. If you have any questions or if we can provide you additional information regarding the TCCC and workforce development, please contact Chris Newman in the Office of Asset Management at (202) 366-2023 or Christopher.newman@dot.gov or Rick Barnaby at the NHI at (703) 235-0520 or Richard.barnaby@dot.gov.

Please advise your State DOT's of these training options and the appropriate use of the funding available.

3 Attachments

Safe, Accountable, Flexible, Efficient, Transportation Equity Act: A Legacy for Users (SAFETEA-LU)

Section 5204(e) Surface Transportation Workforce Development, Training, and Education-

''(e) SURFACE TRANSPORTATION WORKFORCE DEVELOPMENT, TRAINING, AND EDUCATION.—

''(1) FUNDING.—Subject to project approval by the Secretary, a State may obligate funds apportioned to the State under sections 104(b)(1), 104(b)(2), 104(b)(3), 104(b)(4), and 144(e) for surface transportation workforce development, training, and education, including—

''(A) tuition and direct educational expenses, excluding salaries, in connection with the education and training of employees of State and local transportation agencies;

''(B) employee professional development;

''(C) student internships;

''(D) university or community college support; and

''(E) education activities, including outreach, to develop interest and promote participation in surface transportation careers.

''(2) FEDERAL SHARE.—The Federal share of the cost of activities carried out in accordance with this subsection shall be 100 percent.

''(3) SURFACE TRANSPORTATION WORKFORCE DEVELOPMENT, TRAINING, AND EDUCATION DEFINED.—In this subsection, the term 'surface transportation workforce development, training, and education' means activities associated with surface transportation career awareness, student transportation career preparation, and training and professional development for surface transportation workers, including activities for women and minorities.

For more information contact:
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Guidance for
Use of Federal Aid State Core Program Funds for
Training, Education and Workforce Development
SAFETEA-LU Section 5204(e)
Questions and Answers

These questions and answers are intended to provide information and guidance for the application of the Section 5204(e) of the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) as an assist to the FHWA Divisions and State departments of transportation in their efforts to enhance transportation workforce development.

Section 5204(e) of SAFETEA-LU allows funds from five core programs to be used for workforce development activities including employee education and training, and for programs to develop the future transportation workforce through career outreach and preparation. The five core programs are the Surface Transportation Program (STP), National Highway System (NHS), Interstate Maintenance, Bridge Program, and Congestion Mitigation/Air Quality (CMAQ). Program funds used for training, education and workforce development activities receive 100 percent federal funding.

Why was Section 5204(e) included in SAFETEA-LU?
The Transportation Research Board estimates that approximately 50 percent of the State transportation agency workforce will be eligible to retire within the next 10 years. According to the Department of Labor, the labor growth rate has declined from a high of 2.6 percent in the 1980's to a projected rate of 1.2 percent from 2000 to 2015, and an expected growth rate of only 0.2 percent from 2015 to 2025. As the labor shortage becomes more pronounced, so to will the competition for workers from all industries. Those industries that invest in developing their next generation of workers will be in a better position to compete for the smaller labor pool. Section 5204(e) will also provide a greater opportunity to develop the current transportation workforce. As transportation demand continues to grow faster than available resources, the ability to apply innovative technologies, processes and management principles through a skilled, technically competent workforce will provide the greatest return on investment for the transportation industry. The core fund workforce development provisions enhance the States' ability to prepare individuals for transportation careers, and to develop current employees including those with management, technical and vocational job responsibilities.

How do these provisions change what was allowed in TEA-21?
TEA-21 allowed the States to use up to ½ of 1 percent of Surface Transportation Program funds for employee training and included a 20 percent State matching requirement. The SAFETEA-LU provisions supersede TEA-21 and the more limited STP provisions by providing for 100 percent funding for workforce activities, extending eligibility for workforce development activities to the five core programs, and by not limiting the amount of funding available from each program. The SAFETEA-LU language also expands the types of eligible activities beyond training and education for employees to "pipeline" programs that will help students prepare for transportation careers.

What is the significance of the 100% federal funding provision?
The SAFETEA-LU provides for 100 percent federal funding if the core program funds are used for training, education, or workforce development purposes including "pipeline" activities. If used for these purposes, it is not necessary for the State to match the federal funds. The 100 percent funding provision is an indication of the continuing interest in transportation workforce development and provides for enhanced opportunities for the States to invest in employee professional development and student transportation career preparation.

What are some examples of "pipeline" programs that the core funds could be used to support?
Funds may be used for "education activities, including outreach, to develop interest and promote participation in surface transportation careers." Funds may also be used for activities associated with "student transportation career preparation." This could include, but not necessarily be limited to, student transportation related internships; cooperative education programs, university and college support activities, scholarship programs, and other efforts associated with transportation career outreach or that will help students prepare for a career in transportation. Funds could also be used for student outreach and internships associated with a particular project such as the T-REX project in Colorado.

How can the funds be used for employee education, training and professional development?
The core program funds can be used for a wide range of professional development activities that include training programs, academic course study, apprenticeship programs, and support for short-term work details or "rotational" assignments for the purpose of employee development. The core program funds may not used to pay any portion of employee salaries. Core program funds could be also be used for employee training and professional development that is necessary to support a specific surface transportation capital project, such as a major roadway or bridge construction project.

Can the core program funds be used for travel, equipment or materials purchases?
The funds can be used for travel, equipment or materials purchase, however the travel or equipment/materials purchase must be directly related to a defined employee training or professional development need, program or activity, or directly associated with a student transportation career awareness or preparation activity. The travel or equipment/material purchase must be used in primary support of the employee training or professional development, or student career activity. Travel to and from an industry meeting where training was one of several topics of discussion would not qualify for use of core funds. However, core funds could be used to support employee travel to and from a training or professional development program designed to improve the employees' skill, knowledge or abilities in surface transportation management or a technical discipline i.e. travel to a National Highway Institute or other industry training and professional development program.

In the "pipeline" area for example, bus transportation for students to participate in a transportation career awareness or development program, such as Construction Career Days would also be eligible. Materials used to support student transportation programs such as the American Association of State Highway and Transportation Officials (AASHTO) TRAC Program and the Associated General Contractors' (AGC) "Build Up" programs are just two of many types of programs and materials that could be paid for by core funds in support of student transportation career development.

Can the core program funds be used as matching funds for other federal programs such as the Local Technical Assistance Program (LTAP) or the University Transportation Centers (UTC)?
No. Federal funds cannot be used to match federal funds unless specifically provided for in statute and there is no provision in SAFETEA-LU or other statute that allows the core program funds in Section 5204(e) to be used as matching funds.

Are there any restrictions on the use of CMAQ or Transportation Enhancement (TE) funds for workforce development and training programs?
With the continuing emphasis on environmental issues and FHWA's commitment to develop sound environmental policy, should a state or local government choose to use CMAQ or TE funds for workforce development or training purposes, the funds should be directed to activities related to the CMAQ or the TE program area, respectively.

How do the workforce provisions affect the funding available in the core programs?
The use of core program funds for workforce development is discretionary. This allows states the flexibility to determine whether they want to invest these funds in projects or programs directed at addressing their workforce needs. While the use of core funds for workforce development will reduce the funds available for capital projects, the investment will help assure transportation workers have the skills and knowledge they need to be efficient and effective in their work, and to apply new and innovative technologies. In this way, the use of core funds for workforce development will not compete with core program activities, but, in fact, will be an important complement to those programs and as a support to the States' overall transportation mission.

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TPF Studies>>>Detailed View

Solicitation Number:	1205
Status:	Solicitation posted
Title:	Support of the Transportation Curriculum Coordination Council (TCCC)
Sponsoring Agency:	Federal Highway Administration
Sponsor Solicitation Contact:	William Zaccagnino (william.zaccagnino@fhwa.dot.gov) Phone: 202-493-3183 Fax: 202-493-3170
Lead Agency Contact:	Christopher Newman (Christopher.Newman@fhwa.dot.gov) Phone: 202-366-2023 Fax: 202-366-9981
Lead Agency:	Federal Highway Administration
FHWA Technical Liaison:	Christopher Newman (Christopher.Newman@fhwa.dot.gov)
Partners:	IA, MN, MO, TX
Date Posted:	5/20/2008 8:17:36 AM
Solicitation Expires:	3/31/2009
Commitment Start Year:	2008
Commitment End Year:	2013
Duration:	60 months
100% SP&R Approval:	Approved
Commitments Required:	$5,000,000
Commitments Received:	$425,000
Background:	Through contributions to a 5-year pooled fund (TPF 5-(046)), the TCCC has developed a Core Curriculum Matrix comprised of five program areas (Construction, Materials, Maintenance, Safety, and Employee Development) and provided training competencies for a wide variety of disciplines within each subject area. The curriculum is used by State and local DOTs in their efforts to establish training programs and to

develop specific courses for their technical personnel. It is also being used by the TCCC to guide its development of course materials to be shared nationwide. The curriculum matrices are designed to be a living document that will grow as the program continues to evolve; accordingly, the curriculum will be maintained on the TCCC website (www.nhi.fhwa.dot.gov/tccc) and will be updated periodically as new disciplines, course materials, and technologies emerge.

The TCCC also has several new training courses under development or recently completed for use by State and local DOTs:
¿ Managing Construction Workmanship (134055), designed for construction inspectors and resident engineers, provides information on roles and responsibilities, acceptance of work, and construction quality.

¿ The Bridge Construction Inspection (130088) course has been completed and is designed to give basic knowledge of bridge construction procedures to construction inspectors.

¿ Quality Assurance Technologies training developed as a joint effort between FHWA and the NETTCP

¿ The Pavement Preservation Online Guide and Training (131110) is now available for delivery and is intended to provide participants with an introduction to the Pavement Preservation Online Guide (PPOG) created by Caltrans and the National Center for Pavement Preservation. This course is primarily target at individuals unfamiliar with pavement preservation policy and technical information.

¿ The Ethics Awareness for the Transportation Industry course has been developed for online training and is designed to provide employees with the ethical expectations within their respective DOT. It may be modified to any States' codes.

¿ The Geotechnical steering committee has developed two new training courses: Mechanically Stabilized Earth Walls/ Reinforced Soil Slopes, and Subsurface Investigation Qualification Course.

¿ Courses under development include Inspection of Bridge Rehabilitation, Maintenance Leadership Academy, Use of GIS in Construction, and Environmental Factors in Highway Construction. Other recently-identified training needs include Embankment Inspection, Basic Earthwork for Inspectors, Placement and Testing of Self-Consolidating Concrete, and Inspection of Pipe Installation.

¿ The TCCC is partnering with contractor associations in the pavement preservation industry to expand the Maintenance Core Curriculum to provide greater detail on training and qualification contractor personnel and inspectors involved in the placement of pavement preservation treatments.

The objective of this effort is to increase the quality and performance of pavement preservation treatments, and to increase DOT confidence in the application of new "tools in the toolbox."

The pooled funds under the previous TCCC support project have been supplemented by FHWA Office of Asset Management, Office of Pavement Technology, and National Highway Institute to cover expenses related to training development and other activities. Such an extensive level of funding from the FHWA program offices has enabled them to support the TCCC to a level far beyond what was contributed by the States to the pooled fund. Because of current funding levels under FHWA appropriations, a similar level of funding support from FHWA cannot be expected. It is therefore critical that State involvement be secured for a new pooled fund project for continued support of the TCCC.

Objectives: Rebuilding and maintaining the Nation's highways requires agencies and industry to have a trained and qualified workforce from agencies and industry. With the loss of experience in the workforce, training is an industry priority. Agencies and the highway industry across the country face this serious shortage of trained and experienced personnel because of attrition and an aging workforce. We must meet the challenge to preserve the system investments and carry out capital improvements for future growth.

Since 2000, the TCCC, a partnership between the FHWA, State and local DOTs, and private industry, has diligently worked to support the training of transportation industry¿s technical personnel. The TCCC¿s mission is to:
¿ Provide leadership at the national level.
¿ Develop and maintain national curricula for the various transportation disciplines.
¿ Identify training and certification requirements.
¿ Coordinate/facilitate training efforts.

To achieve its mission, the TCCC embraces the following objectives:
¿ Optimize resources through concentrated efforts in the development of core training and qualification mediums.
¿ Improve the skills and abilities of the transportation

technical personnel.
¿ Promote the sharing of technical training resources among government and private transportation industry organizations.
¿ Promote uniformity in training content and qualification requirements to facilitate reciprocity between States, local organizations, and regions.
¿ Optimize the usage of AASHTO standards in training development.

Scope of Work:

This project will be for the creation of a new pooled fund with similar goals to support the TCCC. It will be used for the further development of core curriculum, development of training materials, and tools for sharing training materials. Following is a list of initial activities that will be performed to continue the sharing of training and qualification resources among the transportation industry, while revising and developing identified core training materials and short courses:

¿ Core curriculum materials will be identified under the direction of the TCCC and fully developed under NHI contract or other partner entities (i.e. LTAP Center/State DOT) to address national training needs. Completely developed courses will offer between 80-90 percent of what State agencies, regional groups or industry technical groups need. Recipients may use core materials as developed or individually tailor the materials to meet specific needs for local or regional training programs.

¿ The Core Curriculum materials described above will be made available on the TCCC¿s website and will be searchable through an interactive database. The new pooled funds will be used to continue this development as well as maintain this dynamic, national database.

¿ Continue to promote uniformity in training content and qualification requirements.

¿ Continue to advocate the dissemination of information among training and certifying State, regional and industrial organizations.

¿ Provide a central repository for transportation training resources in the areas of Construction, Employee Development, Maintenance, Materials, and Safety.

¿ Further encourage the development and improvement of AASHTO standards and maximize their usage in member's training and qualification programs.

¿ The TCCC is coordinating with the NHI for hosting a suite of technical training courses to be available in a web-based format.

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Task 2. Pavement Preventive Maintenance Treatment Construction Skill Level Competency Matrices

Input from the Federal Highway Administration was used to identify treatments included in this project. The final list includes treatments for both flexible and rigid pavements, as indicated in table 1.

Table 1. Treatments selected for the development of skill level competency matrices.

Flexible Pavements	Rigid Pavements
Asphalt Crack Fill	Diamond Grinding
Asphalt Crack Seal	Dowel Bar Retrofit
Asphalt Rejuvenation	Diamond Grooving
Chip Seal	Joint Seal Replacement
Fog Seal	
Micro-surface	
Slurry Seal	

Skills were then identified for appropriate responsibilities at four different levels. The levels are described in greater detail in table 2, while the competency matrices are provided in the remainder of this document.

Table 2. Skill level descriptions and example titles.

Skill level	Description	Example level job titles
I—Entry	New employee or trainee with no previous experience in subject area. Performs specific tasks under direct supervision.	New maintenance workers, engineers-in-training, inspector trainees, and construction laborers.
II—Intermediate	Understands and demonstrates skills (is competent) in one or more areas included in the entry-level role. Is able to perform specific tasks under general supervision.	Lead workers, maintenance team leaders, project inspectors, engineers-in-training, construction equipment operators, and construction crew leaders.
III--Advanced	Understands and demonstrates specialized skills in a variety of tasks included in the intermediate level role. Is able to perform specialized tasks in limited areas or broad-based tasks with little to no daily supervision.	Maintenance supervisors, field engineers, and construction foremen.
IV—Project Manager/ Administrator (Project Superintendent)	Prepares/reviews the plans and schedules for specific activities and oversees the management of day-to-day activities in one or more specific tasks, on one or more projects that can cover a wide range of geographical areas. Maintains accountability for resource management. Possesses the competency to be responsible for decision-making on a broad spectrum of tasks, ranging from routine to complex. It is recommended that this level of personnel holds various skills from the previous skill levels.	Maintenance engineers and directors, project engineers, project managers, contractor superintendents, and company owners.

FLEXIBLE PAVEMENTS

Asphalt Crack Fill and Asphalt Crack Seal

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Project Selection		(Agency) Identify pavement candidates and define project limits for a crack treatment operation (determine candidates by differentiating between structural and functional distress, crack severity, and crack density) Identify working cracks and non-working cracks	(Agency) Select projects from submitted candidate pavements for current year or future year crack treatment program, consistent with strategic plan (Agency) Scope project for design and cost estimates, including the review and approval of specifications and traffic control plan (Agency) Prepare bid package Establish proposed work schedule	(Agency) Verify project does not conflict with planned future construction or utility projects (Agency) Direct the programming of project for funding (Agency) Develop guidance documents consistent with strategic plan and directives (including crack fill method) for asphalt crack filling and (including reservoir configuration) asphalt crack sealing program (Contractor) Review project site to estimate sealant quantities prior to bidding (if applicable)
Materials	Receive shipped materials (e.g. sealants, backer rod, anti-tack solution, and so on), verify for quality, quantity, MSDS, and so on, and provide documentation for field engineer Maintain materials storage area	Sample and forward sealant samples to laboratory for testing Regularly check sealants for temperature and visible abnormalities	Estimate material quantities needed for project	Select appropriate sealant for asphalt crack fill and sealant for asphalt crack sealing consistent with environmental conditions and anticipated traffic volume Select approved material suppler prior to bidding (if applicable), receive price quotes, arrange delivery schedule, and sign contract agreement Verify compliance with safety and environmental regulations

Asphalt Crack Fill and Asphalt Crack Seal (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Equipment	Assemble required hand tools (e.g. squeegees) and specified size of cutter heads, bits, blades, and so on prior to starting work Assemble safety equipment and protective wear for project Inventory quantity and condition of traffic control devices to comply with specifications and traffic control plan	Inspect kettles, pumps, hoses, and so on for plugs, leaks, or damaged components that can cause disruption to project Calibrate and check thermometers for accurate material temperature control Maintain sufficient replacements of specified size cutter heads, bits, blades, and other wearing parts Check abrasive cleaning unit (if necessary for sawn reservoir) and air compressor for air dryer and oil trap and inspect for contaminant-free operation	Insure project has necessary equipment on project site prior to starting work, including equipment such as crack vacuums if required for air quality compliance Insure fuel supplies are readily available and convenient for equipment (such as saw, compressor, and kettle) Check that weather radar information is available or accessible to monitor potential adverse weather conditions	Insure equipment is in compliance with OSHA regulations and state air quality plans
Crack Preparation	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, and other selected techniques Deploy protective wear (including respirators if abrasive blasting is necessary) Remove debris and clean cracks for asphalt crack filling on non-working cracks Create reservoir (by rout or saw method), abrasive blast if sawn, blow clean with compressed air for working cracks	Inspect traffic control setup Insure traffic flow and pedestrians are away and protected from potential flying debris Direct necessary preparatory work, such as to clean cracks or create reservoir and clean crack Inspect prepared crack reservoir (if required) for proper size, configuration, and free of loose material Monitor equipment operation used to clean cracks	Initiate public relations outreach to notify residents of pending work Conduct pre-construction meeting with crew to discuss description of working and non-working cracks in addition to safety and traffic control Sequence relocating work zones to coordinate a moving repair operation with planned traffic flow	Coordinate scheduling of project to avoid conflicts with major public events or unrelated work

Asphalt Crack Fill and Asphalt Crack Seal (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Application	Install backer rod if necessary and apply sealant material for specific application and configuration Place an anti-tack solution on sealant material subject to contact with vehicle tires	Inspect prepared crack for cleanliness and absence of any moisture (dryness) Monitor kettle temperature per sealant manufacturer's requirements on a regular basis Inspect backer rod placement (if required) and application in filling reservoir with sealant or to the specified configuration	Note: Usually crack preparation and application are performed as one operation. If not, the same factors will apply as described under crack preparation	Note: Usually crack preparation and application are performed as one operation. If not, the same factors will apply as described under crack preparation
Project Management	Record work hours on project	Maintain detailed Inspector's Daily Reports (IDR) and/or Contractor's Daily Reports (CDR)	Implement and monitor quality assurance program Authorize opening to traffic Direct demobilization of manpower, equipment, and supplies	Implement procedures to update the pavement management system (PMS), maintenance management system (MMS), and cost accounting documents Process payments, overruns, claims, and approve project acceptance

Chip Seal

<table>
<tr><th rowspan="2">Disciplines</th><th colspan="4">Competencies by Skill Level (Agency Staff and Contractor Personnel)</th></tr>
<tr><th>Level I
(New Inspector)</th><th>Level II
(Inspector)</th><th>Level III
(Resident Engineer)</th><th>Level IV
(Maintenance Director)</th></tr>
<tr><td>Project Selection</td><td></td><td>(Agency) Identify pavement candidates (based on ride, rut, friction, and distress) and define project limits for a chip seal</td><td>(Agency) Select projects from submitted candidate pavements for current year or future year chip seal program consistent with strategic plan

(Agency) Scope project for project design and cost estimates, including the review and approval of specifications and traffic control plan

(Agency) Prepare bid package

Establish proposed work schedule</td><td>(Agency) Verify project does not conflict with planned future construction or utility projects

(Agency) Direct the programming of project for funding

(Agency) Develop guidance documents consistent with strategic plan and directives for chip seal program

(Contractor) Review project site to estimate quantities and check for unique conditions prior to bidding (if applicable)</td></tr>
<tr><td>Materials</td><td>Receive shipped materials (e.g. sealants, backer rod, anti-tack solution, and so on), verify for quality, quantity, MSDS, and so on, and provide documentation for field engineer

Maintain materials storage area</td><td>Sample and conduct regular gradation tests on aggregate

Sample and forward emulsion samples to laboratory for testing

Direct and insure good stockpile management of aggregate and regularly check emulsion for temperature and visible abnormalities</td><td>Estimate material quantities needed for project

Determine location and arrange agreements for the project’s material staging area(s)</td><td>Select appropriate type and approved source of emulsion and aggregate gradation for traffic and environmental conditions, including physical and chemical characteristics

Select approved material suppler prior to bidding (if applicable), receive price quotes, and arrange delivery schedule and sign contract agreement

Insure compliance of safety and environmental regulations in material staging area</td></tr>
</table>

Chip Seal (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Equipment	Assemble required hand tools for project Assemble safety equipment and protective wear for project Inventory quantity and condition of traffic control devices to comply with specifications and traffic control plan	Inspect equipment for leaking fluids, mechanical wear, and any items that can cause potential disruption or damage to the project Calibrate and adjust equipment prior to commencement of project to ensure proper application rate	Verify project has necessary equipment on site prior to starting work Verify fuel supplies are readily available and convenient for equipment Verify weather radar is available or accessible to monitor potential adverse weather conditions	Specify sweeping equipment or vacuum equipment for project location consistent with public expectations (rural, urban, recreational, residential, and so on) Verify equipment is in compliance with OSHA regulations
Surface Preparation	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or other approved method Repair major distresses, fill wide cracks, and clean/broom pavement surface Protect structures, castings and place temporary lane markers prior to work	Inspect traffic control setup Delineate areas for pavement repair work, crack fill, and structure protection Operate equipment to prepare and clean the pavement surface in advance of chip seal application Inspect materials and completed surface prep work and verify pavement is free of debris and clean prior to beginning chip seal application Record and/or verify material quantities, man-hours, and equipment use	Initiate public relations outreach to notify residents of pending work Conduct pre-construction meeting with crew to discuss all project elements including safety and traffic control Maintain project records of measured material quantities, man-hours, and equipment use	Coordinate scheduling of project to avoid conflicts with major public events or unrelated work Schedule and, if necessary, prepare contract for major repairs needed at spot locations prior to chip seal application

Chip Seal (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Application	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or other approved method Prepare transverse joint at start-up and stopping	Inspect placement procedures, surface uniformity and periodically measure application rates Operate equipment in a safe and efficient manner consistent with best practices Troubleshoot potential problems	Check weather forecast and radar to insure suitable condition for placement and initial cure Coordinate sequencing and operation of emulsion distributors, aggregate spreaders, rollers, brooms, and haul trucks Sequence relocating work zones to coordinate a moving chip sealing operation with planned traffic flow Troubleshoot potential problems	Troubleshoot potential problems Review project after completion to insure construction debris or unused materials are removed from site or staging area
Brooming	Remove protective covers from structures and where otherwise appropriate	Operate power broom or vacuum sweeper	Direct initiation of brooming operation at the cure time and condition that ensures no damage occurs to the chip seal	Schedule temporary and permanent lane markings after brooming
Project Management	Record work hours on project	Maintain detailed Inspector's Daily Reports (IDR) and/or Contractor's Daily Reports (CDR)	Implement and monitor quality assurance program Authorize opening to traffic Direct demobilization of manpower, equipment, and supplies	Implement procedures to update the PMS, MMS, and cost accounting documents Process payments, overruns, claims, and approve project acceptance

Fog Seal and Asphalt Rejuvenation

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Project Selection		(Agency) Identify fog seal candidates (based primarily on oxidation, early raveling, slowing permeability and traffic volume) or asphalt rejuvenation (based primarily on modifying and improving existing chemical and rheological properties) and define project limits for a fog seal or asphalt rejuvenation	(Agency) Select projects from submitted candidate pavements for current year or future year fog seal or rejuvenation program consistent with strategic plan (Agency) Scope project for project design and cost estimates, including the review and approval of specifications and traffic control plan (Agency) Prepare bid package Establish work proposed schedule	(Agency) Verify project does not conflict with planned future construction or utility projects (Agency) Direct the programming of project for funding (Agency) Develop guidance documents consistent with strategic plan and directives for fog seal or rejuvenation program Establish acceptable friction levels for opening to traffic (Contractor) Review project site to estimate quantities and check for unique conditions prior to bidding (if applicable)
Materials	Receive shipped materials (e.g. sealants, backer rod, anti-tack solution, and so on), verify for quality, quantity, MSDS, and so on, and provide documentation for field engineer Maintain materials storage area	Sample and forward diluted emulsion samples or rejuvenator samples to laboratory for testing Sample and conduct gradation tests on blotting sand (if necessary for fog seal only) Inspect rejuvenator for color dyeing to insure product integrity from supplier Direct and insure good stockpile management of blotting sand and regularly check dilute emulsion for temperature and visible abnormalities	Estimate material quantities needed for project based on surface absorption testing (e.g. ring test or laboratory permeability tests) Determine location and arrange agreements for the project's material staging area(s)	Select appropriate type and approved source of diluted emulsion (fog seal) or rejuvenator and clean blotting sand gradation for traffic and environmental conditions, including physical, chemical, and rheological (rejuvenators) characteristics Select approved material suppler prior to bidding (if applicable), receive price quotes, arrange delivery schedule, and sign contract agreement Verify compliance of safety and environmental regulations in material staging area

Fog Seal and Asphalt Rejuvenation (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Equipment	Assemble safety equipment and protective wear for project Inventory quantity and condition of traffic control devices to comply with specifications and traffic control plan	Inspect asphalt distributor, sanding equipment, and other mechanical devices for leaking fluids, mechanical wear, and any items that can cause potential disruption or damage to the project Calibrate and adjust asphalt distributor prior to commencement of project to ensure proper application	Verify project has necessary equipment on site prior to starting work Verify friction testing equipment is available Verify fuel supplies are readily available and convenient for equipment Verify weather radar is available or accessible to monitor potential adverse weather conditions	If necessary, specify sweeping equipment or vacuum equipment for project location consistent with public expectations (rural, urban, recreational, residential, and so on) Verify equipment is in compliance with OSHA regulations
Surface Preparation	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or other approved method Manually clean pavement surface (if necessary) Protect structures, castings, and other infrastructure elements as necessary prior to fog sealing or applying rejuvenator	Inspect traffic control setup Delineate areas for structure protection Operate equipment to clean the pavement surface in advance of fog seal or rejuvenator application Inspect pavement surface to verify it is clean prior to beginning fog seal or rejuvenator application Record and/or verify material quantities, man-hours and equipment use	Initiate public relations outreach to notify residents of pending work Conduct pre-construction meeting with crew to discuss all project elements, including safety and traffic control	Coordinate scheduling of project to avoid conflicts with major public events or unrelated work

Fog Seal and Asphalt Rejuvenation (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Application		Inspect fog seal absorption of diluted emulsion for correct application and periodically measure application rates Operate equipment in a safe and efficient manner consistent with best practices Perform friction testing to pavement after fog seal or rejuvenator has initially cured Troubleshoot potential problems	Check weather forecast and radar to verify suitable conditions for placement and initial cure Coordinate sequencing and operation of sand spreader, brooms, and other equipment Monitor friction testing of pavement Sequence relocating work zones to coordinate a moving fog sealing operation with planned traffic flow Troubleshoot potential problems	Troubleshoot potential problems Review project on completion (after fully cured product is achieved) to verify excess sand is removed from pavement and staging area
Project Management	Record work hours on project	Maintain detailed Inspector's Daily Reports (IDR) and/or Contractor's Daily Reports (CDR)	Implement and monitor quality assurance program Authorize opening to traffic with posted speed reduction after friction is within acceptable range Direct demobilization of manpower, equipment, and material	Implement procedures to update the PMS, MMS, and cost accounting documents Process payments, overruns, claims, and approve project acceptance

Micro-surfacing and Slurry Seal

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Project Selection		(Agency) Identify pavement candidates (based on ride, rut, friction, distress, and raveling) and define project limits for a micro-surfacing or slurry seal	(Agency) Select projects from submitted candidate pavements for current year or future year micro-surfacing and slurry seal programs consistent with strategic plan (Agency) Scope project for project design and cost estimates, including the review and approval of specifications and traffic control plan (Agency) Prepare bid package Establish proposed work schedule	(Agency) Verify project does not conflict with planned future construction or utility projects (Agency) Direct the programming of project for funding (Agency) Develop guidance documents consistent with strategic plan and directives for micro-surfacing or slurry sealing program (Contractor) Review project site to estimate quantities and check for unique conditions prior to bidding (if applicable)
Materials	Receive shipped materials (e.g. sealants, backer rod, anti-tack solution, and so on), verify for quality, quantity, MSDS, and so on, and provide documentation for field engineer Maintain materials storage area	Sample and conduct regular gradation tests on aggregate Sample and forward emulsion samples to laboratory for testing Direct and verify good stockpile management of aggregate and regularly check emulsion for temperature and visible abnormalities	Estimate material quantities needed for project Determine location and arrange agreements for the project's material staging area(s)	Select appropriate type of slurry system (includes micro and slurry) and approved source of emulsion and aggregate gradation for traffic volume, usage, and environmental conditions, including physical and chemical characteristics Select approved material suppler prior to bidding (if applicable), receive price quotes, arrange delivery schedule, and sign contract agreement Verify compliance of safety and environmental regulations in material staging area

Micro-surfacing and Slurry Seal (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Equipment	Assemble required hand tools for project Assemble safety equipment and protective wear for project Inventory quantity and condition of traffic control devices to comply with specifications and traffic control plan	Inspect equipment for leaking fluids, mechanical wear, and any items that can cause potential disruption or damage to the project Calibrate and adjust equipment prior to commencement of project to ensure proper application rate Insure surface texture is controlled by new drags or strike-offs	Insure project has necessary equipment of sufficient number for uninterrupted operation on project site prior to starting work Insure fuel supplies are readily available and convenient for equipment Insure weather radar is available or accessible to monitor potential adverse weather conditions	Specify rut-box (if necessary) for project Verify equipment is in compliance with OSHA regulations
Surface Preparation	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or other approved method Repair major distresses, fill wide cracks, wash-off animal remains or grease spots, and broom pavement surface Protect structures, castings and remove all thermoplastic pavement markings prior to work	Inspect traffic control setup Delineate areas for pavement repair work, crack fill, and structure protection Operate equipment to prepare and clean the pavement surface in advance of micro-surfacing or slurry seal application Inspect materials and completed surface prep work and insure pavement is free of debris and clean prior to beginning application Record and/or verify material quantities, man-hours, and equipment use	Initiate public relations outreach to notify residents of pending work and provide education on vehicle operation on slurries Conduct pre-construction meeting with crew to discuss all project elements, including safety and traffic control Maintain project records of measured material quantities, man-hours, and equipment use	Coordinate scheduling of project to avoid conflicts with major public events or unrelated work Schedule and, if necessary, prepare contract for major repairs needed at spot locations prior to micro-surfacing or slurry seal application

Micro-surface and Slurry Seal (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Application	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or other approved method Prepare transverse joint at start-up and stopping locations Transfer and adjust spreader box on truck mount operations	Inspect placement procedures, surface texture uniformity, and periodically measure application rates consistent with project specifications Place asphalt tack coat on pavement surfaces with high traffic volumes, concrete surfaces, and polished surfaces prior to micro-surfacing or slurry seal Operate equipment in a safe and efficient manner consistent with best practices Troubleshoot potential problems	Check weather forecast and radar to insure suitable condition for placement and initial cure Coordinate sequencing and operation of haul trucks with operations Sequence relocating work zones to coordinate a moving micro-surfacing or slurry seal operation with planned traffic flow Troubleshoot potential problems	Troubleshoot potential problems Review project after completion to insure construction debris or unused materials are removed from site or staging area
Project Management	Record work hours on project	Maintain detailed Inspector's Daily Reports (IDR) and/or Contractor's Daily Reports (CDR)	Implement and monitor quality assurance program Authorize opening to traffic Direct demobilization of manpower, equipment, and supplies	Implement procedures to update the PMS, MMS, and cost accounting documents Process payments, overruns, claims, and approve project acceptance

RIGID PAVEMENTS

Diamond Grinding and Diamond Grooving

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Project Selection		(Agency) Identify polished, faulted, and/or poor riding concrete pavements for diamond grinding or identify hydroplaning pavements for diamond grooving and define project limits for project candidate	(Agency) Conduct a forensic investigation on the concrete pavement to insure it is a good project candidate (Agency) Select projects from submitted candidate pavements for current year or future year concrete diamond grinding consistent with strategic plan (due to safety concerns, hydroplaning pavements will be prioritized for diamond grooving and polished pavements will be prioritized for diamond grinding) (Agency) Scope project for project design and cost estimates, including approval of specifications for machine cutting width, ride quality, blade spacing (for grooving) and traffic control plan (Agency) Prepare bid package and maintain pavement core(s) in area office for contractor inspection Establish proposed work schedule	(Agency) Verify project does not conflict with planned future construction or utility projects (Agency) Direct the programming of project for funding (Agency) Develop guidance documents consistent with strategic plan and directives for concrete diamond grinding or diamond grooving work (Agency) Establish texture and smoothness requirements for completed diamond grinding or diamond grooving work Insure specification includes training and certification requirement for profilograph or pavement profiler operator (Contractor) Review project site and area office (for core inspection) to determine blade type, blade spacing and production rates and check for unique conditions prior to bidding
Materials			(Agency) Obtain historical information about aggregate used to construct concrete pavement Arrange use of acceptable water supply for obtaining large quantities of water required for concrete diamond grinding or diamond grooving (make contracting arrangements if necessary)	Meet with state environmental regulatory agency to determine disposal method for concrete slurry Contract with approved location for concrete slurry disposal (if necessary) Verify compliance with OSHA and Environmental regulations

Diamond Grinding and Diamond Grooving (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Equipment	Assemble diamond blades and spacer shims on rotating arbor shaft Assemble safety equipment and protective wear for project Inventory quantity and condition of traffic control devices to comply with specifications and traffic control plan	Maintain sufficient replacement blades and spacer shims for grinding or grooving operations Inspect diamond grinding or diamond grooving machines and water tankers for proper operation and or damaged components that can cause disruption to project Inspect diamond blades for wear Inspect blade spacing on rotating arbor of diamond grinding or diamond grooving machine to insure texturing cut meets contract documents	Verify project has necessary equipment on project site prior to starting work Verify fuel supplies are readily available and convenient for equipment Verify and calibrate surface profile measuring equipment to insure it is properly working and provides accurate measurements Based on concrete hardness, determine type of diamond blade and blade spacing on rotating arbor to comply with specifications	Verify equipment is in compliance with OSHA regulations and state air quality requirements

Diamond Grinding and Diamond Grooving (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Pavement Preparation	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or as otherwise specified Delineate obtrusive structures by marking (painting) iron castings or other infrastructure elements that will impede the diamond grinding/grooving proc3ess Remove iron-encased raised pavement markers and patch with concrete repair (if applicable) Initiate all repairs that must be performed prior to beginning diamond grinding or diamond grooving (this work is covered by other skill competencies)	Inspect traffic control setup Monitor and inspect pavement preparation techniques	Initiate public relations outreach to notify residents of pending work Conduct pre-construction meeting with crew to discuss all project elements including safety and traffic control Sequence relocating work zones to coordinate a moving operation with planned traffic flow	Coordinate scheduling of project to avoid conflicts with major public events or unrelated work

Diamond Grinding and Diamond Grooving (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Application	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or as otherwise specified Maintain water tanks with consistent supply of water Remove cement slurry from site for proper disposal (if required) Initiate concrete seal replacement after diamond grinding or diamond grooving (this work is covered by other skill competencies)	Maintain grinding or grooving alignment in a direction parallel with the pavement centerline or as directed in the contract documents Insure air and surface temperatures are predicted above freezing for diamond grinding or diamond grooving Inspect cut surface texture to insure dimensions and tolerances comply with specifications Inspect for uniform cut depth between subsequent passes and for consistent texture throughout project, particularly at overlaps of passes Insure all cement slurry is removed from pavement surface and properly disposed Measure pavement smoothness after diamond grinding or diamond grooving to verify compliance with specification Troubleshoot potential problems	Check weather forecast and radar to insure suitable condition for diamond grinding/grooving work Sequence relocating work zones to coordinate a moving repair operation with planned traffic flow Monitor pavement smoothness to insure compliance with project specification; direct regrinding of pavement surface if non-compliant Troubleshoot potential problems	Troubleshoot potential problems Verify operator of profilograph or pavement profiler meets training and certification requirements of specifications Review project after completion to insure concrete debris and other materials are removed from site
Project Management	Record work hours on project	Maintain detailed Inspector's Daily Reports (IDR) and/or Contractor's Daily Reports (CDR) Receive slurry disposal documentation from authorized waste terminal (if off-site disposal is necessary)	Implement and monitor quality assurance program Authorize opening to traffic Direct demobilization of manpower, equipment, and supplies	Implement procedures to update the PMS, MMS, and cost accounting documents Process payments, overruns, claims, and approve project acceptance

Dowel Bar Retrofit

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Project Selection		(Agency) Identify jointed concrete pavements where transverse joints and full-width/depth transverse cracks lack load transfer capability (faulting) and define the severity and extent for dowel bar retrofit	(Agency) Select projects from submitted candidate pavements for current year or future year concrete dowel bar retrofit consistent with strategic plan (Agency) Scope project for project design and cost estimates, including the review and approval of specifications and traffic control plan (Agency) Prepare bid package Establish proposed work schedule	(Agency) Verify project does not conflict with planned future construction or utility projects (Agency) Direct the programming of project for funding (Agency) Develop guidance documents consistent with strategic plan and directives for concrete dowel bar retrofit work (Contractor) Review project site to estimate quantities and check for unique conditions prior to bidding (if applicable)
Materials	Receive shipped materials and inspect dowel bars, end caps, dowel bar chairs, and compressible foam material for size and damage Inventory abrasives, saw blades, patch material, and other disposable materials for slot preparation, and verify for quality, quantity, MSDS, and so on, and provide documentation for field engineer Prepare slot cementing grout dry mixture Receive sealant shipping documents/certifications and so on for field engineer Maintain materials storage area	Inspect all materials to verify conformance with specifications	Estimate quantities of materials needed for project and insure they are on site prior to starting work	Select appropriate materials from approved sources for performance, safety, and environmental conditions Verify appropriate material specification is used in bid package Select approved material suppler prior to bidding (if applicable), receive price quotes, arrange delivery schedule, and sign contract agreement Verify compliance with safety and environmental regulations in material staging area

Dowel Bar Retrofit (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Equipment	Assemble required hand tools prior to starting work Assemble safety equipment and protective wear for project Inventory quantity and condition of traffic control devices to comply with specifications and traffic control plan	Inspect equipment, mixers, hoses, and other critical construction equipment for proper operation and or damaged components that can cause disruption to project Inspect testing equipment to insure it is properly working and in good condition Maintain sufficient replacement blades for sawing and jackhammer chisel points for removing slot concrete Check abrasive cleaning unit and air compressor for air dryer and oil trap and inspect for contaminant free operation	Verify project has necessary equipment on project site prior to starting work Verify fuel supplies are readily available and convenient for equipment (saw, compressor, mixer, and so on) Verify weather radar is available or accessible to monitor potential adverse weather conditions	Verify equipment is in compliance with OSHA regulations and state air quality plans
Pavement Preparation	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or as otherwise specified Deploy protective wear, including respirators for abrasive blasting, sawing, and so on Remove debris and clean slots, joints, cracks, and other concrete elements to promote bond (this involves operating abrasive blasting nozzles, compressed air wands, or as specified)	Inspect traffic control setup Insure traffic flow and pedestrians are safe from potential flying debris Direct necessary preparatory work, such as to remove and clean old sealant from joint (and full-depth transverse cracks) and dowel bar slots Inspect slots and joint cleaning procedures and techniques Monitor equipment operation used to remove and clean slots and concrete joints (and full-depth transverse cracks)	Initiate public relations outreach to notify residents of pending work Delineate areas by marking (painting) concrete dowel bar retrofit locations consistent with plans Conduct pre-construction meeting with crew to discuss all project elements, including safety and traffic control Sequence relocating work zones to coordinate a moving repair operation with planned traffic flow	Coordinate scheduling of project to avoid conflicts with major public events or unrelated work

Dowel Bar Retrofit (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Application	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or as otherwise specified Clean slots, install dowel chairs, end caps, dowel bars, reform joint Fill slot with cementing grout, finish, apply curing compound, and seal joint/crack consistent with the specified dowel bar retrofit project	Direct slot dowel bar retrofitting and joint sealing operation Inspect dowel bar retrofit slot preparation to insure cleanliness prior to filling, insure proper slot finishing, and removal of pre-formed joint Inspect joint seal replacement consistent with outlined procedures Troubleshoot potential problems	Check weather forecast and radar to insure suitable condition for concrete dowel bar retrofit work Sequence relocating work zones to coordinate a moving repair operation with planned traffic flow Troubleshoot potential problems	Troubleshoot potential problems Review project after completion to insure concrete debris and other materials are removed from site
Project Management	Record work hours on project	Maintain detailed Inspector's Daily Reports (IDR) and/or Contractor's Daily Reports (CDR)	Implement and monitor quality assurance program Authorize opening to traffic Direct demobilization of manpower, equipment, and supplies	Implement procedures to update the PMS, MMS, and cost accounting documents Process payments, overruns, claims, and approve project acceptance

Joint Seal Replacement

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Project Selection		Identify jointed concrete pavement candidates where joint seals have deteriorated sufficiently to warrant replacement. Define extent of joint spalling and seal deterioration and project limits for work	(Agency) Select projects from submitted candidate pavements for current year or future year concrete joint resealing program consistent with strategic plan (Agency) Scope project for project design and cost estimates, including the review and approval of specifications and traffic control plan (Agency) Prepare bid package Establish proposed work schedule	(Agency) Direct the programming of project for funding (Agency) Develop guidance documents and directives consistent with strategic plan for concrete joint resealing program (Contractor) Review project site to estimate quantities
Materials	Receive shipped materials, identify characteristics, and verify shelf life for each type of material used to reseal concrete joints Inventory quality of abrasives for joint cleaning Prepare grout and concrete mixes for spall repairs (if necessary) Receive sealant shipping documents/certifications/MSDS for field engineer Maintain materials storage area	Inspect sealant material and primer (if used) for conformance with specifications Inspect backer rods for conformance with specification Inspect grout and concrete material for conformance with specifications (if necessary for patching prior to joint seal replacement)	Estimate sealant, backer rod, and other material quantities as needed for project and verify the materials are on-site prior to starting work	Select appropriate joint sealant from approved source: e.g., neoprene, hot-poured asphalt, silicone, and so on for site conditions Verify appropriate material specification is used in bid package Select approved material supplier prior to bidding (if applicable), receive price quotes, and arrange delivery schedule and sign contract agreement Verify compliance of safety and environmental regulations in material staging area

Joint Seal Replacement (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Equipment	Assemble required hand tools of correct sizes prior to starting work Assemble safety equipment and protective wear for project Inventory quantity and condition of traffic control devices to comply with specifications and traffic control plan	Inspect equipment, pumps, hoses, etc. for plugs, leaks, or damaged components that can cause disruption to project Inspect testing equipment to insure it is properly working and in good condition Calibrate and check thermometers for accuracy material temperature control Maintain sufficient replacement blades for sawing joint face Check abrasive cleaning unit and air compressor for air dryer and oil trap and inspect for contaminant-free operation	Verify project has necessary equipment on site prior to starting work, including equipment such as vacuums required for air quality compliance Verify fuel supplies are readily available and convenient for equipment (saw, compressor, kettle, and so on) Verify weather radar is available or accessible to monitor potential adverse weather conditions	Verify equipment is in compliance with OSHA regulations and state air quality plans
Joint Preparation	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or as otherwise specified Deploy protective wear, including respirators for abrasive or water blasting, sawing, and so on Remove debris, clean joints, and repair spalls as needed (this may involve operating abrasive blasting nozzles, compressed air wands, small mixers, and so on)	Inspect traffic control setup Verify traffic flow and pedestrians are away and protected from potential flying debris Direct necessary preparatory work, such as remove and clean old sealant from joint, repair spalls, and so on Inspect spall repair and joint cleaning procedures and techniques Monitor equipment operation used to clean concrete joints	Initiate public relations outreach to notify residents of pending work Delineate areas by marking (painting) pavement spall repair and joint resealing locations consistent with plans Conduct pre-construction meeting with crew to discuss all project elements, including safety and traffic control Sequence relocating work zones to coordinate a moving repair operation with planned traffic flow	Coordinate scheduling of project to avoid conflicts with major public events or unrelated work

Joint Seal Replacement (continued)

Disciplines	Competencies by Skill Level (Agency Staff and Contractor Personnel)			
	Level I (New Inspector)	**Level II (Inspector)**	**Level III (Resident Engineer)**	**Level IV (Maintenance Director)**
Application	Deploy traffic control devices consistent with traffic control plan Maintain work zone safety with traffic flagging operation, pilot car, or as otherwise specified Clean joint, install backer rod or a primer (as determined necessary) and seal the joint	Direct joint cleaning and sealing operation Inspect joint preparation to insure very clean and dry joint prior to sealing Troubleshoot potential problems	Check weather forecast and radar to insure suitable condition for joint sealing Sequence relocating work zones to coordinate a moving repair operation with planned traffic flow Troubleshoot potential problems	Troubleshoot potential problems Review project after completion to insure joint debris or materials are removed from site
Project Management	Record work hours on project	Maintain detailed Inspector's Daily Reports (IDR) and/or Contractor's Daily Reports (CDR)	Implement and monitor quality assurance program Authorize opening to traffic Direct demobilization of manpower, equipment and supplies	Implement procedures to update the PMS, MMS, and cost accounting documents Process payments, overruns, claims, and approve project acceptance

Summary of courses cataloged herein:

COURSE	SPONSOR
Portland Cement Concrete (PCC) Pavement Evaluation and Rehabilitation *Course No. or Ref.*: FHWA-NHI-131062	FHWA/NHI
Hot-Mix Asphalt (HMA) Pavement Evaluation and Rehabilitation *Course No. or Ref.*: FHWA-NHI-131063	
TCCC Pavement Preservation: Design and Construction of Quality Preventive Maintenance Treatments *Course No. or Ref.*: FHWA-NHI-131103A, -131103B, -131103C	
TCCC Pavement Preservation: Integrating Pavement Preservation Practices and Pavement Management Course No. or Ref.: FHWA-NHI-131104	
TCCC Pavement Preservation Treatment Construction Course No. or Ref.: FHWA-NHI-131110	
Pavement Preservation: Optimal Timing of Pavement Preservation Treatments Course No. or Ref.: FHWA-NHI-131114	
Pavement Preservation: Preventive Maintenance Treatment, Timing, and Selection Course No. or Ref.: FHWA-NHI-131115	
TCCC Basic Materials for Highway and Structure Construction and Maintenance Course No. or Ref.: FHWA-NHI-131117	
NCPP Chip Seals Best Practices Course No. or Ref.: N/A	NCPP
NCPP Slurry Seal and Micro-Surfacing Course No. or Ref.: N/A	
NCPP Pavement Preservation: Applied Asset Management Course No. or Ref.: N/A	
Maintenance and Rehabilitation Course No. or Ref.: N/A	University of California Berkeley
California Pavement Preservation Conference Course No. or Ref.: IDM-50	
Pavement Preservation: The Preventive Maintenance Concept Course No. or Ref.: N/A	Mid-America Transportation Center, Kansas State University
Bituminous Pavements: Thin Surface Treatments Course No. or Ref.: N/A	
Asphalt Pavement Preservation and Rehabilitation Course No. or Ref.: N/A	
Asphalt Emulsions Technologies Workshop – User Focus Course No. or Ref.: N/A	AEMA
The Black and White of Pavement #2: Preservation Techniques Course No. or Ref.: N/A	APWA
Slurry Systems Workshop Course No. or Ref.: N/A	ISSA
HMA Pavement Evaluation, Preservation, and Rehabilitation Course No. or Ref.: N/A	Florida T^2 Center
Asphalt Pavement Maintenance Course No. or Ref.: N/A	
Asphalt Combo – Inspection and Maintenance Course No. or Ref.: N/A	
Asphalt Pavement Maintenance and Preservation (CTAP) Course No. or Ref.: N/A	Minnesota LTAP
Seal Coat Operations: A Workshop for Practitioners Course No. or Ref.: N/A	
Asphalt Roads Common Maintenance Problems Course No. or Ref.: RS-M03	Pennsylvania LTAP

Asphalt Institute Conference - Essentials of Asphalt Paving and Maintenance Course No. or Ref.: W-6	Asphalt Institute and National Pavement Expo
Iowa Concrete Pavement Preservation Workshop Course No. or Ref.: N/A	NCPTC/Iowa DOT
ACPA Webinar: Strategy Selection for Concrete Pavement Rehabilitation Course No. or Ref.: N/A	ACPA
ACPA Webinar: Diamond Grinding Course No. or Ref.: N/A	

Portland Cement Concrete (PCC) Pavement Evaluation and Rehabilitation *Course No. or Ref.*: FHWA-NHI-131062			**Sponsor:** FHWA/NHI
Description: This course presents state-of-the-practice and state-of-the-art techniques to identify the causes and patterns of different types of pavement distress, and techniques for rehabilitation selection, design, and construction that can be applied for those various types of distress.			
Objective: Upon completion of the course, participants will be able to: ▪ Describe the typical behavior and performance of PCC pavements. ▪ Identify common PCC pavement distress types and be able to describe their mechanisms. ▪ Describe key components of a thorough project-level evaluation. ▪ Describe the variety of rehabilitation techniques available for PCC pavements. ▪ Identify feasible rehabilitation techniques for existing PCC pavements. ▪ Describe a process for selecting the preferred rehabilitation alternative for a given pavement.			
Target Audience: FHWA, State, and local highway engineers in design, construction, and maintenance who are involved in the application of pavement rehabilitation techniques.			**Duration:** 2.5 days **Fee:** $355/participant **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials: Participant workbook			**Last Update:**
Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx			
Suitability		**Additional Information:**	
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)	N
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)	Y
Comments:			

Hot-Mix Asphalt (HMA) Pavement Evaluation and Rehabilitation *Course No. or Ref.*: FHWA-NHI-131063	**Sponsor:** FHWA/NHI

Description:
The course presents state-of-the-practice and state-of-the-art techniques to identify the causes and patterns of different types of pavement distress, and techniques for rehabilitation selection, design, and construction that can be applied to those various types of distress.

Objective:
Upon completion of the course, participants will be able to:
- Describe the typical behavior and performance of HMA pavements.
- Identify common type of HMA pavement distress and be able to describe their mechanisms.
- Describe key components of a thorough project-level evaluation.
- Describe the variety of rehabilitation techniques available and state their deficiencies.
- Identify feasible rehabilitation techniques for HMA pavements exhibiting different distresses and conditions.
- Describe the process for selecting the preferred rehabilitation alternative.

Target Audience: FHWA, State, and local highway engineers in design, construction, and maintenance who are involved in the application of pavement rehabilitation techniques.	**Duration:** 2.5 days **Fee:** $355/participant **Prerequisite(s):** None **Type of Training:** Instructor led
Availability of Course Materials: Participant workbook	**Last Update:**

Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx

Suitability		**Additional Information:**	
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)	N
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)	Y

Comments:

TCCC Pavement Preservation: Design and Construction of Quality Preventive Maintenance Treatments *Course No. or Ref.*: FHWA-NHI-131103A	**Sponsor:** FHWA/NHI

Description:
Developed in partnership with the Transportation Curriculum Coordination Council (TCCC), this course contains modules on all of the categories of preventive maintenance treatments in widespread use today, focusing on design and construction best practices. This course also addresses troubleshooting construction practices, enabling participants to identify the results of poor construction.

This is the 2-day version of this variable-length course, and it consists of Modules 1 and 4, with limited content from Modules 2 and 3 selected by the host organization.

Module 1: Introduction to Preventive Maintenance.

Module 2: Crack Filling and Sealing; Fog Seals, Sand Seals, Scrub Seals, and Rejuvenators; Slurry Seals and Microsurfacing; Chip Seals; In-Place Recycling; Thin and Ultra-Thin HMA Overlays.

Module 3: Joint Resealing and Crack Sealing; Diamond Grinding and Grooving; Full-Depth Repairs; Partial-Depth Repairs; Load Transfer Restoration; Thin PCC Overlays; Undersealing.

Module 4: Course Summary.

Objective:
Upon completion of the course, participants will be able to:
- Describe benefits provided by preventive maintenance treatments.
- Describe critical design factors for preventive maintenance techniques.
- Describe recommended procedures for constructing preventive maintenance techniques.
- Identify critical post-construction/pre-opening inspection objectives.

Target Audience: Construction foremen and highway agency inspectors, up to and including mid-level managers. While the course is aimed at those familiar with the equipment and materials used to construct preventive maintenance treatments, it should also be valuable to those just starting in the maintenance field. Also recommended for asset management team members.	**Duration:** 2 days **Fee:** $320/participant **Prerequisite(s):** None **Type of Training:** Instructor led
Availability of Course Materials: Participant workbook	**Last Update:**

Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx

Suitability		**Additional Information:**	
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)	N
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)	Y

Comments:
The course instructor will assist the host organization in selecting the most appropriate topics for the audience anticipated.

TCCC Pavement Preservation: Design and Construction of Quality Preventive Maintenance Treatments *Course No. or Ref.*: FHWA-NHI-131103B	**Sponsor:** FHWA/NHI

Description:
Developed in partnership with the Transportation Curriculum Coordination Council (TCCC), this course contains modules on all of the categories of preventive maintenance treatments in widespread use today, focusing on design and construction best practices. This course also addresses troubleshooting construction practices, enabling participants to identify the results of poor construction.

This is the 3-day version of this variable-length course, and it consists of content from topics in Modules 2 and 3 selected by the host organization. The length of the course will be determined by the number of topics chosen for discussion.

Module 1: Introduction to Preventive Maintenance.

Module 2: Crack Filling and Sealing; Fog Seals, Sand Seals, Scrub Seals, and Rejuvenators; Slurry Seals and Microsurfacing; Chip Seals; In-Place Recycling; Thin and Ultra-Thin HMA Overlays.

Module 3: Joint Resealing and Crack Sealing; Diamond Grinding and Grooving; Full-Depth Repairs; Partial-Depth Repairs; Load Transfer Restoration; Thin PCC Overlays; Undersealing.

Module 4: Course Summary.

Objective:
Upon completion of the course, participants will be able to:

- Describe benefits provided by preventive maintenance treatments.
- Describe critical design factors for preventive maintenance techniques.
- Describe recommended procedures for constructing preventive maintenance techniques.
- Identify critical post-construction/pre-opening inspection objectives.

Target Audience: Construction foremen and highway agency inspectors, up to and including mid-level managers. While the course is aimed at those familiar with the equipment and materials used to construct preventive maintenance treatments, it should also be valuable to those just starting in the maintenance field. Also recommended for asset management team members.	**Duration:** 3 days **Fee:** $620/participant **Prerequisite(s):** None **Type of Training:** Instructor led
Availability of Course Materials: Participant workbook	**Last Update:**

Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx

Suitability		**Additional Information:**	
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)	N
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)	Y

Comments:
The course instructor will assist the host organization in selecting the most appropriate topics for the audience anticipated.

TCCC Pavement Preservation: Design and Construction of Quality Preventive Maintenance Treatments *Course No. or Ref.*: FHWA-NHI-131103C	**Sponsor:** FHWA/NHI

Description:
Developed in partnership with the Transportation Curriculum Coordination Council (TCCC), this course contains modules on all of the categories of preventive maintenance treatments in widespread use today, focusing on design and construction best practices. This course also addresses troubleshooting construction practices, enabling participants to identify the results of poor construction.

This is the 4-day version of this variable-length course, and it consists of all 4 modules.

Module 1: Introduction to Preventive Maintenance.

Module 2: Crack Filling and Sealing; Fog Seals, Sand Seals, Scrub Seals, and Rejuvenators; Slurry Seals and Microsurfacing; Chip Seals; In-Place Recycling; Thin and Ultra-Thin HMA Overlays.

Module 3: Joint Resealing and Crack Sealing; Diamond Grinding and Grooving; Full-Depth Repairs; Partial-Depth Repairs; Load Transfer Restoration; Thin PCC Overlays; Undersealing.

Module 4: Course Summary.

Objective:
Upon completion of the course, participants will be able to:
- Describe benefits provided by preventive maintenance treatments.
- Describe critical design factors for preventive maintenance techniques.
- Describe recommended procedures for constructing preventive maintenance techniques.
- Identify critical post-construction/pre-opening inspection objectives.

Target Audience: Construction foremen and highway agency inspectors, up to and including mid-level managers. While the course is aimed at those familiar with the equipment and materials used to construct preventive maintenance treatments, it should also be valuable to those just starting in the maintenance field. Also recommended for asset management team members.	**Duration:** 4 days **Fee:** $750/participant **Prerequisite(s):** None **Type of Training:** Instructor led
Availability of Course Materials: Participant workbook	**Last Update:**

Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx

Suitability		**Additional Information:**	
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)	N
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)	Y

Comments:

TCCC Pavement Preservation: Integrating Pavement Preservation Practices and Pavement Management *Course No. or Ref.*: FHWA-NHI-131104	**Sponsor:** FHWA/NHI

Description:
Optimum value from preventive maintenance is only obtained when its activities are fully linked to a pavement management system. There are many opportunities for integration, from identifying/tracking benefits of different treatments/timings to developing management models incorporating effects of maintenance. Using management data for network-level analysis can develop an effective strategy that utilizes reconstruction, rehabilitation, and preventive maintenance. Pavement management used at the project level can assist selecting the best preservation option.

Developed in partnership with the Transportation Curriculum Council (TCCC), this course is intended to communicate the importance of integrating pavement preservation activities into pavement management. Presently, many pavement management systems identify the "worst" case pavements, which typically have condition ratings far below those intended to be addressed by pavement preservation. This course addresses integrating pavement preservation with pavement management in a logical sequence, identifying the necessary steps and the process in which:

1. Management tools are adjusted to support a pavement preservation program.
2. Pavement preservation activities are integrated into "enhanced" pavement management models.
3. Use of these "enhanced" pavement management models support decisions at the project, network, and systems levels.

Objective:
Upon completion of the course, participants will be able to:

- Name ways pavement management tools can support a pavement preservation program at the project, network, and strategic analysis levels.
- List reasons it is important to integrate pavement preservation into pavement management.
- Name ways that pavement preservation techniques can be integrated into pavement management.
- Name some common obstacles to successful integration and strategies for overcoming them.

Target Audience: This course is primarily intended for pavement management engineers, district (or regional) maintenance engineers, local agency engineers, maintenance management engineers, and planning and programming personnel, as well as asset management team members.	**Duration:** 2 days **Fee:** $320/participant **Prerequisite(s):** None **Type of Training:** Instructor led
Availability of Course Materials: Participant workbook	**Last Update:**

Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx

Suitability		**Additional Information:**	
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)	N
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)	Y

Comments:

TCCC Pavement Preservation Treatment Construction *Course No. or Ref.*: FHWA-NHI-131110	**Sponsor:** FHWA/NHI

Description:
The Federal Highway Administration (FHWA), in partnership with Caltrans, the National Center for Pavement Preservation (NCPP), and the Transportation Curriculum Coordination Council (TCCC), created the Pavement Preservation Treatment Construction Guide (PPTCG) as a resource for agency and industry pavement preservation practitioners. The guide covers basic pavement preservation concepts, as well as information on specific treatments to extend the life of asphalt pavements. This course provides an introduction to the PPTCG, so that participants can better use it to become familiar with general information on pavement preservation concepts and techniques.

Objective:
Upon completion of the course, participants will be able to:
- Identify the components and value of a Pavement Preventive Maintenance program.
- Identify pavement conditions and other attributes that suggest whether preventive maintenance is appropriate.
- Identify various pavement preservation strategies, techniques, and materials.
- State the performance characteristics of various pavement preservation strategies, techniques, and materials.
- Select the appropriate strategy(ies), technique(s), and material(s) to extend service life and retard development of pavement distress.

Target Audience:	
The training course is primarily targeted at individuals unfamiliar with pavement preservation policy and technical information. The primary audience is Federal, State, and local highway construction and maintenance teams, specifically highway workers and inspectors involved in the placement of pavement preservation treatments. Design engineers may also benefit.	**Duration:** 6.5 hours **Fee:** $0/participant **Prerequisite(s):** none **Type of Training:** Web-based (asynchronous)
Availability of Course Materials: Online (w/ registration)	**Last Update:**

Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx

Suitability		**Additional Information:**	
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)	N
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)	?

Comments:

<table>
<tr><td colspan="2">Pavement Preservation: Optimal Timing of Pavement Preservation Treatments
Course No. or Ref.: FHWA-NHI-131114</td><td colspan="2">Sponsor:
FHWA/NHI</td></tr>
<tr><td colspan="4">Description:
Many agencies perform pavement preservation or preventive maintenance. An important issue facing such programs is identifying the best time to apply a preventive maintenance (PM) treatment. Applying PM treatments too soon or too late is not effective from a cost, performance, or managerial view. This 4-hour seminar explores some of the work performed under NCHRP Project 14-14, including the OPTime software tool that was developed as part of that project. Topics include defining goals and characteristics of good pavement preservation programs, collecting treatment performance relationship data, and key cost and benefit considerations. This course is taught online in a virtual classroom by a lead author of the report, and includes access to the NCHRP 523 report and a download of the OPTime software add-on.</td></tr>
<tr><td colspan="4">Objective:
Upon completion of the course, participants will be able to:
▪ Describe the methodology for determining the optimal timing of preventive maintenance treatments.
▪ List the factors governing optimal timing of treatments.
▪ Apply the methodology to their agency's pavement preservation program.
▪ Describe the effect of timing on pavement preservation treatment performance and program effect.</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
The target audience includes upper- and mid-level highway agency professionals responsible for pavement preservation and management. Responsibilities may include: choosing rehabilitation, reconstruction, and preservation treatments for highways; determining which projects to schedule; deciding which projects fall under a particular program; allotment of funds; and scheduling.

Finally, possible attendees include those who are currently using preservation methods, but may not be using the entire "toolbox" of techniques.</td><td colspan="2">Duration:
4 hours</td></tr>
<tr><td colspan="2">Fee:
$100/participant</td></tr>
<tr><td colspan="2">Prerequisite(s):
none

Recommended: Participants have completed high school, or obtained a GED, read at a 12th grade level, and have basic knowledge of pavement maintenance, pavement preservation, and how they differ.</td></tr>
<tr><td colspan="2">Type of Training:
Web-based (synchronous)</td></tr>
<tr><td colspan="2">Availability of Course Materials: Online (w/ registration): NCHRP Report 523, OPTime, Optime User guide</td><td colspan="2">Last Update: 2008</td></tr>
<tr><td colspan="4">Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)

As a TCCC resource? (Y/N)</td><td>
Y

Y</td><td>Additional Information:
Certification awarded? (Y/N)

End-of-course assessment? (Y/N)</td><td>
N

N</td></tr>
<tr><td colspan="4">Comments:</td></tr>
</table>

<table>
<tr><td colspan="2">Pavement Preservation: Preventive Maintenance Treatment, Timing, and Selection
Course No. or Ref.: FHWA-NHI-131115</td><td colspan="2">Sponsor:
FHWA/NHI</td></tr>
<tr><td colspan="4">Description:
The purpose of this course is to improve the skills of those involved in implementing pavement preservation programs. This includes improving the selection of pavement preventive maintenance projects and the selection of preventive maintenance treatments.</td></tr>
<tr><td colspan="4">Objective:
Upon completion of the course, participants will be able to:
▪ Describe the different types of pavements and how they perform in response to traffic and environmental loading.
▪ Identify concepts of a preventive maintenance program and its role in pavement management.
▪ Identify pavement conditions and other attributes that indicate whether preventive maintenance is appropriate.
▪ Describe preventive maintenance treatments and materials.
▪ Determine when is the most appropriate time during the life of a pavement to apply a preventive maintenance treatment.
▪ Select the most appropriate (or "best") preventive maintenance treatment for a given pavement based on a combination of timing, anticipated benefits, economic considerations, and other key factors.</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
The target audience for this course is mid- or upper-level highway agency professionals responsible for pavement preservation/ maintenance and management; responsibilities may include: evaluating pavements, selecting pavements and treatments for preservation projects, and/or making budget determinations for pavement preservation projects (preservation vs. reconstruction).

At state highway agencies this might include: Pavement Engineers, Project Managers, Maintenance Engineers, Materials Lab personnel, Planning/Programming staff, Pavement Management Engineers/ Managers, Road Superintendents, and Region Directors.

For local agencies, this might include: public works directors or chief engineers of cities, towns, counties, and metropolitan planning organizations (MPOs).</td><td colspan="2">Duration:
2 days</td></tr>
<tr><td colspan="2">Fee:
$320/participant</td></tr>
<tr><td colspan="2">Prerequisite(s):
none

Recommended: basic understanding of information in a pavement condition report, ability to interpret the results of a pavement condition report, ability to visually identify deterioration/distress and determine causes.</td></tr>
<tr><td colspan="2">Type of Training:
Instructor led</td></tr>
<tr><td colspan="2">Availability of Course Materials:</td><td colspan="2">Last Update: 2008</td></tr>
<tr><td colspan="4">Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx</td></tr>
<tr><td colspan="2">Suitability</td><td colspan="2">Additional Information:</td></tr>
<tr><td>For inspectors and technicians? (Y/N)</td><td>Y</td><td>Certification awarded? (Y/N)</td><td>N</td></tr>
<tr><td>As a TCCC resource? (Y/N)</td><td>Y</td><td>End-of-course assessment? (Y/N)</td><td>Y</td></tr>
<tr><td colspan="4">Comments:</td></tr>
</table>

TCCC Basic Materials for Highway and Structure Construction and Maintenance *Course No. or Ref.*: FHWA-NHI-131117	**Sponsor:** FHWA/NHI

Description:
This training was developed by the Transportation Curriculum Coordination Council (TCCC) in partnership with NHI to review basic materials for highway and structure construction and maintenance. The training was prepared by State DOT personnel for State DOT personnel. It contains good practices from various agencies; each State agency/company has its own specifications, which the participant needs to review and follow.

This course is focused on the three basic materials used in construction and maintenance for both highways and structures: Aggregate, Portland Cement Concrete (PCC), and Hot Mix Asphalt (HMA). Course modules will address procedures used in production and sampling.

Module 1, "Basic Aggregates," includes quarry inspection, sand operation, stockpiling, and sampling. Module 2 covers Portland cement, including the production of Portland cement, the hydration process, as well as other cementing materials used in PCC, such as water, admixtures, and aggregates. Module 3 reviews HMA, including the asphalt binder and aggregates used in production.

Objective:
Upon completion of the course, participants will be able to:

- Identify aggregate production and sampling procedures.
- Recognize the ingredients of PCC and the part each plays in concrete production.
- Recognize the ingredients of HMA and the part each plays in hot mix asphalt production.

Target Audience:	
This training is designed for entry- and intermediate-level state/local public agency personnel, and their industry counterparts, involved in construction, maintenance, and testing for highways and structures. Entry-level personnel are those with little to no experience in the subject area and perform tasks under direct supervision. Intermediate-level personnel are those that understand and demonstrate skills in one or more areas of Level I and perform tasks under general supervision.	**Duration:** 4 hours **Fee:** $0/participant **Prerequisite(s):** None **Type of Training:** Web-based
Availability of Course Materials: Online (w/ registration)	**Last Update:**

Contact: NHI Scheduler (703) 235-0534, http://www.nhi.fhwa.dot.gov/training/train_catalog.aspx

Suitability		**Additional Information:**	
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)	N
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)	?

Comments:

NCPP Chip Seals Best Practices *Course No. or Ref.*: N/A		**Sponsor:** NCPP

Description:
The National Center for Pavement Preservation (NCPP) Chip Seal course is intended to provide a thorough and comprehensive understanding of chip seal design, construction, equipment, and inspection best practices. As a preventive maintenance treatment, chip seals offer superior pavement distress mitigation; retard the downward action of water, snow and ice; and provide improved skid resistance when applied to the right roadway. Because the primary causes of chip seal failure are poor construction practices and inappropriate roadway candidate selection, this course is designed to provide the essential skills needed to ensure success.

Content:
Introduction—The Preventive Maintenance Concept. Chip Seal Terminology. Basic Concepts. Engineering Principles.
Chip Seal Design—Chip Seal Programming. Design Methods. Types of Chip Seals. Traffic and Climate Considerations. Selecting the Right Seal.
Contract Administration—Contract Types. Contract Management. Risks and Warranties.
Materials Selection—Aggregate Selection. Binder Selection. Aggregate-Binder Compatibility. Geotextile and Reinforced Seals. Specifications and Best Practices.
Equipment Practices—Asphalt Binder Distributor. Aggregate Spreader. Dump Trucks and Haulers. Rollers. Sweeping Equipment. Specialty Equipment.
Construction Practices—Weather. Road Preparation. Spraying Operations. Aggregate Spreading. Rolling and Sweeping. Traffic Control. QA/QC Practices. Lab Design and Material Testing. Field Testing.
Chip Seal Performance Measures—Engineering-Based Performance Measures. Qualitative Performance Indicators.

Target Audience:	
The course has been geared for a broad audience, and is appropriate for anyone involved in chip seal design, construction, or inspection, including engineers, foremen, superintendents, designers, inspectors, and equipment operators. The material is well-suited for state and local transportation personnel, as well as consultants and contractors.	**Duration:** 1 day **Fee:** $100 – $200/participant (typical) **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials: Course notebook	**Last Update:**

Contact: (517) 432-8220, ncpp@egr.msu.edu, http://www.pavementpreservation.org

Suitability		**Additional Information:**
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)
As a TCCC resource? (Y/N)	Y	*End-of-course assessment?* (Y/N)

Comments:
The course is designed to provide intensified training with an emphasis on practicality. Instructors are experienced chip seal practitioners with extensive backgrounds working with state and local transportation agencies. Participants will receive a notebook containing all course materials.

<table>
<tr><td colspan="2">NCPP Slurry Seal and Micro-Surfacing
Course No. or Ref.: N/A</td><td colspan="2">Sponsor:
NCPP</td></tr>
<tr><td colspan="4">Description:
This one-day course is intended to provide a comprehensive understanding of slurry seal and micro-surfacing systems: selecting good candidate pavements, designing and estimating projects, and gaining awareness of good construction practices.</td></tr>
<tr><td colspan="4">Content:
Introduction—History of and differences between slurry seals and micro-surfacing. Typical Specification Requirements. Project Selection. Also: The Pavement Preservation Concept and associated advantages with respect to improved performance and lower overall costs.
Materials—Primary Material Components. Asphalt Emulsions (including use of polymer-modifiers and other additives). Aggregate Gradations, Types, and Chemical and Physical Properties. Importance of Water Quality. Role of Portland Cement, Fly Ash, and Other Setting Additives.
Design—Material Availability Issues. Mixing Proportions. Test Requirements. Unique Mix Design Requirements (e.g. wet cohesion and Schulze-Breuer). Field Sampling and Testing. Estimating Material Quantities and Expected Field Production Rates.
Equipment—Placement: continuous-run and truck-mounted. Spreading: spreader boxes, rut-filling boxes, and strike-offs and drags. Support Equipment: feeder trucks, loaders, screeners, and sweepers.
Construction—Practical Best Practices. Including: project pre-planning, traffic control, material stockpile management and handling, surface preparation (crack filling, dig-outs, pavement marker removal, sweeping, and casting protection), appropriate ambient conditions for treatment application, types of applications (tack coats, full-width courses, scratch coats, and rut filling), accelerated project requirements, inspection points, troubleshooting, and post construction evaluation.
Business Processes—Payment methods. Warranties. The Value of Innovations (product enhancements). Project Documentation.</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
This course is suitable for the following groups:
State Highway Agencies, County Road Commissions, Municipal Street Departments, Consultants.
Engineers, Managers, Superintendents, Designers, Estimators, Technicians, and Inspectors.</td><td colspan="2">Duration:
1 day</td></tr>
<tr><td colspan="2">Fee:
$100 – $200/participant (typical)</td></tr>
<tr><td colspan="2">Prerequisite(s):
none</td></tr>
<tr><td colspan="2">Type of Training:
Instructor led</td></tr>
<tr><td colspan="2">Availability of Course Materials: Resource notebook</td><td colspan="2">Last Update:</td></tr>
<tr><td colspan="4">Contact: (517) 432-8220, ncpp@egr.msu.edu, http://www.pavementpreservation.org</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)
As a TCCC resource? (Y/N)</td><td>
Y
Y</td><td colspan="2">Additional Information:
Certification awarded? (Y/N)
End-of-course assessment? (Y/N)</td></tr>
<tr><td colspan="4">Comments:
Course facilitators are experienced in both network- and project-level pavement preservation practices gained in operating agencies and academia. Certificates of completion will be provided to all who complete the class. Participants will receive a resource notebook containing all class materials.</td></tr>
</table>

NCPP Pavement Preservation: Applied Asset Management *Course No. or Ref.*: N/A	**Sponsor:** NCPP
Description: This course provides transportation officials and practitioners with a comprehensive understanding of pavement preservation by identifying efficiencies gained developing and selecting strategies that reduce long-term operating costs and improve safety, pavement condition, and user satisfaction.	
Content: **Preservation Nomenclature**—Discussion of preservation and its components; what preservation means at the network level; investment and management benefits; reasons for undertaking preservation; differences between condition and performance; extended pavement life; distress indices; measures of pavement life; windows of opportunity; and cost effectiveness. **Need for Asset Management**—An introduction to the fundamental problems of managing pavements and the need for a corresponding management tool. The concept of treating pavements as assets; asset management principles and their applicability to manage pavements within networks. **Data Inventories**—Information needed to successfully apply asset management principles to highway networks, introduction to the various types of highway data, why they are important, and how they can be used to effectively manage highway networks. **Distress Identification**—Project-level descriptions of structural and functional pavement problems. **Distress Analysis**—Ways to measure distress, an introduction to various indices, including Remaining Service Life (RSL), and their applicability in various strategies. **Network and Project Level Management—**Differences between management levels, introduction to prioritization and optimization, making tradeoffs, development and application of strategies at the network level, distress causes, treatment timing, pavement preservation, and benefits and costs. **Pavement Preservation Strategy—**A strategic view of highway systems, proactive policies, how to implement a pavement preservation program, the long term perspective with its issues and barriers, and general discussion of various treatment types.	
Target Audience: This course is suitable for the following: ▪ Policy-level administrators and managers, planners, economists, and others interested in highways or streets at the network level. ▪ Engineers, technicians, and specialists with a technical interest in preservation techniques and their application at the project level.	**Duration:** 2 days **Fee:** $250 – $400/participant (typical) **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials: Resource notebook	**Last Update:**
Contact: (517) 432-8220, ncpp@egr.msu.edu, http://www.pavementpreservation.org	
Suitability *For inspectors and technicians?* (Y/N) Y *As a TCCC resource?* (Y/N) Y	**Additional Information:** *Certification awarded?* (Y/N) *End-of-course assessment?* (Y/N)
Comments: Course facilitators are experienced in both network- and project-level pavement preservation practices gained in operating agencies and academia. Certificates of completion will be provided to all who complete the class. All participants will receive a resource notebook containing all class materials.	

Maintenance and Rehabilitation *Course No. or Ref.*: N/A	**Sponsor:** University of California Berkeley (LTAP)

Description:
This course provides descriptions of the activities associated with maintenance and rehabilitation of flexible pavements, including the following:

- Maintenance: crack seals, fog seals, slurry seals, bituminous surface treatments, overlays, and patches.
- Rehabilitation: structural overlays, in-place recycling

Objective:
Upon completion of the course, participants will be able to:

- List and describe the major types of activities associated with flexible pavement maintenance.
- List and describe the two major types of flexible pavement rehabilitation.
- Discuss the four primary design approaches for structural overlays.
- Assess the pros and cons associated with the different options for maintenance and rehabilitation.

Target Audience: Contractor and agency technicians and engineers.	**Duration:** 1 hour **Fee:** $99/participant **Prerequisite(s):** none **Type of Training:** Web-based (asynchronous)
Availability of Course Materials: Online (w/ registration)	**Last Update:**

Contact: Pavia Systems (206) 428-3094, http://training.paviasystems.com/catalog/view.php?id=13

Suitability	**Additional Information:**
For inspectors and technicians? (Y/N)	*Certification awarded?* (Y/N)
As a TCCC resource? (Y/N)	*End-of-course assessment?* (Y/N)

Comments:

California Pavement Preservation Conference *Course No. or Ref.*: IDM-50	**Sponsor:** University of California Berkeley (LTAP)

Description:
A pavement preservation program enables local agencies to maintain roads in better condition for less money over a longer life-span, in addition to improving citizen satisfaction. The Pavement Preservation Conference is designed to inform participants how their roadway systems can benefit from a pavement preservation program and what pavement preservation strategies will work best them. This conference provides an opportunity to meet federal, state, regional, and local agency personnel and industry representatives, discover innovations in pavement preservation, discuss methods used by others to maintain and improve pavement in the region, and learn how to partner and where to go for pavement preservation training and resources.

Objective:
Asphalt Fundamentals—Participants learn how to choose the best materials and applications for different situations and environments. Topics include: sources, production, and types and properties of asphalt materials; key specifications; and effective uses of different types of asphalt materials.
Asphalt Pavement Maintenance—Participants learn how to save time and money by extending pavement service life through proper maintenance and repairs. Topics include: types and causes of pavement problems; maintenance vs. rehabilitation; asphalt materials review and safety considerations; crack sealing; patching; and surface treatment options, including fog seals, chip seals and slurry seals.
Concrete Pavement Maintenance—Participants learn the basics of diamond grinding, crack repair, slab replacement, rapid strength concrete use, joint maintenance, subgrade stabilization, and utility cuts.
Pavement Management Fundamentals—This half-day workshop covers the benefits of a Pavement Management System by sharing success stories, identifying potential roadblocks, discussing ongoing maintenance concerns, and addressing other practical issues.
Pavement Preservation Concepts—Brought to you by the California Pavement Preservation Center, this half-day workshop teaches the fundamentals of pavement preservation and gives an overview needed to better understand the more technical information presented during the conference.

Target Audience: Street supervisors and road foremen, maintenance engineers and managers, construction engineers and inspectors, materials engineers and technicians, and pavement management engineers.	**Duration:** 2 days **Fee:** $175/participant (public) $225/participant (private) **Prerequisite(s):** none **Type of Training:** Conference
Availability of Course Materials:	**Last Update:**

Contact: UC-Berkeley Tech Transfer (510) 665-3466, conferences@techtransfer.berkeley.edu, http://www.techtransfer.berkeley.edu/training/index.php

Suitability		**Additional Information:**
For inspectors and technicians? (Y/N)	Y	*Certification awarded?* (Y/N)
As a TCCC resource? (Y/N)		*End-of-course assessment?* (Y/N)

Comments:

<table>
<tr><td colspan="2">Asphalt Emulsions Technologies Workshop – User Focus
Course No. or Ref.: N/A</td><td>Sponsor:
AEMA</td></tr>
<tr><td colspan="3">Description:
The Asphalt Emulsion Technologies Workshop is two, 2-day workshops, one focusing on the user and the other focusing on the producer.

Road agencies are presently experiencing unprecedented high costs to build, rebuild and maintain their road systems due to increased material costs and ever increasing environmental restrictions being placed on the hot mix industry. Pavement preservation, cold mix technologies, and other cost effective and environmentally friendly road building and maintenance techniques are the future. Asphalt emulsions have been and are increasingly being used as an economical and environmentally friendly road building and maintenance material.

Topics include: emulsions, emulsified prime coats and dust palliatives, micro-surfacing and slurry seals, chips seals and fog seals, cold mix paving, and RAP mix.</td></tr>
<tr><td colspan="3">Objective:
Participants should gain a better understanding of new technologies and how the old tried and true applications will allow them to better maintain and improve road systems in an economical and environmentally friendly manner.</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
AEMA members and non-members who are contractors, manufacturers, engineers, consultants and government agencies.</td><td>Duration:
2 days</td></tr>
<tr><td>Fee:
$500/member
$800/non-member
$150/government agency</td></tr>
<tr><td>Prerequisite(s):
None</td></tr>
<tr><td>Type of Training:
Conference</td></tr>
<tr><td colspan="2">Availability of Course Materials:</td><td>Last Update:</td></tr>
<tr><td colspan="3">Contact: (410) 267-0023, cerone@aema.org, http://www.aema.org</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)

As a TCCC resource? (Y/N)</td><td colspan="2">Additional Information:
Certification awarded? (Y/N)

End-of-course assessment? (Y/N)</td></tr>
<tr><td colspan="3">Comments:</td></tr>
</table>

Slurry Systems Workshop *Course No. or Ref.*: N/A		**Sponsor:** ISSA
Description: This study course offers an informative program on slurry seal, micro-surfacing, chip seals, and crack sealing with "hands-on" operation demonstrations and workshop-type discussions.		
Objective: Qualified professionals will cover topics including: materials and equipment, specifications, hand mixes, calibration, quality control, and inspection. Attendees will also be able to view state-of-the-art slurry, micro-surfacing, chip seals, and crack sealing equipment, independent of the paving demonstrations.		
Target Audience: ISSA members and non-members who are contractors, suppliers, engineers, consultants, as well as government agencies.		**Duration:** 4 days **Fee:** $550/member $1100/non-member $400/government agency **Prerequisite(s):** none **Type of Training:** Conference
Availability of Course Materials: Course notebook and CD-ROM		**Last Update:**
Contact: (410) 267-0023, krissoff@slurry.org, http://www.slurry.org		
Suitability *For inspectors and technicians?* (Y/N) *As a TCCC resource?* (Y/N)	**Additional Information:** *Certification awarded?* (Y/N) N *End-of-course assessment?* (Y/N) N	
Comments: A certificate of achievement will be awarded to all participants at the completion of the workshop.		

<table>
<tr><td colspan="2">Pavement Preservation: The Preventive Maintenance Concept
Course No. or Ref.: N/A</td><td>Sponsor:
Mid-America Transportation Center, Kansas State University</td></tr>
<tr><td colspan="3">Description:
This course is intended for those who are or will be involved in the implementation of preventive maintenance for pavement preservation concepts or the substantial maintenance program of the Kansas Department of Transportation (KDOT). Instruction is provided by a select group of instructors from the industry.</td></tr>
<tr><td colspan="3">Objective:
The following topics are covered:
▪ Theory & Philosophy of Pavement Preservation and Pavement Preventive Maintenance
▪ Benefits & Challenges of Pavement Preservation
▪ Selecting the Right Pavement for Preventive Maintenance
▪ Preventive Maintenance Treatments
▪ Determining Preventive Maintenance Treatment Feasibility
▪ KDOT Preventive Maintenance/Substantial Maintenance Program
▪ KDOT Results and What the Future Holds</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
Administrators, engineers, and maintenance superintendents who make decisions and implement thin surface treatments for the KDOT substantial maintenance program.</td><td>Duration:
1 day</td></tr>
<tr><td>Fee:
$100/participant + $10 for CEUs</td></tr>
<tr><td>Prerequisite(s):
None</td></tr>
<tr><td>Type of Training:
Instructor led</td></tr>
<tr><td colspan="2">Availability of Course Materials:</td><td>Last Update:</td></tr>
<tr><td colspan="3">Contact: training director (785) 532-1576, registration (785) 532-5569, http://www.dce.k-state.edu/conf/pavement/</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)   N
As a TCCC resource? (Y/N)</td><td colspan="2">Additional Information:
Certification awarded? (Y/N)
End-of-course assessment? (Y/N)</td></tr>
<tr><td colspan="3">Comments:</td></tr>
</table>

Bituminous Pavements: Thin Surface Treatments *Course No. or Ref.*: N/A	**Sponsor:** Mid-America Transportation Center, Kansas State University

Description:
This course is intended to educate and certify engineers, maintenance superintendents, inspectors, technicians, and other personnel who will be involved in the implementation of thin surface treatments for pavement preservation or substantial maintenance in Kansas. Instruction is provided by a select group of instructors from the industry.

Objective:
It is envisioned that in the near future successful completion of the course and certification will be required to work on maintenance projects. The following modules are covered:

- Introduction to Thin Surface Treatment and Project Selection
- Micro-surfacing
- Surface Recycling
- Chip Seal
- Ultra-Thin Bonded Asphalt Surface (UBAS)

Target Audience: Engineers, maintenance superintendents, inspectors, technicians and other personnel who will be involved in the implementation of thin surface treatments for pavement preservation or the substantial maintenance program of the Kansas Department of Transportation.	**Duration:** 2 days **Fee:** $150/participant + $10 for CEUs **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials:	**Last Update:**

Contact: training director (785) 532-1576, registration (785) 532-5569, http://www.dce.k-state.edu/conf/pavement/

Suitability	**Additional Information:**	
For inspectors and technicians? (Y/N)	*Certification awarded?* (Y/N)	
As a TCCC resource? (Y/N)	*End-of-course assessment?* (Y/N)	Y

Comments:

<table>
<tr><td colspan="2">Asphalt Pavement Preservation and Rehabilitation
Course No. or Ref.: N/A</td><td>Sponsor:
Mid-America Transportation Center, Kansas State University</td></tr>
<tr><td colspan="3">Description:
This course will assist an engineer in the development of the most reliable and cost-effective rehabilitation alternatives for asphalt pavements.</td></tr>
<tr><td colspan="3">Objective:
The video is broken into two units: pavement management concepts and pavement rehabilitation procedures. The first unit addresses pavement management concepts at the project level which will include an overview of pavement management, pavement structural and condition assessment, distress mechanisms for hot mix asphalt and project evaluation. The second unit provides information on pavement rehabilitation through pavement maintenance techniques, surface rehabilitation procedures, and recycling of asphalt pavements and asphalt overlays.</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:</td><td>Duration:
5 hours</td></tr>
<tr><td>Fee:
$240/individual
$725/organization</td></tr>
<tr><td>Prerequisite(s):
none</td></tr>
<tr><td>Type of Training:
Video (DVD)</td></tr>
<tr><td colspan="2">Availability of Course Materials: Ordered online</td><td>Last Update:</td></tr>
<tr><td colspan="3">Contact: training director (785) 532-1576, registration (785) 532-5569, http://www.dce.k-state.edu/conf/pavement/</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)
As a TCCC resource? (Y/N)</td><td colspan="2">Additional Information:
Certification awarded? (Y/N)
End-of-course assessment? (Y/N)</td></tr>
<tr><td colspan="3">Comments:</td></tr>
</table>

Asphalt Roads Common Maintenance Problems *Course No. or Ref.*: RS-M03		**Sponsor:** Pennsylvania LTAP
Description: This course discusses causes and repair procedures for common problems such as cracking, potholes, rutting, corrugations, etc. The procedures cover materials, equipment and techniques for lasting repairs.		
Objective:		
Target Audience: All municipal employees and officials (including road crews, public works personnel, and so on) responsible for road maintenance and safety in their community.		**Duration:** 4 hours **Fee:** **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials:		**Last Update:**
Contact: registration (785) 532-5569, LTAP@state.pa.us, https://www.dot7.state.pa.us/LTAP/		
Suitability *For inspectors and technicians?* (Y/N) *As a TCCC resource?* (Y/N)	**Additional Information:** *Certification awarded?* (Y/N) *End-of-course assessment?* (Y/N) Y	
Comments:		

<table>
<tr><td colspan="2">The Black and White of Pavement #2: Preservation Techniques
Course No. or Ref.: N/A</td><td>Sponsor:
APWA</td></tr>
<tr><td colspan="3">Description:
This second webcast in the Pavement series (http://www.apwa.net/events/eventdetail.asp?ID=4040) gives an overview of maintenance and preservation techniques to help maintain and preserve asphalt and concrete pavements. Speakers address options for maintaining asphalt using crack, fog, slurry and chip seal; micro-surfacing, asphalt overlay; and hot and cold in-place recycling techniques. While cement maintenance options include joint sealing, diamond grinding, dowel bar retrofit and partial and full depth repair.</td></tr>
<tr><td colspan="3">Objective:
Upon completion of the course, participants will be able to:
▪ Identify and compare hot mix asphalt pavement maintenance options.
▪ Identify and compare portland cement concrete pavement maintenance options.</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:</td><td>Duration:
3 hours</td></tr>
<tr><td>Fee:
$225/site (members)
$275/site (non-members)</td></tr>
<tr><td>Prerequisite(s):
none</td></tr>
<tr><td>Type of Training:
Web-based (synchronous)</td></tr>
<tr><td colspan="2">Availability of Course Materials: Online (w/ registration)</td><td>Last Update:</td></tr>
<tr><td colspan="3">Contact: (800) 848-2792, education@apwa.net, http://www.apwa.net/Education/</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)
As a TCCC resource? (Y/N)</td><td colspan="2">Additional Information:
Certification awarded? (Y/N)
End-of-course assessment? (Y/N)</td></tr>
<tr><td colspan="3">Comments:</td></tr>
</table>

<table>
<tr><td colspan="2">Hot-Mix Asphalt Pavement Evaluation, Preservation, and Rehabilitation
Course No. or Ref.: N/A</td><td>Sponsor:
Florida T2 Center</td></tr>
<tr><td colspan="3">Description:
The course presents state-of-the-practice and state-of-the-art techniques to identify the causes and patterns of different types of pavement distress, as well as techniques for preservation and rehabilitation selection, design, and construction that can be applied to those various types of distress.</td></tr>
<tr><td colspan="3">Objective:
Upon completion of the course, participants will be able to:
▪ Describe typical behavior and performance of HMA pavements.
▪ Identify common types of HMA pavements distress and be able to describe their mechanisms.
▪ Describe key components of a thorough project-level evaluation.
▪ Describe the variety of preservation and rehabilitation techniques and state their deficiencies.
▪ Identify feasible preservation and rehabilitation techniques for HMA pavements exhibiting different distresses and conditions.
▪ Develop the process for selecting the preferred preservation and rehabilitation alternative.</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
Highway engineers and technicians in design, construction, and maintenance who are involved in the application of pavement preservation and rehabilitation techniques.</td><td>Duration:
12 hours</td></tr>
<tr><td>Fee:
$150/participant (public)
$225/participant (private)</td></tr>
<tr><td>Prerequisite(s):
none</td></tr>
<tr><td>Type of Training:
Instructor led</td></tr>
<tr><td colspan="2">Availability of Course Materials:</td><td>Last Update:</td></tr>
<tr><td colspan="3">Contact: (352) 273-1685, http://t2.ce.ufl.edu/workshops.asp</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)
As a TCCC resource? (Y/N)</td><td colspan="2">Additional Information:
Certification awarded? (Y/N)
End-of-course assessment? (Y/N) Y</td></tr>
<tr><td colspan="3">Comments:</td></tr>
</table>

Asphalt Pavement Maintenance Course No. or Ref.: N/A	**Sponsor:** Florida T^2 Center
Description: This workshop focuses on the materials, construction details, and maintenance practices that help extend the service life of asphalt pavements. Videos and sketches will be shown to demonstrate proper and improper maintenance techniques. Ample time will be allowed to address questions on specific maintenance issues.	
Content: The workshop will include discussions on: ▪ Pavement evaluation to determine the types and causes of pavement distress. ▪ Techniques, equipment, and materials for effective crack sealing. ▪ Procedures, equipment, and materials for effective asphalt patching and utility repair. ▪ Procedures, materials, and equipment for quality and cost-effective asphalt surface treatments.	
Target Audience: Highway engineers and technicians in design, construction, and maintenance who are involved in the application of pavement preservation and rehabilitation techniques.	**Duration:** 6 hours **Fee:** $125/participant (public) $210/participant (private) **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials:	**Last Update:**
Contact: (352) 273-1685, http://t2.ce.ufl.edu/workshops.asp	

Suitability	**Additional Information:**	
For inspectors and technicians? (Y/N)	*Certification awarded?* (Y/N)	Y
As a TCCC resource? (Y/N)	*End-of-course assessment?* (Y/N)	Y

Comments:
Participants attending this course are eligible for the Elective Training Certificate Program: Levels II and III (http://t2.ce.ufl.edu/view.asp?view=certprog)

Asphalt Combo – Inspection and Maintenance Course No. or Ref.: N/A	**Sponsor:** Florida T^2 Center
Description: This workshop teaches the basic concepts of asphalt materials and inspection techniques that are key to obtaining high quality work on new construction and resurfacing projects, pointing out common problem areas and how to properly repair them. This workshop focuses on the materials, construction details, and maintenance practices that help extend the service life of asphalt pavements. Videos and sketches will be shown to demonstrate proper and improper maintenance techniques. Ample time will be allowed to address questions on specific maintenance issues.	
Content: The workshop will include discussions on: ▪ Pavement evaluation to determine the types and causes of pavement distress techniques. ▪ Equipment and materials for effective crack sealing procedures. ▪ Equipment and materials for effective asphalt patching and utility repair procedures. ▪ Equipment and materials for quality and cost-effective asphalt surface treatments.	
Target Audience:	**Duration:** 6 hours **Fee:** $175/participant (public) $250/participant (private) **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials:	**Last Update:**
Contact: (352) 273-1685, http://t2.ce.ufl.edu/workshops.asp	
Suitability *For inspectors and technicians?* (Y/N) *As a TCCC resource?* (Y/N)	**Additional Information:** *Certification awarded?* (Y/N) *End-of-course assessment?* (Y/N)
Comments:	

Asphalt Pavement Maintenance and Preservation Course No. or Ref.: N/A	**Sponsor:** Minnesota LTAP

Description:
This workshop is designed to provide an overview of the available technology and tools that make implementing a pavement preservation program feasible. An effective pavement preservation program encompasses a full range of maintenance strategies and rehabilitation treatments, with the goal of enhancing pavement performance (ride, quality, safety, service life, etc.). Pavement preservation takes the maintenance process one step further by carefully choosing and timing pavement maintenance applications to extend the life of the pavement. Additionally, the workshop will introduce some new preventative maintenance technologies.

Content:
The workshop includes discussions on:
- Techniques for asphalt pavement evaluation, including pavement condition rating.
- Selecting the best maintenance strategy; choosing the right treatment at the right time on the right project.
- Overview of the various maintenance treatments and their construction practices: fog seal, chip seal, double chip seal, slurry seal, micro-surfacing, Macro®–Surfacing, bonded thin overlay, thin overlay, new technology or processes.
- Overview of material properties—how they are produced, and proper handling and storage of aggregates, emulsions, cutbacks, and asphalts.
- Review of new *Minnesota Best Practices Handbook on Asphalt Pavement Maintenance*.

Target Audience: Engineers, managers, supervisors, and technicians responsible for asphalt pavement maintenance, design, and construction.	**Duration:** 1 day **Fee:** **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials:	**Last Update:**

Contact: (612) 626-1077, mnltap@umn.edu, http://www.mnltap.umn.edu/Events/Topics.html

Suitability	**Additional Information:**
For inspectors and technicians? (Y/N)	*Certification awarded?* (Y/N)
As a TCCC resource? (Y/N)	*End-of-course assessment?* (Y/N)

Comments:

<table>
<tr><td colspan="2">Asphalt Pavement Maintenance and Preservation (CTAP)
Course No. or Ref.: N/A</td><td colspan="2">Sponsor:
Minnesota LTAP</td></tr>
<tr><td colspan="4">Description:
Complementing the “Asphalt Pavement Maintenance and Preservation” workshop, this CTAP workshop increases participants’ awareness of the various pavement maintenance alternatives available today. It also highlights the benefits of performing preventive maintenance on roadways to extend their service life, improve rideability, and reduce long-term costs. This workshop is continuously supplemented with new information and incorporates best practices from the new Minnesota Best Practices Handbook on Asphalt Pavement Maintenance and Asphalt Pavement Maintenance Field Guide.</td></tr>
<tr><td colspan="4">Content:
The workshop includes discussions on:
▪ Evaluating pavement condition
▪ Crack sealing, filling, and repair
▪ Correct methods for pothole patching
▪ Chip and slurry seals
▪ Micro-surfacing
▪ Seal coats
▪ Spray injection patching
▪ Selecting the right treatment for the distress
▪ New techniques and equipment</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
Street or road superintendents, supervisors, and roadway maintenance workers responsible for asphalt pavement construction or maintenance.</td><td colspan="2">Duration:
4 hours</td></tr>
<tr><td colspan="2">Fee:
$40/participant</td></tr>
<tr><td colspan="2">Prerequisite(s):
none</td></tr>
<tr><td colspan="2">Type of Training:
Instructor led</td></tr>
<tr><td colspan="2">Availability of Course Materials:</td><td colspan="2">Last Update:</td></tr>
<tr><td colspan="4">Contact: (612) 626-1077, mnltap@umn.edu, http://www.mnltap.umn.edu/Events/Topics.html</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)
As a TCCC resource? (Y/N)</td><td colspan="3">Additional Information:
Certification awarded? (Y/N)
End-of-course assessment? (Y/N)</td></tr>
<tr><td colspan="4">Comments:</td></tr>
</table>

<table>
<tr><td colspan="2">Seal Coat Operations: A Workshop for Practitioners
Course No. or Ref.: N/A</td><td colspan="2">Sponsor:
Minnesota LTAP</td></tr>
<tr><td colspan="4">Description:
This workshop will provide attendees with an overview of current best practices for seal-coat operations in Minnesota, including how to design and implement a seal-coat operation.</td></tr>
<tr><td colspan="4">Content:
The workshop includes the following topics:
▪ The updated chip-seal handbook
▪ What a chip seal is
▪ Project selection
▪ Why we design chip seals
▪ Aggregates: more than just stone
▪ Binder: it sticks to your road
▪ Construction methods: more than just driving the equipment
▪ Fog sealing: not just a shot in the dark</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
County or city engineers, or their technical staff, who have responsibility for designing and/or managing seal-coat operations.</td><td colspan="2">Duration:
4 hours</td></tr>
<tr><td colspan="2">Fee:
$50/participant</td></tr>
<tr><td colspan="2">Prerequisite(s):
none</td></tr>
<tr><td colspan="2">Type of Training:
Instructor led</td></tr>
<tr><td colspan="2">Availability of Course Materials:</td><td colspan="2">Last Update:</td></tr>
<tr><td colspan="4">Contact: (612) 626-1077, mnltap@umn.edu, http://www.mnltap.umn.edu/Events/Topics.html</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)

As a TCCC resource? (Y/N)</td><td colspan="3">Additional Information:
Certification awarded? (Y/N)

End-of-course assessment? (Y/N)</td></tr>
<tr><td colspan="4">Comments:</td></tr>
</table>

Asphalt Institute Conference - Essentials of Asphalt Paving and Maintenance Course No. or Ref.: W-6	**Sponsor:** Asphalt Institute and National Pavement Expo
Description: This one-day training conference highlights the fundamentals and "best practices" for producing high-quality, long lasting asphalt pavements. Conference presentations are geared toward smaller paving operations.	
Content: The workshop includes the following topics: ▪ Materials: asphalt binders and emulsions, aggregates, and asphalt mixtures ▪ Construction: mix properties, placement, compaction, and quality control ▪ Maintenance and Surface Preparation: patching, crack sealing, and surface treatments ▪ Practical Troubleshooting: for paving and pavement maintenance	
Target Audience: The conference is intended to provide basic information on asphalt materials, paving and maintenance to prepare NPE seminar and workshop attendees for the more specialized topical training offered in other sessions.	**Duration:** 1 day **Fee:** $200/participant **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials:	**Last Update:**
Contact: Cygnus Expositions (800) 827-8009, http://www.nationalpavementexpo.com/	
Suitability *For inspectors and technicians?* (Y/N) *As a TCCC resource?* (Y/N)	**Additional Information:** *Certification awarded?* (Y/N) *End-of-course assessment?* (Y/N)
Comments:	

Iowa Concrete Pavement Preservation Workshop Course No. or Ref.: N/A		**Sponsor:** National Concrete Pavement Technology Center/Iowa DOT
Description: The course has been prepared to provide guidance on the design, construction, and selection of concrete pavement preservation treatments.		
Content: The workshop includes the following topics: ▪ Pavement preservation concepts ▪ Concrete pavement evaluation ▪ Slab stabilization ▪ Partial-depth repair ▪ Full-depth repair ▪ Retrofitted edge drains ▪ Load transfer restoration ▪ Diamond grinding and grooving ▪ Joint resealing		
Target Audience: DOT engineers, City engineers and managers, County engineers and managers, consulting engineers, and contractors		**Duration:** 2 days **Fee:** $65/participant **Prerequisite(s):** none **Type of Training:** Instructor led
Availability of Course Materials:		**Last Update:**
Contact: CPTech Center (515) 294-0550, http://www.cptechcenter.org/		
Suitability *For inspectors and technicians?* (Y/N) *As a TCCC resource?* (Y/N)	**Additional Information:** *Certification awarded?* (Y/N) *End-of-course assessment?* (Y/N)	
Comments:		

ACPA Webinar: Strategy Selection for Concrete Pavement Rehabilitation Course No. or Ref.: N/A	**Sponsor:** ACPA

Description:
Building on the information presented in, "Evaluating Existing Concrete Pavements for Rehabilitation," this course provides information aimed at selecting a strategy for concrete pavement rehabilitation.

A concrete pavement can be rehabilitated at any point during its specified design life, or after, as is often the case. Repairs, ranging from minor preventive maintenance techniques to structural overlays, must factor in many variables to select the most cost effective, viable technique.

This webinar provides practical information about the factors to consider when selecting rehabilitation options, including both pavement and non-pavement related criteria. The program also will cover methods for determining relative rankings of the alternatives.

The rehabilitation options considered include full-depth repairs, partial-depth repairs, diamond grinding, dowel bar retrofit, slab stabilization and others. The webinar also will describe overlay options, including concrete over existing concrete pavements, asphalt pavements, and composite pavements.

Objective:

Target Audience: Contractors, consultants, engineers, Federal agencies (including FHWA and FAA), Military and Dept. of Defense contractors and Engineers, municipal and county public works officials, Metropolitan Planning Organization (MPO) officials, Academia, et al. Some courses are also ideal for materials engineers, inspectors, and other agency personnel. A few webinars are also well suited for facilities managers, architects, owners' representatives and other asset managers and planning professionals needing to enhance their knowledge of pavements.	**Duration:** 1 hour **Fee:** $65/participant (non-members) $35/participant (members) $25/participant (government employees) **Prerequisite(s):** none **Type of Training:** Instructor led web-based seminar
Availability of Course Materials: Online (w/ registration)	**Last Update:**

Contact: CPTech Center (217) 621-3438, http://www.pavement.com/

Suitability	**Additional Information:**
For inspectors and technicians? (Y/N)	*Certification awarded?* (Y/N)
As a TCCC resource? (Y/N)	*End-of-course assessment?* (Y/N)

Comments:

<table>
<tr><td colspan="2">ACPA Webinar: Diamond Grinding
Course No. or Ref.: N/A</td><td colspan="2">Sponsor:
ACPA</td></tr>
<tr><td colspan="4">Description:
Diamond grinding can be used for correcting surface deficiencies, including roughness, faulting, polishing, noise, friction, splash and spray, among others.

This technique can be used shortly after construction to provide the final riding surface with improved surface characteristics or correct localized roughness. However, the most common use of diamond grinding is to improve the functional performance of older pavements.

This webinar will focus on many aspects diamond grinding including project selection, setting baseline values and expected level of improvement, equipment selection, cutting head and blade configuration, historical performance data and other important elements for a successful project.

Current research focusing on development of the next generation surface texture will also be discussed.</td></tr>
<tr><td colspan="4">Objective:</td></tr>
<tr><td colspan="2" rowspan="4">Target Audience:
Contractors, consultants, engineers, Federal agencies (including FHWA and FAA), Military and Dept. of Defense contractors and Engineers, municipal and county public works officials, Metropolitan Planning Organization (MPO) officials, Academia, et al.

Some courses are also ideal for materials engineers, inspectors, and other agency personnel. A few webinars are also well suited for facilities managers, architects, owners' representatives and other asset managers and planning professionals needing to enhance their knowledge of pavements.</td><td colspan="2">Duration:
1 hour</td></tr>
<tr><td colspan="2">Fee:
$65/participant (non-members)
$35/participant (members)
$25/participant (government employees)</td></tr>
<tr><td colspan="2">Prerequisite(s):
none</td></tr>
<tr><td colspan="2">Type of Training:
Instructor led web-based seminar</td></tr>
<tr><td colspan="2">Availability of Course Materials: Online (w/ registration)</td><td colspan="2">Last Update:</td></tr>
<tr><td colspan="4">Contact: CPTech Center 217-621-3438 (217) 621-3438, http://www.pavement.com/</td></tr>
<tr><td>Suitability
For inspectors and technicians? (Y/N)

As a TCCC resource? (Y/N)</td><td colspan="3">Additional Information:
Certification awarded? (Y/N)

End-of-course assessment? (Y/N)</td></tr>
<tr><td colspan="4">Comments:</td></tr>
</table>

Summary of LTAP Courses

The following courses are those listed with relatively detailed descriptions. Many state LTAP centers note having courses regarding pavement preservation and/or preventive maintenance, as well as construction inspection—typically as part of their Road Scholar program—however, not all are provided descriptions. Some Road Scholar course descriptions are included herein (Kansas, Louisiana, and Texas).

<table>
<tr><th colspan="2">Illinois
http://www.dot.state.il.us/blr/training.asp</th></tr>
<tr><td rowspan="3">COURSE(S)</td><td>Pavement Preservation: Design and Construction of Quality Preventive Maintenance
PURPOSE: To provide an understanding of how the types of treatments and timing of their application extends pavement performance.
TOPICS COVERED: Critical design factors, benefits provided, recommended construction procedures, and critical post-construction inspection objectives.
DURATION: 3 days
FEE: $620/participant</td></tr>
<tr><td>Pavement Maintenance
PURPOSE: To enable recognizing the causes of pavement failure and to make and/or recommend corrective measures alleviating the cause, including selecting the proper materials and methods and documenting the work accomplished. Discuss various types of road surfaces with the emphasis on flexible bases and developing a pavement management system.
TOPICS COVERED: Drainage and subsurface maintenance; patching and resurfacing material; street patching methods; portland cement concrete, utility cuts; seal coats and crack sealing; and developing a systematic approach to pavement maintenance.
DURATION: 1 day</td></tr>
<tr><td>Seal Coats (Oil & Chipping)
PURPOSE: To enable selection of the best type of bituminous material and aggregate for prime, seal, and cover coats. To understand proper construction methods in preparing the surface, placement of bituminous materials and aggregates, and to recognize typical problems.
TOPICS COVERED: Types of bituminous materials and aggregates, proper preparation of the existing surface, proper construction of seal coats, and typical problems.
DURATION: 1/2 day</td></tr>
</table>

<table>
<tr><th colspan="2">Wisconsin
http://tic.engr.wisc.edu/Workshops/Listing.lasso</th></tr>
<tr><td>COURSE(S)</td><td>Road Maintenance (Course #K233)
PURPOSE: To recognize problems early and apply the right method at the right time to maintain good roads cost effectively, focusing on:
• Basics of a good road
• Patching and crack filling done right
• Chip seals and other surface treatments
• Selecting the right maintenance treatment for your road
• Spring maintenance checklist
AUDIENCE: Those responsible for maintaining local streets, roads and highways: elected officials, engineers, superintendents, foremen, equipment operators, et al.</td></tr>
</table>

Missouri http://www.moltap.org/trainings/index.html	
COURSE(S)	**Preventive Pavement Maintenance** PURPOSE: This course studies various methods available to restore roads, including equipment and materials available and how best to apply modern techniques. AUDIENCE: Public Works employees, City and County Road and Bridge Maintenance employees along with Road Supervisors and/or Superintendents. DURATION: 1 day FEE: $25/participant

Iowa http://www.ctre.iastate.edu/ltap/workshop.htm	
COURSE(S)	**Iowa Maintenance Training Expo** DESCRIPTION: With tight transportation budgets and limited staff, all-season maintenance has become especially challenging, but Iowa is committed to satisfying public expectations for safe travel in all kinds of weather and on all roadways. AUDIENCE: City, county, and state staff responsible for maintenance: roadway maintenance supervisors, maintenance equipment operators, technology and equipment providers, public works superintendents, city and county engineers, airport maintenance staff, et al. DURATION: 2 days FEE: $65/participant

Louisiana https://www.ltrc.lsu.edu/ltap/training.html	
COURSE(S)	**Louisiana Road Scholar Program** DESCRIPTION: Consists of fifteen three to six hour road and bridge maintenance training sessions. Each session is offered on several occasions over a three-year period in convenient, centrally located sites. To become a Louisiana Roads Scholar and receive a Louisiana Roads Scholar Certificate, each participant must attend ten of the fifteen training courses approved for the program. Six of the courses are required and nine are elective. The student can pick a minimum of four of the seven elective courses, or if desired, may attend all nine. COURSES OFFERED: **Road Scholar Course #2 - Asphalt Roads: Common Maintenance Problems** This course includes a review of the causes of potholes, rutting, corrugations, alligator cracking, and other common maintenance problems. This session covers proper methods, materials, and equipment which should be used in making lasting repairs. **Road Scholar Course #7 - Seal Coats and Slurry Seals** This course will cover the proper selection and use of seal coats and slurry seals. Participants will learn correct methods of application, as well as practical tips on how to avoid trouble. Materials, equipment, and calibrations are discussed.

<table>
<tr><th colspan="2">Texas
http://teexcit.tamu.edu/texasltap/training.html</th></tr>
<tr><td>COURSE(S)</td><td>Texas Road Scholar Training
DESCRIPTION: Provides training about quality maintenance and management practices.
AUDIENCE: county and municipal road and bridge maintenance personnel, supervisors, directors, and engineers
COURSES OFFERED:
Maintenance Problems on Asphalt Roads: This course includes a review of the causes of potholes, rutting, corrugations, alligator cracking and the appropriate repair procedures for each. Proper repair techniques, materials and equipment used to produce a lasting repair are also covered.
Asphalt Pavements - Preventive Maintenance Using Surface Treatments: Covered in this course are when, where and how to apply chip and slurry seals on existing asphalt roads, including the correct application methods. Also discussed are practical tips on how to avoid trouble, and a discussion on equipment and calculations.</td></tr>
</table>

<table>
<tr><th colspan="2">Oklahoma
http://clgt.okstate.edu/Secondary%20Page%20Layout/roadsscholar.htm (Road Scholar Program)
http://clgt.okstate.edu/Secondary%20Page%20Layout/otherclasses.htm (Other Courses)</th></tr>
<tr><td>COURSE(S)</td><td>Chip Seal Class & Demonstration
This class consists of two parts: 1) classroom instruction detailing the proper procedures to use when doing chip seal operations; 2) demonstration of these methods in the field.</td></tr>
</table>

<table>
<tr><th colspan="2">Kansas
http://www.ksltap.org/ (General information)
http://www.kutc.ku.edu/cgiwrap/kutc/ltap/rs_index.php (Road Scholar Program)</th></tr>
<tr><td>COURSE(S)</td><td>Kansas Road Scholar Program
Level I: Road Scholar
Technical Skills Program
Asphalt Road/Street Maintenance
OBJECTIVE: Provide basic maintenance information on patching, sealing, drainage, etc.; discuss new maintenance and road rehabilitation techniques.
DURATION: 6 hours (minimum)
Concrete Road/Street Maintenance
OBJECTIVE:TBD
DURATION: 6 hours (minimum)</td></tr>
</table>

Wyoming http://wwweng.uwyo.edu/wyt2/index.php	
COURSE(S)	**Chip Seals, Surface Treatments, and Maintenance of Roads of Asphalt Roads** TOPICS COVERED: *Common Maintenance Problems*—This workshop reviews the causes of localized distresses such as potholes, rutting, corrugations, alligator cracking, etc., and the correct repair procedures. This course will cover proper methods, materials, and equipment which should be used in making lasting repairs. *Chip Seals and Surface Treatments*—When and where to apply global treatments such as seal coats and slurry seals will be discussed. Participants will learn correct methods of application, as well as practical tips on how to avoid trouble. Materials, equipment, and calibrations will also be discussed. AUDIENCE: This workshop should be attended by city, county and WYDOT engineers, street maintenance supervisors, construction inspectors, engineering technicians, maintenance crew leaders, and street maintenance workers. DURATION: 1 day FEE: $45/participant

Utah http://www.utahltap.org/Services/Training.php	
COURSE(S)	**Flexible Pavement Preservation** OBJECTIVE: Upon completion of the course the participant should be able to: • Identify asphalt pavement distress types, conduct condition surveys and determine root causes of these distresses. • Determine the appropriate and most cost effective preservation technique to use. • Determine proper timing for applying the various preservation treatments. • Apply effective quality control and construction practice in each treatments' application. AUDIENCE: state and local highway agency design, construction and maintenance engineers; other road personnel involved in the design, construction and maintenance of asphalt roads; and consultant/contractor personnel.

Washington State http://www.wsdot.wa.gov/LocalPrograms/Training/default.htm	
COURSE(S)	**Pavement Preservation: Slurry Seal & Micro-Surfacing** DESCRIPTION: This course provides participants with a comprehensive understanding of slurry seal and micro-surfacing systems, offering the essential skills for selecting good candidate pavements, designing and estimating projects, and gaining awareness of good construction practices. AUDIENCE: State Highway Agencies, County Road Commissions, Municipal Street Departments, Consultants, and others interested in maintaining and preserving good pavement conditions, including: engineers, managers, superintendents, designers, estimators, technicians, and inspectors. DURATION: 1 day FEE: $125/participant
	Modern Chip Seal Techniques TOPICS COVERED: asphalt chemistry, purpose of chip sealing, types of asphalt, aggregate, chip seal design, supervising chip seal crews, equipment, construction, weather conditions, cost management AUDIENCE: Supervisors, lead workers and key employees on chip seal crews. May also be of interest to inspectors on chip seal projects. DURATION: 1 day FEE: $75/participant

Maryland http://www.mdt2center.umd.edu/courses/course-catalog.html	
COURSE(S)	**Preventive Pavement Maintenance** <u>DESCRIPTION</u>: This course covers preventive maintenance treatments such as chip seals, slurry seals, and microsurfacing, discussing when and where each technique can be effective. It presents application methods, including preparation, materials, equipment, operations and safety, along with practical tips to avoid trouble. <u>FEE</u>: $75/Local Government, $95/State Government, $110/Private or Out of State

Ohio http://www.dot.state.oh.us/DIVISIONS/LTAP/Pages/technicalcourseinformation.aspx	
COURSE(S)	**Asphalt Pavement Preservation** <u>DESCRIPTION</u>: This workshop will address: (a) basic performance characteristics of HMA pavements; (b) evaluation of common distress types and their causes; and (c) available maintenance and rehabilitation treatments, including preventive maintenance, such as chip seals, microsurfacing and thin overlays. <u>AUDIENCE</u>: Road/street maintenance personnel, supervisors/managers, engineers and others responsible for maintaining and preserving asphalt pavements. <u>DURATION</u>: 1 day <u>FEE</u>: $55/participant

New Hampshire http://www.t2.unh.edu/training/index.html	
COURSE(S)	**Crack Sealing** <u>OBJECTIVE</u>: To learn how to plan a crack sealing project, prepare the cost estimate, and apply crack sealing material. <u>TOPICS COVERED</u>: • Identifying problems and causes. • Determining possible and best solutions. • Contract specifications and inspection. • Crack preparation and material application. <u>AUDIENCE</u>: Public Works Directors, Road Agents, and other municipal engineers, as well as private engineers that assist municipalities with repair of municipal roads. <u>FORMAT</u>: *In the classroom*—estimating cost and discussion. *In the field*—assess problems and solutions, and demonstrate best practices. <u>DURATION</u>: 1 day <u>FEE</u>: $60/participant

	Pavement Preservation OBJECTIVE: To learn the basics on pavement properties, products, and management. TOPICS COVERED: • Properties of asphalt products: hot and cold mix. • Construction techniques. • Materials testing. • Pavement Management. • Choosing the right treatment at the right time. • Why you don't repair worst first. AUDIENCE: Those involved with repairing roads or highway maintenance, such as: Road Agents and Directors, highway workers, and other municipal officials. FORMAT: Classroom instruction with exercises in solving common problems. DURATION: 1 day FEE: $60/participant

State Highway Agency Manuals and Guidelines

Alaska
Asphalt Surface Treatment Guide, http://www.dot.state.ak.us/stwddes/research/assets/pdf/fhwa_ak_rd_01_03.pdf Cost-Effective Rut Repair Methods, http://www.dot.state.ak.us/stwddes/research/assets/pdf/fhwa_ak_rd_01_04.pdf

Arizona
Construction Manual http://www.dot.state.az.us/Highways/ConstGrp/Construction_Manual/ Chapter 4 - Surface Treatments and Pavements, http://www.dot.state.az.us/Highways/ConstGrp/Construction_Manual/Chapters/PDF/Chapter_4.pdf

California
Maintenance Technical Advisory Guide (MTAG) Volume I - Flexible Pavement Preservation, http://www.dot.ca.gov/hq/maint/MTA_GuideVolume1Flexible.html MTAG Volume I Training Modules, http://www.dot.ca.gov/hq/maint/MTA_GuideVolume1FlexibleTrainingModules.html MTAG Volume 2 - Rigid Pavement Preservation, http://www.dot.ca.gov/hq/maint/MTA_GuideVolume2Ridgid.html MTAG Volume 2 Training Modules, http://www.dot.ca.gov/hq/maint/MTA_GuideVolume2RidgidTrainingModules.html *Other* Fog Seal Guidelines, http://www.dot.ca.gov/hq/maint/TAGFogSealsGuidelines.pdf

Colorado
Development of a Pavement Preventive Maintenance Program for the Colorado DOT, http://www.dot.state.co.us/publications/PDFFiles/preventivemaintenance.pdf

Georgia
Asphalt Pavement Selection Guidelines, http://www.dot.state.ga.us/doingbusiness/Materials/Documents/GeorgiaasphaltpavementresurfacingguideMay06.pdf

Hawaii
Pavement Preventive Maintenance Guidelines, http://state.hi.us/dot/highways/hwy-l/PREVENTIVE_MAINTENANCE_GUIDELINE.doc

Idaho
Maintenance Manual - Section 411 Preventive Maintenance Program http://itd.idaho.gov/manuals/Online_Manuals/Current_Manuals/Maintenance/Mtce_Manual_Section_400.pdf Materials Manual - Section 542 Preventive Maintenance, http://itd.idaho.gov/manuals/Online_Manuals/Current_Manuals/Materials/Sec_542.pdf

Illinois
Policy Memorandum - Guidelines for Pavement Preservation, http://dot.state.il.us/desenv/pdf/pm47_05.pdf Special Provisions, Bituminous Surface Treatment, http://www.dot.il.gov/desenv/pdf/80218.pdf Cape Seal, http://www.dot.il.gov/desenv/pdf/80219.pdf Micro-Surfacing, http://www.dot.il.gov/desenv/pdf/80220.pdf Slurry Seal, http://www.dot.il.gov/desenv/pdf/80221.pdf

Kentucky
Pavement Management Field Book - KYTC Pavement Distress Identification Manual & Guideline for Preventive Maintenance Treatments, http://transportation.ky.gov/Maintenance/PM_Reports/PM_Field_Manual09.pdf

Michigan
Pavement Design and Selection Manual**,** http://www.michigan.gov/documents/mdot/MDOT_Pavement_Design_and_Selection_Manual_257723_7.pdf See Chapter 7, Pavement Preservation Strategies Construction Advisories, http://www.michigan.gov/mdot/0,1607,7-151-9622_11044_36358---,00.html CA-2005-13, Testing Requirements for Capital Preventive Maintenance HMA Projects, http://www.michigan.gov/documents/MDOT_CA-2005-13_129619_7.pdf CA-2008-12, PCC Pavement Joint Sealing, http://www.michigan.gov/documents/mdot/MDOT_CA_2008-12_244023_7.pdf CA-2009-03, HMA Crack Treatment on Roads with Centerline or Shoulder Rumble Strips http://www.michigan.gov/documents/mdot/MDOT_CA_2009-03_268134_7.pdf

Minnesota
State Aid Concrete Pavement Rehabilitation Best Practices Manual, http://www.pavementpreservation.org/toolbox/links/200631.pdf Minnesota Seal Coat Handbook, http://www.wsdot.wa.gov/NR/rdonlyres/4A21ECE8-114B-434D-B967-0927541CE042/0/AsphaltSealCoats.pdf Best Practices Handbook on Asphalt Pavement Maintenance, http://www.mnltap.umn.edu/Publications/Handbooks/documents/asphalt.pdf Asphalt Pavement Maintenance Field Guide, http://www.mnltap.umn.edu/Publications/Handbooks/documents/2002AsphaltPavementMaint.pdf

Missouri
Engineering Policy Guide - Surface Treatments and Preventive Maintenance, http://epg.modot.org/index.php?title=Category:413_Surface_Treatments_and_Preventive_Maintenance Engineering Policy Guide - Portland Cement Concrete Pavement Maintenance, http://epg.modot.mo.gov/index.php?title=Category:570_Portland_Cement_Concrete_Pavement_Maintenance

Montana
Maintenance Manual, http://www.mdt.mt.gov/publications/manuals/maint_manual.shtml Chapter 3 - Asphalt Pavement Program, http://www.mdt.mt.gov/publications/docs/manuals/mmanual/chapt3c.pdf Chapter 4 - Concrete Pavement Program, http://www.mdt.mt.gov/publications/docs/manuals/mmanual/chapt4c.pdf Chip Seal Manual, http://www.mdt.mt.gov/publications/docs/manuals/chipseal.pdf Crack Seal Manual, http://www.mdt.mt.gov/publications/docs/manuals/crackseal.pdf

Nebraska
Pavement Maintenance Manual, http://www.nebraskatransportation.org/docs/pavement.pdf

New York
Comprehensive Pavement Design Manual, https://www.nysdot.gov/portal/page/portal/divisions/engineering/design/dqab/cpdm/ Chapter 10 - Preventive Maintenance, https://www.nysdot.gov/divisions/engineering/design/dqab/cpdm/repository/chapter10.pdf

North Dakota
Chip Seal Coat Manual, http://www.dot.nd.gov/manuals/maintenance/chip-seal.pdf

Ohio
Pavement Preventive Maintenance Program Guidelines, http://www.dot.state.oh.us/Divisions/Planning/Pavement/PM_Guidelines/PM_Guide.pdf

Oregon
Pavement Preservation Project Guidelines for Local Governments http://www.oregon.gov/ODOT/HWY/LGS/docs/Local_Pavement_Preservation_Guidelines.doc

Pennsylvania
Publication 242 - Pavement Design and Analysis, http://www.dot.state.pa.us/Internet/Bureaus/pdBOMO.nsf/Pub242?OpenForm Go to Appendix G - Pavement Preservation Guidelines & NEPP Guidelines

Texas
Maintenance Operations Manual, http://onlinemanuals.txdot.gov/txdotmanuals/ope/ope.pdf Seal Coat and Surface Treatment Manual, ftp://ftp.dot.state.tx.us/pub/txdot-info/gsd/manuals/scm.pdf Texas Pavement Preservation Program, http://www.utexas.edu/research/tppc/pubs/sims.pdf Field Manual for Crack Sealing in Asphalt Pavements, http://www.utexas.edu/research/ctr/pdf_reports/0_4061_P3.pdf

Utah
Technical Bulletins MT-05.07, Selecting and Using Asphalt Pavement Rejuvenating Agents, http://www.udot.utah.gov/main/uconowner.gf?n=200511230851001 MT-04.02, Chip Seal Emulsions, http://www.udot.utah.gov/main/uconowner.gf?n=200511230814561 MT-04.05, Concrete Fast Repair, http://www.udot.utah.gov/main/uconowner.gf?n=200511230816321 MT-03.02, Dowel Bar Retrofit,

http://www.udot.utah.gov/main/uconowner.gf?n=200511230817271

MT-04.01, Scrub Seal Coat,
http://www.udot.utah.gov/main/uconowner.gf?n=200511230849131

MT-03.01, Tire Rubber Modified Hot-Applied Chip Seal Coat,
http://www.udot.utah.gov/main/uconowner.gf?n=200511230855321

MT-02.01, Ultra-Thin Whitetopping,
http://www.udot.utah.gov/main/uconowner.gf?n=200511230857191

MT-05.06, Selecting and Using Asphalt Emulsions,
http://www.udot.utah.gov/main/uconowner.gf?n=200511230852161

Federal Land Management Agencies
Project Development and Design Manual**,** http://www.wfl.fhwa.dot.gov/design/manual/PDDM.zip Chapter 11, Section 11.7 - Pavement Preservation, http://www.wfl.fhwa.dot.gov/design/manual/Chapter_11.pdf#11.7 **Note:** This section of the Design Manual has not yet been developed; in the interim, designers are recommended to refer to the websites of the National Center for Pavement Preservation (www.pavementpreservation.org), CalTrans Maintenance Technical Advisory Guides (www.dot.ca.gov/hq/maint/roadway.htm), and FHWA (www.fhwa.dot.gov/preservation/).

Virginia
Slurry Surfacing Certification Study Guide, http://www.virginiadot.org/business/resources/Materials/SlurryManual_2009.pdf Surface Treatment Certification Study Guide, http://www.virginiadot.org/business/resources/Materials/SurfaceManual_2009.pdf

Washington
Construction Manual, http://www.wsdot.wa.gov/Publications/Manuals/M41-01.htm Chapter 5, Surface Treatments and Pavements, http://www.wsdot.wa.gov/publications/manuals/fulltext/M41-01/Chapter5.pdf Technology Transfer - Asphalt Seal Coats, http://www.wsdot.wa.gov/NR/rdonlyres/4A21ECE8-114B-434D-B967-0927541CE042/0/AsphaltSealCoats.pdf

Attachment 11

TPF logo | Transportation Pooled Fund Program

TPF logo | H | About | Authorized | B | S | Sign Up To Be

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- General Definitions
- Organization Roles
- Person Roles
- Full Glossary

- **General Help:**
- FAQs
- FHWA Division Office Contacts
- FHWA Division Office Planning, Environment, Realty & Research Staff
- FHWA Division Office Financial Staff

TPF Sponsors

TPF Studies>>>Detailed View

Solicitation Number:	1233
Status:	Solicitation posted
Title:	Validation and Implementation of Hot-Poured Crack Sealant
Sponsoring Agency:	Virginia Department of Transportation
Sponsor Solicitation Contact:	Bill Kelsh (Bill.Kelsh@VDOT.Virginia.gov) Phone: 434-293-1934 Fax: 434-293-1990
Technical Contact:	Kevin McGhee (Kevin.McGhee@VirginiaDOT.org) Phone: 434-293-1956 Fax: 434-293-1990
Lead Agency Contact:	Kevin McGhee (Kevin.McGhee@VirginiaDOT.org) Phone: 434-293-1956 Fax: 434-293-1990
Lead Agency:	Virginia Department of Transportation
Partners:	NY, VA
Date Posted:	2/27/2009 9:57:36 AM
Solicitation Expires:	6/30/2009
Commitment Start Year:	2009
Commitment End Year:	2012
Duration:	48 months
100% SP&R Approval:	Pending Approval
Commitments Required:	$1,000,000
Commitments Received:	$200,000
Background:	The developed guidelines were the outcome of the North American Consortium conducted by University of Illinois a Urbana-Champaign and the National Research Council of Canada. Thirteen states, led by VDOT/VTRC, sponsored the pool-fund research project TPF-5(045) along with 13 Canadian transportation agencies and industry. New test methods, which make use of the existing SHRP devices, were developed.
Objectives:	The University of Illinois at Urbana-Champaign, in collaboration with Virginia Transportation Research

- FHWA Federal Highway Administration
- TRB Transportation Research Board
- AASHTO American Association of State Highway and Transportation Officials

Council, proposes to validate and implement the recently developed Performance-Based Guidelines for the Selectio of Hot-Poured Crack Sealants. The test variations within laboratories were successfully verified. The developed tests and new guidelines will be submitted to AASHTO fo consideration as new specifications

Scope of Work: Round robin tests at five to seven various laboratories wi be conducted.
As an outcome of the TPF-5(045)study preliminary threshold(s)for each test were established based on extensive laboratory testing and limited field data. Therefore, a comprehensive field study is urgently neede to validate and to fine-tune the threshold values. Eight test sections in various climatic regions (dry-freeze, dry-nonfreeze, wet-freeze and wet-nonfreeze) will be include in the study. Representative crack sealants will be installed in these field sections and monitored for three years. At least five field surveys will be conducted. The field surveys will include sealant inspection and data and sample collection. Collected samples will be used to validate the laboratory tests and the proposed parametei threshold values. The following tasks are proposed in thi: study:

Task I : Laboratory Validation
Conduct round robin testing to establish test precision ar bias for the recently developed six tests.
Develop training program that includes detailed testing procedures.
Task II: Field Validation
Construct eight test sections in the four environmental regions (Wet-Freeze, Wet-Nonfreeze, Dry-Freeze, Dry-Nonfreeze).
Install two sealant types at each test section.
Task III: Monitoring Test Section for Four Years
Conduct field inspection of crack sealant five times durin the project duration.
Collect sealant samples annually from the test sections t measure their rheological properties and identify any changes.
Monitor crack movement and temperature variation to provide insight into the selection of the current temperature shift used in the proposed guidelines.

Task IV: Threshold Value Fine-Tuning
Use field performance to fine-tune the testing parameter thresholds in the proposed guidelines.

Task V: Quantify the Cost Effectiveness of Utilizing Crack Sealants
Measure pavement condition annually, in accordance witl SHRP Distress Manual, to examine the cost effectiveness of crack sealant.

Comments: Suggested contribution is $25,000/yr for each of 4 years total of $100,000.

Documents: http://www.pooledfund.org/documents/solicitations/123;

ILLINOIS CENTER FOR TRANSPORTATION

ILLINOIS
UNIVERSITY OF ILLINOIS AT URBANA-CHAMPAIGN

October 24, 2007

Validation and Implementation of Hot-Poured Crack Sealant Performance-Based Guidelines

The University of Illinois at Urbana-Champaign, in collaboration with Virginia Transportation Research Council, proposes to validate and implement the recently developed Performance-Based Guidelines for the Selection of Hot-Poured Crack Sealants. The developed guidelines were the outcome of the North American Consortium conducted by University of Illinois at Urbana-Champaign and the National Research Council of Canada. Thirteen states, led by VDOT/VTRC, sponsored the pool-fund research project TPF-5(045) along with 13 Canadian transportation agencies and industry. New test methods, which make use of the existing SHRP devices, were developed. The test variations within laboratories were successfully verified. The developed tests and new guidelines will be submitted to AASHTO for consideration as new specifications. However, to establish test precision and bias, round robin tests at five to seven various laboratories need to be conducted.

As an outcome of the aforementioned study, preliminary threshold(s) (Table 1) for each test were established based on extensive laboratory testing and limited field data. Therefore, a comprehensive field study is urgently needed to validate and to fine-tune the threshold values. Eight test sections in various climatic regions (dry-freeze, dry-nonfreeze, wet-freeze and wet-nonfreeze) will be included in the study. Representative crack sealants will be installed in these field sections and monitored for three years. At least five field surveys will be conducted. The field surveys will include sealant inspection and data and sample collection. Collected samples will be used to validate the laboratory tests and the proposed parameter threshold values. The following tasks are proposed in this study:

- **Task I : Laboratory Validation**
    - Conduct round robin testing to establish test precision and bias for the recently developed six tests.
    - Develop training program that includes detailed testing procedures.
- **Task II: Field Validation**
    - Construct eight test sections in the four environmental regions (Wet-Freeze, Wet-Nonfreeze, Dry-Freeze, Dry-Nonfreeze).
    - Install two sealant types at each test section.
- **Task III: Monitoring Test Section for Four Years**
    - Conduct field inspection of crack sealant five times during the project duration.
    - Collect sealant samples annually from the test sections to measure their rheological properties and identify any changes.
    - Monitor crack movement and temperature variation to provide insight into the selection of the current temperature shift used in the proposed guidelines.

- **Task IV: Threshold Value Fine-Tuning**
  - Use field performance to fine-tune the testing parameter thresholds in the proposed guidelines.
- **Task V: Quantify the Cost Effectiveness of Utilizing Crack Sealants**
  - Measure pavement condition annually, in accordance with SHRP Distress Manual, to examine the cost effectiveness of crack sealant.

The outcomes of the study consist of a validation of the test methods developed under the TPF-5(045) project, development of precision and bias for each test, and fine-tuned parameter thresholds proposed as part of the performance-based guidelines for the selection of hot-poured crack sealants. The participants of the recently concluded TPF-5(045) project recognized the need for this research and requested a Phase II of the study on field validation and implementation. Virginia will serve as the lead state, and Professor Imad Al-Qadi of the University of Illinois will serve as the principal investigator.

Figure 1 Four Environmental Zones Defined by SHRP-LTPP Experiment

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Table 1 Crack Sealant Performance Grade (SC-1)

Crack Sealant Performance Grade	SG-46							SG-52							SG-58							SG-64							SG-70							SG-76							SG-82						
	-46	-40	-34	-28	-22	-16	-10	-46	-40	-34	-28	-22	-16	-10	-46	-40	-34	-28	-22	-16	-10	-46	-40	-34	-28	-22	-16	-10	-46	-40	-34	-28	-22	-16	-10	-46	-40	-34	-28	-22	-16	-10	-46	-40	-34	-28	-22	-16	-10
Apparent Viscosity, SC-2	Installation Temperature																																																
Maximum Viscosity (Pa.s)	3.5																																																
MinimumViscosity (Pa.s)	1																																																
Vacuum Oven Residue (SC-3)																																																	
Dynamic Shear, SC-4	46							52							58							64							70							76							82						
Minimum Flow Coeffi. (kPa.s)	4																																																
Minimum Shear Thinning	0.7																																																
Crack Sealant BBR, SC-5	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4
Maximum Stiffness (MPa)	25																																																
Minimum Avg. Creep Rate	0.31																																																
Crack Sealant DTT, SC-6	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4
Minimum Extendibility (%)	85	85	70	55	40	25	10	85	85	70	55	40	25	10	85	85	70	55	40	25	10	85	85	70	55	40	25	10	85	85	70	55	40	25	10	85	85	70	55	40	25	10	85	85	70	55	40	25	10
Crack Sealant AT, SC-7	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4	-40	-34	-28	-22	-16	-10	-4
Minimum Load (N)*																																																	

* This parameter is not completed yet

Attachment 11B

FINAL
CONTRACT REPORT
VTRC 09-CR7

DEVELOPMENT OF PERFORMANCE-BASED GUIDELINES FOR SELECTION OF BITUMINOUS-BASED HOT-POURED PAVEMENT CRACK SEALANT: AN EXECUTIVE SUMMARY REPORT

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Standard Title Page—Report on Federally Funded Project

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7. Author(s): Imad Al-Qadi, Jean-Francois Masson, Eli Fini, Shih-Hsien Yang, and Kevin K. McGhee		8. Performing Organization Report No.: VTRC 09-CR7
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This study is based on work supported by the Federal Highway Administration and the U.S.–Canadian Crack Sealant Consortium under pool fund award No. TPF-5(045). Virginia serves as the Lead State; the Virginia Transportation Research Council of the Virginia Department of Transportation managed the project.

16. Abstract:

This report summarizes research presented in separate technical reports, papers, and journal articles that collectively document the development of a systematic process to aid in the selection of appropriate bituminous hot-poured sealants for pavement cracks and joints.

The following process elements are summarized herein: *Apparent Viscosity Test for Hot-Poured Crack Sealants*, *Development of a Short-Term Aging Test and Low-Temperature Testing Bibliography*, *Sealant Flow and Deformation by Dynamic Shear Rheometry in Summer Temperatures*, *Characterization of Low Temperature Creep Properties of Crack Sealants Using Crack Sealant Bending Beam Rheometry*, *Characterization of Low Temperature Mechanical Properties of Crack Sealants Using Crack Sealant Direct Tension Test*, and *Development of Adhesion Tests for Crack Sealants at Low Temperature.*

This report brings the results of this cumulative research together to introduce a set of tests and performance parameters for sealant at installation and service temperatures; an aging procedure to simulate sealant weathering; and most important, a simplified chart with thresholds for all performance parameters for the straightforward selection of crack sealant.

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In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

January 2009
VTRC 09-CR7

DISCLAIMER

The project that is the subject of this report was done under contract for the Virginia Department of Transportation, Virginia Transportation Research Council. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Virginia Department of Transportation, the Commonwealth Transportation Board, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. Any inclusion of manufacturer names, trade names, or trademarks is for identification purposes only and is not to be considered an endorsement.

Each contract report is peer reviewed and accepted for publication by Research Council staff with expertise in related technical areas. Final editing and proofreading of the report are performed by the contractor.

ABSTRACT

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INTRODUCTION

ASTM Standard D5535 defines a sealant as a material that possesses both adhesive and cohesive properties to form a seal, which prevents liquid and solid from penetrating into the pavement system. Crack sealing has been widely accepted as a routine preventative maintenance practice. Given a proper installation is achieved, crack sealant can extend pavement service life by a period ranging from three to five years (Chong and Phang, 1987). Numerous studies also demonstrated the cost- effectiveness of crack sealants (Joseph, 1990; Cuelho et al., 2002, 2003; Fang et al., 2003; Ward, 2001; Chong and Phang, 1987; Chong, 1990).

Crack sealant is produced so that it keeps its shape as applied and hardens through chemical and/or physical processes to form a viscoelastic rubber-like material that withstands extension or compression (crack movement) and weathering (Al-Qadi et al., 2007). However, in many cases, premature failure of crack sealants may be observed in one of the following scenarios. During the sealant installation, if the viscosity of the sealant is too high, sealant might not be able to fill the crack properly; hence, it will affect the interface bonding between sealant and pavement substrate. If the viscosity is too low, sealant might flow out from the cracks.

In the field, a sealant extends at low temperature and compresses at high temperature to accommodate pavement crack openings which increase with decreasing temperature and decreases with rising temperature. At high service temperature, sealant might fail due to pull out from the crack by tire passing. At low service temperature, the crack opening may increase from 10% to more than 90% depending on the environmental location; hence, one of the two mechanisms might be observed: cohesive or adhesive failure. The former occurs in the sealant, and the latter occurs at the sealant-pavement crack wall interface. At low temperature, sealant becomes more brittle due to physical hardening and is subjected to short-duration loading due to crack movements associated with stick-slip motions and truck trafficking as well as long periods of environmental loading.

In order to achieve a cost-effective crack sealing/filling operation and proper field performance, two factors must be closely controlled: quality of sealant installation and sealant mechanical and rheological properties (such as viscosity, bulk stiffness, and adhesive bonding). Regardless of sealant quality, improper installation will cause premature failure and, hence, reduced sealant service life.

Standards and specifications for selecting crack sealant have been established by several organizations, including ASTM International; the American Association of State Highway and Transportation Officials (AASHTO); and U.S. and Canadian federal, state, provincial, and municipal agencies. The objective of the specifications is to select materials that have the necessary properties to perform adequately in the field. However, these specifications are generally empirical and do not measure sealant fundamental properties. Hot-poured bituminous crack sealants are typically selected based on standard empirical tests such as penetration, resilience, flow, and bond to cement concrete briquettes (ASTM D6690). ASTM Standard D5329-04 (Standard Test Methods for Sealants and Fillers, Hot-Applied, for Joints and Cracks in Asphaltic and Portland Cement Concrete Pavements) summarizes most of these tests. These include non-immersed cone penetration, fuel-immersed cone penetration, the flow test, the non-immersed bond test, the water-immersed bond test, the fuel-immersed bond test, the resilience test, the oven-aged resilience test, the asphalt compatibility test, the artificial weathering test, the tensile adhesion test, the solubility test, and the flexibility test.

These tests are used by most state highway agencies in selecting their crack sealing materials; but the specification limits may vary from one state to another. These differences create difficulties for crack sealant suppliers because many states with the same environmental conditions specify different limits for the measured properties. These tests were also reported to poorly characterize the rheological properties of bituminous-based crack sealants and to predict sealant performance in the field.

Researchers have widely reported that current specifications for selection of hot-poured crack sealants are based on tests whose results showed no correlation with field performance (Masson, 2000; Belangie and Anderson, 1985; Masson and Lacasse, 1999; Smith and Romine, 1993; 1999). In addition, over the past two decades, a new generation of highly modified crack sealants has been introduced to the market (Zanzotto, 1996). These sealants exhibit quite complex behavior compared to those of traditional sealant materials (Belangie and Anderson, 1985). This necessitates the development of a new set of specifications.

PURPOSE AND SCOPE

The most effective way to evaluate the performance of crack sealants would be to perform field tests. However, the results from field tests are sometimes controversial because a sealant can perform well in one site and fail in another simply because of differences in environmental conditions. Therefore, the main objectives of this project were to develop laboratory tests that measure bituminous-based crack sealants rheological properties and to develop performance-based guidelines for the selection of hot-poured crack sealants. Meeting these objectives requires the development of new tests to measure the rheological properties of hot-poured crack sealants over a wide range of service temperatures. The developed tests need to be practical, repeatable and reproducible. Thresholds for each test should be identified to ensure desirable sealant field performance. A special effort was given to make use of the equipment originally developed during the five-year Strategic Highway Research Program (SHRP), which were used to measure binder rheological behavior as part of the performance grading (PG) system.

This executive summary report introduces a systematic process to help users select appropriate bituminous hot-poured crack sealants. This document summarizes research that is presented in separate technical reports, papers, and journal articles that collectively chronicle the development of this process. The process includes a new set of tests and performance parameters for sealant at installation and service temperatures. It also proposes a new aging procedure to simulate sealant weathering.

METHODS

To develop performance-based guidelines for the selection of hot-poured crack sealants that meet the aforementioned requirements and minimize the cost of possessing new testing equipment, the research group made use of the SuperPave™ binder performance grading (PG) equipment. Modifications to the existing viscosity test, bending beam rheometer, and direct tension test devices, specimen size and preparation, and testing procedures were made to accommodate the testing of crack sealants. In addition, new tests for sealant aging and sealant evaluation at high service temperatures were introduced. Upon the completion of test validation, test measured performance parameters were recommended for implementation as part of the newly developed "Sealant Grade" (SG) system. Appendix A briefly catalogues the sealants that were used in the extensive laboratory and field tests as well as their ASTM testing results that supported this research.

Apparent Viscosity

Sealant viscosity is among the parameters that affect initial bonding. Therefore, applying a sealant at the appropriate viscosity provides for better crack filling and enhances interface bonding. Several factors affect the measured viscosity of hot-poured crack sealant. Therefore, it is essential to identify the material characteristics that influence the rheological behavior of hot-poured crack sealant at installation. These characteristics need to be set at reasonable limits, to simulate field installation as closely as possible. Although standard tests to examine sealant consistency exist, these standard tests have not been proven to predict field performance. As part of an effort to bridge the gap between sealant fundamental properties and field performance, a test procedure was developed to measure apparent sealant viscosity using the same rotational viscometer equipment used in the SuperPave™ PG system. The development of this procedure is described in detail in a supporting document (Al-Qadi et al., 2008b). The procedure for measuring apparent viscosity is summarized in this report under "Results and Discussion."

Sealant Aging

For an aging test to be effective, it must quickly provide an aging as close as possible to reality. Figure 1 illustrates the basic process. To this effect, true aging was determined from the physico-chemical analysis of 12 sealants weathered in Montreal, Canada, for nine years (Table 1). As expected, sealants with good performance contain components resistant to weathering, whereas sealants with poor performance oxidize quickly. Figure 2 shows an example of sealant stiffening due to weathering.

Because of sealant's complex mixture, each sealant shows unique aging characteristics. To mimic the effect of weathering on sealants, several accelerated aging methods were compared (alone or in combination) after various aging periods and temperatures, including small-kettle aging, microwave aging, pressure aging, oven aging, and vacuum oven aging. The results of physico-chemical analysis of sealants weathered in the field were compared to those of sealants aged quickly in the laboratory (Al-Qadi et al., 2004).

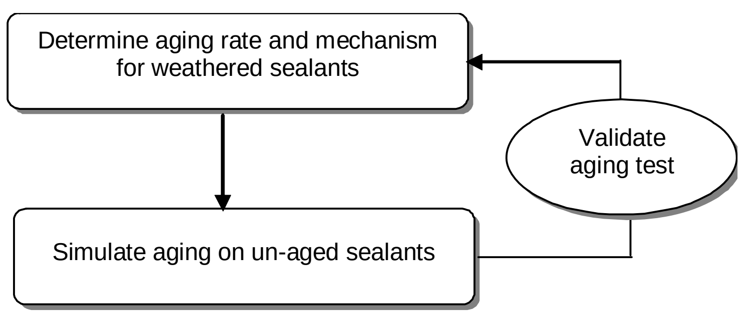


Figure 1. Schematic of Aging Procedure

Table 1. Physico-chemical Method to Characterize Crack Sealant Aging

Method*	Output	Use
GPC	Separation of bitumen and polymer	Quantify polymer; degradation rate and mechanism
FTIR	Fingerprint of composition	Oxidation; identification of polymer and filler; semi-quantitative analysis; degradation mechanism
TG	Weight loss upon heating	Contents of filler and light, medium, and heavy hydrocarbon components
DSR	Stiffness, relaxation	Effect of temperature and aging on mechanical properties

*GPC: Gel permeation chromatography; FTIR: Fourier transform infrared spectroscopy; TG: Thermogravimetric analysis; and DSR: Dynamic shear rheometry (DSR).

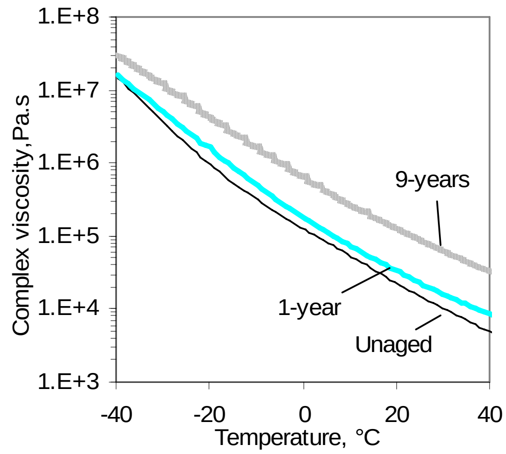


Figure 2. Complex Viscosity Increase for a Field-Aged Sealant at One and Nine Years

Sealant Flow and Deformation

Bituminous sealants applied to cracked pavements sometimes fail due to deformation under the combined action of shear stresses and high service temperatures (Masson et al., 2007). In an attempt to define performance parameters, 21 sealants were tested with a dynamic shear rheometer (DSR) and subjected to increasing stresses at temperatures between 46°C and 82°C. These conditions were meant to mimic the effects of various traffic levels and maximum temperature in the field.

Flexural Creep

The bending beam rheometer (BBR) is used in most pavement laboratories nowadays to measure binder stiffness at low temperature. A modified BBR test, a crack sealant bending beam rheometer (CSBBR), was introduced to measure the flexural creep of crack sealant at temperatures as low as -40°C. The development of this procedure is described in detail in a supporting document (Al-Qadi et al, 2008d). The resulting procedure is summarized later in this report.

Low Temperature Tensile Properties

Four typical types of stress-strain curves of crack sealant are shown in Figure 3. Depending on sealant composition and test temperature, sealants may behave as a brittle plastic for which the stress-strain curve is linear up to fracture with little percentage elongation (Curve A). Low-polymer and high-crumb-rubber-modified sealants behave this way. A brittle-ductile failure may be observed for crack sealants as well. When the tensile load reaches a maximum, sealant may fracture as shown in curve B, or the specimen may continue to stretch after the maximum load as shown in curve C (Figure 3). Sealant may also experience ductile failure (curves D and E). Typically, this type of sealant exhibits a yield point, followed by extensive elongation at a constant stress. This is referred to as a plastic flow region, and is clearly a region

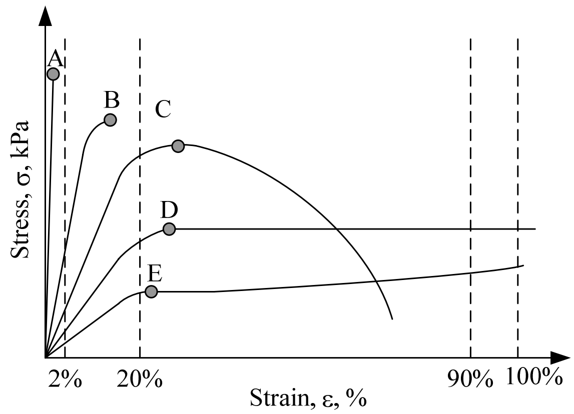


Figure 3. Stress-Strain Behavior of Crack Sealant Observed in the Direct Tension Test

of nonlinear viscoelasticity. After the plastic flow region, sealant might exhibit strain hardening. This type of sealant usually has a relatively high polymer concentration.

To investigate whether a sealant can survive in a particular service conditions, the SuperPave™ Direct Tension Test (DTT) was considered and modified for crack sealants. The development of crack sealant direct tension test (CSDTT) is described in detail in a supporting document (Al-Qadi et al, 2008c). The resulting procedure is summarized later in this report.

Adhesive Properties at Low Temperature

The adhesion capability of hot-poured bituminous sealants is usually evaluated using a standard test of an empirical nature (ASTM D5329). There is, however, no indication that the results of this test pertain to field performance. In addition, this test examines adhesion of sealant to Portland cement concrete, and the test result does not account for aggregate composition, which is the main component of HMA. Therefore, a reliable test method, which is based on sealant rheology, accounts for aggregate composition, and correlates with field performance, is urgently needed. This study proposes three laboratory tests to predict interface bonding of crack sealant to aggregate at service temperatures ranging from -4ºC to -40ºC (Al-Qadi et al., 2008a). The three tests are designed to address the needs of manufacturers, transportation organizations, including contractors, transportation agencies and consultants, and researchers, respectively.

The first laboratory approach addresses the compatibility of sealant with a specific substrate, by measuring the free energy of the bond, *work of adhesion*. The second test makes use of the *direct tension* test (DTT) device. The third test is a fracture type test that uses a fracture mechanics approach to derive a fundamental property of the bond, *interfacial fracture energy* (IFE).

RESULTS AND DISCUSSION

Apparent Viscosity Testing Procedure

Numerous factors affect the measured viscosity of hot-poured crack sealant. Due to the relatively high polymer content, sealant responses to changes in temperature and loading can be quite complex. Since hot-poured crack sealants behave as non-Newtonian fluids, variation in the experimental parameters can affect the measured values; hence, a test setup and testing parameters were identified (Al-Qadi et al., 2006; 2008b). Laboratory conditions should simulate sealant installation conditions as closely as possible. A critical issue was the shear rate imposed on the material during application. It was determined that a spindle speed ranging from 115 rpm to 5536 rpm should be used to simulate the shearing of the sealant as it enters the crack during installation. However, a significant reduction in the shear rate may occur as the sealant exits the applicator wand, due to the sharp temperature drop as well as the high friction with the crack walls.

Although the Brookfield Thermosel system (as adopted from SuperPave™) is not a high-shear rheometer (its maximum allowable spindle speed is 250 rpm), it was found to be sufficient for the sealant testing. After extensive testing, a Brookfield rotational viscometer was adopted; modification of the test procedure and equipment was implemented (Figure 4). SC4-27 spindle at a speed of 60 rpm (shear rate of $20.4s^{-1}$) at the recommended installation temperature is used. The spindle is attached to a newly developed rigid rod. The rod is a replacement for the current hook; it prevents a rubber particle from disturbing the spindle rotation which results in better test repeatability. A conditioning time of 20 min and a waiting time of 30 s before collecting data are also recommended to ensure that the measured viscosity has stabilized.

In this test, sealants are cut in small pieces and placed directly in an aluminum chamber, then sealant was melted inside the chamber. Cutting sealant without melting it improved test results significantly. This measured apparent viscosity is expected to be an acceptable indication of the sealant's rheological behavior at installation temperature, assuming that the suggested procedure and equipment are used. Fifteen virgin sealants were tested in accordance with the developed testing procedure; the apparent viscosity of several sealants at various temperatures is presented in Figure 5.

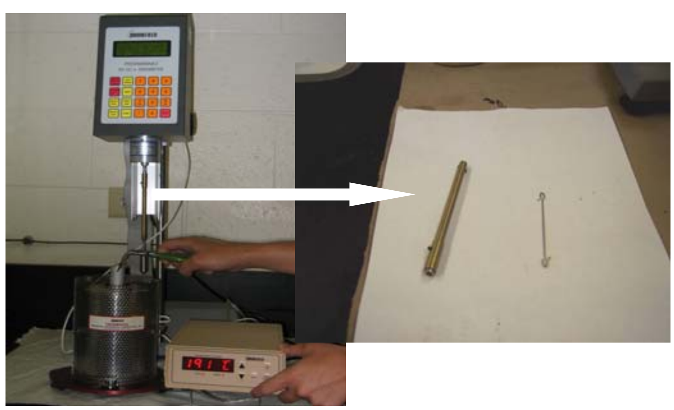


Figure 4. Brookfield Thermosel System and Rigid Rod Used for Crack Sealant Testing Compared to the Rod Used for Asphalt Binder

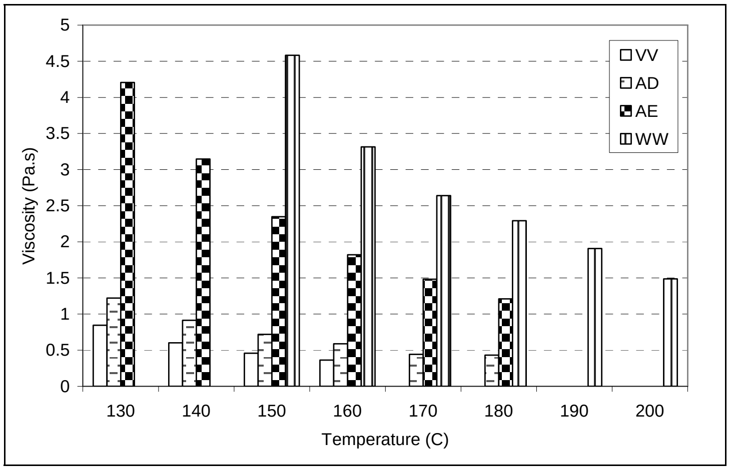


Figure 5. Apparent Viscosity of Several Sealants at Various Temperatures Weathering and Accelerated Aging of Hot-Poured Bituminous Sealants

Seven laboratories conducted a round-robin test. The repeatability of the measured apparent viscosities was determined through statistical analysis. The average coefficients of variation within and between laboratories were found to be 1.6% and 6%, respectively. Maximum permissible differences within a laboratory and between laboratories are 5.4% (among the best three readings out of four) and 17% (between the test conducted in two different laboratories), respectively. These values are comparable to those of asphalt binder: 3.5% and 14.5% based on ASTM D4402-02 and 3.5% and 12.1% based on AASHTO 2006 T316, respectively. Because viscosity plays an essential role in predicting field performance of hot-poured crack sealant, upper and lower viscosity limits are recommended. An upper limit of 3.5 Pa.s ensures that sealant is liquid enough to pour, whereas a lower limit of 1 Pa.s controls the potential of using excessively fluid sealant. Hence, the sealant apparent viscosity should be between 1.0 and 3.5 Pa.s when measured at recommended installation temperature.

Sealant Aging

The effect of hot-poured crack sealant oxidation and the change in polymer molecular weight on the sealant complex viscosity between -40°C and 40°C served as the two parameters in examining the applicability of pressure aging method (method 1) and vacuum oven method (method 2) (Figure 6). Microwave heating was found to mimic the aging of sealants that contain mineral filler; but not other sealants. The microwave method thus lacked general application. Pressure aging was also found to be inappropriate because it often led to insufficient bitumen oxidation, and excessive thermo-degradation of the polymer. Vacuum oven aging proved to be the most appropriate method to simulate sealant weathering. In this method, sealants are cut into slices and placed on a stainless steel pan; each pan contains 35 g of sealant. The pan is transferred into a conventional temperature controlled oven which is maintained at 180°C for approximately 5 min to allow sealant to melt and form a film. The sealant is then removed from the oven and cooled to room temperature. Once it cools, sealant is placed in a vacuum oven preheated at 115°C for 16 hr. After 16 hr, the vacuum is released and sealant is placed in a conventional oven at 180°C for 5 min or until the sealant is fluid enough to pour.

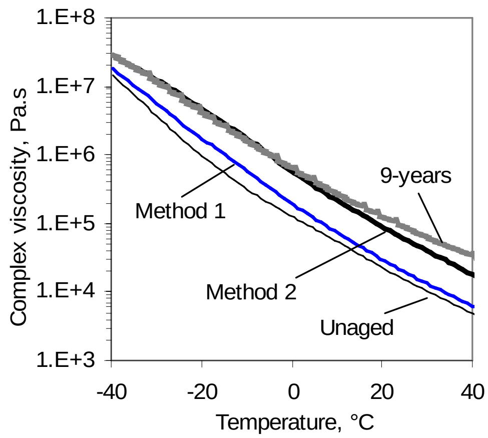


Figure 6. Complex Viscosity of a Field-Aged Sealant Compared to that after Accelerated Aging

Sealant Flow and Deformation by Dynamic Shear Rheometry

Plots of low shear viscosity (η_L) versus shear rate ($\dot{\gamma}$), Equation 1, indicated that many sealants were susceptible to shear thinning. Their apparent viscosity decreased with an increase in shear rate and/or temperature (Figure 7). This indicated that at high service temperatures, high traffic loads or volumes would affect the extent of sealant flow when it is under stress.

$$\sigma = C\,\dot{\gamma}^{P} \qquad \text{(Eq. 1)}$$

where,

σ = stress;

$\dot{\gamma}$ = shear rate;

C = flow coefficient; and

P = shear-thinning coefficient

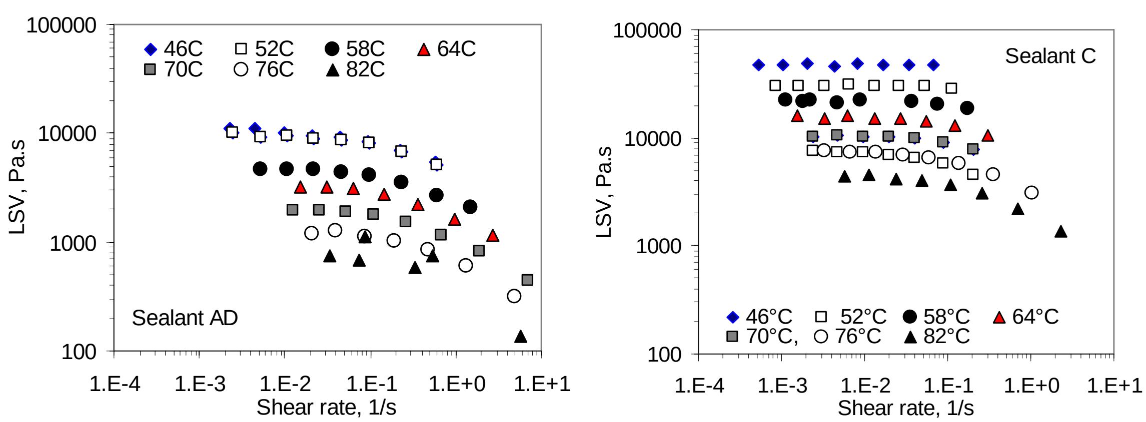


Figure 7. Low Shear Viscosity as a Function of Shear Rate; The Stress Doubles for Each Point from Left to Right

Plots of η_L vs $\dot{\gamma}$ were interpreted based on the Ostwald power law model. This model provides two parameters: a flow coefficient (C) and a shear-thinning coefficient (P). These coefficients correlated well with sealant pseudo-field performance as measured by tracking (Collins et al., 2007). Figure 8 shows the relationship between these factors and performance during the pseudo-field test. The solid markers indicate the sealants that did not fail during the pseudo-field test, and the open markers show those that failed. The semi-log scale in Figure 8 serves to highlight the high-failure regions (open markers). Limiting values for P and C can be established to limit the risk of sealant failure. Each pair of C and P represents performance criteria.

In the absence of effective limiting criteria (area A in Figure 8), the sealant failure rate due to tracking is 39% (Table 2). As C and P limits are raised, the risk of tracking failure is reduced. Area E defines the limits within which no tracking failure is observed. With such demanding criteria, 33% of the sealants are above the pass limits. Any limits in C and P can be used to define the level of sealant performance, but the most appropriate performance criteria may be that defined by area D, where limits of C = 4000 Pa.s and P = 0.70 provide for a failure risk of only 3% and a sealant acceptance rate greater than 50%. The other limits have greater acceptance rates, but the risk of failure is disproportionately higher (Table 2).

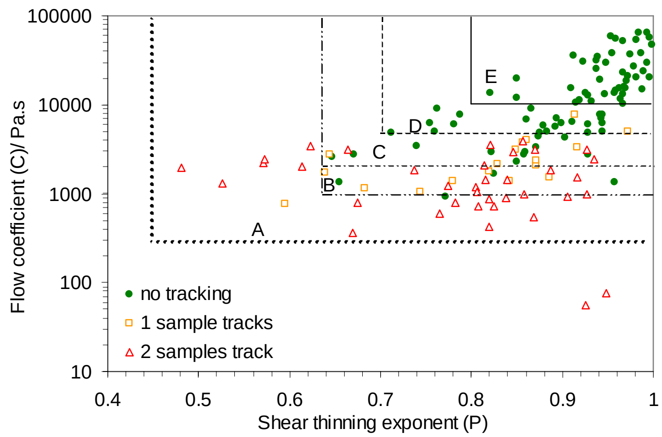


Figure 8. Semi-Log Plot of the Ostwald Parameters and Possible Performance Limits, Areas A to D

Table 2. Lower Limits for C and P and their Relationship with Tracking Performance

Area[a]	C (Pa.s)	P	Passing[b]	Tracking[c]
A	300	0.46	145 (99%)	39 % failure
B	1000	0.64	123 (84%)	24% failure
C	2500	0.64	98 (66%)	14% failure
D	4000	0.70	76 (52%)	3% failures
E	10000	0.80	49 (33%)	No failures

[a]Area in Figure 8.

[b]From DSR: Number of samples above C and P limits out of a total of 147.

[c]From the pseudo-field test: ratio empty/all marker within the given plot area.

Characterization of Low Temperature Mechanical Properties Using Modified Bending Beam Rheometry

The principle of the crack sealant bending beam rheometer (CSBBR) test is applying a constant load of 980 mN (100g) to a sealant beam and then measuring the beam deflection. The crack sealant is a much softer material compared to asphalt binder; therefore, excessive deflection was encountered during the testing of some sealants with the SuperPave™ BBR device. Consequently, several modifications were made to the SuperPave™ BBR test. First, the specimen thickness was doubled to overcome the excessive deflection. Second, the device was modified to accommodate the new specimen geometry. Additionally, a modified testing procedure, a new aging procedure, a validated testing period, and stiffness determination time were introduced. Linearity verification was conducted in order to verify that bituminous crack sealants behave as linear viscoelastic materials within CSBBR testing range.

The CSBBR beam thickness was doubled from 6.35 to 12.7 mm. However, because of the increase in the beam's thickness, the deflection at the center of the beam due to shear would increase. The analysis shows that the center deflection contributed by shear force is only 4%, which was deemed acceptable. Figure 9 shows the softest tested sealant loaded with 980 mN and resulted in final deflection of 3.2 mm after 240 s of loading (Al-Qadi et al., 2005). To adjust the device to accommodate the newly developed specimen geometry, the specimen support and calibration kits of the BBR were modified, as shown in Figure 10. The new design specimen supports are 6.35 mm shorter than the SuperPave™ BBR specimen supports and can easily be replaced. The calibration kits of the system were also modified as shown in Figure 11. The compliance beam for the crack sealant test was modified by adding two footers at each end of the beam (Al-Qadi et al., 2008d).

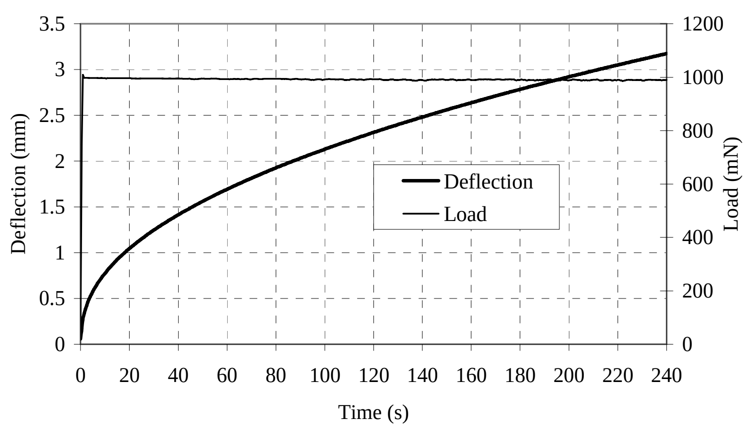


Figure 9. Deflection and Load versus Time for Sealant BB Using Beam Thickness 12.7 mm at -40°C

(a) (b) (c)

Figure 10. (a) SuperPave^{TM} BBR Specimen Supports; (b) Crack Sealant BBR Specimen Supports; and (c) Modified Specimen Supports

Figure 11. Calibration Beam: In the Front, Compliance Beam for SuperPave^{TM} BBR Test and in the Back, Modified Compliance Beam for Crack Sealant BBR Test

The test procedure modification was completed in two parts: the silicon-based release agent was used to replace the Mylar strip because the Mylar strip melted at sealant pouring temperature; second, each specimen was poured from an individual container which has the same weight. Because bituminous-based sealants are composed by asphalt binder, SBS copolymer, crumb rubber, and various additives, the variability between specimens is high. By controlling the pouring weight of each specimen, the test repeatability was greatly improved (Al-Qadi et al., 2006). Prior to pouring specimens into the molds, sealant was aged in accordance with the aging method developed in this study.

The assumption that crack sealant follow linear viscoelastic behavior was verified (Elseifi et al., 2006). Test results indicated that stiffness was independent of the applied stress level, as shown in Figure 12; three levels of loading (250 mN, 490 mN, and 980 mN) were applied on sealant specimens. The second condition of linearity, the experimental deflection at time (t) and the recovery deflection at time 240s + (t) should be equal or within a 5% difference could not be verified experimentally due to the sealant's softening behavior. Therefore, finite element (FE) was used to investigate the second condition of linearity. Figure 13 shows that the second condition of linear viscoelasticity can be verified.

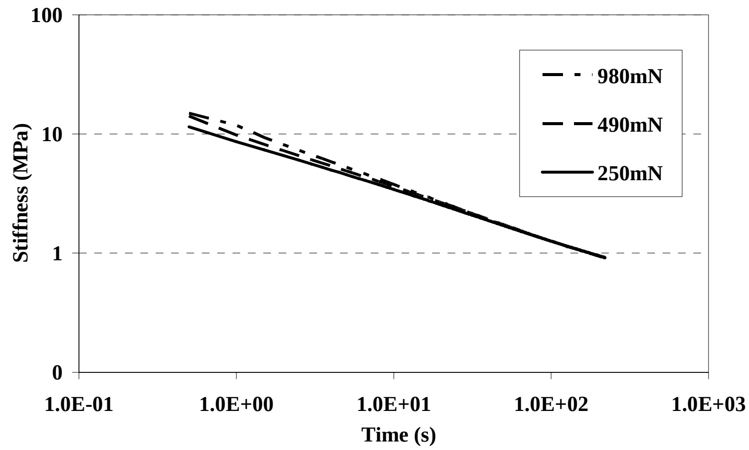


Figure 12. Measurements of Creep Stiffness for Sealant NN at -40°C

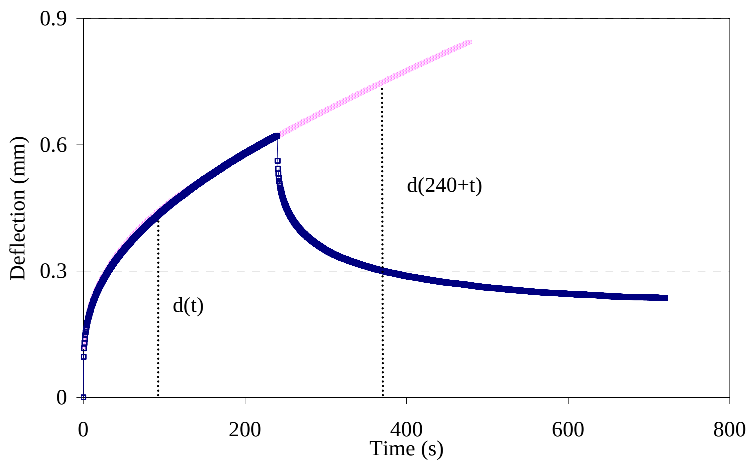


Figure 13. Schematic Diagram Illustrates the Extrapolated Data after 240s, Using FE

The viscoelastic model, Prony series, was fitted to the experimental data to obtain Prony series parameters. Prony series expansion was found to be adequate in describing the mechanical behavior of crack sealants at low temperature (Elseifi et al., 2006). Fitting parameters were then incorporated into a three-dimensional FE model of the CSBBR specimen. The resulting calculated creep deflections agreed with the measured values.

Fifteen sealants were tested at temperatures ranging from -4°C to -40°C. In this test, 35 g of sealant is first heated at its recommended pouring temperature and then poured into an aluminum assembled mold. A rectangular sealant beam is cast with dimension of 12.7 mm in height, 12.7 mm in width, and 102 mm in length. The beam is then placed in a fluid environmental chamber and the specimen is placed on a two-point support and subjected to a point creep loading. The specimen is exposed to a creep loading for 240 s then followed by 480 s unloading. The load and deflection of the sealant beam is recorded during the period of loading and unloading.

Each sealant was tested at three temperatures. These testing results were used to develop performance parameters. The selected performance parameters had to satisfy these four criteria: ability to describe the sealant's rheological behavior, ease of measurement and calculation, repeatability, and correlation with field performance. In addition, it was found that the critical loading time for crack sealant material at low temperature is after 5 hr of loading. If the temperature superposition principle is applied, the stiffness at 240 s for a given temperature can be used to predict the stiffness after 5 hr of loading at a temperature of approximately 12°C greater. Given the variation in sealant response to temperature change, 6°C shift is deemed appropriate. The stiffness at 240 s (Figure 14), the average creep rate (Figure 15) and dissipated energy ratio (Figure 16) were the performance parameters selected to distinguish between sealants. These new tests are repeatable, and the coefficient of variation between operators is less than 4%. However, there is a difference in measured values was noted when using devices from different manufacturers (Al-Qadi et al., 2007).

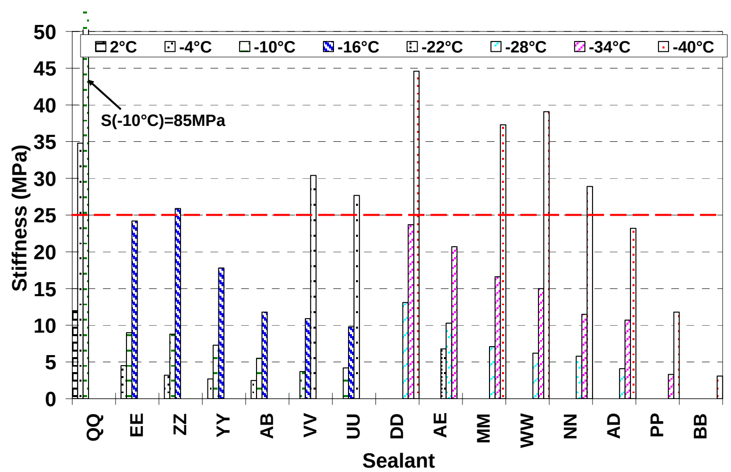


Figure 14. Stiffness at 240 s at Various Testing Temperatures for 15 Sealants

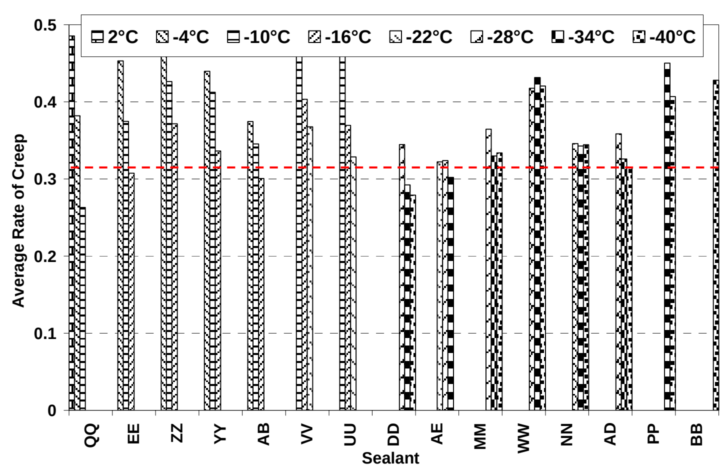


Figure 15. Average Creep Rate at Various Testing Temperatures for 15 Sealants

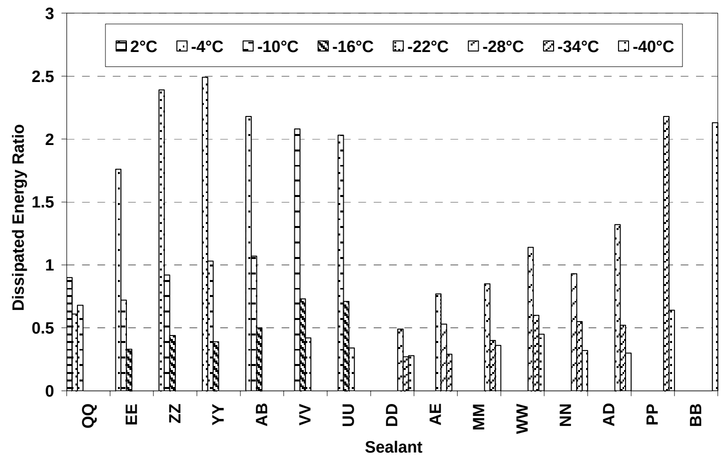


Figure 16. Dissipated Energy Ratio at 240 s at Various Testing Temperatures for 15 Sealants

Five sealants that were previously installed in Montreal as part a long-term field performance evaluation survey were tested using CSBBR. The results were used to establish the selection criteria for the CSBBR test. The test results recommend two performance criteria for application: stiffness at 240s and average creep rate. The recommended thresholds, which are temperature independent, for the two criteria are a maximum stiffness of 25 MPa and minimum average creep rate 0.31, respectively.

Characterization of Low Temperature Tensile Properties Utilizing Direct Tension Tester

The principle of the crack sealant DTT (CSDTT) is to slowly pull a crack sealant specimen in tension until it breaks. The dog-bone shaped specimen used in the DTT has a rectangular cross section. Its ends are enlarged so that when crack sealant is poured into the mold, it has a large adhesive area between the crack sealant and end tabs. The end tabs are made from Phenolic G-10 material, to provide good bonding. The SuperPave™ DTT specimen geometry can only extend the sealant specimen up to 32% strain. This is significantly smaller than the expected crack sealant extension in the field. Therefore, for crack sealant testing, the specimen geometry and preparation procedure were modified.

A FE analysis was conducted to determine the optimized specimen geometry which provides uniform stress distribution within specimen while allowing sufficient extension (Figure 17). This led to a new geometry; the new specimen dimensions are the following: 24 mm long, 6 mm wide and 3 mm thick; the effective gauge length is 20.3 mm. The maximum extension that can be achieved using this specimen is 19 mm which is equivalent to approximately 94% strain. This meets the extreme service conditions that sealants may experience in the field. Table 3 presents the geometry comparison of SuperPave™ binder DTT specimen, transition specimen and CSDTT specimen (Al-Qadi et al., 2007).

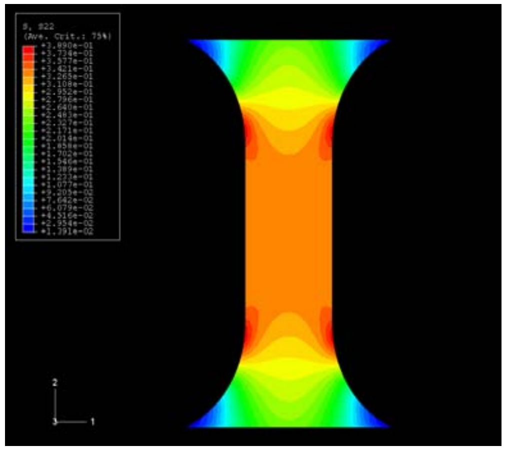


Figure 17. Uniform Stress Distribution along the Web of the Dog-Bone Shape CSDTT Specimen

Table 3. Comparison of Direct Tension Specimen Dimensions

Specimen Type	Width (mm)	Thickness (mm)	Nominal Length (mm)	Effective Gauge Length (mm)
SuperPave™	6	6	40	33.8
Transition	6	3	40	33.8
Crack Sealant	6	3	24	20.2

The effects of geometry and loading rate on sealant tensile behavior were investigated. Results obtained from testing SuperPave™ (6 x6 x 33.8 mm^3), transition (6 x 3 x 33.8 mm^3) and crack sealant (6 x 3 x 20.2 mm^3) high-polymer-content sealant specimens were used to evaluate the effect of cross-section area on the stress-strain relationship. As illustrated in Figure 18, high-polymer-content sealants WW and PP showed no major effects due to changes in specimen cross-section areas. The effect of specimen length on stress-strain response for effective gauge length of 33.8 mm and 20.3 mm specimens is shown in Figure 19 a and b). A greater stress response was noted in the 20.3 mm specimen than in the 33.8 mm specimen when tested at the same elongation rate. To compare the stress response at the same strain level, two elongation rates, 3 mm/min and 1.8 mm/min, were applied to 33.8 mm and 20.3 mm gauge length specimens, respectively. Considering the corresponding specimen length, the variation in elongation rate resulted in an identical strain rate of 8.8%/min for each type. Figure 19 illustrates that the crack sealant specimen elongates about two to three times longer than the SuperPave™ binder specimen. In addition, high-polymer-content sealants have shown 10 to 20 times more elongation and up to twice the tensile strength of crumb-rubber sealants. Regardless of specimen geometry, high-polymer-content sealants have shown equivalent peak stress at its maximum elongation state. Hence, the length effect is negligible in the sealant tensile strength when high-polymer-content products were used (Al-Qadi et al., 2007).

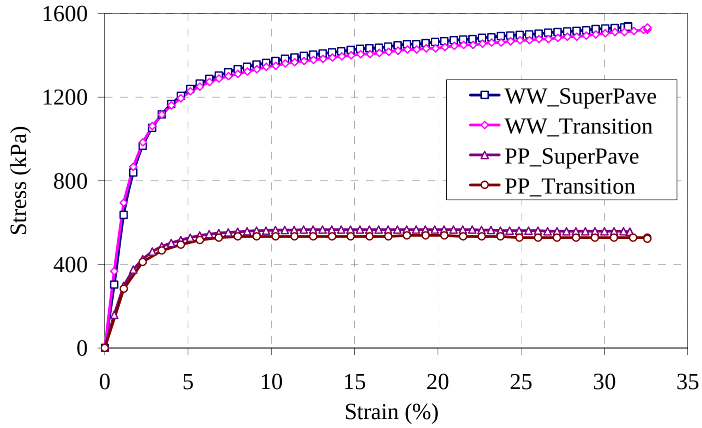
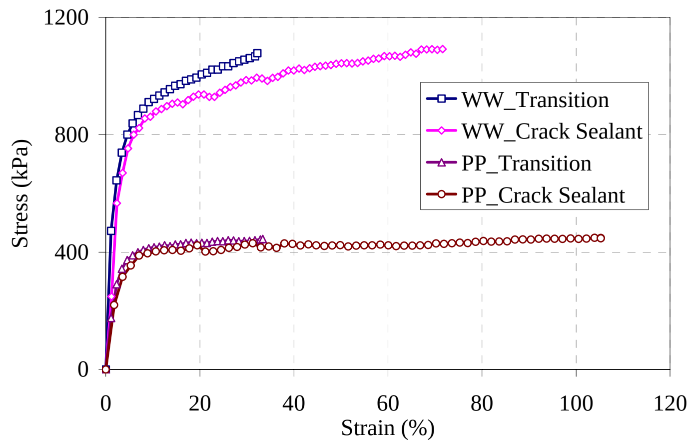
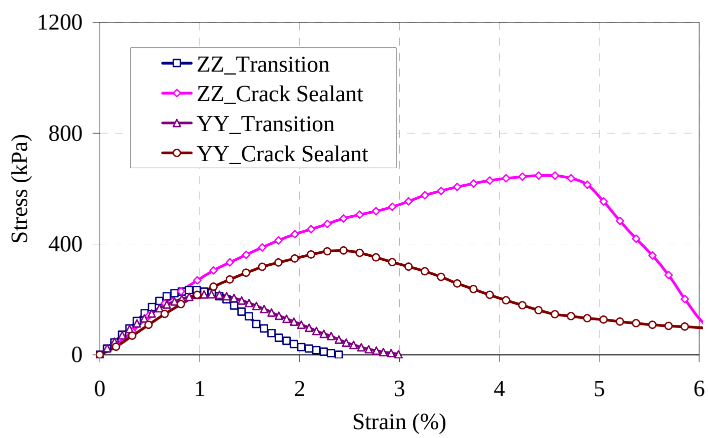


Figure 18. Effect of Cross-Section Area on Stress-Strain Relationship for Two High-Polymer-Content Sealants at 4.5 mm/min



(a)



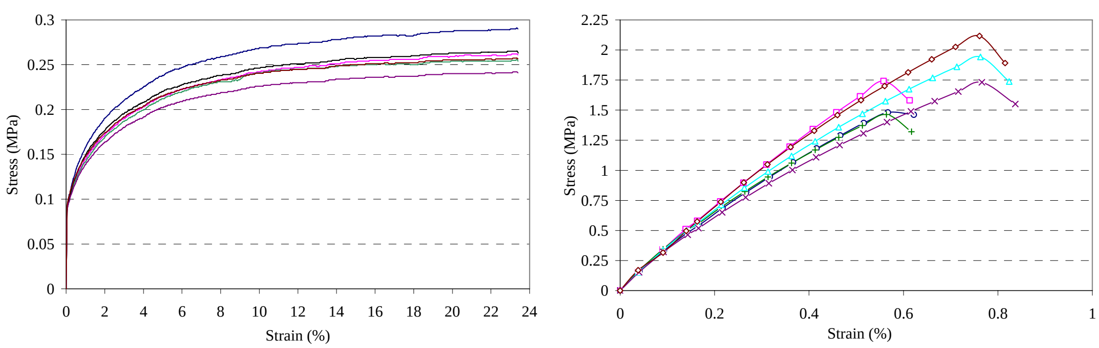
(b)

Figure 19. Effect of Specimen Length on Stress-Strain Relationship at the Same Strain Rate for (a) High-Polymer-Content Sealant, and (b) Crumb-Rubber Sealant

The specimen preparation procedure was also modified to accommodate various sealant compositions to improve the workability while pouring the sealant into a mold. The mold was heated to 50°C lower than sealant pouring temperature prior to pouring the sealant. Right after the sealant was poured into the mold, a spatula was used to slightly tap the sealant to ensure that sealant filled the mold. Figure 20 shows the typical stress-strain relationship of six replicates for stiff sealant QQ and soft sealant BB, which were tested at -10°C and -40°C, respectively.

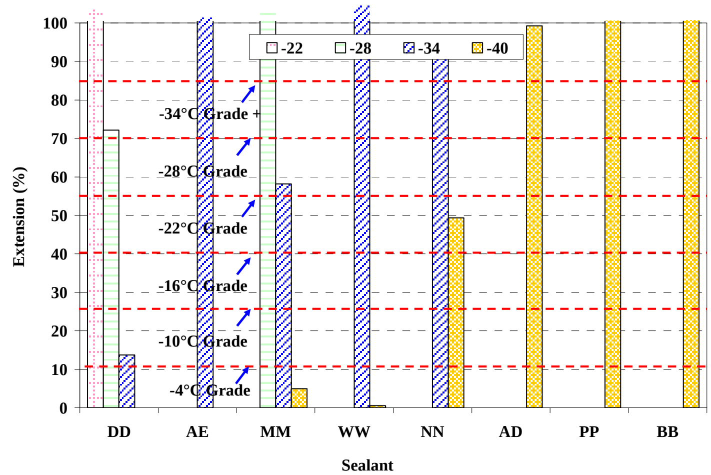
Fifteen sealants were tested at low temperatures ranging from -40 to -4°C. The extendibility of the sealant was measured and used as a performance parameter (see Figure 21). It was found that extendibility is a good criterion for identifying and distinguishing among sealants. In addition, a viscoelastic model was fitted to tensile stress-strain test results to obtain Prony series of crack sealants. The model was used to estimate the stress relaxation modulus for the crack sealant. The fundamental property, relaxation modulus, can be used to relate to sealant's field performance as well (Yang and Al-Qadi, 2008).

The study recommends using the DTT as a standard test to evaluate the bituminous-based hot-poured crack sealant at low temperature. The performance parameter, extendibility, was recommended for use in the specification. The threshold for the extendibility depends on the sealants' lowest application temperature and is presented in Table 4. In addition, because the test is conducted under a relatively higher deformation rate compared to real crack movement, the research team recommends a +6°C shift in the crack sealant grading system. Therefore, for instance, if the lowest service temperature is determined as -16°C, the test would then be conducted at -10°C. If the extendibility of such sealant is over 25%, the sealant passes the criteria and is approved for use.

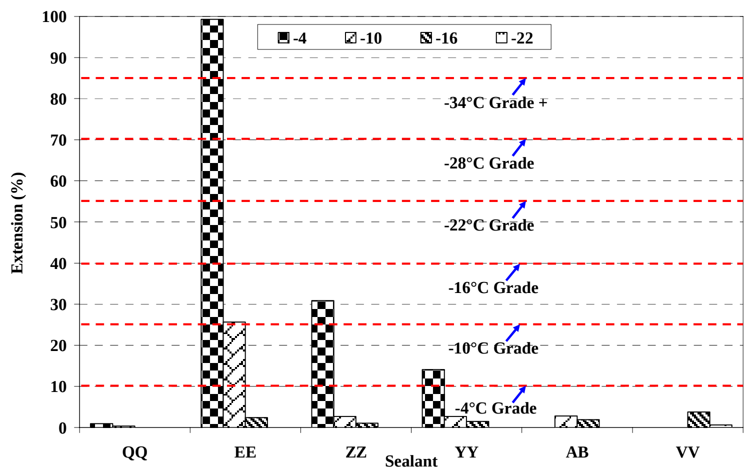


(a) **(b)**

Figure 20. Replicate Stress-Strain Curves for Sealants: (a) QQ at -10°C; and (b) BB at -40°C



(a)



(b)

Figure 21. Extendibility of Selected Sealant at (a) -22, -28, -34, and -40°C; and (b) -4, -10, -16, -22°C

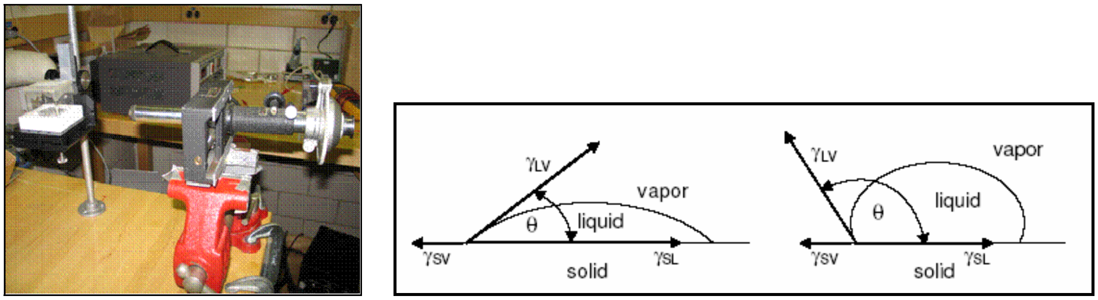
Table 4. Thresholds for Crack Sealant Extendibility at Various Temperatures

Temperature (°C)	-4	-10	-16	-22	-28	-34	-40
Extendibility (%)	10	25	40	55	70	85	85

Adhesive Properties of Crack Sealant at Low Temperature

Work of Adhesion

Even a quality sealant may fail if used with an incompatible aggregate. A compatibility test can be performed using the Sessile drop method, Figure 22, which is used to determine both the surface energy of the hot-poured crack sealant and its wettability (contact angle) with respect to aggregate. For this test, sealant is heated and mixed at the manufacturer's recommended installation temperature and poured onto an aluminum sheet to form a thin, smooth surface. The sealant is cooled at room temperature to solidify and make thin plates. A five-micrometer pipette is used to manually apply liquid drops from three probe liquids (water, formamide, and glycerol) onto the sealant plate. The image of each drop is captured by microscope within 15 s after it is



(a) **(b)**

Figure 22. Surface Energy Method: (a) Sessile Drop Equipment and Microscope; and (b) Contact Angle of a Drop of Liquid on a Solid

applied. The resulting contact angle is used to determine the work of adhesion between sealant and substrate.

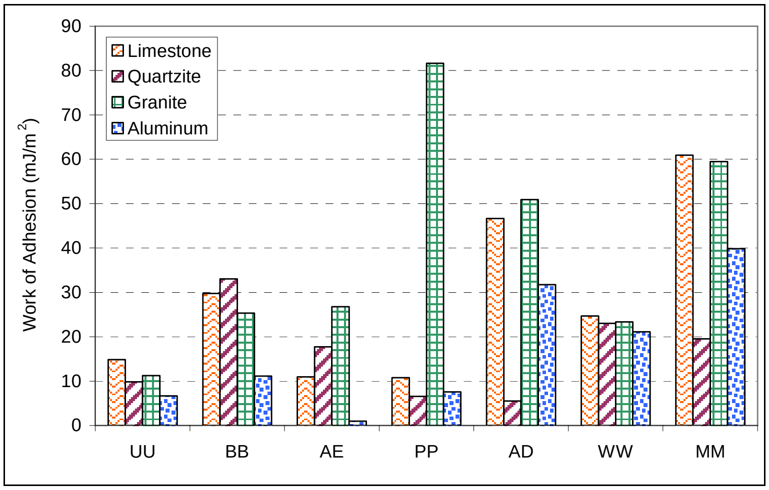
Due to the high variation among aggregate properties, replacing aggregate substrate with a standard material would be beneficial. Various potential reference materials were examined and aluminum was selected because it has compatible thermal expansion, smoothest surface, and similar surface chemistry to natural aggregate (Fini et al., 2006). Figure 23 presents the calculated work of adhesion between sealant and four substrates: aluminum, granite, quartzite, limestone.

Direct Tension

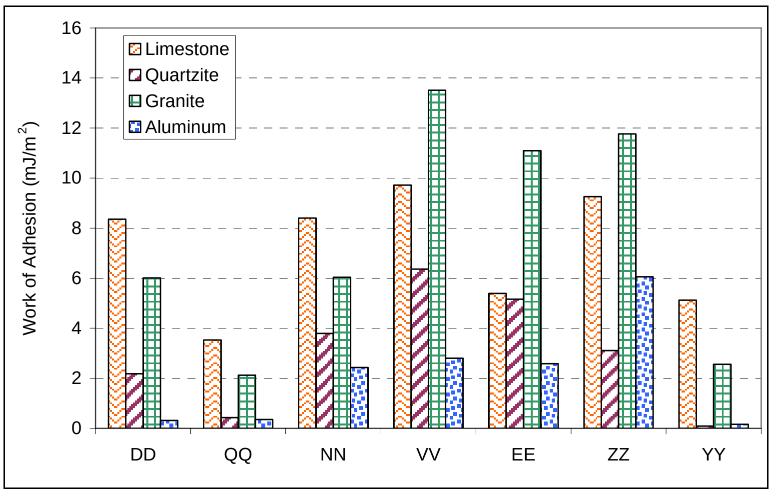
The premise of the DTT for adhesion is to detach sealant from its aggregate counterpart by applying a tensile force. A new test fixture that simulates sealant pouring condition and loading mechanism in the field was developed. The briquette assembly consists of two aluminum half-cylinders of 25 mm diameter and 12 mm thickness; aluminum is conservatively selected as a substrate reference material. Each aluminum briquette is confined within an aluminum grip designed to work with the DTT sitting posts. The assembly has a half cylinder mold, open at the upper part. The mold is placed between the two aluminum half cylinders on an even surface. In order to ensure that adhesive failure occurs, and to define failure's location, a notch is made at one side of the sealant-aggregate interface. A 12.5 x 2 mm shim is placed at one aggregate-sealant interface. The assembly then is placed in the DT machine so the notch is placed at the non-moving side of the DT machine.

To conduct the test, sealant is heated at its recommended installation temperature and poured into the half cylinder mold. After one hour of annealing at room temperature, the specimen is trimmed and placed in the cooling bath for 15 min . The specimen is then removed from the bath, demolded, and placed back in the bath for another 45 min before testing. Using the DT device, the end pieces are pulled apart by moving one of the end pieces at a speed of 0.05 mm/s, strain rate of 0.005 mm/mm/s (Figure 24). The results of the test are the maximum load and calculated energy, which is the area under the load-displacement curve up to failure, are reported as indications of bond strength (Al-Qadi et al., 2008a).

The interfacial bonding of combinations of eight sealants and four substrates were measured. Figure 25 presents the maximum load and energy required to break the bond between



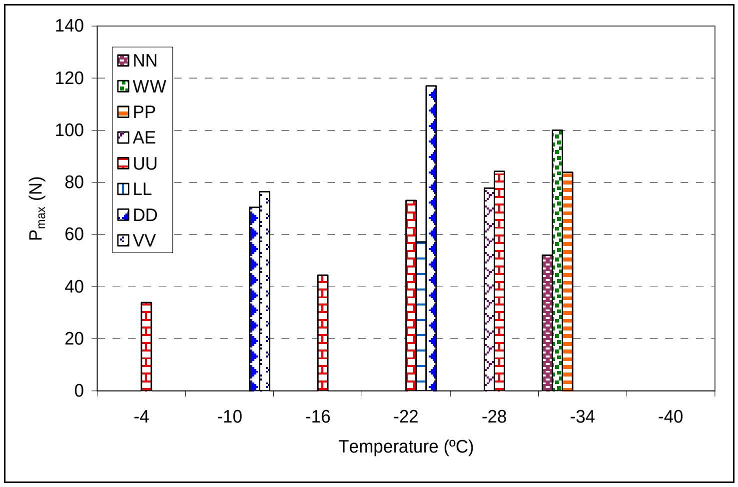
(a)



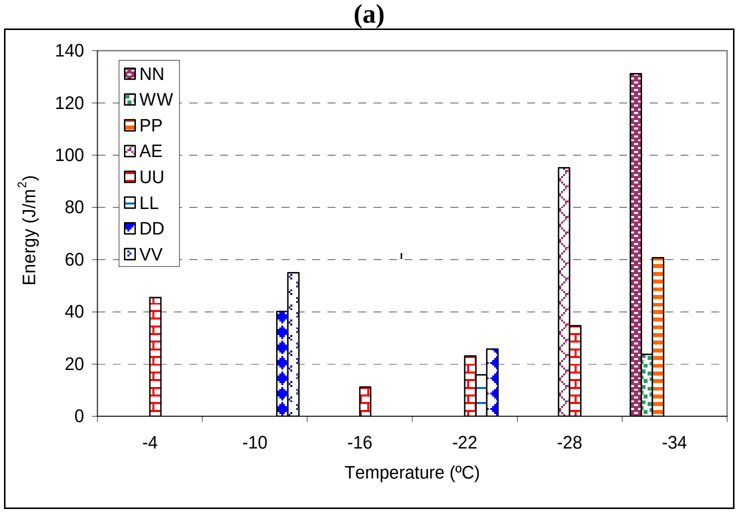
(b)

Figure 23. Work of Adhesion between Sealants and Limestone, Quartzite, Granite and Aluminum for Sealants a) UU, BB, AE, PP, AD, WW, MM; and b) DD, QQ, NN, VV, EE, ZZ, YY

Figure 24. Pulling End Piece Apart at a Constant Displacement Rate Using DT Device



(a)



(b)

Figure 25. Maximum Load and Energy Required to Break the Bond between Sealant and Aluminum at Various Temperatures

the eight sealants and aluminum. The maximum load showed better repeatability, and it was able to clearly distinguish among different sealant-aggregate pairs. Variation between operators and setups were checked, and no significant variations were found. The maximum load was selected as the test performance parameter (Al-Qadi et al., 2008a). Figure 26 presents this parameter for several pairs of sealant-aggregate. Using comparison between test results of laboratory-aged specimens and field data, a minimum 50N at tested temperature was selected as the performance threshold.

Interfacial Fracture Energy

The third test of the low-temperature adhesive properties of sealants is a fracture-type test that utilizes fracture mechanics to derive a fundamental property of the bond. The fundamental property is the interfacial fracture energy (IFE). A geometry-independent, pressure blister test was developed (Fini et al., 2007). The intrinsically stable interface debonding process makes this test attractive and allows calculation of fundamental properties of the interface (Gent and Lewandowski, 1987; Shirani and Liechti, 1998; Penn and Defex, 2002). The blister test, which

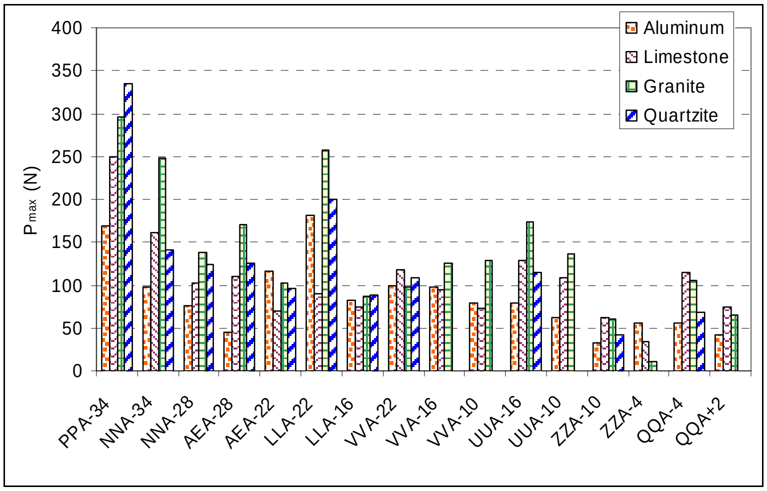


Figure 26. Maximum Load Measured for Bonding between Sealants and Substrates

is an enclosed system, exposes the interface to simultaneous loading and environmental condition.

In this test, a servo-hydraulic pump displaces a piston at a constant rate. The upward movement of the piston injects a liquid medium (alcohol) at a constant rate of 0.1L/hr through a channel that is connected to the specimen (Figure 27). The specimen is composed of an annular (donut-shaped) substrate plate (aggregate or a standard material) covered with binder or sealant on one side. Alcohol pushes the adhesive (binder or sealant) away from the substrate creating a blister which continues to grow until the adhesive separates from the substrate. The blister height and the pressure are recorded during the test and they are used to calculate the IFE. In simplified form, IFE can be calculated as half of the product of the maximum pressure and the corresponding blister height. In addition to IFE, adhesive modulus can be determined from this test using the test data before debonding occurs. In addition, residual stress developed at the interface during the sample preparation process can be obtained.

Figure 28 shows the IFE values for several sealant-substrates. It clearly shows that IFE can differentiate among sealants at different temperatures. The crack sealant blister test was

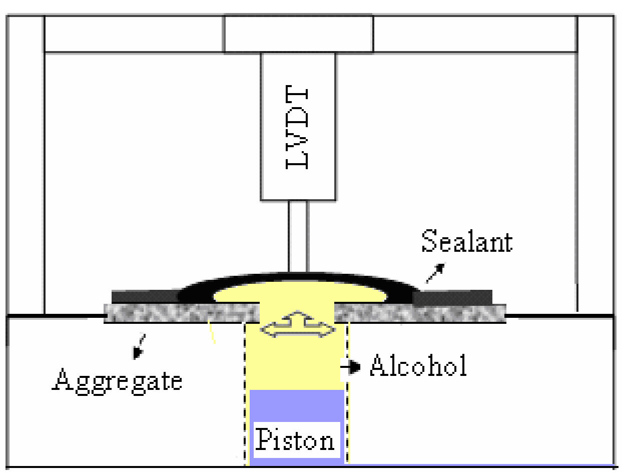
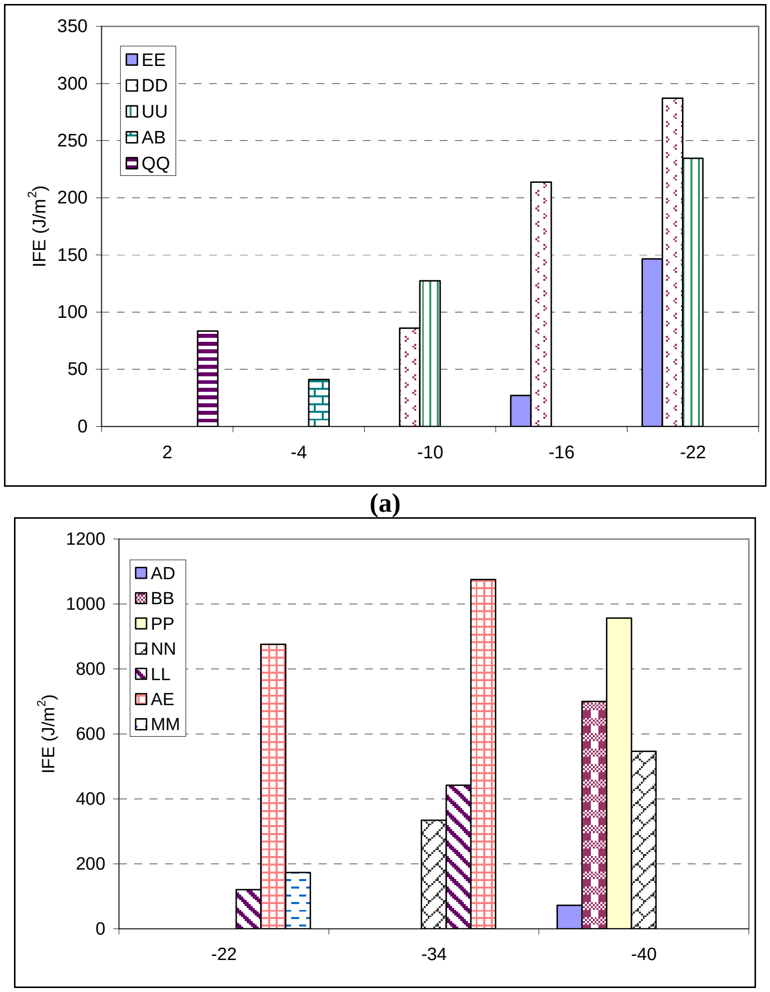


Figure 27. A Schematic of Blister Apparatus



(a)

(b)

Figure 28. IFE for 12 Sealants at Temperature Ranging from a) 2 to -22ºC; and b) -22 to -40ºC

further used to study the effect of sealant aging, temperature, loading rate, viscosity, and curing time on interface bonding. Al-Qadi et al. (2008a) found that aging significantly affects interface bonding; depending on the sealant property, IFE may increase/ decrease due to aging (Figure 29). In addition, within-laboratory variation was checked; and no significant difference was found between operators.

Sealant IFE strongly depends on temperature and loading rate; this dependence varies based on material condition (rubbery vs. glassy stages). The IFE of eight aged sealants and three binders were determined at various temperatures. A master curve was constructed for sealant UU which could be tested at the widest temperature range varying from -4 to -34ºC. It was found that when the adhesives are in their rubbery stage, the IFE increases as temperature decreases. However, in the glassy stage, the opposite trend was observed (Figure 30). This study concluded that an optimum interface bonding can be achieved at specific temperature and loading rate identified for each sealant. Knowing the expected loading rate in the field and using a loading rate-temperature master curve, an IFE range can be identified for a specific temperature to ensure acceptable bonding (Fini et al., 2008). In addition, testing results showed that high-viscosity sealants adhere better as long as substrate surface is adequately wetted (Al-

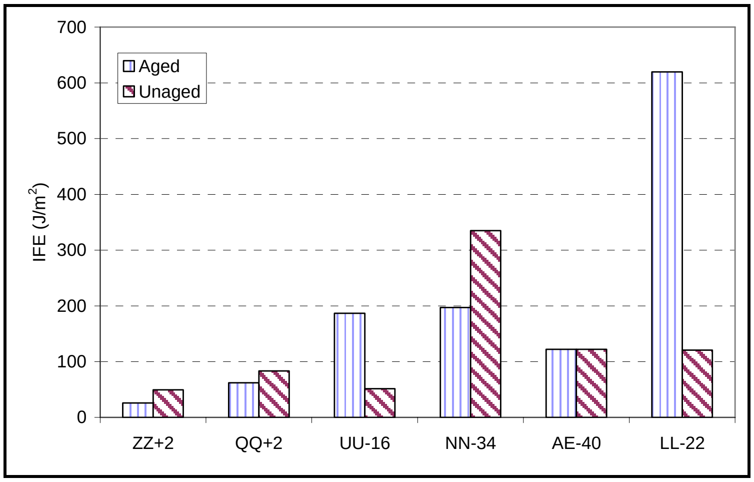


Figure 29. IFE for Aluminum with Aged and Non-Aged Sealants

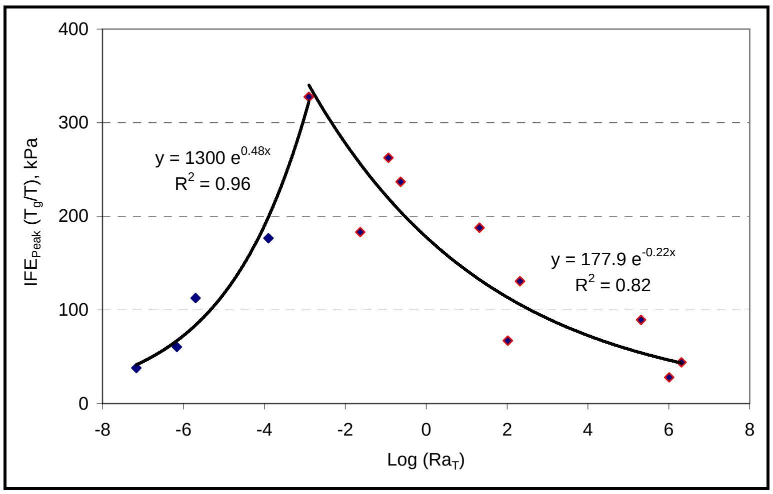


Figure 30. IFE versus Reduced Loading Rate for Sealant UU Bonded to Aluminum

Qadi et al., 2008e). Effect of curing time was also investigated; sealant cured for 24 hr at room temperature has significantly higher life than that cured for 1 hr (Al-Qadi et al., 2007).

SUMMARY AND CONCLUSIONS

New sealant tests were developed based on the performance of sealants tested in the field and on the characterization of other sealants widely used in North America. The newly developed procedures provide fundamental sealant properties that include apparent viscosity at the recommended installation temperature, vacuum oven aging to simulate sealant weathering in the field, a DSR test to assess sealant's tracking resistance at high service temperatures, the CSBBR test to evaluate sealant's creep properties at low temperatures, the CSDTT to characterize sealant's low temperature extendibility, and low temperature adhesive (surface energy, direct adhesion, and blister) tests to evaluate the bonding between sealant and its substrate.

Extensive laboratory testing, regular consultation with the project's 26-member technical advisory committee, and limited field testing support conclusions that were used to develop a draft set of performance criteria for crack sealants. Those conclusions and criteria are summarized as follow:

- An apparent viscosity at installation temperature of between 1 and 3.5 Pa.s provides for good crack filling while not being excessively fluid. *Note that this test is the only test performed on un-aged material.*

- Resistance to tracking at high service temperatures can be controlled using a minimum flow coefficient of 4 kPa.s and a shear thinning exponent of 0.7, as determined using a dynamic shear rheometer (DSR).

- Using the modified BBR test (CSBBR), a maximum stiffness at 240 s of 25 MPa and minimum average creep rate of 0.31 will promote good field performance of sealant materials that must withstand low-temperature service conditions.

- Extendibility, as measured with the CSDTT, is another good measure of expected low-temperature performance of crack sealants. The threshold for good expected performance is tied to the lowest application temperature (plus a 6°C shift), which is reported in Table 4.

- At this time, the direct adhesion test is best suited (of the three adhesion tests that were developed) for use in a practical performance-based guideline. A minimum load of 50 N at tested temperature coincides with good field performance for sealant adhesion.

RECOMMENDATIONS

A systematic process for the selection of hot-poured crack sealants is proposed. This process, including performance guidelines for crack sealant grading, is described in detail in Appendix B. The following recommendations relate to these newly developed guidelines.

1. *AASHTO representatives from the sponsoring member states of this pooled fund study (No. TPF-5[045]) should submit the developed tests and guidelines for consideration as provisional AASHTO specifications.*

2. *The Virginia Transportation Research Council in collaboration with the University of Illinois should pursue a second pooled funded research program to validate and implement the guidelines for selection of hot-poured crack sealants.*

COSTS AND BENEFITS ASSESSMENT

In this study, a systematic approach is proposed to help state agencies select more effective and durable crack sealing material. A study conducted by Ontario's Ministry of Transportation (MTO) during the 1970s to 1980s, which included several field test sections to investigate the influence of crack sealing on pavement distress and performance, shows that with effective crack sealing techniques, at least two years of service life extension can be added to flexible pavements (Chong, 1987). Ponniah and Kennepohl (1996) indicate that most of the premature sealant failure occurs after the first year of installation and is mainly due to unsuitable material and inappropriate installation techniques. In their study, the authors also show a cost-benefit ratio of 1.48 through effective sealing of pavement.

A recent survey on the pavement preventive maintenance programs (PPM) of 18 transportation agencies in North America shows that 13 transportation agencies have established PPM programs, and crack sealing is routinely used as treatment in the PPM program (AASHTO, 2006). The budgets for the PPM programs of each transportation agency are listed in Table 5. For example, the Commonwealth of Virginia spends approximately $20M/year on crack sealing (Liston, 2002). If sealant life is doubled through better selection procedures, then annual savings of $20M/year are possible. More conservatively, the cost-benefit multiplier from Ponniah and Kennepohl's work can be applied to the North American survey to determine a nation-wide estimate of savings. If each PPM program with a dedicated budget devoted only 10% to crack sealing, the total annual savings for these 18 states would be nearly $30M/year.

Table 5. Budget and Miles Covered by Pavement Preventive Maintenance Program for 18 Transportation Agencies in North America

Transportation Agency	PPM	Budget ($)	Miles cover by PPM
Alberta Transportation	Yes	12M	16875
Alaska DOT	Yes	100M	14500
Missouri DOT	No	Not dedicated	10000
Illinois DOT	Yes	50M	14292
South Dakota DOT	No	Not dedicated	7500
Washington DOT	Yes	25M	17800
Hawaii DOT	No	Not dedicated	1000
Idaho DOT	No	4M	12000
New York DOT	Yes	70M	28925
New Jersey DOT	Yes	60M	8500
Vermont DOT	Yes	10-20M	1600
Georgia DOT	Yes	89M	18000
Oregon DOT	No	Not dedicated	18000
Delaware DOT	Yes	40-45M	5700
Louisiana DOT	Yes	12M	12400
Iowa DOT	Yes	11M	9350
Michigan DOT	Yes	81M	1203
Rhode Island DOT	Yes	4M	1100

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APPENDIX A: BITUMINOUS-BASED CRACK SEALANT TYPES AND IDENTIFICATIONS

Sealant products used at University of Illinois were designated by a two-character code, which identifies the sealant type (Table A.1). Sealants with one character code are sealants installed in the field in Canada. In addition, three typical test results of the sealants used in this study based on current sealant specification (penetration at 25°C, flow test at 60°C and resilience at 25°C) were also reported.

Table A.1. Sealants Description and Designation

ID	Notes	Penetration	Flow	Resilience
		25°C (dmm)	60°C (mm)	25°C
QQ	Stiffest crack sealant	22	0	36
EE	Expected low temperature grade is -22°C	47	0	51
ZZ	Used in San Antonio, TX	42	N/A	N/A
YY	Used in San Antonio, TX	42	N/A	N/A
AB	Used in San Antonio, TX	40	N/A	23
VV	Modified with fiber	N/A	N/A	N/A
UU	Used by SHRP H106	62	1.5	N/A
AE	Widely used in NY, VA, and NH	N/A	N/A	N/A
DD	Expected low temperature grade is -34°C	80	1.5	50
MM	For aging study	120	1	70
WW	Field data available	N/A	N/A	N/A
NN	Field data available	75	0	70
AD	SHRP H106 field data available	N/A	1	80
PP	Field data	130	1	44
BB	Softest crack sealant	148	0	80
SS	For preliminary test	122	0.1	63
CC	Field data available	N/A	0	65
GG	For preliminary test	66	0	75
HH	SHRP H106 field data	N/A	0	44
A	Field data available	86	0.5	57
B	Field data available	68	0.5	64
C	Field data available	78	0	59
E	Field data available	124	1	73
G	Field data available	50	0.5	51
J	Field data available	66	6	48

APPENDIX B: PROPOSED PROCESS FOR THE SELECTION OF HOT-POURED CRACK SEALANTS

A systematic process for the selection of hot-poured crack sealant is described in Figure B.1., and the guidelines for crack sealant grading (SC) are presented in Table B.1. For example, SG 52-34 means the sealant can be used at a high service temperature of 52°C and low temperature of -34°C. The apparent viscosity test (SC-2) helps to ensure that sealant installation goes smoothly. The DSR test (SC-4) sets the sealant's high temperature grade to prevent tracking. If a sealant does not meet the performance criteria at a selected temperature, the test is repeated at a lower temperature until it does. At low temperature, the CSBBR (SC-5) and CSDTT (SC-6) tests predict cohesive performance of sealants. The direct adhesion test (SC-7) addresses the expected bond performance. The sealant is first tested using CSBBR and CSDTT tests at 6°C higher than its lowest service temperature to examine its low temperature cohesive character. If the sealant's cohesive property passes, it is tested using the direct adhesion test to predict its bond strength at the same test temperature as CSBBR and CSDTT. The low temperature grading is determined if the sealant passes the three low temperature tests. If the sealant does not pass the bond test but the extendibility is still appropriate at a lower grade, then the low temperature grade is determined by the cohesive tests (CSBBR and CSDTT). Otherwise, the sealant is rejected for use at the testing temperature. The low temperature cohesive grade is selected at 6°C below the low testing temperature. The reliability approach used by SuperPave™ may be applied.

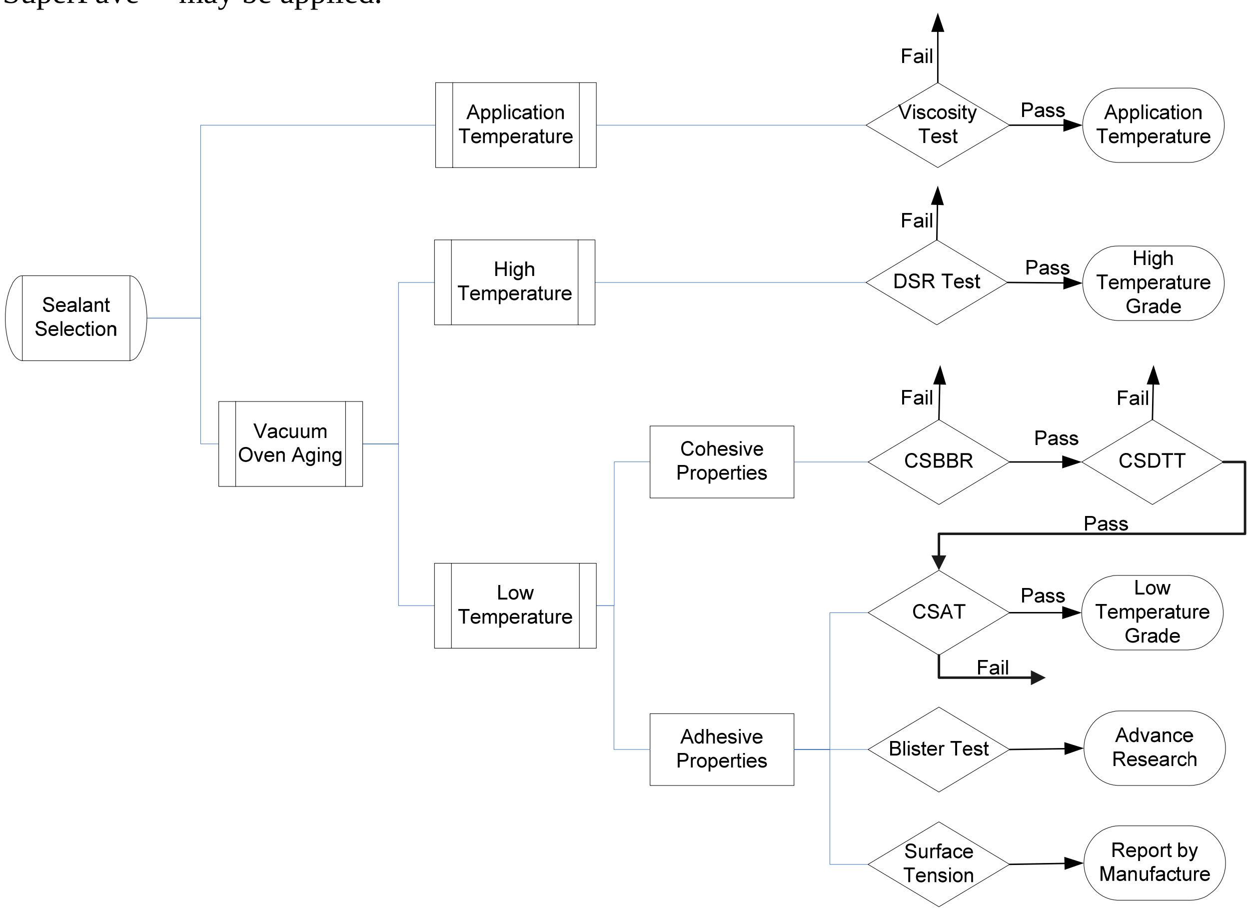

Figure B.1. Process for the Selection of Bituminous-Based Sealants

Table B.1. Crack Sealant Performance Grade

<table>
<tr><td rowspan="2">Crack Sealant Performance Grade</td><td colspan="7">SG 46</td><td colspan="7">SG 52</td><td colspan="7">SG 58</td><td colspan="7">SG 64</td><td colspan="7">SG 70</td><td colspan="7">SG 76</td><td colspan="7">SG 82</td></tr>
<tr><td>-46</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-46</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-46</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-46</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-46</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-46</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-46</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td></tr>
<tr><td>Apparent Viscosity, SC-2</td><td colspan="49">Installation Temperature</td></tr>
<tr><td>Maximum Viscosity (Pa.s)</td><td colspan="49">3.5</td></tr>
<tr><td>Minimum Viscosity (Pa.s)</td><td colspan="49">1</td></tr>
<tr><td colspan="50">Vacuum Oven Residue (SC-3)</td></tr>
<tr><td>Dynamic Shear, SC-4</td><td colspan="7">46</td><td colspan="7">52</td><td colspan="7">58</td><td colspan="7">64</td><td colspan="7">70</td><td colspan="7">76</td><td colspan="7">82</td></tr>
<tr><td>Minimum Flow Coeff. (kPa.s)</td><td colspan="49">4</td></tr>
<tr><td>Minimum Shear Thinning</td><td colspan="49">0.7</td></tr>
<tr><td>Crack Sealant BBR, SC-5</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td></tr>
<tr><td>Maximum Stiffness (MPa)</td><td colspan="49">25</td></tr>
<tr><td>Minimum Avg. Creep Rate</td><td colspan="49">0.31</td></tr>
<tr><td>Crack Sealant DTT, SC-6</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td></tr>
<tr><td>Minimum Extendibility (%)</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td></tr>
<tr><td>Crack Sealant AT, SC-7</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td></tr>
<tr><td>Minimum Load (N)</td><td colspan="49">50</td></tr>
</table>

Note: Crack sealant surface energy is provided by manufacturer.

FINAL CONTRACT REPORT - Draft

DEVELOPMENT OF PERFORMANCE-BASED GUIDELINES FOR SELECTION OF BITUMINOUS-BASED HOT-POURED PAVEMENT CRACK SEALANT: AN EXECUTIVE SUMMARY REPORT

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Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

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NOTICE

The project that is the subject of this report was completed under contract for the Virginia Department of Transportation, Virginia Transportation Research Council. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Virginia Department of Transportation, the Commonwealth Transportation Board, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Each contract report is peer reviewed and accepted for publication by Research Council staff with expertise in related technical areas. Final editing and proofreading of the report are performed by the contractor.

ABSTRACT

This report summarizes research presented in separate reports on a systematic process developed to help users select appropriate bituminous hot-poured sealants for pavement cracks and joints. The following reports are summarized herein: *Apparent Viscosity Test for Hot-Poured Crack Sealants*, *Development of a Short-Term Aging Test and Low-Temperature Testing Bibliography*, *Sealant Flow and Deformation by Dynamic Shear Rheometry in Summer Temperatures*, *Characterization of Low Temperature Creep Properties of Crack Sealants Using Crack Sealant Bending Beam Rheometry*, *Characterization of Low Temperature Mechanical Properties of Crack Sealants Using Crack Sealant Direct Tension Test*, and *Development of Adhesion Tests for Crack Sealants at Low Temperature.* As a result of this cumulative research, this report introduces a set of tests and performance parameters for sealant at installation and service temperatures, an aging procedure to simulate sealant weathering, and most importantly, a simplified chart with thresholds for all performance parameters for the straightforward selection of crack sealant.

TABLE OF CONTENTS

DEVELOPMENT OF PERFORMANCE-BASED GUIDELINES FOR SELECTION OF BITUMINOUS-BASED HOT-POURED PAVEMENT CRACK SEALANT: AN EXECUTIVE SUMMARY REPORT
(Draft Document – May 2008))

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INTRODUCTION

ASTM standard D5535 defines a sealant as a material that possesses both adhesive and cohesive properties to form a seal, which prevents liquid and solid from penetrating into the pavement system. Crack sealing has been widely accepted as a routine preventative maintenance practice. Given a proper installation is achieved, crack sealant can extend pavement service life by a period ranging from three to five years (Chong and Phang, 1987). Numerous studies also demonstrated the cost effectiveness of crack sealants (Joseph, 1990; Cuelho et al., 2002, 2003; Fang et al., 2003; Ward, 2001; Chong and Phang, 1987; Chong, 1990).

Crack sealant is produced so that it keeps its shape as applied and hardens through chemical and/or physical processes to form a viscoelastic rubber-like material that withstands extension or compression (crack movement) and weathering (Al-Qadi et al., 2007). However, in many cases, premature failure of crack sealants may be observed in one of the following scenarios. During the sealant installation, if the viscosity of the sealant is too high, sealant might not be able to fill the crack properly; hence, it will affect the interface bonding between sealant and pavement substrate. If the viscosity is too low, sealant might flow out from the cracks.

In the field, a sealant extends at low temperature and compresses at high temperature to accommodate pavement crack openings which increase with decreasing temperature and decreases with rising temperature. At high service temperature, sealant might fail due to pull out from the crack by tire passing. At low service temperature, the crack opening may increase from 10% to more than 90% depending on the environmental location; hence, one of the two mechanisms might be observed: cohesive or adhesive failure. The former occurs in the sealant, while the latter occurs at the sealant-pavement crack wall interface. At low temperature, sealant becomes more brittle due to physical hardening; and is subjected to short-duration loading due to crack movements associated with stick-slip motions and truck trafficking as well as long periods of environmental loading.

In order to achieve a cost-effective crack sealing/filling operation and proper field performance, two factors must be closely controlled: quality of sealant installation and sealant mechanical and rheological properties (such as viscosity, bulk stiffness, and adhesive bonding).

Regardless of sealant quality, improper installation will cause premature failure and, hence, reduced sealant service life.

Standards and specifications for selecting crack sealant have been established by several organizations, including American Society for Testing and Materials (ASTM), American Association of State Highway and Transportation Officials (AASHTO), and U.S. and Canadian federal, state, provincial, and municipal agencies. The objective of the specifications is to select materials that have the necessary properties to perform adequately in the field. However, these specifications are generally empirical and do not measure sealant fundamental properties. Hot-poured bituminous crack sealants are typically selected based on standard empirical tests such as penetration, resilience, flow, and bond to cement concrete briquettes (ASTM D6690). ASTM Standard D5329-04 (Standard Test Methods for Sealants and Fillers, Hot-Applied, for Joints and Cracks in Asphaltic and Portland Cement Concrete Pavements) summarizes most of these tests. These include non-immersed cone penetration, fuel-immersed cone penetration, the flow test, the non-immersed bond test, the water-immersed bond test, the fuel-immersed bond test, the resilience test, the oven-aged resilience test, the asphalt compatibility test, the artificial weathering test, the tensile adhesion test, the solubility test, and the flexibility test.

These tests are used by most state highway agencies in selecting their crack sealing materials; but the specification limits may vary from one state to another. These differences create difficulties for crack sealant suppliers because many states with the same environmental conditions specify different limits for the measured properties. These tests were also reported to poorly characterize the rheological properties of bituminous-based crack sealants and to predict sealant performance in the field.

PURPOSE AND SCOPE

Researchers have widely reported that current specifications for selection of hot-poured crack sealants are based on tests whose results showed no correlation with field performance (Masson, 2000; Belangie and Anderson, 1985; Masson and Lacasse, 1999; Smith and Romine, 1993; 1999). In addition, over the past two decades, a new generation of highly modified crack sealants has been introduced to the market (Zanzotto, 1996). These sealants exhibit quite complex behavior compared to those of traditional sealant materials (Belangie and Anderson, 1985). This necessitates the development of a new set of specifications.

The most effective way to evaluate the performance of crack sealants would be to perform field tests. However, the results from field tests are sometimes controversial because a sealant can perform well in one site and fail in another simply because of differences in environmental conditions. Therefore, the main objectives of this project were to develop laboratory tests that measure bituminous-based crack sealants rheological properties and to develop performance-based guidelines for the selection of hot-poured crack sealants. Meeting these objectives requires the development of new tests to measure the rheological properties of hot-poured crack sealants over a wide range of service temperatures. The developed tests need to be practical, repeatable and reproducible. Thresholds for each test should be identified to ensure desirable sealant field performance. A special effort was given to make use of the equipment originally developed during the five-year Strategic Highway Research Program (SHRP), which were used to measure binder rheological behavior as part of the performance grading (PG) system.

This executive summary report introduces a systematic process to help users select appropriate bituminous hot-poured crack sealants. This document summarizes research that is presented in separate reports on the methods developed for this process (Al-Qadi et al., 2008a, b, c, d). The process includes a new set of tests and performance parameters for sealant at installation and service temperatures. It also proposes a new aging procedure to simulate sealant weathering.

METHODS

To develop performance-based guidelines for the selection of hot-poured crack sealants that meet the aforementioned requirements and minimize the cost of possessing new testing equipment, the research group made use of the SuperPave™ binder performance grading (PG) equipment. Modifications to the existing viscosity test, bending beam rheometer, and direct tension test devices, specimen size and preparation, and testing procedures were made to accommodate the testing of crack sealants. In addition, new tests for sealant aging and sealant evaluation at high service temperatures were introduced. Upon the completion of test validation, test measured performance parameters were recommended for implementation as part of the newly developed "Sealant Grade" (SG) system. The appendix briefly catalogues the sealants that were used in the extensive laboratory and field tests as well as their ASTM testing results that supported this research.

Apparent Viscosity

Sealant viscosity is among the parameters that affect initial bonding. Therefore, applying a sealant at the appropriate viscosity provides for better crack filling and enhances interface bonding. Several factors affect the measured viscosity of hot-poured crack sealant. Therefore, it is essential to identify the material characteristics that influence the rheological behavior of hot-poured crack sealant at installation. These characteristics need to be set at reasonable limits, to simulate field installation as closely as possible. While standard tests to examine sealant consistency exist, these standard tests have not been proven to predict field performance. As part of an effort to bridge the gap between sealant fundamental properties and field performance, a test procedure was developed to measure apparent sealant viscosity using the same rotational viscometer equipment used in the SuperPave™ PG system. The development of this procedure is described in detail in a supporting document (Al-Qadi et al., 2008b). The procedure for measuring apparent viscosity is summarized in this report under "Findings and Discussion."

Sealant Aging

For an aging test to be effective, it must quickly provide an aging as close as possible to reality. Figure 1 illustrates the basic process. To this effect, true aging was determined from the physico-chemical analysis of 12 sealants weathered in Montreal, Canada, for nine years (Table

1). As expected, sealants with good performance contain components resistant to weathering, whereas sealants with poor performance oxidize quickly. Figure 2 shows an example of sealant stiffening due to weathering.

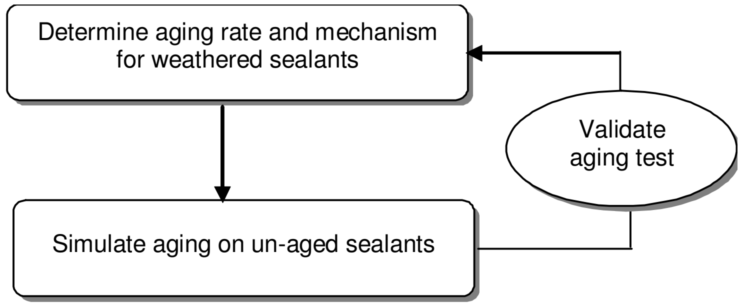


Figure 1 Schematic of Aging Procedure

Because of sealant's complex mixture, each sealant shows unique aging characteristics. To mimic the effect of weathering on sealants, several accelerated aging methods were compared (alone or in combination) after various aging periods and temperatures, including small-kettle aging, microwave aging, pressure aging, oven aging, and vacuum oven aging. The results of physico-chemical analysis of sealants weathered in the field were compared to those of sealants aged quickly in the laboratory (Masson et al., 2003).

Table 1 Physico-chemical Method to Characterize Crack Sealant Aging

Method*	Output	Use
GPC	Separation of bitumen and polymer	Quantify polymer; degradation rate and mechanism
FTIR	Fingerprint of composition	Oxidation; identification of polymer and filler; semi-quantitative analysis; degradation mechanism
TG	Weight loss upon heating	Contents of filler and light, medium, and heavy hydrocarbon components
DSR	Stiffness, relaxation	Effect of temperature and aging on mechanical properties

*** GPC: Gel permeation chromatography; FTIR: Fourier transform infrared spectroscopy; TG: Thermogravimetric analysis; and DSR: Dynamic shear rheometry (DSR).**

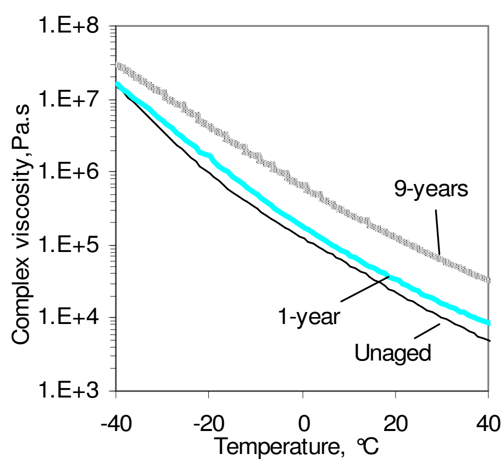


Figure 2 Complex Viscosity Increase for a Field-Aged Sealant at One and Nine Years

Sealant Flow and Deformation

Bituminous sealants applied to cracked pavements sometimes fail due to deformation under the combined action of shear stresses and high service temperatures (Masson et al., 2007). In an attempt to define performance parameters, 21 sealants were tested with a dynamic shear rheometer (DSR) and subjected to increasing stresses at temperatures between 46℃ and 82℃. These conditions were meant to mimic the effects of various traffic levels and maximum temperature in the field.

Flexural Creep

The bending beam rheometer (BBR) is used in most pavement laboratories nowadays to measure binder stiffness at low temperature. A modified BBR test, a crack sealant bending beam rheometer (CSBBR), was introduced to measure the flexural creep of crack sealant at temperatures as low as -40°C. The development of this procedure is described in detail in a supporting document (Al-Qadi et al, 2008d). The resulting procedure is summarized later in this report.

Low Temperature Tensile Properties

Four typical types of stress-strain curves of crack sealant are shown in Figure 3. Depending on sealant composition and test temperature, sealants may behave as a brittle plastic for which the stress-strain curve is linear up to fracture with little percentage elongation (Curve A). Low-polymer and high-crumb-rubber-modified sealants behave this way. A brittle-ductile failure may be observed for crack sealants as well. When the tensile load reaches a maximum, sealant may fracture as shown in curve B, or the specimen may continue to stretch after the maximum load as shown in curve C (Figure 3). Sealant may also experience ductile failure (curves D and E). Typically, this type of sealant exhibits a yield point, followed by extensive elongation at a constant stress. This is referred to as a plastic flow region, and is clearly a region of nonlinear viscoelasticity. After the plastic flow region, sealant might exhibit strain hardening. This type of sealant usually has a relatively high polymer concentration.

To investigate whether a sealant can survive in a particular service conditions, the SuperPave™ Direct Tension Test (DTT) was considered and modified for crack sealants. The development of crack sealant direct tension test (CSDTT) is described in detail in a supporting document (Al-Qadi et al, 2008-c). The resulting procedure is summarized later in this report.

Adhesive Properties at Low Temperature

The adhesion capability of hot-poured bituminous sealants is usually evaluated using a standard test of an empirical nature (ASTM D5329). There is, however, no indication that the results of this test pertain to field performance. In addition, this test examines adhesion of sealant to Portland cement concrete, and the test result does not account for aggregate composition, which is the main component of HMA. Therefore, a reliable test method, which is based on sealant rheology, accounts for aggregate composition, and correlates with field performance, is urgently needed. This study proposes three laboratory tests to predict interface bonding of crack sealant to aggregate at service temperatures ranging from -4ºC to -40ºC. The three tests are designed to address the needs of manufacturers, transportation organizations, including contractors, transportation agencies and consultants, and researchers, respectively.

The first laboratory approach addresses the compatibility of sealant with a specific substrate, by measuring the free energy of the bond, *work of adhesion*. The second test makes use of the *direct tension* test (DTT) device. The third test is a fracture type test that utilizes a fracture mechanics approach to derive a fundamental property of the bond, *interfacial fracture energy* (IFE).

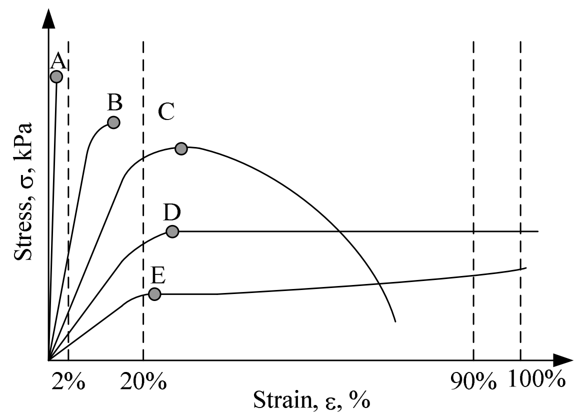


Figure 3 Stress-Strain Behavior of Crack Sealant Observed in the Direct Tension Test

RESULTS AND DISCUSSION

Apparent Viscosity Testing Procedure

Numerous factors affect the measured viscosity of hot-poured crack sealant. Due to the relatively high polymer content, sealant responses to changes in temperature and loading can be quite complex. Since hot-poured crack sealants behave as non-Newtonian fluids, variation in the experimental parameters can affect the measured values; hence, a test setup and testing parameters were identified (Al-Qadi et al., 2006; 2008b). Laboratory conditions should simulate sealant installation conditions as closely as possible. A critical issue was the shear rate imposed on the material during application. It was determined that a spindle speed ranging from 115rpm to 5536rpm should be used to simulate the shearing of the sealant as it enters the crack during installation. However, a significant reduction in the shear rate may occur as the sealant exits the applicator wand, due to the sharp temperature drop as well as the high friction with the crack walls.

Although the Brookfield Thermosel system (as adopted from SuperPave™) is not a high-shear rheometer (its maximum allowable spindle speed is 250rpm), it was found to be sufficient for the sealant testing. After extensive testing, a Brookfield rotational viscometer was adopted;

modification of the test procedure and equipment was implemented (Figure 4). SC4-27 spindle at a speed of 60rpm (shear rate of $20.4s^{-1}$) at the recommended installation temperature is used. The spindle is attached to a newly developed rigid rod. The rod is a replacement for the current hook; it prevents a rubber particle from disturbing the spindle rotation which results in better test repeatability. A conditioning time of 20min and a waiting time of 30s before collecting data are also recommended to ensure that the measured viscosity has stabilized.

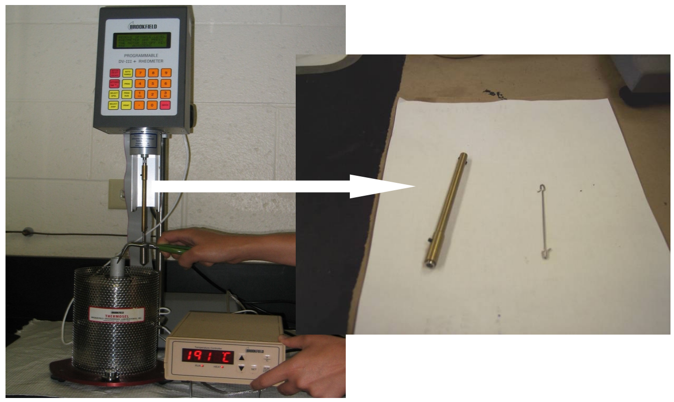


Figure 4 Brookfield Thermosel System and Rigid Rod Used for Crack Sealant Testing Compared to the Rod Used for Asphalt Binder

In this test, sealants are cut in small pieces and placed directly in an aluminum chamber, then sealant was melted inside the chamber. Cutting sealant without melting it improved test results significantly. This measured apparent viscosity is expected to be an acceptable indication of the sealant's rheological behavior at installation temperature, assuming that the suggested procedure and equipment are used. Fifteen virgin sealants were tested in accordance with the developed testing procedure; the apparent viscosity of several sealants at various temperatures is presented in Figure 5.

Seven laboratories conducted a round-robin test. The repeatability of the measured apparent viscosities was determined through statistical analysis. The average coefficients of variation within and between laboratories were found to be 1.6% and 6%, respectively. Maximum permissible differences within a laboratory and between laboratories are 5.4% (among the best three readings out of four) and 17% (between the test conducted in two different laboratories), respectively. These values are comparable to those of asphalt binder: 3.5% and 14.5% based on ASTM D4402-02 and 3.5% and 12.1% based on AASHTO 2006 T316, respectively. Because viscosity plays an essential role in predicting field performance of hot-poured crack sealant, upper and lower viscosity limits are recommended. An upper limit of 3.5Pa.s ensures that sealant is liquid enough to pour, whereas a lower limit of 1Pa.s controls the potential of using excessively fluid sealant. Hence, the sealant apparent viscosity should be between 1.0 and 3.5Pa.s when measured at recommended installation temperature.

Sealant Aging

The effect of hot-poured crack sealant oxidation and the change in polymer molecular weight on the sealant complex viscosity between -40°C and 40°C served as the two parameters in examining the applicability of pressure aging method (method 1) and vacuum oven method (method 2) [Figure 6]. Microwave heating was found to mimic the aging of sealants that contain mineral filler; but not other sealants. The microwave method thus lacked general application. Pressure aging was also found to be inappropriate because it often led to insufficient bitumen oxidation, and excessive thermo-degradation of the polymer. Vacuum oven aging proved to be the most appropriate method to simulate sealant weathering. In this method, sealants are cut into slices and placed on a stainless steel pan; each pan contains 35g of sealant. The pan is transferred into a conventional temperature controlled oven which is maintained at 180°C for approximately 5min to allow sealant to melt and form a film. The sealant is then removed from the oven and cooled to room temperature. Once it cools, sealant is placed in a vacuum oven preheated at 115°C for 16hr. After 16hr, the vacuum is released and sealant is placed in a conventional oven at 180°C for 5min or until the sealant is fluid enough to pour.

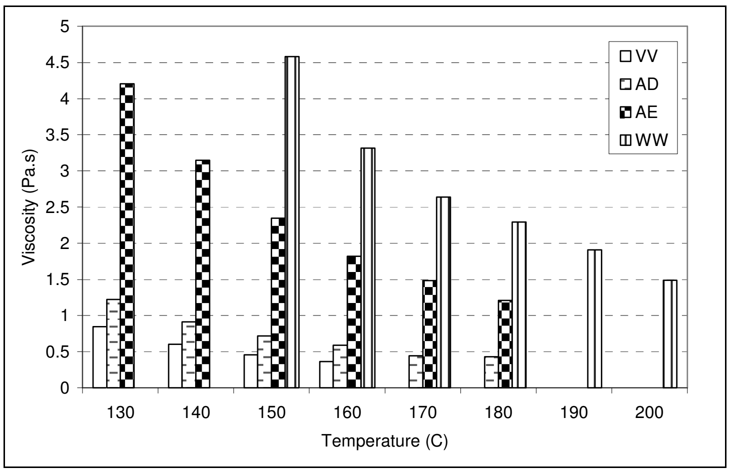


Figure 5 Apparent Viscosity of Several Sealants at Various Temperatures
Weathering and Accelerated Aging of Hot-Poured Bituminous Sealants

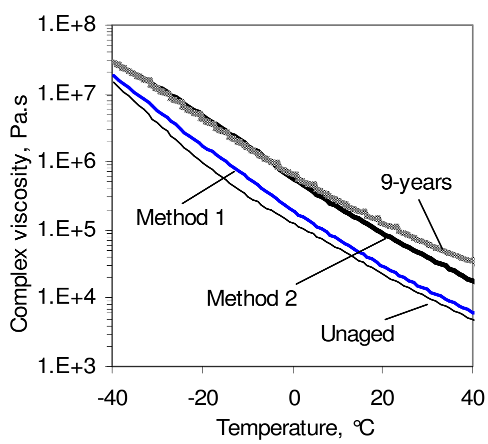


Figure 6 Complex Viscosity of a Field-Aged Sealant Compared to that after Accelerated Aging

Sealant Flow and Deformation by Dynamic Shear Rheometry

Plots of low shear viscosity (η_L) versus shear rate ($\dot{\gamma}$), Equation 1, indicated that many sealants were susceptible to shear thinning. Their apparent viscosity decreased with an increase in shear rate and/or temperature (Figure 7). This indicated that at high service temperatures, high traffic loads or volumes would affect the extent of sealant flow when it is under stress.

$$\sigma = C\,\dot{\gamma}^{P} \tag{1}$$

where,

$\sigma =$	stress;
$\dot{\gamma} =$	shear rate;
$C =$	flow coefficient; and
$P =$	shear-thinning coefficient

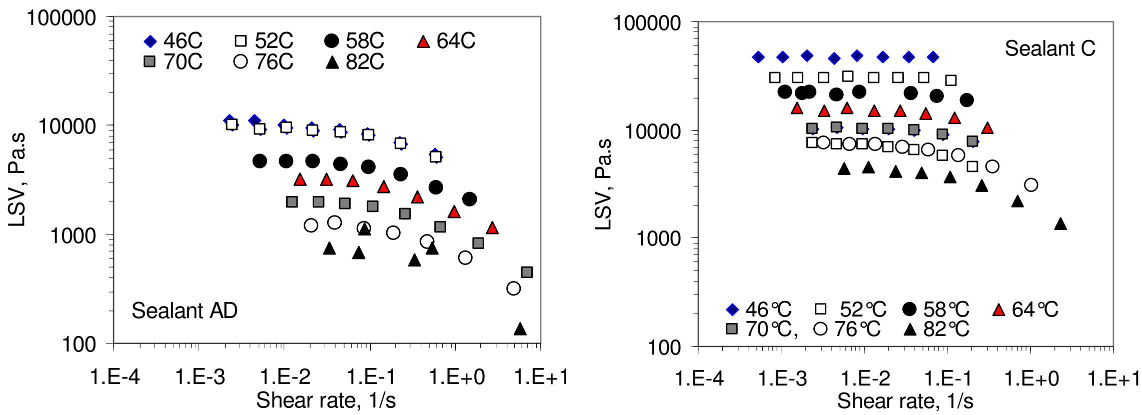


Figure 7 Low Shear Viscosity as a Function of Shear Rate; The Stress Doubles for Each Point from Left to Right

Plots of η_L vs $\dot{\gamma}$ were interpreted based on the Ostwald power law model. This model provides two parameters: a flow coefficient (C) and a shear-thinning coefficient (P). These coefficients correlated well with sealant pseudo-field performance as measured by tracking (Collins et al., 2007). Figure 8 shows the relationship between these factors and performance during the pseudo-field test. The solid markers indicate the sealants that did not fail during the pseudo-field test, and the open markers show those that failed. The semi-log scale in Figure 8 serves to highlight the high-failure regions (open markers). Limiting values for P and C can be established to limit the risk of sealant failure. Each pair of C and P represents performance criteria.

In the absence of effective limiting criteria (area A in Figure 8), the sealant failure rate due to tracking is 39% (Table 2). As C and P limits are raised, the risk of tracking failure is reduced. Area E defines the limits within which no tracking failure is observed. With such demanding criteria, 33% of the sealants are above the pass limits. Any limits in C and P can be used to define the level of sealant performance, but the most appropriate performance criteria may be that defined by area D, where limits of C = 4000Pa.s and P = 0.70 provide for a failure risk of only 3% and a sealant acceptance rate greater than 50%. The other limits have greater acceptance rates, but the risk of failure is disproportionately higher (Table 2).

Table 2 Lower Limits for C and P and their Relationship with Tracking Performance

Area[a]	C (Pa.s)	P	Passing[b]	Tracking[c]
A	300	0.46	145 (99%)	39 % failure
B	1000	0.64	123 (84%)	24% failure
C	2500	0.64	98 (66%)	14% failure
D	4000	0.70	76 (52%)	3% failures
E	10000	0.80	49 (33%)	No failures

[a]Area in Figure 8.

[b]From DSR: Number of samples above C and P limits out of a total of 147.

[c]From the pseudo-field test: ratio empty/all marker within the given plot area.

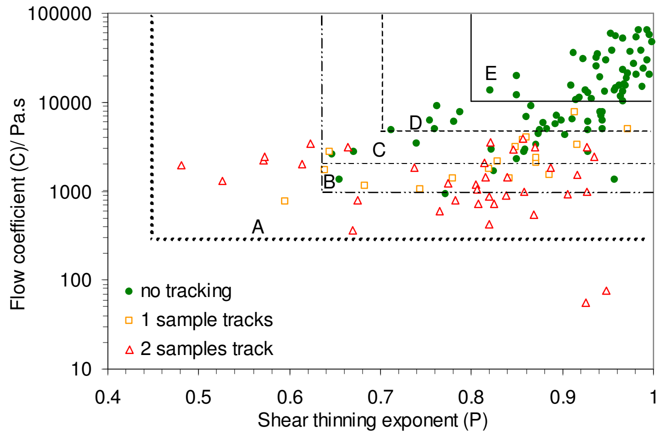


Figure 8 Semi-Log Plot of the Ostwald Parameters and Possible Performance Limits, Areas A to D

Characterization of Low Temperature Mechanical Properties Using Modified Bending Beam Rheometry

The principle of the crack sealant bending beam rheometer (CSBBR) test is applying a constant load of 980mN (100g) to a sealant beam and then measuring the beam deflection. The crack sealant is a much softer material compared to asphalt binder; therefore, excessive deflection was encountered during the testing of some sealants with the SuperPave™ BBR device. Consequently, several modifications were made to the SuperPave™ BBR test. First, the specimen thickness was doubled to overcome the excessive deflection. Second, the device was modified to accommodate the new specimen geometry. Additionally, a modified testing procedure, a new aging procedure, a validated testing period, and stiffness determination time were introduced. Linearity verification was conducted in order to verify that bituminous crack sealants behave as linear viscoelastic materials within CSBBR testing range.

The CSBBR beam thickness was doubled from 6.35 to 12.7mm. However, because of the increase in the beam's thickness, the deflection at the center of the beam due to shear would increase. The analysis shows that the center deflection contributed by shear force is only 4%, which was deemed acceptable. Figure 9 shows the softest tested sealant loaded with 980mN and resulted in final deflection of 3.2mm after 240s of loading (Al-Qadi et al., 2005). To adjust the device to accommodate the newly developed specimen geometry, the specimen support and calibration kits of the BBR were modified, as shown in Figure 10. The new design specimen supports are 6.35mm shorter than the SuperPave™ BBR specimen supports and can easily be replaced. The calibration kits of the system were also modified as shown in Figure 11. The compliance beam for the crack sealant test was modified by adding two footers at each end of the beam (Al-Qadi et al., 2008d).

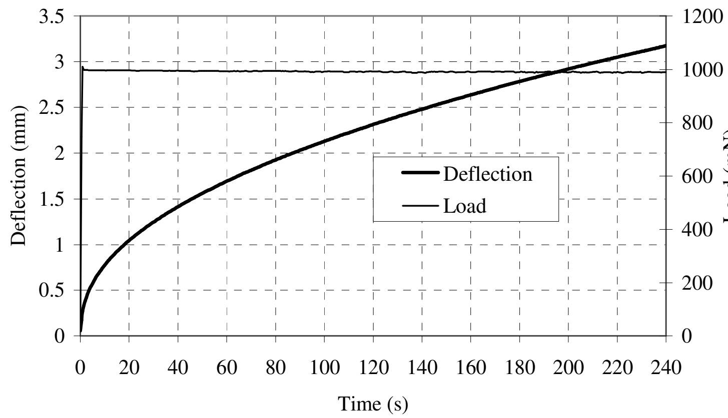
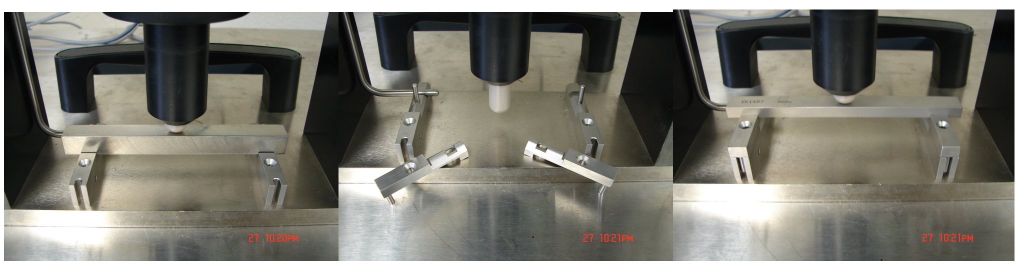


Figure 9 Deflection and Load versus Time for Sealant BB Using Beam Thickness 12.7mm at -40°C



(a) (b) (c)

Figure 10 (a) SuperPave™ BBR Specimen Supports; (b) Crack Sealant BBR Specimen Supports; and (c) Modified Specimen Supports

Figure 11 Calibration Beam: In the Front, Compliance Beam for SuperPave™ BBR Test and in the Back, Modified Compliance Beam for Crack Sealant BBR Test

The test procedure modification was completed in two parts: the silicon-based release agent was used to replace the Mylar strip because the Mylar strip melted at sealant pouring temperature; second, each specimen was poured from an individual container which has the same weight. Because bituminous-based sealants are composed by asphalt binder, SBS copolymer, crumb rubber, and various additives, the variability between specimens is high. By controlling the pouring weight of each specimen, the test repeatability was greatly improved (Al-Qadi et al., 2006). Prior to pouring specimens into the molds, sealant was aged in accordance with the aging method developed in this study.

The assumption that crack sealant follow linear viscoelastic behavior was verified (Elseifi et al., 2006). Test results indicated that stiffness was independent of the applied stress level, as shown in Figure 12; three levels of loading (250mN, 490mN, and 980mN) were applied on sealant specimens. The second condition of linearity, the experimental deflection at time (t) and the recovery deflection at time 240s + (t) should be equal or within a 5% difference could not be verified experimentally due to the sealant's softening behavior. Therefore, finite element (FE) was used to investigate the second condition of linearity. Figure 13 shows that the second condition of linear viscoelasticity can be verified.

The viscoelastic model, Prony series, was fitted to the experimental data to obtain Prony series parameters. Prony series expansion was found to be adequate in describing the mechanical behavior of crack sealants at low temperature (Elseifi et al., 2006). Fitting parameters were then incorporated into a three-dimensional FE model of the CSBBR specimen. The resulting calculated creep deflections agreed with the measured values.

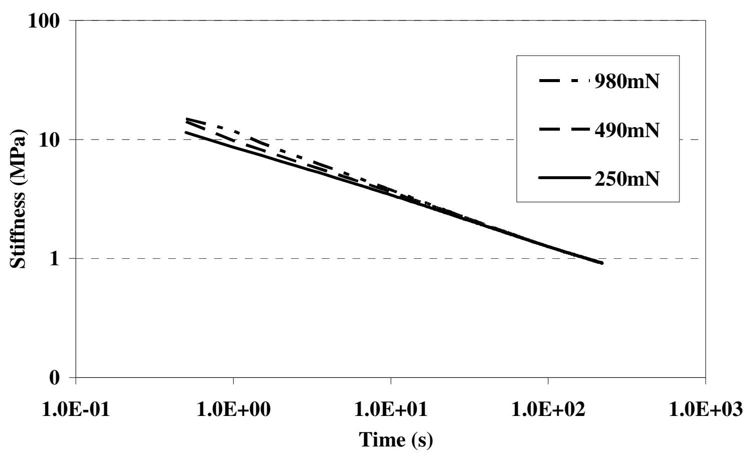


Figure 12 Measurements of Creep Stiffness for Sealant NN at -40°C

Fifteen sealants were tested at temperatures ranging from -4°C to -40°C. In this test, 35g of sealant is first heated at its recommended pouring temperature and then poured into an aluminum assembled mold. A rectangular sealant beam is cast with dimension of 12.7mm in height, 12.7mm in width, and 102mm in length. The beam is then placed in a fluid environmental chamber and the specimen is placed on a two-point support and subjected to a point creep loading. The specimen is exposed to a creep loading for 240s then followed by 480s unloading. The load and deflection of the sealant beam is recorded during the period of loading and unloading.

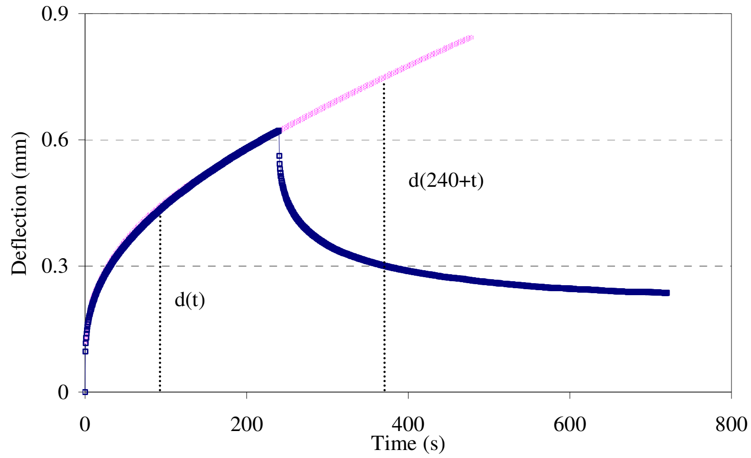


Figure 13 Schematic Diagram Illustrates the Extrapolated Data after 240s, Using FE

Each sealant was tested at three temperatures. These testing results were used to develop performance parameters. The selected performance parameters had to satisfy these four criteria: ability to describe the sealant's rheological behavior, ease of measurement and calculation, repeatability, and correlation with field performance. In addition, it was found that the critical loading time for crack sealant material at low temperature is after 5hr of loading. If the temperature superposition principle is applied, the stiffness at 240s for a given temperature can be used to predict the stiffness after 5hrs of loading at a temperature of approximately 12°C greater. Given the variation in sealant response to temperature change, 6°C shift is deemed appropriate. The stiffness at 240s (Figure 14), the average creep rate (Figure 15) and dissipated energy ratio (Figure 16) were the performance parameters selected to distinguish between sealants. These new tests are repeatable, and the coefficient of variation between operators is less than 4%. However, there is a difference in measured values was noted when using devices from different manufacturers (Al-Qadi et al., 2007).

Five sealants that were previously installed in Montreal as part a long-term field performance evaluation survey were tested using CSBBR. The results were used to establish the selection criteria for the CSBBR test. The test results recommend two performance criteria for application: stiffness at 240s and average creep rate. The recommended thresholds, which are temperature independent, for the two criteria are a maximum stiffness of 25MPa and minimum average creep rate 0.31, respectively.

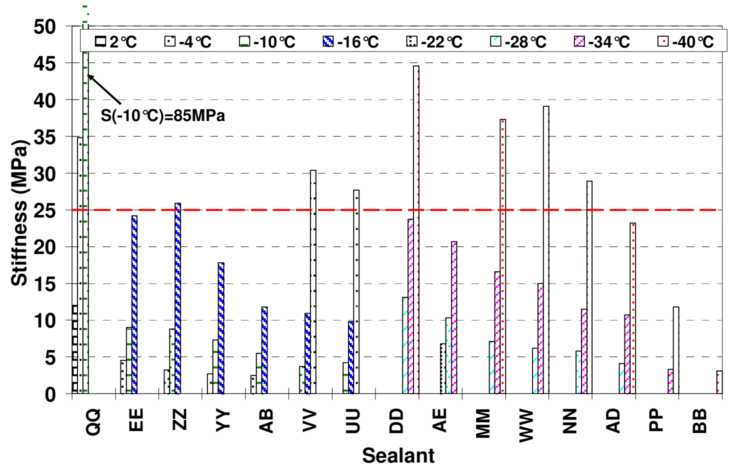


Figure 14 Stiffness at 240s at Various Testing Temperatures for 15 Sealants

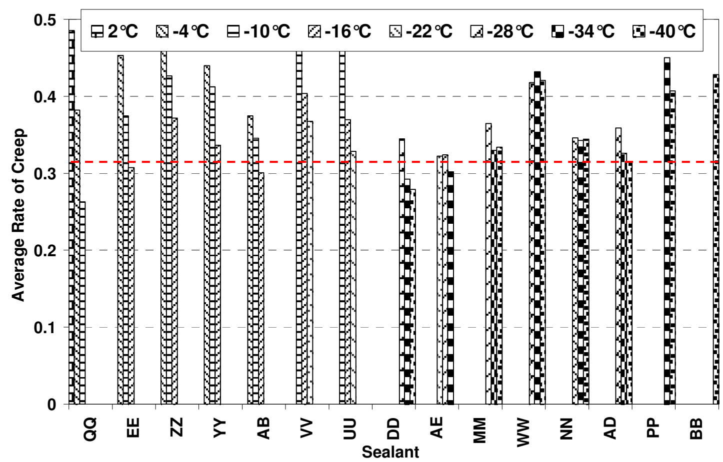


Figure 15 Average Creep Rate at Various Testing Temperatures for 15 Sealants

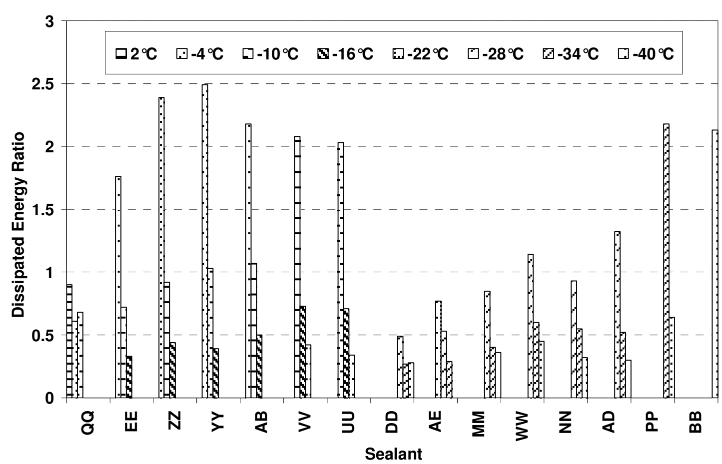


Figure 16 Dissipated Energy Ratio at 240s at Various Testing Temperatures for 15 Sealants

Characterization of Low Temperature Tensile Properties Utilizing Direct Tension Tester

The principle of the crack sealant DTT (CSDTT) is to slowly pull a crack sealant specimen in tension until it breaks. The dog-bone shaped specimen used in the DTT has a rectangular cross section. Its ends are enlarged so that when crack sealant is poured into the mold, it has a large adhesive area between the crack sealant and end tabs. The end tabs are made from Phenolic G-10 material, to provide good bonding. The SuperPave™ DTT specimen geometry can only extend the sealant specimen up to 32% strain. This is significantly smaller than the expected crack sealant extension in the field. Therefore, for crack sealant testing, the specimen geometry and preparation procedure were modified.

A FE analysis was conducted to determine the optimized specimen geometry which provides uniform stress distribution within specimen while allowing sufficient extension (Figure 17). This led to a new geometry; the new specimen dimensions are the following: 24-mm-long, 6-mm-wide and 3-mm-thick; the effective gauge length is 20.3mm. The maximum extension that can be achieved using this specimen is 19mm which is equivalent to approximately 94% strain. This meets the extreme service conditions that sealants may experience in the field. Table 3 presents the geometry comparison of SuperPave™ binder DTT specimen, transition specimen and CSDTT specimen (Al-Qadi et al., 2007).

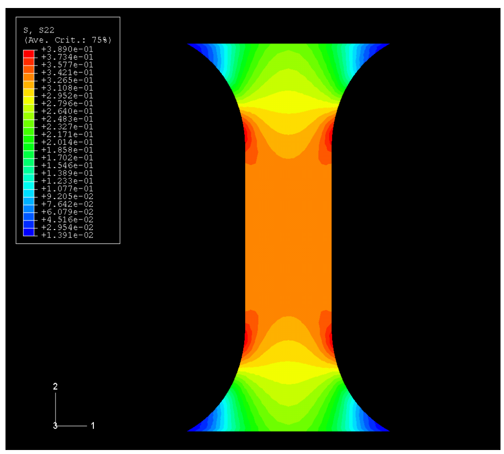


Figure 17 Uniform Stress Distribution along the Web of the Dog-Bone Shape CSDTT Specimen

Table 3 Comparison of Direct Tension Specimen Dimensions

Specimen Type	Width (mm)	Thickness (mm)	Nominal Length (mm)	Effective Gauge Length (mm)
SuperPave™	6	6	40	33.8
Transition	6	3	40	33.8
Crack Sealant	6	3	24	20.2

The effects of geometry and loading rate on sealant tensile behavior were investigated. Results obtained from testing SuperPave™ (6x6 x 33.8mm^3), transition (6x3 x 33.8mm^3) and crack sealant (6x3x20.2mm^3) high-polymer-content sealant specimens were used to evaluate the effect of cross-section area on the stress-strain relationship. As illustrated in Figure 18, high-polymer-content sealants WW and PP showed no major effects due to changes in specimen cross-section areas. The effect of specimen length on stress-strain response for effective gauge length of 33.8mm and 20.3mm specimens is shown in Figure 19 a and b). A greater stress response was noted in the 20.3mm specimen than in the 33.8mm specimen when tested at the same elongation rate. To compare the stress response at the same strain level, two elongation rates, 3mm/min and 1.8mm/min, were applied to 33.8mm and 20.3mm gauge length specimens, respectively. Considering the corresponding specimen length, the variation in elongation rate resulted in an identical strain rate of 8.8%/min for each type. Figure 19 illustrates that the crack sealant specimen elongates about two to three times longer than the SuperPave™ binder specimen. In addition, high-polymer-content sealants have shown 10 to 20 times more elongation and up to twice the tensile strength of crumb-rubber sealants. Regardless of specimen geometry, high-polymer-content sealants have shown equivalent peak stress at its maximum elongation state. Hence, the length effect is negligible in the sealant tensile strength when high-polymer-content products were used (Al-Qadi et al., 2007).

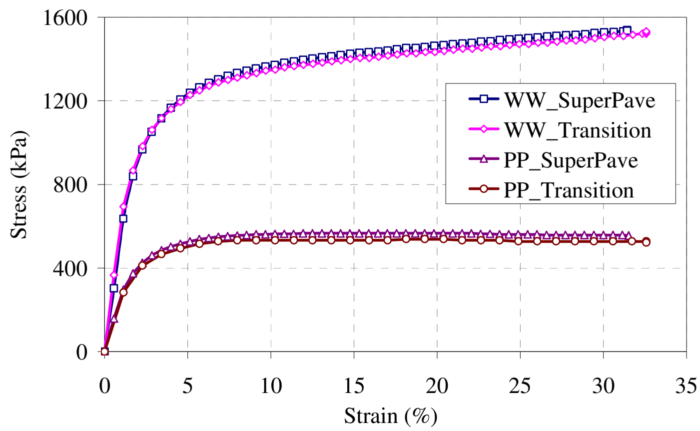
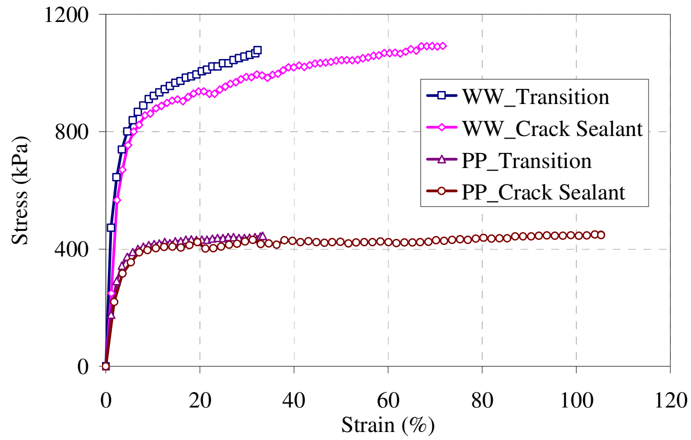


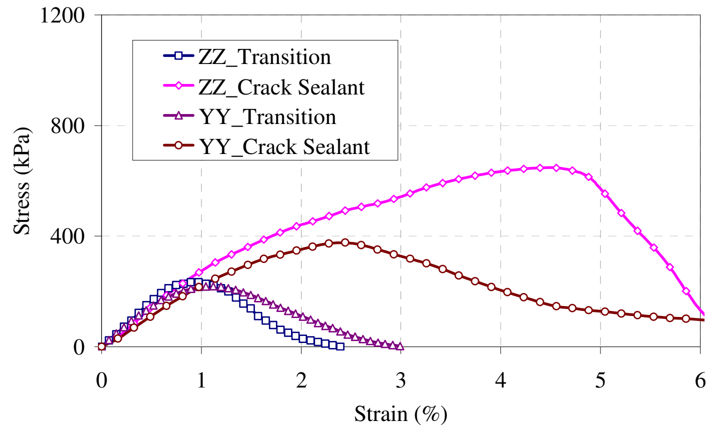
Figure 18 Effect of Cross-Section Area on Stress-Strain Relationship for Two High-Polymer-Content Sealants at 4.5mm/min

The specimen preparation procedure was also modified to accommodate various sealant compositions to improve the workability while pouring the sealant into a mold. The mold was heated to 50°C lower than sealant pouring temperature prior to pouring the sealant. Right after the sealant was poured into the mold, a spatula was used to slightly tap the sealant to ensure that sealant filled the mold. Figure 20 shows the typical stress-strain relationship of six replicates for stiff sealant QQ and soft sealant BB, which were tested at -10°C and -40°C, respectively.

Fifteen sealants were tested at low temperatures ranging from -40 to -4°C. The extendibility of the sealant was measured and used as a performance parameter. It was found that extendibility is a good criterion for identifying and distinguishing among sealants. In addition, a viscoelastic model was fitted to tensile stress-strain test results to obtain Prony series of crack sealants. The model was used to estimate the stress relaxation modulus for the crack sealant. The fundamental property, relaxation modulus, can be used to relate to sealant's field performance as well (Yang and Al-Qadi, 2008).

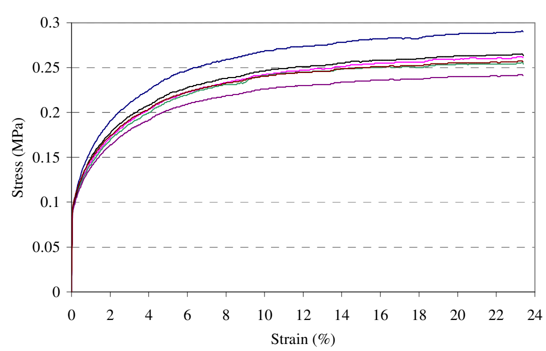


(a)

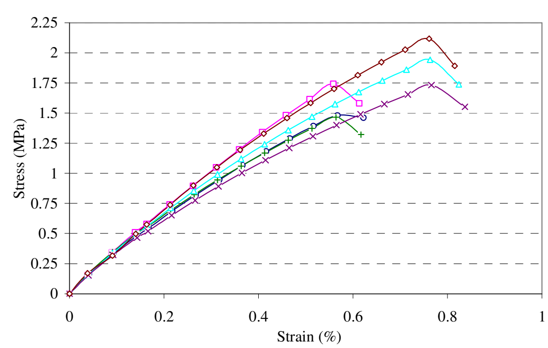


(b)

Figure 19 Effect of Specimen Length on Stress-Strain Relationship at the Same Strain Rate for (a) High-Polymer-Content Sealant, and (b) Crumb-Rubber Sealant



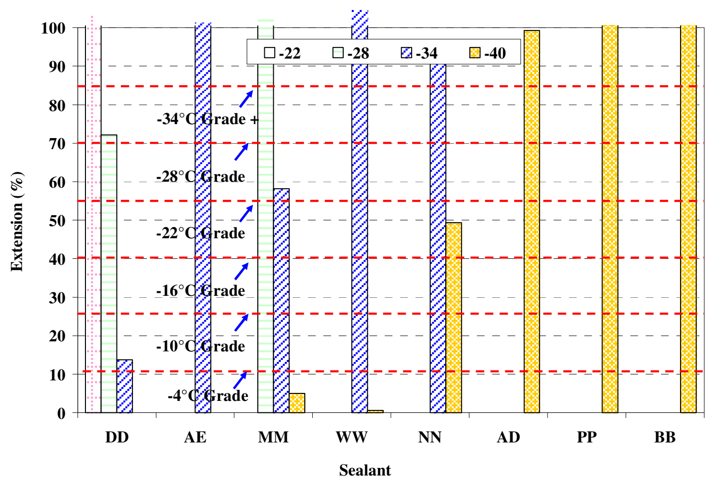
(a)



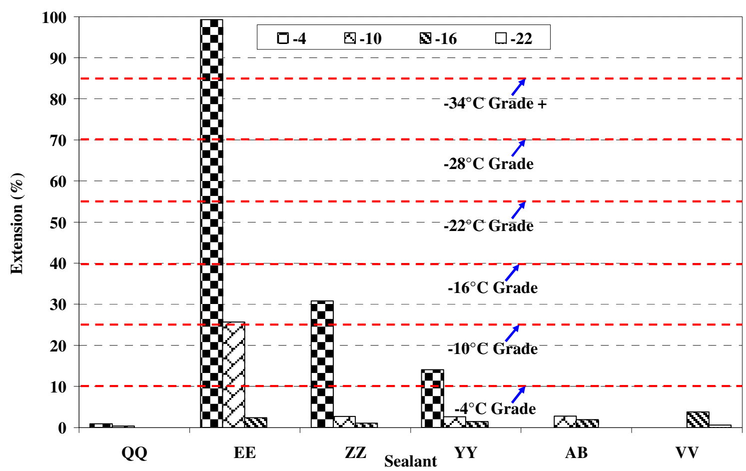
(b)

Figure 20 Replicate Stress-Strain Curves for Sealants: (a) QQ at -10°C; and (b) BB at -40°C

The study recommends using the DTT as a standard test to evaluate the bituminous-based hot-poured crack sealant at low temperature. The performance parameter, extendibility, was recommended for use in the specification. The threshold for the extendibility depends on the sealants' lowest application temperature and is presented in Table 4. In addition, because the test is conducted under a relatively higher deformation rate compared to real crack movement, the research team recommends a +6°C shift in the crack sealant grading system. Therefore, for instance, if the lowest service temperature is determined as -16°C, the test would then be conducted at -10°C. If the extendibility of such sealant is over 25%, the sealant passes the criteria and is approved for use.



(a)



(b)

Figure 21 Extendibility of Selected Sealant at (a) -22, -28, -34, and -40°C; and (b) -4, -10, -16, -22°C

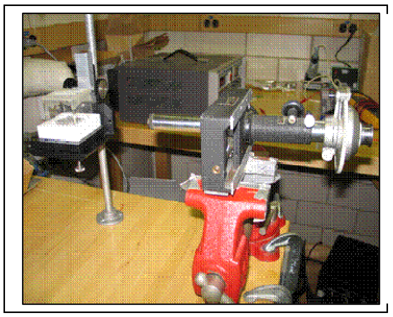
Table 4 Thresholds for Crack Sealant Extendibility at Various Temperatures

Temperature (°C)	-4	-10	-16	-22	-28	-34	-40
Extendibility (%)	10	25	40	55	70	85	85

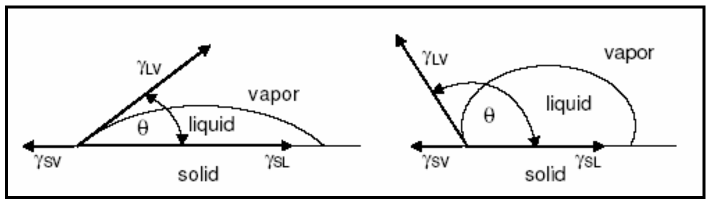
Adhesive Properties of Crack Sealant at Low Temperature

Work of Adhesion

Even a quality sealant may fail if used with an incompatible aggregate. A compatibility test can be performed using the Sessile drop method, Figure 22, which is used to determine both the surface energy of the hot-poured crack sealant and its wettability (contact angle) with respect to aggregate. For this test, sealant is heated and mixed at the manufacturer's recommended installation temperature and poured onto an aluminum sheet to form a thin, smooth surface. The sealant is cooled at room temperature to solidify and make thin plates. A five-micrometer pipette is used to manually apply liquid drops from three probe liquids (water, formamide, and glycerol) onto the sealant plate. The image of each drop is captured by microscope within 15s after it is applied. The resulting contact angle is used to determine the work of adhesion between sealant and substrate.



(a)



(b)

Figure 22 Surface Energy Method: (a) Sessile Drop Equipment and Microscope; and (b) Contact Angle of a Drop of Liquid on a Solid

Due to the high variation among aggregate properties, replacing aggregate substrate with a standard material would be beneficial. Various potential reference materials were examined and aluminum was selected because it has compatible thermal expansion, smoothest surface, and similar surface chemistry to natural aggregate (Fini et al., 2006). Figure 23 presents the calculated work of adhesion between sealant and four substrates: aluminum, granite, quartzite, limestone.

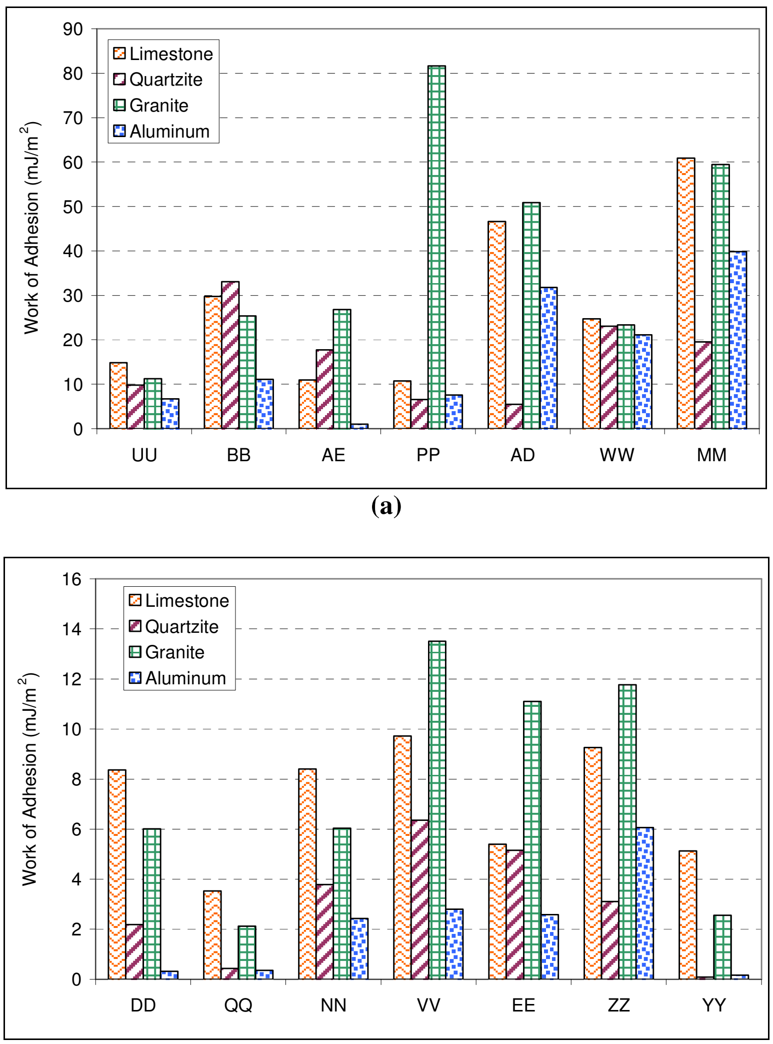
Direct Tension

The premise of the DTT for adhesion is to detach sealant from its aggregate counterpart by applying a tensile force. A new test fixture that simulates sealant pouring condition and loading mechanism in the field was developed. The briquette assembly consists of two aluminum half-cylinders of 25mm diameter and 12mm thickness; aluminum is conservatively selected as a substrate reference material. Each aluminum briquette is confined within an aluminum grip designed to work with the DTT sitting posts. The assembly has a half cylinder mold, open at the upper part. The mold is placed between the two aluminum half cylinders on an even surface. In order to ensure that adhesive failure occurs, and to define failure's location, a notch is made at one side of the sealant-aggregate interface. A 12.5x2mm shim is placed at one aggregate-sealant interface. The assembly then is placed in the DT machine so the notch is placed at the non-moving side of the DT machine.

To conduct the test, sealant is heated at its recommended installation temperature and poured into the half cylinder mold. After one hour of annealing at room temperature, the specimen is trimmed and placed in the cooling bath for 15min. The specimen is then removed from the bath, demolded, and placed back in the bath for another 45min before testing. Using the DT device, the end pieces are pulled apart by moving one of the end pieces at a speed of

0.05mm/s, strain rate of 0.005mm/mm/s (Figure 24). The results of the test are the maximum load and calculated energy, which is the area under the load-displacement curve up to failure, are reported as indications of bond strength (Al-Qadi et al., 2008a).

The interfacial bonding of combinations of eight sealants and four substrates were measured. Figure 25 presents the maximum load and energy required to break the bond between the eight sealants and aluminum. The maximum load showed better repeatability, and it was able to clearly distinguish among different sealant-aggregate pairs. Variation between operators and setups were checked, and no significant variations were found. The maximum load was selected as the test performance parameter (Al-Qadi et al., 2008a). Figure 26 presents this parameter for several pairs of sealant-aggregate. Using comparison between test results of laboratory-aged specimens and field data, a minimum 50N at tested temperature was selected as the performance threshold.

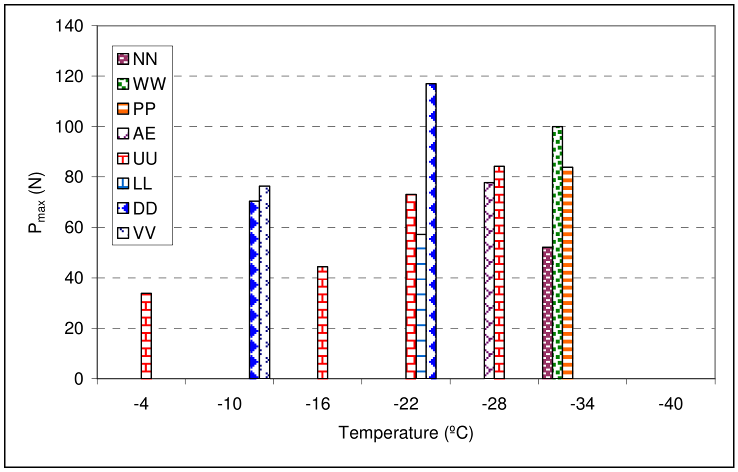


(a)

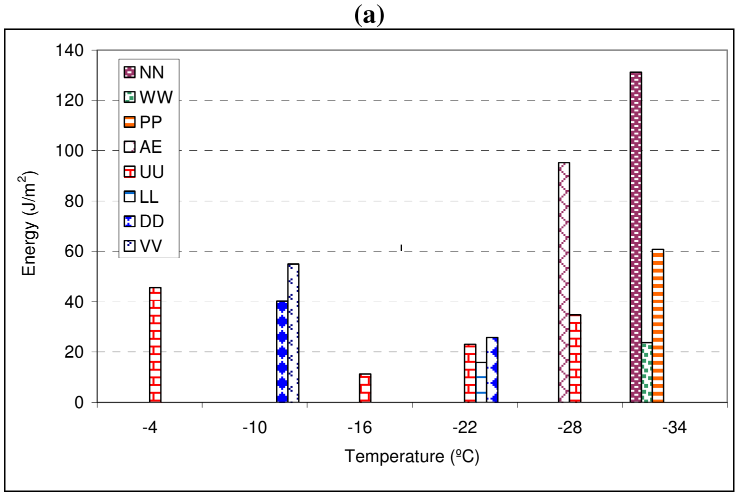
(b)

Figure 23 Work of Adhesion between Sealants and Limestone, Quartzite, Granite and Aluminum for Sealants a) UU, BB, AE, PP, AD, WW, MM; and b) DD, QQ, NN, VV, EE, ZZ, YY

Figure 24 Pulling End Piece Apart at a Constant Displacement Rate Using DT Device



(a)



(b)

Figure 25 Maximum Load and Energy Required to Break the Bond between Sealant and Aluminum at Various Temperatures

Interfacial Fracture Energy

The third test of the low-temperature adhesive properties of sealants is a fracture-type test that utilizes fracture mechanics to derive a fundamental property of the bond. The fundamental property is the interfacial fracture energy (IFE). A geometry-independent, pressure blister test was developed (Fini et al., 2007). The intrinsically stable interface debonding process makes this test attractive and allows calculation of fundamental properties of the interface (Gent and Lewandowski, 1987; Shirani and Liechti, 1998; Penn and Defex, 2002). The blister test, which is an enclosed system, exposes the interface to simultaneous loading and environmental condition.

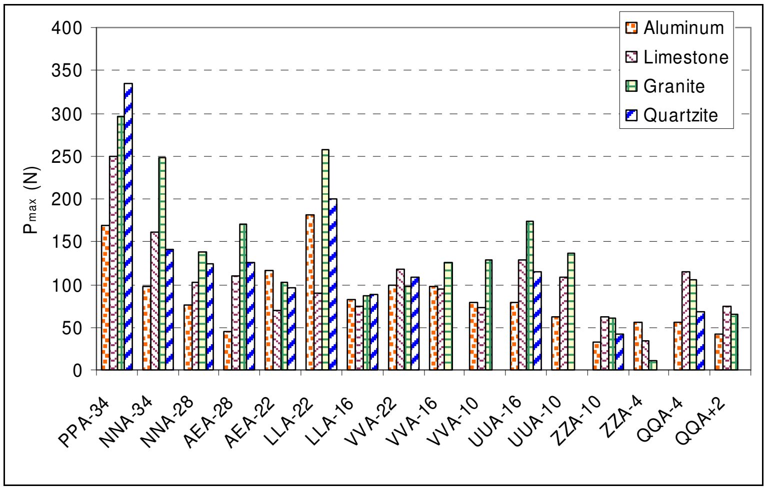


Figure 26 Maximum Load Measured for Bonding between Sealants and Substrates

In this test, a servo-hydraulic pump displaces a piston at a constant rate. The upward movement of the piston injects a liquid medium (alcohol) at a constant rate of 0.1L/hr through a channel that is connected to the specimen (Figure 27). The specimen is composed of an annular (donut-shaped) substrate plate (aggregate or a standard material) covered with binder or sealant on one side. Alcohol pushes the adhesive (binder or sealant) away from the substrate creating a blister which continues to grow until the adhesive separates from the substrate. The blister height and the pressure are recorded during the test and they are used to calculate the IFE. In simplified form, IFE can be calculated as half of the product of the maximum pressure and the corresponding blister height. In addition to IFE, adhesive modulus can be determined from this test using the test data before debonding occurs. In addition, residual stress developed at the interface during the sample preparation process can be obtained.

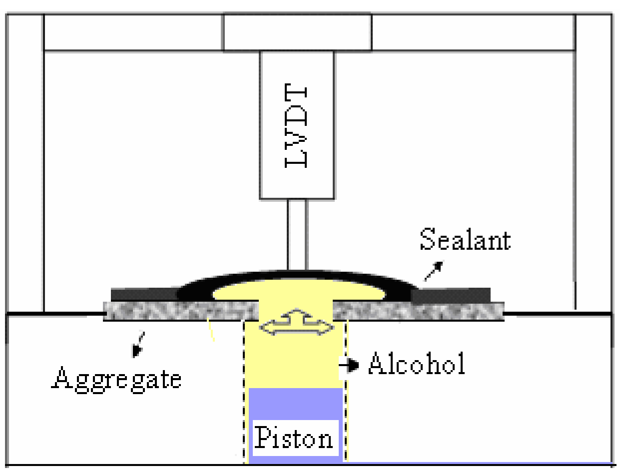
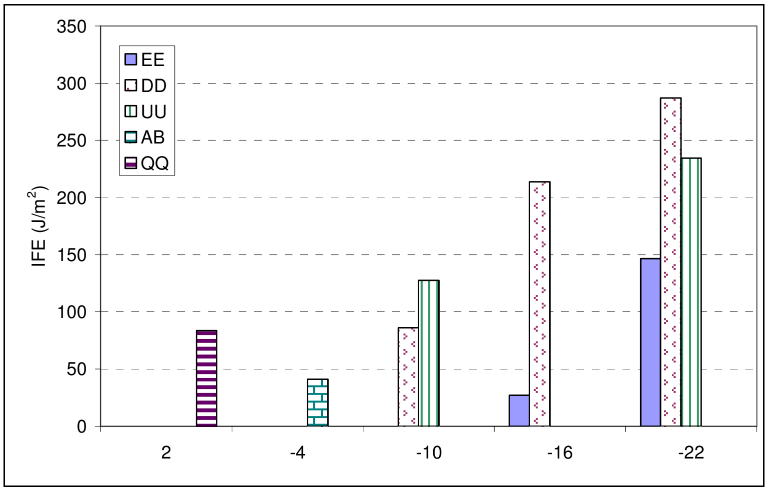
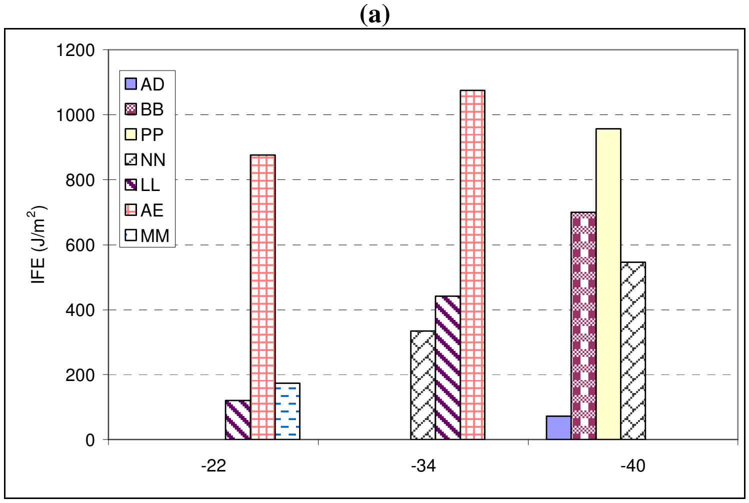


Figure 27 A Schematic of Blister Apparatus

Figure 28 shows the IFE values for several sealant-substrates. It clearly shows that IFE can differentiate among sealants at different temperatures. The crack sealant blister test was further used to study the effect of sealant aging, temperature, loading rate, viscosity, and curing time on interface bonding. Al-Qadi et al. (2008a) found that aging significantly affects interface bonding; depending on the sealant property, IFE may increase/ decrease due to aging (Figure 29). In addition, within-laboratory variation was checked; and no significant difference was found between operators.



(a)



(b)

Figure 28 IFE for 12 Sealants at Temperature Ranging from a) 2 to -22ºC; and b) -22 to -40ºC

Sealant IFE strongly depends on temperature and loading rate; this dependence varies based on material condition (rubbery vs. glassy stages). The IFE of eight aged sealants and three binders were determined at various temperatures. A master curve was constructed for sealant UU which could be tested at the widest temperature range varying from -4 to -34ºC. It was found that when the adhesives are in their rubbery stage, the IFE increases as temperature decreases. However, in the glassy stage, the opposite trend was observed (Figure 30). This study concluded that an optimum interface bonding can be achieved at specific temperature and loading rate identified for each sealant. Knowing the expected loading rate in the field and using a loading rate-temperature master curve, an IFE range can be identified for a specific temperature to ensure acceptable bonding (Fini et al., 2008). In addition, testing results showed that high-viscosity sealants adhere better as long as substrate surface is adequately wetted (Al-Qadi et al., 2008). Effect of curing time was also investigated; sealant cured for 24hrs at room temperature has significantly higher IFE than that cured for 1hr (Al-Qadi et al., 2007).

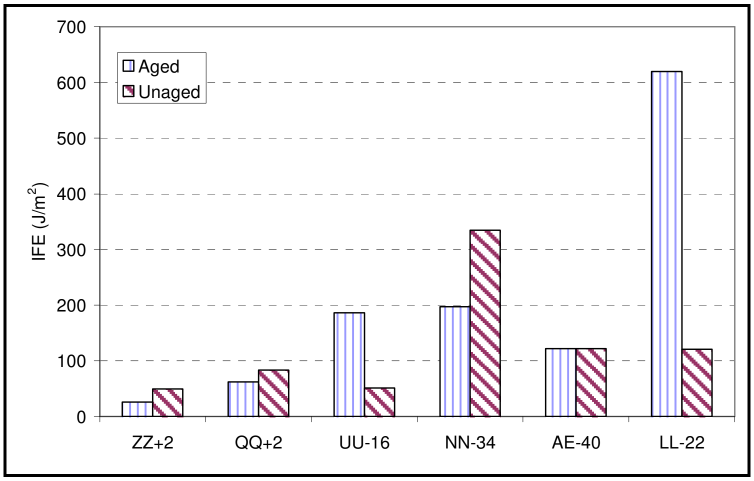


Figure 29 IFE for Aluminum with Aged and Non-Aged Sealants

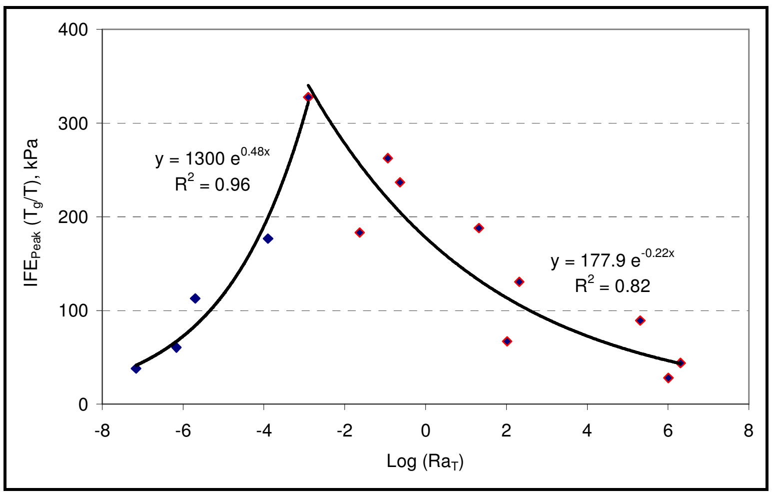


Figure 30 IFE versus Reduced Loading Rate for Sealant UU Bonded to Aluminum

SUMMARY AND CONCLUSIONS

New sealant tests were developed based on the performance of sealants tested in the field and on the characterization of other sealants widely used in North America. The newly developed procedures provide fundamental sealant properties that include: apparent viscosity at the recommended installation temperature, vacuum oven aging to simulate sealant weathering in the field, a DSR test to assess sealant's tracking resistance at high service temperatures, the CSBBR test to evaluate sealant's creep properties at low temperatures, the CSDTT to characterize sealant's low temperature extendibility, and low temperature adhesive (surface energy, direct adhesion, and blister) tests to evaluate the bonding between sealant and its substrate.

Extensive laboratory testing, regular consultation with the project's 26-member technical advisory committee, and limited field testing support conclusions that were used to develop a draft set of performance criteria for crack sealants. Those conclusions and criteria are summarized as follow:

- An apparent viscosity at installation temperature of between 1-3.5Pa.s provides for good crack filling while not being excessively fluid. *Note that this test is the only test performed on un-aged material.*
- Resistance to tracking at high service temperatures can be controlled using a minimum flow coefficient of 4kPa.s and a shear thinning exponent of 0.7, as determined using a dynamic shear rheometer (DSR).
- Using the modified BBR test (CSBBR), a maximum stiffness at 240s of 25 MPa and minimum average creep rate of .31 will promote good field performance of sealant materials that must withstand low-temperature service conditions.
- Extendibility, as measured with the CSDTT, is another good measure of expected low-temperature performance of crack sealants. The threshold for good expected performance is tied to the lowest application temperature (plus a 6°C shift), which is reported in Tables 4 and 5.
- At this time, the direct adhesion test is best suited (of the three adhesion tests that were developed) for use in a practical performance-based guideline. A minimum load of 50N at tested temperature coincides with good field performance for sealant adhesion.

RECOMMENDATIONS

A systematic process for the selection of hot-poured crack sealant is proposed as shown in Figure 31. The performance-based guidelines for crack sealant grading (SC) are presented in Table 5. For example SG 52-34, means the sealant can be used at a high service temperature of 52°C and low temperature of -34°C. The apparent viscosity test (SC-2) helps to ensure that sealant installation goes smoothly. The DSR test (SC-4) sets the sealant's high temperature grade to prevent tracking. If a sealant does not meet the performance criteria at a selected temperature, the test is repeated at a lower temperature until it does. At low temperature, the CSBBR (SC-5) and CSDTT (SC-6) tests predict cohesive performance of sealants. The direct adhesion test (SC-7) addresses the expected bond performance. The sealant is first tested using CSBBR and CSDTT tests at 6°C higher than its lowest service temperature to exam its low temperature cohesive property. If the sealant is passed the cohesive property, it is tested using direct adhesion test to predict its bond strength at the same test temperature as CSBBR and CSDTT. The low temperature grading is determined if sealant passed three low temperature tests. If sealant does not pass the bond test but the extendibility is still appropriate at a lower grade, then the low temperature grade is determined by the cohesive test (CSBBR and CSDTT). Otherwise, the sealant is rejected for use at the testing temperature. The low temperature cohesive grade is selected at -6°C below the low testing temperature. The reliability approach used by SuperPave™ may be applied.

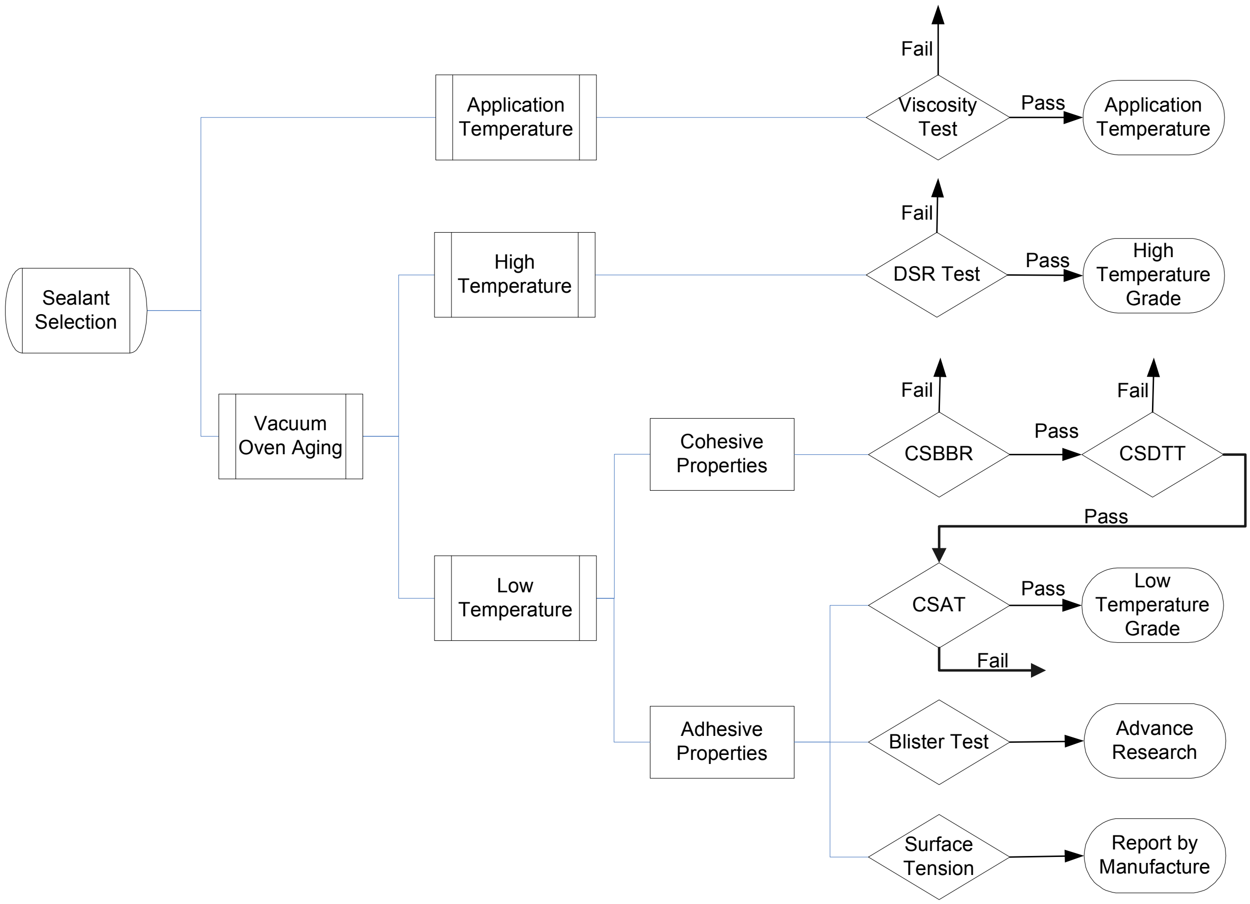


Figure 31 Process for the Selection of Bituminous-Based Sealants

Table 5 Crack Sealant Performance Grade

<table>
<tr><td rowspan="2">Crack Sealant Performance Grade</td><td colspan="7">SG 46</td><td colspan="7">SG 52</td><td colspan="7">SG 58</td><td colspan="7">SG 64</td><td colspan="7">SG 70</td><td colspan="7">SG 76</td><td colspan="7">SG 82</td></tr>
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<tr><td colspan="50">Vacuum Oven Residue (SC-3)</td></tr>
<tr><td>Dynamic Shear, SC-4</td><td colspan="7">46</td><td colspan="7">52</td><td colspan="7">58</td><td colspan="7">64</td><td colspan="7">70</td><td colspan="7">76</td><td colspan="7">82</td></tr>
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<tr><td>Maximum Stiffness (MPa)</td><td colspan="49">25</td></tr>
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<tr><td>Crack Sealant DTT, SC-6</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td></tr>
<tr><td>Minimum Extendibility (%)</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td><td>85</td><td>85</td><td>70</td><td>55</td><td>40</td><td>25</td><td>10</td></tr>
<tr><td>Crack Sealant AT, SC-7</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td><td>-40</td><td>-34</td><td>-28</td><td>-22</td><td>-16</td><td>-10</td><td>-4</td></tr>
<tr><td>Minimum Load (N)</td><td colspan="49">50</td></tr>
</table>

Note: Crack sealant surface energy is provided by manufacturer.

COSTS AND BENEFITS ASSESSMENT

In this study, a systematic approach is proposed to help state agencies select more effective and durable crack sealing material. A study conducted by Ontario's Ministry of Transportation (MTO) during the 1970s to 1980s, which included several field test sections to investigate the influence of crack sealing on pavement distress and performance, shows that with effective crack sealing techniques, at least two years of service life extension can be added to flexible pavements (Chong, 1987). Ponniah and Kennepohl (1996) indicate that most of the premature sealant failure occurs after the first year of installation and is mainly due to unsuitable material and inappropriate installation techniques. In their study, the authors also show a cost-benefit ratio of 1.48 through effective sealing of pavement.

A recent survey on the pavement preventive maintenance programs (PPM) of 18 transportation agencies in North America shows that 13 transportation agencies have established PPM programs, and crack sealing is routinely used as treatment in the PPM program (AASHTO, 2006). The budgets for the PPM programs of each transportation agency are listed in Table 6. For example, the Commonwealth of Virginia spends approximately $20M/year on crack sealing (Liston, 2002). If sealant life is doubled through better selection procedures, then annual savings of $20M/year are possible. More conservatively, the cost-benefit multiplier from Ponniah and Kennepohl's work can be applied to the North American survey to determine a nation-wide estimate of savings. If each PPM program with a dedicated budget devoted only 10% to crack sealing, the total annual savings for these 18 states is nearly $30M/year.

Table 6 Budget and Miles Covered by Pavement Preventive Maintenance Program for 18 Transportation Agencies in North America

Transportation Agencies	PPM	Budget ($)	Miles cover by PPM
Alberta Transportation	Yes	12M	16875
Alaska DOT	Yes	100M	14500
Missouri DOT	No	Not dedicated	10000
Illinois DOT	Yes	50M	14292
South Dakota DOT	No	Not dedicated	7500
Washington DOT	Yes	25M	17800
Hawaii DOT	No	Not dedicated	1000
Idaho DOT	No	4M	12000
New York DOT	Yes	70M	28925
New Jersey DOT	Yes	60M	8500
Vermont DOT	Yes	10-20M	1600
Georgia DOT	Yes	89M	18000
Oregon DOT	No	Not dedicated	18000
Delaware DOT	Yes	40-45M	5700
Louisiana DOT	Yes	12M	12400
Iowa DOT	Yes	11M	9350
Michigan DOT	Yes	81M	1203
Rhode Island DOT	Yes	4M	1100

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APPENDIX: BITUMINOUS-BASED CRACK SEALANT TYPES AND IDENTIFICATIONS

Sealant products used at University of Illinois were designated by a two-character code, which identifies the sealant type (Table A.1). Sealants with one character code are sealants installed in the field in Canada. In addition, three typical test results of the sealants used in this study based on current sealant specification (penetration at 25°C, flow test at 60°C and resilience at 25°C) were also reported.

Table A.1 Sealants Description and Designation

ID	Notes	Penetration	Flow	Resilience
		25°C (dmm)	60°C (mm)	25°C
QQ	Stiffest crack sealant	22	0	36
EE	Expected low temperature grade is -22°C	47	0	51
ZZ	Used in San Antonio, TX	42	N/A	N/A
YY	Used in San Antonio, TX	42	N/A	N/A
AB	Used in San Antonio, TX	40	N/A	23
VV	Modified with fiber	N/A	N/A	N/A
UU	Used by SHRP H106	62	1.5	N/A
AE	Widely used in NY, VA, and NH	N/A	N/A	N/A
DD	Expected low temperature grade is -34°C	80	1.5	50
MM	For aging study	120	1	70
WW	Field data available	N/A	N/A	N/A
NN	Field data available	75	0	70
AD	SHRP H106 field data available	N/A	1	80
PP	Field data	130	1	44
BB	Softest crack sealant	148	0	80
SS	For preliminary test	122	0.1	63
CC	Field data available	N/A	0	65
GG	For preliminary test	66	0	75
HH	SHRP H106 field data	N/A	0	44
A	Field data available	86	0.5	57
B	Field data available	68	0.5	64
C	Field data available	78	0	59
E	Field data available	124	1	73
G	Field data available	50	0.5	51
J	Field data available	66	6	48

Test Method for Apparent Viscosity of Hot-poured Crack Sealant Using Brookfield Rotational Viscometer RV Series Instrument
Sealant Consortium Designation: SC-2

1. SCOPE

1.1. This test method outlines the procedure for measuring the viscosity of hot-poured bituminous crack sealant at elevated temperature from 150ºC to 200ºC using a Rotational Viscometer.

1.2. The rotational viscometer is a rotating spindle-type viscometer that meets the requirements of the AASHTO T 316, Standard Viscosity Determination of Asphalt Binder. This test method can be used for general specification and is especially convenient for use in a field laboratory or a plant site.

1.3. This standard may involve hazardous material, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the application of regulatory limitations prior to use.

2. REFERENCED DOCUMENTS

2.1. AASHTO Standards:

- T316, AASHTO T316, Viscosity Determination of Asphalt Binder Using Rotational Viscometer.

2.2. ASTM Standards:

- D5167-03, Standard Practice for Melting Hot-Applied Joint and Crack Sealant and Filler for Evaluation.
- D4402-06, Standard Test Method for Viscosity Determination of Asphalt at Elevated Temperature Using a Rotational Viscometer.
- E220-07, Test Method for Calibration of Thermocouples by Comparison Techniques.
- E1, Specification for ASTM Thermometers
- E145-94(2006), Standard Specification for Gravity-Convection and Forced-Ventilation Ovens.
- C670, Practice for preparing Precision and Bias Statements for Test Methods for Construction Materials.

2.4. Sealant Consortium (SC) Standards:

- SC-1, Guidelines for Graded Bituminous Sealants.
- SC-2, Test Method for Measuring Apparent Viscosity of Hot-poured Crack Sealant Using Brookfield Rotational Viscometer RV Series Instrument
- SC-3, Method for the Accelerated Aging of Bituminous Sealants.

- SC-4, Method to Measure Low Temperature Sealant Flexural Creep Stiffness at Low Temperature by Bending Beam Rheometer.
- SC-5, Method to Evaluate Sealant Extensibility at Low Temperature by Direct Tension Test.
- SC-6, Blister Method to Predict the Adhesion of Bituminous Sealants.

3. TERMINOLOGY

3.1. Hot-poured crack sealants are hot-poured modified asphaltic materials used in pavement cracks and joints.

3.2. Apparent viscosity is the ratio of shear stress to shear rate for a liquid. This parameter is a measure of the resistance to flow of the liquid. The SI unit of viscosity is the Pascal second (Pa.s).

4. SUMMARY OF METHOD

4.1. Crack sealant material is homogenized according to ASTM D5167-03, cut into pieces not larger than 5mm (the largest dimension), and placed into standard containers. Apparent viscosity is measured utilizing the Brookfield viscometer using Spindle #SC4-27; the spindle is attached to the rigid hook attachment and rotates at the speed of 60 rpm. The test is conducted at the manufacturer's recommended installation temperature.

5. SIGNIFICANCE AND USE

5.1. This test is intended for bituminous sealants applied to roadway joints and cracks.

5.2. This procedure is designed to simulate the viscosity of crack sealants while pouring in the cracks.

5.3. Sealants must be homogenized (ASTM D5167-03) before measuring the apparent viscosity by this method.

6. APPARATUS

6.1. Brook-field rotational viscometer RV Series Instrument

6.2. Brookfield Thermosel, maintaining a temperature ranging from 170°C to 193°C ± 1°C.

6.3. Laboratory oven – any laboratory standard oven capable of producing and maintaining a temperature ranging from 170°C to 193°C ± 1°C.

6.4. Rigid hook attachment especially designed as an attachment in Brookfield viscometer to measure hot-poured crack sealant viscosity.

6.5. Disposal aluminum containers or standard Brookfield containers.

6.6. The rotational viscometer contains sensors that monitors the applied torque and automatically displays the calculated apparent viscosity. The keypad on the instrument is used to enter the spindle number, zero the signal, and run the test at a selected speed. Torque and viscosity can be recorded manually, or an interface can be used to send the signal from the instrument to a personal computer. Optional software is also available that can be used to program preselected thermal profiles. This software is not needed for the specification test. However, the Thermosel must be used to control the temperature and thereby obtain acceptable reproducibility.

7. HAZARDS

7.1. Standard laboratory caution should be used in handling hot sealant in accordance to ASTM D5167-03, and when using the Brookfield Thermosel. Required safety procedures should be followed when chemical agents are used.

8. PREPARATION OF APPARATUS

8.1. The rotational viscometer must be leveled to function properly. A bubble-type level is normally located on top of the viscometer and is adjusted by using leveling screws located on the base. Preparing the device, leveling and aligning of the viscometer on the stand, and setting the temperature of the Thermosel are explained in the operation instructions provided by the manufacturer. The detailed steps for testing are specified in the AASHTO Standard Test Method T316-06.

9. CALIBRATION AND STANDARDIZATION

9.1. Temperatures of the ovens should be calibrated in accordance with each user's quality assurance program.

9.2. Thermometer (temperature detector) should be calibrated every six months to ensure precision of +/-1°C.

9.3. The accuracy of the viscometer should be checked annually using a certified reference fluid of known viscosity following the procedure recommended by manufacturer.

10. PREPARATION OF SAMPLES AND TEST SPECIMENS

All apparent viscosity measurements must be performed on homogenized sealant. Sealant homogenization is conducted in accordance with the procedure presented in ASTM D5167-03, Melting of Hot-Applied Joint and Crack Sealant and Filler for Evaluation.

10.1. Once homogenized, hot sealant should be cooled down to room temperature and stored for 24hr before usage. It is recommended that a can or plastic-lined box be used. The container must be of sufficient size so that the sealant depth is no greater than 100 mm to allow for rapid cooling.

10.2. 10.5g of the homogenized sealant should be cut into small pieces not larger than 5mm and placed in aluminum chambers. Disposable chambers able be installed in the Thermosel shall be used. .

10.3. Preheat Thermosel to test temperature, and unless otherwise noted, use temperature recommended by the producer.

10.4. Place aluminum chamber including sealant in Thermosel.

10.5. Turn on the viscometer and zero it.

10.6. Allow 5 minutes for sealant to melt.

10.7. Assemble spindle # SC4-27 and attach to a rigid rod, see Note 1.

Note 1—The current hook which is used for asphalt cement may not be applied to asphalt binder that contains rubber fillers, which would affect the spindle's rotation. Figures 1 through 3 illustrate the spindle and the rigid rod.

10.8. Allow 20min to stabilize the temperature; adjust stirring speed of the spindle to 60rpm.

10.9. Start testing and record the data right after 30sec of stirring. After the data is recorded, stop the test and clean the spindle and remove the aluminum chamber.

10.10. Insert the next specimen and repeat steps 10.4 to 10.9 until four replicates are tested for each sealant.

11. CALCULATION OF RESULTS

11.1. The viscosity is reported as the average of best three out of four readings. The Brookfield viscometer measures the apparent of viscosity in centipoise. The measured viscosity may be converted to Pascal seconds by using the conversion factor 1 cps = 0.001 Pa-s.

12. REPORT

12.1. Report the following information: sealant identification and supplier, lot number, date received, date of apparent viscosity measurement, recommended pouring temperature, safe heating temperature, and any deviations from test temperature.

13. PRECISION AND BIAS

13.1. *Single Operator Precision (Repeatability)*—The figures in Column 2 of Table are the Coefficient of variation that have been found to be appropriate for the conditions of test described in Column 1. Two Results obtained in the same laboratory, by the same operator using the same equipment, in the shortest practical period of time, should not be considered suspect unless the difference in the two results, expressed as a percent of their mean, exceeds the value given Table 1. Column 3.

13.2. *Multilaboratory Precision (Reproducibility)*—The figures in Column 2 of Table are the Coefficient of variation that have been found to be appropriate for the conditions of test described in Column 1. Two results submitted by two different operators testing the same material in different laboratories shall not be considered suspect unless the difference in the two results, expressed as percent of their mean, exceeds the values given in Table 1, Column 3.

13.3. The rotational viscometer test is an AASHTO standard method (T 316). The reader is referred to the standard method for points of caution and details regarding the test method.

13.4. Viscosity data obtained with this test method are used to ensure crack sealant's apparent viscosity is low enough to fill cracks and at the same time high enough not to flow out of the crack. Ideally, the shear rates during the test should match the shear rates sealant experiences during installation. The rotational speed of the spindle was selected at 60rpm to resemble field pouring conditions. Changing spindle sizes and rotational speeds affects both the shear rate and the measured apparent viscosity.

13.5. Data should be collected after a specific rotation time. Excessive mixing may cause segregation; especially in the case of rubber modified sealant.

13.6. Excessive heating may cause volatiles to be lost from the sample or polymer chains to be degraded which leads to reduction in measured apparent viscosity. In general, during testing, the sample should not be heated to temperatures greater than the pouring temperature, as recommended by the manufacturer.

Table 1—Precision Estimates

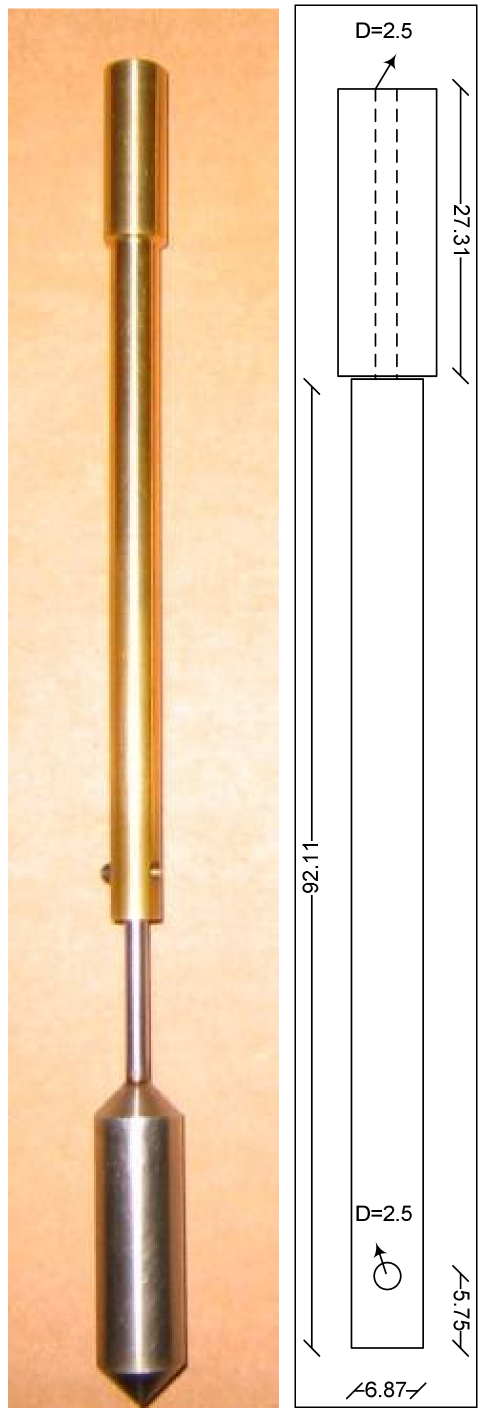
Condition	Coefficient of Variation (1s%)a	Acceptable Range of Three Test Results (d2s%)a
Single-Operator Precision:		
Average Viscosity (Pa.s)	1.62	5.4
Multilaboratory Precision		
Average Viscosity (Pa.s)	5.9	16.9

Note 2—The precision estimates given in Table 2 are based on the analysis of test results from seven sealant with a wide range of rheological properties. The data analyzed includes results from seven laboratories who conducted each test in four replicates.

Note 3—As an example, two tests conducted on the same material yield viscosity results of 3.12, 3.05, 3.15Pa.s, respectively. The average of these three measurements is 3.11 Pa.s. The acceptable range of results is then 5.4 percent of 3.11 or 0.17 Pa.s. As the greatest difference between each two, 0.1 is less than 0.17, the results are within the acceptable range.

14. KEYWORDS

14.1. Hot-poured bituminous sealant; fillers; joint; crack; apparent viscosity rotational viscometer.



(a) (b)

Figure 1. Rigid rod with spindle assembled (a) Schematic of rigid rod (b)

Figure 2. Lower opening of 2.5mm to screw the spindle to the rigid rod

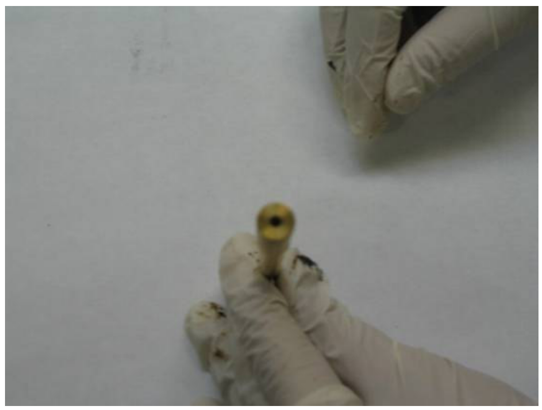


Figure 3. Upper opening of 2.5mm to screw the rod to the viscometer head

Method to Measure Low Temperature Flexural Creep Stiffness of Bituminous Sealants and Fillers by Bending Beam Rheometer (BBR)
Sealant Consortium Designation: SC-5

1. SCOPE

14.2. This method applies to bituminous sealants used in the construction and maintenance of roadways.

14.3. The method is used to determine the bituminous sealant flexural stiffness. It can be used on unaged material or on material aged using Test Method SC-3 (Vacuum Oven Aging). The test apparatus is designed for testing within the temperature range from -4°C to -40°C.

14.4. The values stated in SI units are to be regarded as the standard.

14.5. This practice covers the determination of flexural stiffness in bituminous sealants using the bending beam rheometer and by conducting the creep test.

15. REFERENCED DOCUMENTS

15.1. AASHTO Standards:

- T313, Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR).

2.2. ASTM Standards

- D6648-01, Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR).
- D5167-03, Standard Practice for Melting Hot-Applied Joint and Crack Sealant and Filler for Evaluation.
- D6373-99, Standard Specification for Performance Graded Asphalt Binder.
- E77-98(2003), Test Method for the Inspection and Verification of Thermometers.
- E145-94(2006), Standard Specification for Gravity-Convection and Forced-Ventilation Ovens.
- E1-05 Standard Specifications for ASTM Liquid-in-Glass Thermometers

2.3. Documents of the Sealant Consortium (SC):

- SC-1, Guidelines for Graded Bituminous Sealants.
- SC-2, Test Method for Measuring Apparent Viscosity of Hot-poured Crack Sealant Using Brookfield Rotational Viscometer RV Series Instrument
- SC-3, Method for the Accelerated Aging of Bituminous Sealants.

- SC-4, Method to Evaluation of the Tracking Resistance of Bituminous Sealants and Fillers by Dynamic Shear Rheometry.
- SC-5, Method to Measure Low Temperature Sealant Flexural Creep Stiffness at Low Temperature by Bending Beam Rheometer.
- SC-6, Method to Evaluate Sealant Extensibility at Low Temperature by Direct Tension Test.
- SC-7, Blister Method to Measure the Adhesion of Bituminous Sealants.

16. TERMINOLOGY

16.1. Bituminous sealants are hot-poured modified asphaltic materials used in pavement cracks and joints.

16.2. Definitions of Terms Specific to This Standard:

3.2.1 Contact load, n – the load, Pc, required to maintain positive contact between the test specimen, supports, and the loading shaft; 35 ± 10mN.

3.2.2 Flexural creep compliance, D(t), n – the ratio obtained by dividing the maximum bending strain in a beam by the maximum bending stress. The flexural creep stiffness is the inverse of the flexural creep compliance.

3.2.3 Flexural creep stiffness, S(t), n – the creep stiffness obtained by fitting a second order polynomial to the logarithm of the measured stiffness at 8.0, 15.0, 30.0 60.0, 120.0, and 240.0s and the logarithm of time.

3.2.4 Measured flexural creep stiffness, Sm (t), n – the ratio obtained by dividing the measured maximum bending stress by the measured maximum bending strain. Flexural creep stiffness has been used historically in asphalt technology while creep compliance is commonly used in studies of viscoelasticity.

3.2.5 Average creep rate – the average creep rate obtained by fitting the power law model of the logarithm of the strain versus the logarithm of time. The average creep rate is the absolute value of the exponents of the power law model.

3.2.6 Test load, n – the load, Pt, of 240s duration is used to determine the stiffness of the crack sealant being tested; 980 ± 50mN.

17. SUMMARY OF PRACTICE

17.1. The bending beam rheometer is used to measure the midpoint deflection of a simply supported prismatic beam of bituminous crack sealant subjected to a constant load applied to the midpoint of the test specimen. The device operates only in the loading mode.

17.2. A prismatic test specimen is placed in the controlled temperature fluid bath and loaded with a constant test load for 240.0s and unloaded for 480.0s. The test load (980 ± 50mN) and the midpoint deflection of the test specimen are monitored versus time using a computerized data acquisition system.

18. SIGNIFICANCE AND USE

18.1. This test is intended for bituminous sealants applied to roadway joints and cracks.

18.2. The test temperature is determined as the lowest temperature experienced by the pavement surface in the geographical area for which the sealant is intended.

18.3. The flexural creep stiffness or flexural creep compliance, determined from this test, describes the low-temperature stress-strain-time response of crack sealant at the test temperature.

18.4. The average creep rate determined from this test gives an indication of the rate of deformation of crack sealant at the test temperature.

18.5. Sealants must be homogenized before being used to conduct this test.

19. APPARATUS

19.1. A crack sealant bending beam rheometer (CSBBR) test system consists of the following: (1) a modified bending beam rheometer with a controlled temperature liquid bath which maintains the test specimen at the test temperature, (2) test specimen molds, and (3) items for verifying and calibrating the system.

19.2. A Modified Bending Beam Rheometer – A CSBBR is modified from a typical BBR. The CSBBR has a modified loading frame system which can accommodate a specimen 12.7mm in height to operate a three-point bending beam test that applies a constant test load for 240.0s and unloads for 480.0s The specification required by the CSBBR system is in accordance with Test Method T313. The updated version software can be obtained from the instrument manufactures.

19.3. Test Specimen Molds – Test specimen molds with interior dimensions of 12.70 ± 0.05mm wide by 12.70 ± 0.05mm deep by 102.0 ± 0.5mm long fabricated from aluminum or stainless steel (Fig. 1).

6.3.1 The thickness of the two spacers used for each mold (small end pieces used in the metal molds) shall be measured with a micrometer and shall not vary from each other in thickness by more than 0.05mm.

NOTE 1 – Small errors in the thickness of the test specimen can have a significant effect on the calculated stiffness because the calculated stiffness is a function of the thickness, h, raised to the third power.

19.4. Items for Calibration – items remain the same as AASHTO Test Method T313, except the dimension of the stainless steel (thick) beam use for calibration. The new calibration kits can be obtained from the instrument manufactures.

19.5. Calibrated Thermometers – calibrated liquid-in-glass thermometers to verify the temperature transducer of suitable range with subdivisions of 0.1°C.

19.6. Laboratory Ovens – two standard laboratory ovens capable of producing and maintaining a temperature of 200 ± 0.5°C.

20. REAGENTS AND MATERIALS

20.1. Bath Fluid – A bath fluid that is not absorbed by or does not affect the properties of the crack sealant being tested. The bath fluid shall be optically clear at the test temperature.

20.2. Binder Clip – A binder clip is used to hold the aluminum mold to maintain the size of the sample to prevent shrinkage during sealant cooling.

20.3. Release Agent – A proper release agent prevents bituminous crack sealant from sticking to the mold. Using a spray type silicon-based release agent is recommended.

20.4. Solvent – A solvent can properly clean the molds, end tabs, and plates. The parts cleaned by the solvent shall be submerged in the ethanol prior to use. Cleaning ensures the proper bond between sealant and end tabs.

20.5. Cleaning Cloths – Cloths for wiping molds, end tabs and plates.

21. HAZARDS

21.1. Standard laboratory caution should be used in handling hot bituminous sealant in accordance with ASTM D5167, and required safety procedures should be followed when chemical agents are used.

22. VERIFICATION AND CALIBRATION

22.1. BBR – Follow the procedure as stated in AASHTO T313.

22.2. Oven and freezer – Calibrate the temperature with a thermometer that meets the requirements of ASTM E1. The thermometer calibration can be verified according to ASTM E77.

23. SAMPLES PREPARATION

23.1. Preparation of Molds.

10.1.1 Spread a very thin layer of release agent on the interior faces of four mold sections to prevent the crack sealant from sticking to the metal end pieces. Assemble the mold and use binder clips to hold the pieces of the mold together.

10.1.2 Preheat the oven to a temperature 50°C lower than recommended pouring temperature at least one hour before testing. Place the mold on the ceramic tiles into the oven 15mins before pouring the crack sealant.

23.2. Preparation of Test Specimens.

10.2.1 Laboratory-aged samples shall be obtained in accordance with appropriate test methods.

10.2.2 Heat 4 cans of bituminous crack sealant, which contain 35g of bituminous sealant each, in an oven set at the sealant manufacturer-recommended pouring temperature until the sealant is sufficiently fluid to pour (Do not heat the sealant more than 30mins.) Each can of sealant will be poured into its own mold.

23.3. Molding Test Specimens – 4 replicates should be prepared for each tested sealant. Prior to pouring the sealant, take one preheated mold and one ceramic tile from the oven. With the preheated mold on the ceramic tile, firmly stir the sealants prior to pouring into the molds to ensure the homogeneity of the sealant. Begin pouring the sealant from one end of the mold and move toward the other end, slightly overfilling the mold. When pouring, hold the sample container 20 to 30mm from the top of the mold, pouring continuously toward the other end in a single pass. Repeat the same procedure for the other three molds. Place the filled mold on the preheated ceramic tile and allow the mold to cool for one hour to room temperature. After cooling to room temperature, trim the exposed face of the cooled specimens even with the top of the mold using a hot knife.

23.4. Storing and Demolding Test Specimens.

10.4.1 Store all test specimens in their molds at room temperature prior to testing. Testing shall be completed within 4hrs after specimens are poured.

10.4.2 Just prior to demolding, cool the molds containing the test specimens in a cold fluid bath which has the same temperature as the selected test temperature for no longer than 5min, but only long enough to stiffen the test specimen so that it can be readily demolded without distortion. A 15-minute interval between each sample is desired prior to placing the sample into the cold chamber bath. Do not cool the molds containing the specimens in the test bath because it may cause temperature fluctuations in the bath to exceed ± 0.2°C.

10.4.3 Immediately demold the specimen when it is sufficiently stiff to demold without distortion, by disassembling the mold. To avoid distorting the specimen, demold the specimen by sliding the metal side pieces from the specimen.

NOTE – During demolding, handle the specimen with care to prevent distortion. Full contact at specimen supports is assumed in the analysis. A warped test specimen may affect the measured stiffness.

24. PROCEDURE

24.1. All sealants to be tested must undergo the aging process. Follow the procedure described SC-3, Method for the Accelerated Aging of Bituminous Sealants It is recommended that a minimum of 150g of bituminous sealant be prepared for a set of tests.

24.2. Select the appropriate test temperature for the crack sealant specimen. After demolding, immediately place the test specimen in the testing bath and condition it at the testing temperature. The test specimen shall remain submerged in the bath fluid at the test temperature ± 0.1°C for the entire 60 ± 5min.

24.3. Check the adjustment of the contact load and test load prior to testing each set of test specimens. The 12.7-mm thick stainless steel beam shall be used for checking the contact load and test load.

11.3.1 Place the 12.7mm steel beam in position on the beam supports. Using the test load regulator valve, gently increase the force on the beam to 980 ± 50mN.

11.3.2 Switch from the test load to the contact load and adjust the force on the beam to 35±10mN. Switch between the test load and contact load four times to ensure that the load is stable.

11.3.3 When switching between the test load and contact load, watch the loading shaft and platform for visible vertical movement. The loading shaft shall maintain contact with the steel beam when switching between the contact load and test load, and the contact load and test load shall be maintained at 35±10mN and 980±50mN, respectively.

24.4. Enter the specimen identification information, including the elapsed time the specimen has been conditioned in bath at the test temperature, and other information as appropriate into the computer that controls the test system.

24.5. After conditioning, place the test specimen on the test supports and gently position the back side of the test specimen against the alignment pins. Initiate the test.

24.6. The bath temperature shall be maintained at the selected test temperature ± 0.1°C during the test; otherwise the test shall be rejected.

24.7. The contact load shall be applied by gently increasing the load to 35±10mN. While applying the contact load, the load on the beam shall not exceed 45mN, and the time to apply and adjust the contact load shall be no greater than 10s.

24.8. With the contact load applied to the test specimen, activate the automatic test system, which is programmed to proceed as follows:

11.8.1 Apply a 980±50mN seating load for 1±0.1s.

11.8.2 Reduce the load to the 35±10mN contact load and allow the test specimen to recover for 480±0.1s. At the end of the test, the operator shall monitor the computer screen to verify that the load on the test specimen returns to 35±10mN. If it does not, the test shall be rejected.

11.8.3 Apply a 980±50mN test load to the test specimen. The software shall record the test load at 0.5-second intervals from 0.5 to 240s and calculate the average of the recorded load values. Between 0.5 and 5s, the test load shall be within ±50mN of the average test load and for the remaining times within ±10mN of the average test load. The actual load on the test specimen as measured by the load cell shall be used to calculate the stress in the test specimen.

11.8.4 Remove the test load and return to the 35 ± 10mN contact load and collect the data for 480s.

11.8.5 Remove the specimen from the supports and proceed to the next test.

25. CALCULATIONS

25.1. Deflection of an Elastic Beam – Using the elementary bending theory, the midspan deflection of an elastic prismatic beam of constant cross-section loaded in three-point loading can be obtained by applying Equations 12.1 and 12.2 as follows:

$$\delta = PL^3/48EI \qquad (12.1)$$

where:

δ = deflection of beam at midspan, mm,
P = load applied, N,
L = span length, mm,
E = modulus of elasticity, MPa, and
I = moment of inertia, mm^4.
and,

$$I = bh^3/12 \qquad (12.2)$$

where:

b = width of beam, mm, and
h = thickness of beam, mm.

NOTE – The test specimen has a span to depth ratio of 10 to 1, and the contribution of shear to deflection of the beam can be neglected.

25.2. Elastic Flexural Modulus – According to elastic theory, calculates the flexural modulus of a prismatic beam of constant cross-section loaded at its midspan. Therefore:

$$E = PL^3/4bh^3\delta \qquad (12.3)$$

where:

E = flexural creep stiffness, MPa,
P = load, N,
L = span length, mm,
b = width of beam, mm,
h = depth of beam, mm, and
δ = deflection of beam, mm.

25.3. Maximum Bending Stress – the maximum bending stress occurs at the top and bottom of the beam at its midspan. Therefore:

$$\sigma = 3PL/2bh2 \qquad (12.4)$$

where:

σ = maximum bending stress in beam, MPa,
P = constant load, N,
L = span length, mm,
b = width of beam, mm, and
h = depth of beam, mm.

25.4. Maximum Bending Strain – the maximum bending strain in the beam occurs at the top and bottom of the beam at its midspan. Therefore:

$$\varepsilon = 6\delta h/L^2 \text{ mm/mm} \qquad (12.5)$$

where:

ε = maximum bending strain in beam, mm/mm,
δ = deflection of beam, mm,
h = thickness of beam, mm, and
L = span length, mm.

25.5. Linear Viscoelastic Stiffness Modulus – According to the elastic-viscoelastic correspondence principle, it can be assumed that if a linear viscoelastic beam is subjected to a constant load applied at t = 0 and held constant, the stress distribution in the beam would be the same as that in a linear elastic beam under the same load. Further, the strains and displacements depend on time and are derived from those of the elastic case by replacing E with 1/D(t). Since 1/D(t) is numerically equivalent to S(t), rearranging the elastic solution results in the following relationship for stiffness:

$$S(t) = PL^3/4bh^3\delta(t) \tag{12.6}$$

where:

S(t) = time-dependent flexural creep stiffness, MPa,
P = constant load, N,
L = span length, mm,
b = width of beam, mm,
h = depth of beam, mm,
$\delta(t)$ = deflection of beam, at time t, mm, and

$\delta(t)$ and S(t) indicate that the deflection and stiffness, respectively, are functions of time.

26. REPORT

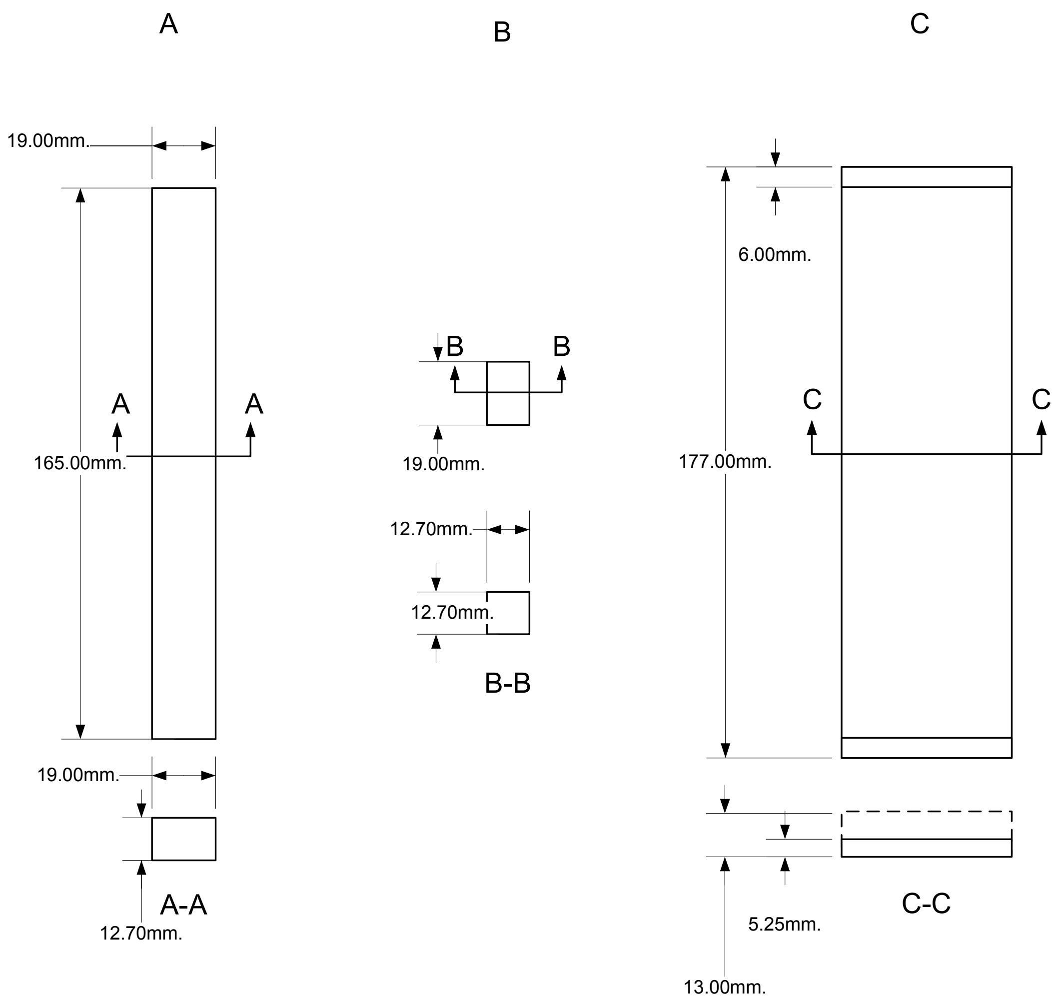
26.1. Report the following information: sealant name and supplier, test sample ID, date of aging (dd/mm/yy), date of test (dd/mm/yy), time of demolding (h, m), time of conditioning (h, m), time test load applied for each sample (h, m), test temperature, maximum and minimum temperature during the test, any deviations from test load and temperature, measured stiffness modulus and average creep rate.

27. PRECISION AND BIAS

27.1. Confidence intervals of 95% should be constructed around the average of the calculated stiffness from the results of the four replicates. The closest three measurements will then be used to calculate the coefficient of variation while the fourth replicate will be discarded. A coefficient of variation less than 10% is desirable.

28. KEYWORDS

28.1. Hot-poured bituminous sealant; joint; crack; direct tension test; stiffness; average creep rate; creep compliance.



IT	QTY	DESCRIPTION
A	2	Side Bar
B	2	Spacer Bar
C	1	Bottom Plate

Figure 1. Dimension for crack sealant Bending Beam Rheometer mold and modified specimen support

Evaluation of the Low Temperature Tensile Property of Bituminous Sealants by Direct Tension Test
Sealant Consortium Designation: SC-6

29. SCOPE

29.1. This method applies to bituminous sealants used in the construction and maintenance of roadways.

29.2. The method is used to determine the extensibility and strain energy density (SED) of sealants at low temperature. It can be used with unaged material or with material aged using Test Method SC-3 (Vacuum Oven Aging). The test apparatus is designed for testing within the temperature range from -4°C to -40°C.

29.3. This practice covers the determination of extensibility and percent modulus decay in bituminous sealants with the use of direct tension testing and by applying tensile stress-strain test.

30. REFERENCED DOCUMENTS

30.1. AASHTO Standards:

- T314, Determining the Fracture Properties of Asphalt Binder in Direct Tension (DT).

2.2. ASTM Standards:

- D6723, Standard Test Method for Determining the Fracture Properties of Asphalt Binder in Direct Tension (DT).
- D5167, Standard Practice for Melting Hot-Applied Joint and Crack Sealant and Filler for Evaluation.
- D6373, Standard Specification for Performance Graded Asphalt Binder.
- E77, Test Method for the Inspection and Verification of Thermometers
- E145, Standard Specification for Gravity-Convection and Forced-Ventilation Ovens.

2.3. N. E. Dowling. Mechanical Behavior of Materials (Second Edition). Prentice Hall, Upper Saddle River, NJ, 1999.

2.4. Documents of the Sealant Consortium (SC):

- SC-1, Guidelines for Graded Bituminous Sealants
- SC-2, Test Method for Measuring Apparent Viscosity of Hot-poured Crack Sealant using Brookfield Rotational Viscometer RV.
- SC-3, Method for the Accelerated Aging of Bituminous Sealants.
- SC-4, Method to Evaluation of the Tracking Resistance of Bituminous Sealants and Fillers by Dynamic Shear Rheometry.

- SC-5, Method to Measure Low Temperature Sealant Flexural Creep Stiffness at Low Temperature by Bending Beam Rheometer.
- SC-6, Method to Evaluate Sealant Extensibility at Low Temperature by Direct Tension Test.
- SC-7, Blister Method to Predict the Adhesion of Bituminous Sealants.

31. TERMINOLOGY

31.1. Bituminous sealants are hot-poured modified asphaltic materials used in pavement cracks and joints.

31.2. Effective gauge length. Elongation of a standard dog bone shaped test specimen due to an applied axial load P is equivalent to that of a simple rectangular specimen with the same cross-sectional dimensions of the restricted section. Effective gauge length, L_{eff}, is defined as the length of the simple rectangular specimen and has been determined to be 20.3mm.

31.3. Tensile stress. Tensile load divided by the true area of cross-section of the specimen.

31.4. Tensile strain. Change in the effective gauge length by the application of tensile load divided by the original unloaded effective gauge length.

31.5. Brittle material. The stress-strain curve is linear up to fracture at about 1% to 2% elongation.

31.6. Brittle-ductile material. The stress-strain curve is curvelinear and the stress is gradually reduced after the peak point. The failure happens by gradually breaking the molecular bond within the material.

31.7. Ductile material. The material does not rupture in the direct tension test but elongates due to high strain.

31.8. Rubbery behavior. Materials that exhibit rubbery behavior can be stretched to extreme elongation without rupture.

31.9. Percent modulus decay. The percentage modulus deduction after 10sec of loading.

32. SUMMARY OF PRACTICE

32.1. This practice contains the procedure to measure the extensibility and the strain energy density of a bituminous sealant or filler using direct tension test (DTT). The material is bonded between two end-tabs made by Plexiglass and subjected to a constant strain rate at a specific temperature.

32.2. The test method is developed to select the bituminous sealant at temperatures where they exhibit rubbery behavior.

32.3. A linear variable differential transformer (LVDT) is used to measure the elongation of the test specimen as it is pulled in tension at a constant strain rate of 6%/min (1.2mm/min). A load cell is used to monitor the load during the test. The stress and strain at the point of rupture or peak load are reported.

33. SIGNIFICANCE AND USE

33.1. This test is intended for bituminous sealants applied to roadway joints and cracks.

33.2. The test temperature is determined to be the lowest temperature experienced by the pavement surface in the geographical area for which the sealant is intended.

33.3. The sealant extensibility is a parameter of the capacity of sealant to sustain large deformations due to crack expansion at low temperature without fracture.

33.4. The percent modulus decay is an indication of how fast the sealant can release the imposed loading. A higher percentage decay represents that the sealant can relax the load faster.

33.5. This method is intended for aged sealants, which could become stiffer or softer with age.

34. APPARATUS

34.1. Direct Tension Test (DTT) Device – The DTT system consists of two metal grips to hold the specimen, an environment chamber, a loading device, and a control and data acquisition system. The instrument must meet the requirements stated in AASHTO T314.

34.2. Specimen End Tabs and Gripping System – End tabs made from Plexiglass material having dimensions as described in Figure 1 that shall be bonded to both ends of the test specimen to transfer the tensile load to the sealant. The manufacturing requirement of the end tabs and the gripping system shall meet the requirement in AASHTO T314.

34.3. Chiller and test chamber – A calibrated circulated temperature control system shall have temperature range from -4°C to -40°C. The insulated test chamber shall be capable of maintaining a temperature of ± 0.1°C.

34.4. Specimen molds – The specimen molds should be made from aluminum. Molds shall have dimension as specified in Figure 1. A silicon-based release agent as described later in 7.2 shall be used to prevent sealant from adhering to the aluminum molds.

34.5. Laboratory Ovens – two standard laboratoryOven – Two forced-air convection ovens capable of producingreaching and maintaining a temperature of 200 ± 0.± 5°C. for heating sealant and molds.

35. REAGENTS AND MATERIALS

35.1. Fluid for Test Chamber – A fluid that is not absorbed by or does not affect the properties of the crack sealant being tested. The bath fluid shall be optically clear at the test temperature. Ethyl alcohol is suggested to use as a fluid for temperature control. The aqueous mixture of potassium acetate and deionized water used in the AASHTO T314 has been found to form turbid solution at temperature of -40°C.

35.2. Release Agent – A proper release agent to prevent crack sealant sticking to the mold. A silicon-based released agent is recommended.

35.3. Solvent – A solvent can properly clean the molds, end tabs, and plates. The parts cleaned by the solvent shall be submerged in the ethyl alcohol prior to use. This ensures the proper bond between sealant and end tabs.

35.4. Cleaning Cloths – Cloths for wiping molds, end tabs, and plates.

36. HAZARDS

36.1. Standard laboratory caution should be used in handling hot sealant in accordance to ASTM D5167, and required safety procedures should be followed when chemical agents are used.

37. VERIFICATION AND CALIBRATION

37.1. DTT – Follow the procedure as stated in AASHTO T314.

37.2. Oven and freezer – Calibrate the temperature with a thermometer that meets the requirements of ASTM E1. The thermometer calibration can be verified according to ASTM E77.

38. SAMPLES PREPARATION

38.1. Sample and prepare sealant according to ASTM D5167. See Note 1.

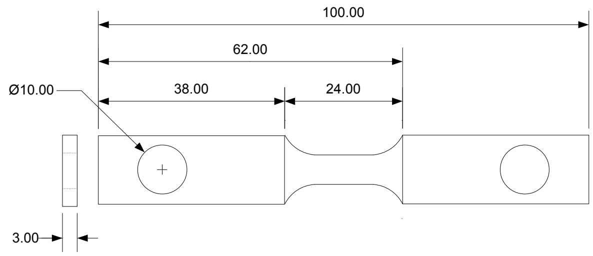
Note 1 – It is advantageous to sample about 500g sealant and sequentially pour specimens for all the tests, including the aging test (SC-3), the low temperature tests (SC-4 and SC-5), and the adhesion test (SC-6).

38.2. Anneal the sealant from which the test specimen is obtained by heating for 30 minutes. After 15 minutes, place the sealant in the oven, remove the sealant from the oven shortly, and stir the sealant by spatula to prevent segregation.

38.3. Follow the procedure 9.2 to 9.6 in AASHTO T314 with the following modification. See note 2 and 3.

Note 2 – If spray-type silicon based release agent is used, start from one side of the mold and slowly move toward the other side. Only one spray should be applied to the mold.

Note 3 – Place the molds and end tab assembly on top of a ceramic tile heated to 50°C lower than sealant pouring temperature. The ceramic tile should be placed in the preheated oven for 15 minutes.



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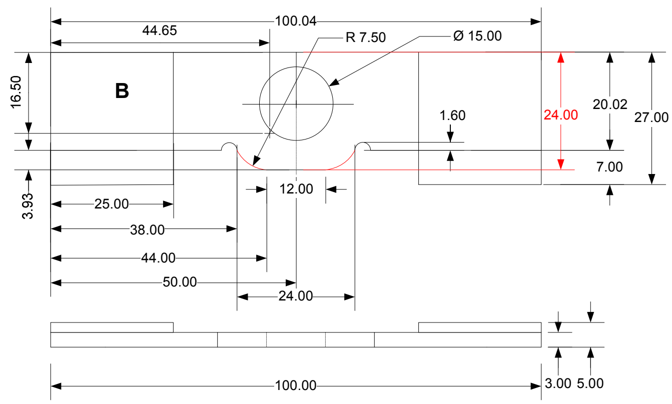
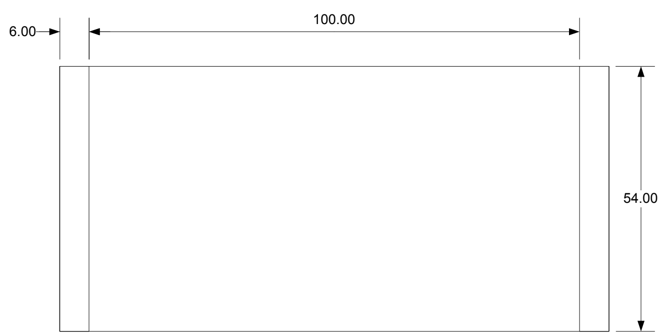


Mold Half



Mold Base

Figure 1. Dimension for DTT, end insert, and mold

39. CONDITIONING

39.1. Follow the procedure as stated in AASHTO T314.

40. PROCEDURE

40.1. Bring the DTT chamber to the test temperature (see Note 4).

Note 4: Select test temperatures in accordance with the material specification, e.g., SC-1, ASTM D6373-99.

40.2. Prepare four test specimens according to section 10.

40.3. Follow the procedure 12.2 to 12.3 in AASHTO T314 with the modification as in notes 5 and 6.

Note 5 – Adjust the load frame to allow 20mm traveling distance then place the specimen on the loading pin. Remove the slack between the specimen and the loading pins.

Note 6 – Manually adjust the thumb wheel on the control box to apply tension in the specimen until a load of 1±0.5 N is shown on the screen. Then calibrate the stroke and load back to zero.

40.4. Set the strain rate to 6%/min (This is equivalent to 1.2mm/min) and start the test.

40.5. After the specimen fractures, degradation is observed, or maximum traveling distance is reached (whichever comes first), stop the test and remove the specimen from the loading frame.

40.6. The extensibility is identified as follows: When the specimen fractures (breaks into two pieces), the extensibility is easily identified as the strain at peak load (maximum stress). When the specimen does not fracture, but reaches a maximum stress and then flows without fracture, the extensibility is recorded as the strain corresponding to the maximum stress. When the specimen does not fracture or load reduction is not observed, the extensibility is recorded as the strain at the end of the traveling distance.

40.7. Repeat 12.3 to 12.6 for the remaining three specimens.

40.8. After testing is complete, discard the bituminous portions of the spent specimens and clean the end tabs by soaking them in solvent and wiping with a soft cloth. After wiping the end tabs, use a detergent soap solution to remove any oil film residue left by the cleaner material. Alternatively, use a degreasing spray cleaner. Clean the end tabs thoroughly. A grease film on the sealant bonding area can create a weak bond causing bond failures.

41. CALCULATIONS

41.1. For each test result, calculate the engineering stress-strain

$$\sigma_f = \frac{P_f}{A_0} \qquad (13.1)$$

$$\varepsilon_f = \frac{\Delta L_f}{L_0} \tag{13.2}$$

where,

σ_f = peak stress;
P_f = measured load at peak;
A_0 = original cross-sectional area (=18mm^2);
ε_f = measured strain at peak load;
ΔL_f = measured elongation at failure (ΔL); and
L_e = gauge length (=20.3mm).

41.2. For each test result, calculate the true stress-strain

$$\tilde{\varepsilon} = \frac{\Delta L_f}{L_0}. \tag{13.3}$$

$$\tilde{\sigma} = \frac{P_f}{A_i} = \frac{P \times e^{(\dot{\varepsilon}t)}}{A_0} \tag{13.4}$$

where,

$\tilde{\sigma}$ = true stress;
$\tilde{\varepsilon}$ = true strain;
P_f = measured load at peak;
A_0 = original cross-sectional area (=18mm^2);
$\dot{\varepsilon}$ = strain rate.

41.3. The extensibility is identified as $\tilde{\varepsilon}$.

41.4. Select the best three test results which give the best coefficient of variation of the extensibility. Calculate the mean and standard deviation for SED from the selected three test results.

41.5. Calculate the percent modulus decay.

13.5.1 From the Boltzman superposition principle, the stress-strain relationship for a viscoelastic material can be expressed as Equation 13.5.

$$\sigma(t) = \int_0^t E(t-t')\frac{d\varepsilon(t')}{dt'}dt' \tag{13.5}$$

where,

σ(t) is stress history;
ε(t) is strain history; and
E(t) is the relaxation modulus.

13.5.2 The Prony series (generalized Kelvin model) is used to describe the viscoelastic behavior of hot-poured crack sealants as presented in Equation 13.6.

$$E(t) = E_0 - \sum_{i=1}^{K} E_i (1 - e^{-t/\tau_i}) \tag{13.6}$$

where,

E(t) = the relaxation modulus at time t,

Ei = material constants, and

τ_i = retardation times.

13.5.3 Substituting Equation 13,6 into Equation 13.5, the expression of the stress becomes

$$\sigma(t) = E_0 \varepsilon(t) - \int_0^t \sum_i^N E_i (1 - e^{\frac{-(t-t')}{\tau_i}}) \frac{d\varepsilon(t')}{dt'} dt' \tag{13.7}$$

13.5.4 In the DT test, sealant is subjected to a constant strain rate beginning at time zero, $\varepsilon(t) = \begin{Bmatrix} 0 \text{ for } t < 0 \\ \dot{\varepsilon}t \text{ for } t \geq 0 \end{Bmatrix}$, with $\dot{\varepsilon}$ as the strain rate. The above convolution integral then can be solved as follows:

$$\sigma(t) = E_0 \varepsilon(t) - \sum_i^N E_i R \left[t - \tau_i \left(1 - e^{\frac{-t}{\tau_i}} \right) \right] \tag{13.8}$$

13.5.5 The equation is used to fit the experimental data by means of the nonlinear least squares (NLS) technique to obtain the material constants E0, Ei, and τ_i.

41.6. The percent modulus reduction after 10sec loading is calculated as following

$$M_{10} = \frac{E(10) - E(0)}{E(0)} \times 100 \tag{13.9}$$

42. REPORT

42.1. Report the sealant name and supplier, lot number, date received, date sampled according to ASTM D5167.

42.2. Report the date and time of test, test temperature, rate of elongation, average extensibility, average SED and their standard deviation, peak load, and type of fracture (fracture or no fracture).

43. PRECISION AND BIAS

43.1. Confidence intervals of 95% should be constructed around the average of the calculated extensibility from the results of the four replicates. The closest three measurements will then be used to calculate the coefficient of variation while the fourth replicate will be discarded. A coefficient of variation less than 15% is desirable.

44. KEYWORDS

44.1. Hot-poured bituminous sealant; joint; crack; direct tension test; extensibility; strain energy density; low temperature; pavement maintenance.

Test Method for Measuring Adhesion of Hot-poured Crack Sealant Using Direct Adhesion Tester
Sealant Consortium Designation: SC-7

45. SCOPE

45.1. The direct adhesion test is used to determine the adhesion strength of hot-poured crack sealant at the application temperatures.

45.2. The adhesion test is a test of fracture. The object of the test is to apply tensile forces to the interface between sealant and aggregate. Sealant is confined between two half cylindrical aggregate (aluminum can be used for standard test). The applied force and displacement can be recorded as functions of time. Energy required to break the bond can be calculated by measuring the area under the load-displacement curve. This energy can be considered a measure of bonding. In addition, the maximum force to failure can be reported as adhesion strength.

45.3. These guidelines do not purport to address all of the safety concerns, if any, associated with their use. It is the responsibility of the user of this standard to establish and follow appropriate health and safety practices and to determine the applicability of regulatory limitations prior to use.

46. REFERENCED DOCUMENTS

2.1. ASTM Standards:

- D5167-03, Standard Practice for Melting Hot-Applied Joint and Crack Sealant and Filler for Evaluation.
- D5329-04, Standard Test Methods for Sealants and Fillers, Hot-Applied, for Joints and Cracks in Asphaltic and Portland Cement Concrete Pavements
- D6690-06, Standard Specification for Joint and Crack Sealants, Hot Applied, for Concrete and Asphalt Pavements
- D4541-02, Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers
- E220-07, Test Method for Calibration of Thermocouples by Comparison Techniques.
- E1, Specification for ASTM Thermometers
- C670-03, Practice for preparing Precision and Bias Statements for Test Methods for Construction Materials.

2.2. Sealant Consortium (SC) Standards:

- SC-1, Guidelines for Graded Bituminous Sealants.

- SC-2, Test Method for Measuring Apparent Viscosity of Hot-poured Crack Sealant Using Brookfield Rotational Viscometer RV Series Instrument
- SC-3, Method for the Accelerated Aging of Bituminous Sealants.
- SC-4, Test Method to Measure Tracking Resistance of Bituminous Sealants
- SC-5, Method to Measure Low Temperature Sealant Flexural Creep Stiffness at Low Temperature by Bending Beam Rheometer.
- SC-6, Method to Evaluate Sealant Extensibility at Low Temperature by Direct Tension Test.
- SC-8, Blister Method to Predict the Interfacial Fracture Energy of Bituminous Sealants.

47. TERMINOLOGY

47.1. Hot-poured crack sealants are hot-poured modified asphaltic materials used in pavement cracks and joints.

47.2. Adhesion is the maximum force and energy required to separate bituminous sealant from a standard substrate.

48. SUMMARY OF METHOD

48.1. Crack sealant material is homogenized, following the procedure given in ASTM D5176. For each test including four replicates, 40g of sealant will be cut and heated to the manufacturer's recommended pouring temperature. Sealant will be poured in the mold placed between the two half cylindrical aggregate samples. The mold confines the sealant at the bottom and between the two aggregate samples at the sides.

49. SIGNIFICANCE AND USE

49.1. This procedure is designed to measure the adhesion of hot-poured sealant to aggregate.

49.2. Sealants must be rehomogenized (ASTM D5176) before measuring the adhesion by this method.

50. APPARATUS

50.1. Modified DTT machine

50.2. Chiller which can reach -40°C± 0.5°C

50.3. Laboratory oven — any standard laboratory oven capable of producing and maintaining temperature ranging from 170°C to 193°C ± 0.5°C

50.4. Release agent

50.5. Four test setups, four molds and rubber band

51. HAZARDS

51.1. Standard laboratory caution should be used in handling hot sealant in accordance to ASTM D5167-03, and when using the Direct Adhesion Tester (DAT). Required safety procedures should be followed when chemical agents are used.

52. PREPARATION OF APPARATUS

52.1. The Direct Adhesion Tester (DAT) bath must be adjusted to specific temperature. The sitting posts must be leveled to function properly.

53. CALIBRATION AND STANDARDIZATION

53.1. Temperature of the ovens should be calibrated according to each user's quality assurance program.

53.2. Temperature of the chiller should be calibrated according to each user's quality assurance program.

53.3. Thermometer (temperature detector) — verify the calibration of the temperature sensing device to +/-0.1°C every six months.

54. PREPARATION OF SAMPLES AND TEST SPECIMENS

54.1. All adhesion strength measurements must be performed on rehomogenized sealant. Follow the procedure for homogenization given in ASTM D5176, Melting of Hot-Applied Joint and Crack Sealant and Filler for Evaluation. It is recommended that a minimum of 400g of sealant be homogenized.

54.2. Once homogenized, hot sealant should be molded, cooled, and stored for later usage. To store the sealant, it is recommended that a can or plastic-lined box be used. The mold must be of sufficient size that the sealant depth is no greater than 100mm, to allow for rapid cooling.

54.3. Adjust the oven's temperature to recommended pouring temperature for sealant being tested.

54.4. Turn on the DTT machine, load the program and cool the chiller to test temperature.

54.5. Place a half cylinder of aggregate in each grip and tighten it.

54.6. Assemble the setup which is composed of an aggregate sample on each side and an aluminum mold in between. Wrap a rubber band around the setup to keep all the components in place.

54.7. Place the notch on the edge of one of the aggregates/ aluminum.

54.8. Prepare a can of sealant by cutting 40g of homogenized sealant for each set of four samples.

54.9. Place the can in the oven for 15 minutes, remove it from the oven, stir the sealant thoroughly, and place it back in the oven for another 15 minutes.

54.10. Remove the can from the oven, stir the sealant thoroughly, and pour into all the assembled setups.

54.11. Use four replicates for each sealant; care should be taken in filling up the molds to prevent any trapped air bubbles in the sample.

54.12. Let samples sit one hour at room temperature.

54.13. Trim the excessive sealant away with a hot spatula.

54.14. Move spatula once over and parallel to the interface of the sample; trimming direction shall not be changed during trimming.

54.15. Use well-heated spatula to prevent any shearing of the sealant.

54.16. Use two tanks to grab the plate underneath the specimen, and place the specimens in the cooling bath.

54.17. Remove the plates underneath each specimen and leave specimens in the bath for 15 minutes.

54.18. Remove one specimen at a time from the bath; place it on a flat surface.

54.19. Flip the specimen, keep the two end pieces still using point fingers and remove the mold with your thumb.

54.20. Place the specimen back in the bath.

54.21. Repeat 8.16 until all specimens are demolded.

54.22. Leave the samples for 45 minutes in the bath prior to testing.

54.23. Turn on the DAT's test builder, adjust the machine so the sample can sit freely on the posts, and place the specimen on the posts. Care should be taken not to disturb the specimen.

54.24. Tare the load to zero.

54.25. Run the test and record the data.

55. CALCULATION OF RESULTS

55.1. Find the maximum load to failure and its correspondent displacement.

55.2. Report the maximum load as adhesion strength.

55.3. Calculate the area under the Load-Displacement curve up to the maximum load.

55.4. Divide the area calculated in 11.3 by the cross section of one the end pieces.

55.5. Report the calculated value in 11.4 as the bonding energy.

56. REPORT

56.1. Report the following information: sealant identification and supplier, lot number, date received, date of apparent viscosity measurement, recommended pouring temperature, safe heating temperature, and any deviations from test temperature.

57. PRECISION AND BIAS

57.1. *Single Operator Precision (Repeatability)*—The Results obtained in the same laboratory, by the same operator using the same equipment, in the shortest practical period of time, should not be considered suspect unless the difference in the two results, expressed as a percent of their mean, exceeds the value given Table 1.

Table 1—Precision Estimates

Condition	Coefficient of Variation (1s%)a
Single-Operator Precision:	
Average Viscosity (Pa.s)	19.3

Note 2—The precision estimates given in Table 2 are based on the analysis of test results from seven sealants with a wide range of rheological properties. The data analyzed includes results from two operators in the same laboratories who conducted each test in four replicates.

58. KEYWORDS

58.1. Hot-poured bituminous sealant; fillers; joint; crack; adhesion; bond

Figure 1. Heat 35 gram of homogenized sealant at recommended pouring temperature for 30 minutes

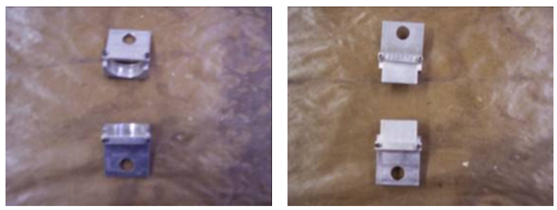


Figure 2. Placing each end pieced in one grip and tighten it

Figure 3. Spray release agent on the mold

Figure 4. Place two end pieces on the mold

Figure 5. Hold the assembly in place using a rubber band


Figure 8. Remove the sealant from the oven and mix thoroughly and pour it in the mold from one corner


Figure 9. Let sealant set for one hour at room temperature


Figure 10. Trim the extra sealant away using a heated spatula


Figure 11. Grab the base plate with two tangs and place it in cooling bath


Figure 12. Remove the base plates

Figure 13. Leave the specimens in cooling bath for 15 minutes

Figure 14. Remove one specimen at a time holding its center using a tang

Figure 15. Flip the specimen on a flat surface

Figure 16. Remove the rubber band

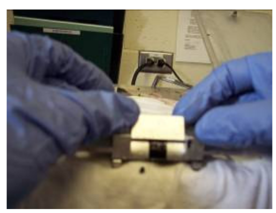


Figure 17. Remove the mold while keeping the two end pieces in place

Figure 18. Flip back the specimen and push the notch horizontally until it comes off

Figure 19. Grab the specimen from two corners and place it back in the bath

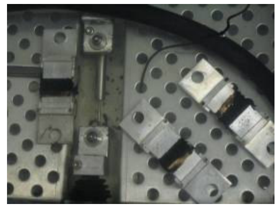


Figure 20. Leave the specimens for 45 minutes in the bath

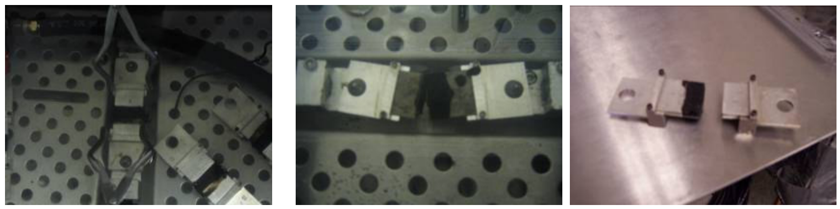


Figure 21. Turn on the DTT machine, load adhesion program, mount the specimen, tare the load to zero, run the test and record the data

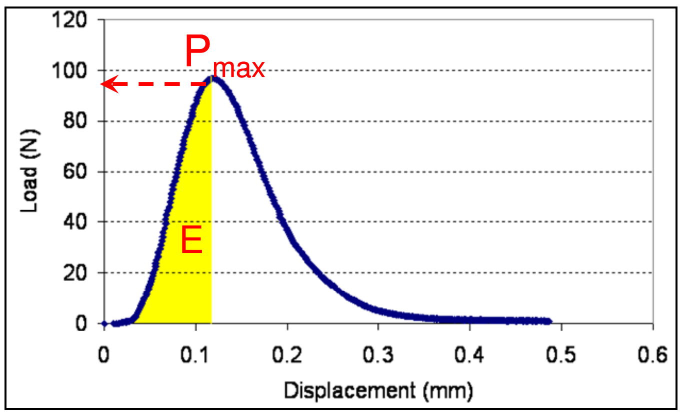


Figure 22. Calculate the P_{max} and E as explained before

Test Method for Measuring Interfacial Fracture Energy of Hot-poured Crack Sealant Using a Blister Test

Sealant Consortium Designation: SC-8

59. SCOPE

59.1. The blister test is used to determine the adhesion strength of hot-poured crack sealant at the application temperatures..

59.2. The blister test is a test of fracture. The objective of the test is to inject alcohol between a substrate (aluminum plate) and hot-poured crack sealant in such a way that the sealant is detached from the substrate in the form of a blister. The energy balance principle is used to calculate the adhesive fracture energy, which is a fundamental and unique property of each individual interface. Pressure of the injected alcohol is measured using a pressure transducer, while the height of the blister in the center of the dome will be measured through an LVDT. Using the data collected from the test period and the energy balance principle, one can calculate the adhesive fracture energy.

59.3. These guidelines do not purport to address all of the safety concerns, if any, associated with their use. It is the responsibility of the user of this standard to establish and follow appropriate health and safety practices and to determine the applicability of regulatory limitations prior to use.

60. REFERENCED DOCUMENTS

2.1. ASTM Standards:

- D5167-03, Standard Practice for Melting Hot-Applied Joint and Crack Sealant and Filler for Evaluation.
- D5329-04, Standard Test Methods for Sealants and Fillers, Hot-Applied, for Joints and Cracks in Asphaltic and Portland Cement Concrete Pavements
- D6690-06, Standard Specification for Joint and Crack Sealants, Hot Applied, for Concrete and Asphalt Pavements
- D4541-02, Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers
- E220-07, Test Method for Calibration of Thermocouples by Comparison Techniques.
- E1, Specification for ASTM Thermometers
- C670-03, Practice for preparing Precision and Bias Statements for Test Methods for Construction Materials.

2.2. Sealant Consortium (SC) Standards:

- SC-1, Guidelines for Graded Bituminous Sealants.
- SC-2, Test Method for Measuring Apparent Viscosity of Hot-poured Crack Sealant Using Brookfield Rotational Viscometer RV Series Instrument
- SC-3, Method for the Accelerated Aging of Bituminous Sealants.
- SC-4, Test Method to Measure Tracking Resistance of Bituminous Sealants
- SC-5, Method to Measure Low Temperature Sealant Flexural Creep Stiffness at Low Temperature by Bending Beam Rheometer.
- SC-6, Method to Evaluate Sealant Extensibility at Low Temperature by Direct Tension Test.
- SC-7, Test Method for Measuring Adhesion of Hot-poured Crack Sealant Using a Direct Adhesion Test.

61. TERMINOLOGY

61.1. Hot-poured crack sealants are hot-poured modified asphaltic materials used in pavement cracks and joints.

61.2. Interfacial Fracture Energy is the energy required to separate bituminous sealant from a standard substrate.

62. SUMMARY OF METHOD

62.1. Crack sealant material is homogenized, following the procedure given in ASTM D5176. For each specimen, 80g of sealant will be cut and heated to the manufacturer's recommended pouring temperature. To prepare the specimen an aluminum mold is assembled on top of the annular-shaped substrate. An aluminum plug is inserted to close the orifice. Sealant will be poured on top of the aluminum plate to provide a film of 4.6mm thickness. After cooling and conditioning the specimen, the test will be conducted using a servo-hydraulic machine.

63. SIGNIFICANCE AND USE

63.1. This procedure is designed to measure the adhesion strength of hot-poured sealant to aggregate.

63.2. Sealants must be rehomogenized (ASTM D5176) before measuring adhesion by this method.

64. APPARATUS

64.1. Servo-hydraulic machine

64.2. Chiller which can reach -40°C± 0.5°C

64.3. Blister test software

64.4. Laboratory oven — any standard laboratory oven capable of producing and maintaining a temperature ranging from 170°C to 193°C ± 0.5°C

64.5. Silicon-based release agent

64.6. Adhesive-backed fluoropolymer (FEP) film

64.7. Sitting plate for cooling period

65. HAZARDS

65.1. Standard laboratory caution should be used in handling hot sealant in accordance to ASTM D5167-03, and when using the Blister Tester. Required safety procedures should be followed when chemical agents are used.

66. PREPARATION OF APPARATUS

66.1. The Blister Test bath must be adjusted to specific temperature. The LVDT and sitting base must be leveled to function properly.

67. CALIBRATION AND STANDARDIZATION

67.1. Temperature of the ovens should be calibrated according to each user's quality assurance program.

67.2. Temperature of the chiller should be calibrated according to each user's quality assurance program.

67.3. Thermometer (temperature detector) — verify the calibration of the temperature sensing device to +/-0.1°C every six months.

67.4. Blister machine needs to be calibrated and trapped air be removed.

67.4.1.1. For calibration of the equipment, follow the specifications defined by the manufacturer.

68. PREPARATION OF SAMPLES AND TEST SPECIMENS

68.1. All adhesion strength measurements must be performed on rehomogenized sealant. Follow the procedure for homogenization given in ASTM D5176, Melting of Hot-Applied Joint and Crack Sealant and Filler for Evaluation.

68.2. Once homogenized, hot sealant should be molded, cooled, and stored for later usage. To store the sealant, it is recommended that a can or plastic-lined box be used. The mold must be of sufficient size that the sealant depth is no greater than 100mm, to allow for rapid cooling.

68.3. Prepare 4 cans of sealant by cutting 80g of homogenized sealant for each.

68.4. Heat the sealant to the recommended pouring temperature for 15 minutes.

68.5. Remove the can from the oven, stir the sealant thoroughly, and place it back in the oven for another 15 minutes.

68.6. Use a compass to draw a circle on fluoropolymer (FEP) film.

68.7. Punch out the circle using a sharp manual punch.

68.8. Peel FEP film and place it on a level surface, adhesive side up.

68.9. Place the needle on the center of the FEP film.

68.10. Place the flat side of the plug on top of the needle and let the needle go through the plug.

68.11. Press the plug on the film gently to make sure it has adhered to the film.

68.12. Spray release agent on the plug while keeping it inclined.

68.13. Place the plug on top of annular disk; care should be taken not to contaminate the surface of the disk.

68.14. Place the disk and the plug on the sitting plate.

68.15. Assemble the mold on top of the disk.

68.16. Stack two molds on top of each other to achieve double the thickness.

68.17. Wrap the mold with a rubber band stretched through the groove around the mold, to keep the components in place.

68.18. Pour the sealant on the top-center of the plate to fill the mold.

68.19. Let the specimen sit at room temperature for one hour (curing time).

68.20. Trim the extra sealant away using a heated potty knife.

68.21. Place the specimen in the cooling bath for 15 minutes.

68.22. Demold the specimen.

68.23. Remove the rubber band, pull out the plug, dissemble the mold, and place the specimen back in the bath.

68.24. Wait for 45 minutes before running the test.

68.25. Open the outlet valve to prevent any pressure being applied to the sample while assembling.

68.26. Lower the piston.

68.27. Plug the sample in the setup, and clamp/screw it tightly.

68.28. Adjust the moving rate of the piston to 0.12mm/s.

68.29. Start running the test.

68.30. Stop the test when pressure drops to 40% of the peak pressure.

68.31. Find the peak pressure and the corresponding height of the blister.

69. CALCULATION OF RESULTS

69.1. Calculate initial interfacial fracture energy (IFE_{ini}) by multiplying half of the peak pressure by the blister height.

Note. If the height of the blister is more than 0.5"; stop the test, report sealant is too ductile to fail.

69.2. Calculate IFE for all pairs of pressure and blister height recorded after the peak pressure by multiplying half of the pressure by blister height for each pair of recorded data.

69.3. Calculate IFE_{prop} by taking the average of all IFE values including IFE_{ini}

69.4. Run four specimens for each sealant, average values of interfacial fracture energy, and report the average of the best three out of four results as the bond energy.

Note. It is recommended that failure surfaces be examined for each replicate to ensure adhesive failure occurred. If sealant breakage occurred discard the data.

70. REPORT

70.1. Report the following information: sealant identification and supplier, lot number, date received, date of apparent viscosity measurement, recommended pouring temperature, safe heating temperature, and any deviations from test temperature.

71. PRECISION AND BIAS

71.1. *Single Operator Precision (Repeatability)*—The Results obtained in the same laboratory, by the same operator using the same equipment, in the shortest practical period of time, should not be considered suspect unless the difference in the two results, expressed as a percent of their mean, exceeds the value given Table 1.

Table 1—Precision Estimates

Condition	Coefficient of Variation (1s%)a
Single-Operator Precision:	
Average Viscosity (Pa.s)	8.77

Note 2—The precision estimates given in Table 2 are based on the analysis of test results from 12 sealants with a wide range of rheological properties. The data analyzed includes results from two operators in the same laboratories who conducted each test in four replicates.

72. KEYWORDS

72.1. Hot-poured bituminous sealant; joint; crack; adhesion; bond, blister, Interfacial fracture energy

Figure 1. Heat four cans of 80g homogenized sealant for 15 minutes at the recommended pouring temperature

Figure 2. Remove the can, mix the sealant thoroughly and place it back in the oven for another 15 minutes

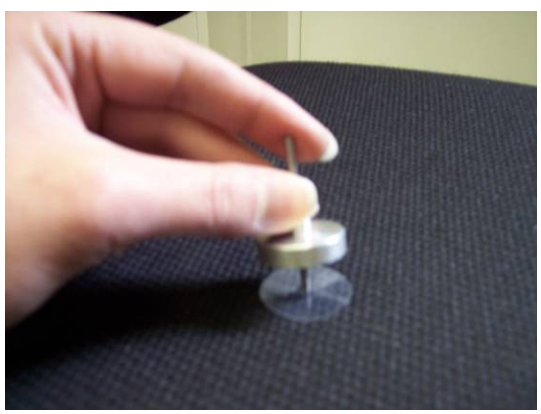


Figure 3. Peel the transparent film, and place it on a flat surface with adhesive side up, place the pin on its center and let the plug slide through the pin to the film

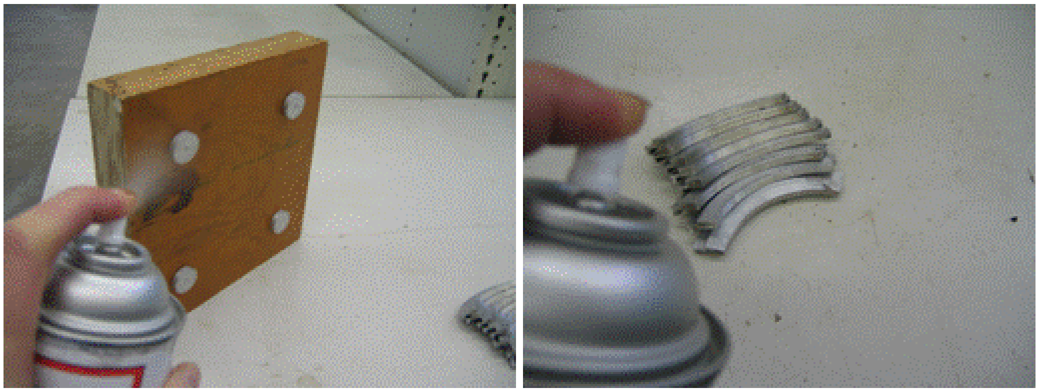


Figure 4. Spray release agent on the plugs and the molds

Figure 5. Place the plug on the annular plate, and pull it from underneath the plate until it fits in the orifice and the film edges sits on the plate

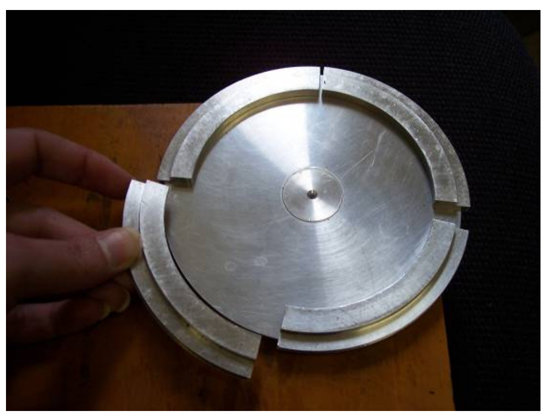


Figure 6. Assemble the molds on top of the plate

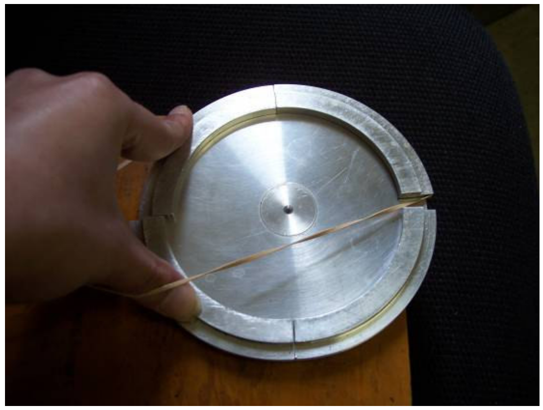


Figure 7. Use rubber band to keep the mold together

Figure 8. Mix sealant thoroughly and pour on center top of the plug and let it flow to fill the mold

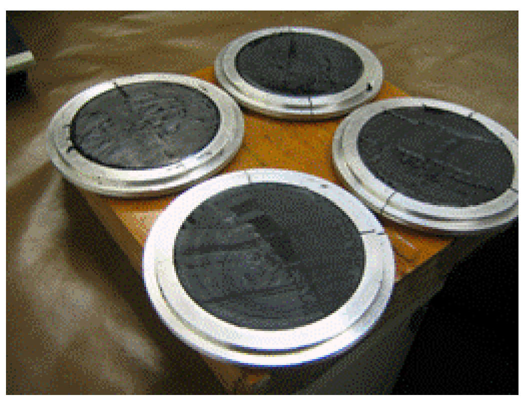


Figure 9. Let the sealant set for one hour at room temperature

Figure 10. Trim the extra sealant away with a heated potty knife

Figure 11. Remove the rubber band

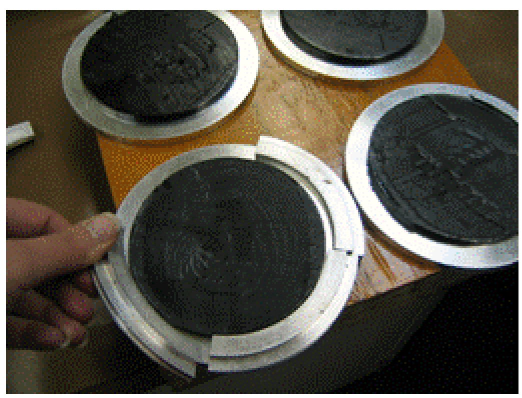


Figure 12. Remove the molds gently

Figure 13. Place the plated on the rack; allow enough clearance between each two specimens

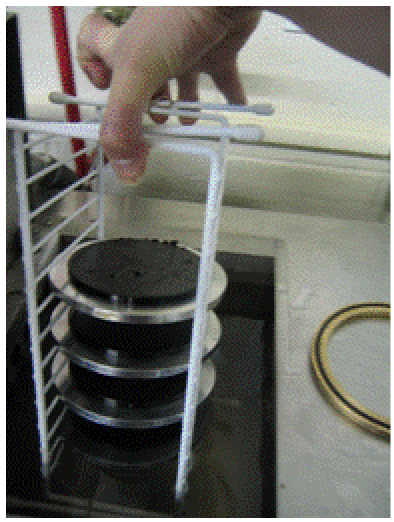


Figure 14. Place the rack in the bath for 15 minutes

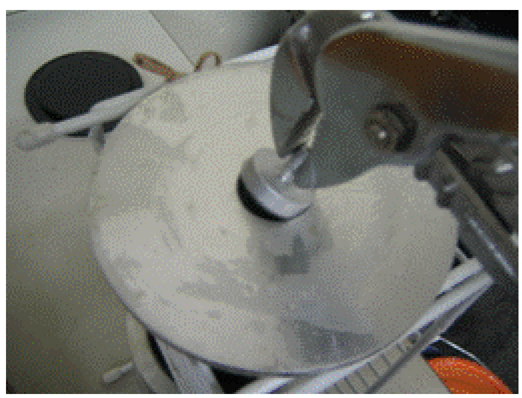


Figure 15. Deplug the specimen and place them back on the rack

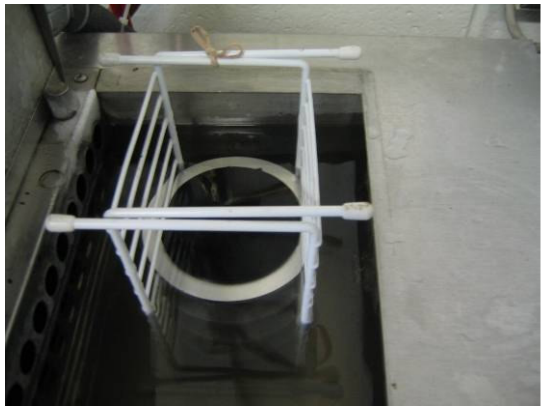


Figure 16. Condition the specimens in the bath for 20 minutes

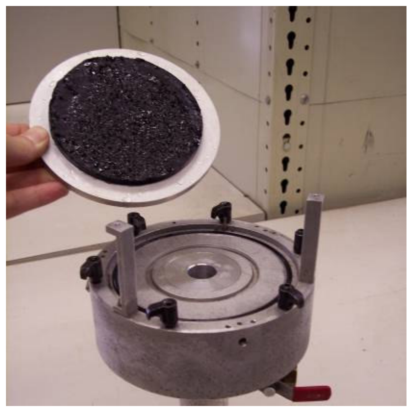


Figure 17. Place the first specimen on the base setup

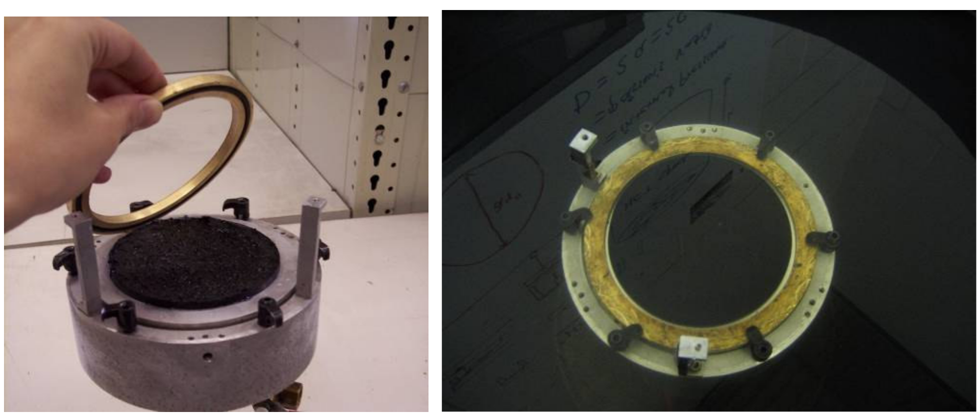


Figure 18. Place the supporting ring on top and tighten the screws

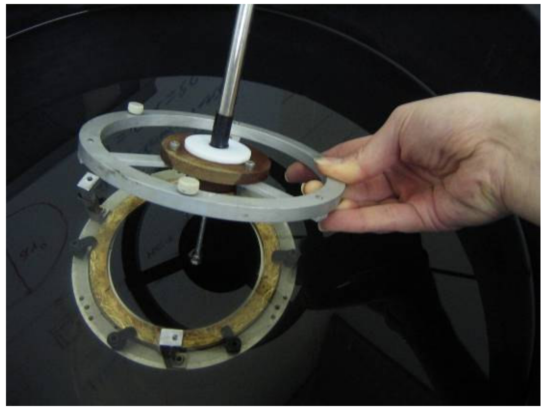


Figure 19. Assemble the LVDT

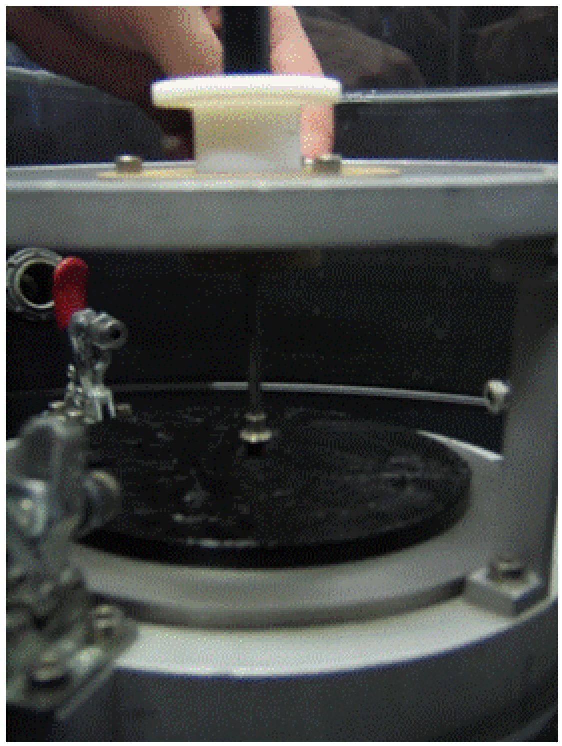


Figure 20. Close the outlet valve and start the test

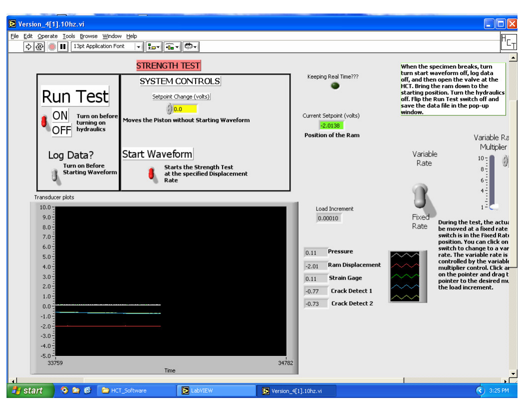


Figure 21. Run the test

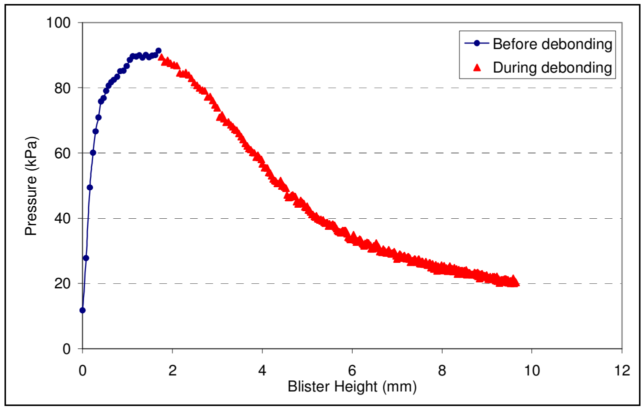
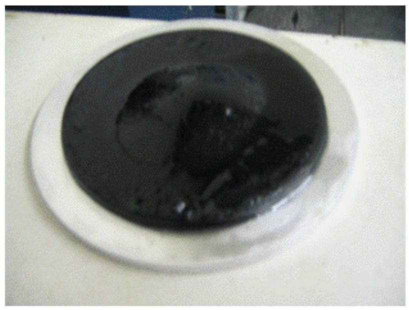


Figure 22. Record the pressure and displacement of the blister as functions of time



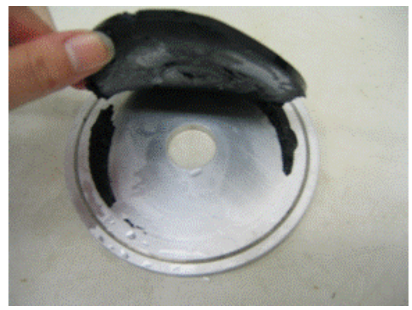


Figure 23. Remove the sample and examine if the failure type was adhesive in which sealant separates from the plate

Practice for the

Accelerated Aging of Bituminous Sealants and Fillers With a Vacuum Oven

Sealant consortium designation: SC-3 (May 2nd, 2007)

1 Scope

1.1 This practice applies to bituminous crack sealants and fillers used in the construction and maintenance of roadways.

1.2 The practice covers the accelerated aging of the bituminous materials by means of elevated temperatures and vacuum.

2 Referenced documents

2.1 ASTM Standards

- D573 Standard Test Method for Rubber—Deterioration in an Air Oven
- D5167 Standard Practice for Melting Hot-Applied Joint and Crack sealant and Filler for Evaluation
- D6521 Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)
- E1 Standard Specification for ASTM Liquid-in-Glass Thermometers
- E77 Test Method for the Inspection and Verification of Thermometers
- E145 Standard Specification for Gravity-Convection and Forced-Ventilation Ovens

2.2 Documents of the Sealant Consortium (SC)

- SC-1 Guidelines for graded bituminous sealants
- SC-2 Method for the evaluation of the tracking resistance of bituminous sealants by dynamic shear rheometry.
- SC-4 Bending beam method to measure low temperature sealant stiffness.
- SC-5 Direct tensile method to measure low temperature sealant elongation.
- SC-6 Blister method to measure the adhesion of bituminous sealants

3 Terminology

3.1 Bituminous sealants and fillers. Polymer- or rubber-modified bitumens most often formulated with a mineral filler.

4 Summary of practice

4.1 Sealant is sampled according to ASTM D5167 and poured in a stainless steel pan to provide a film about 2 mm thick. The sealant is then aged at 115°C for 16 hours under vacuum.

5 Significance and Use

5.1 This procedure is designed to simulate the aging and weathering of bituminous sealants and fillers.

5.2 Materials aged with this procedure are best used to evaluate sub-zero characteristics.

5.3 For materials with different bitumen source, polymer grade and filler types and content, there is no unique correlation between the accelerated conditions and the time of in-service weathering. The accelerated aging leads to sealant rheology typical of sealants weathered one to ten years in the field.

5.4 Sealants must be sampled according to ASTM D5167 before being aged.

6 Apparatus

6.1 Vacuum oven – Calibrated Type IA vacuum oven specified in ASTM E145 with a vacuum valve, a bleed valve and a pressure gauge in inches of mercury. The oven must be capable of maintaining a temperature of 115°C ± 1°C at the sample shelf under a vacuum of 29.9 inches of mercury (see note 1). Refer to ASTM E145 to verify temperature uniformity. The oven should be of a size sufficient to accommodate a minimum of 8 sample pans of 6 inches on two shelves.

6.2 Vacuum pump – a one- or two-stage mechanical pump capable of achieving a vacuum of 99.9% or better in 10 minutes or less. See note 1.

Note 1. At sea level, the achievable vacuum is 29.92 inches of mercury. See the appendix for the effect of elevation on the achievable vacuum reading.

6.3 Laboratory oven – calibrated Type IA oven specified in ASTM E145.

6.4 Stainless steel pans – Pans of sufficient dimensions such that 30 g of melted sealant will provide a film about 2 mm thick. See note 2.

Note 2. PAV pans, used in ASTM D6521, work well for the purpose of 6.4.

7 Hazards

7.1 Standard laboratory caution should be used in handling and remixing hot sealant in accordance to ASTM D5167.

7.2 This practice does not purport to address all of the safety concerns, if any, associated with their use. It is the responsibility of the user of this standard to establish and follow appropriate health and safety practices and to determine the applicability of regulatory limitations prior to use.

8 Verification and Calibration

8.1 Temperature and vacuum control of the ovens should be calibrated according to each user's quality assurance program.

8.2 Temperature detector – verify the calibration of the temperature sensing device to 0.1°C every six months

8.3 Vacuum gauge – calibrate the vacuum gauge to an accuracy of 1% every six months.

8.4 Verify that after preheating the oven, the aging temperature is obtained within about 90 min after the application of vacuum. An example is shown in Figure 1.

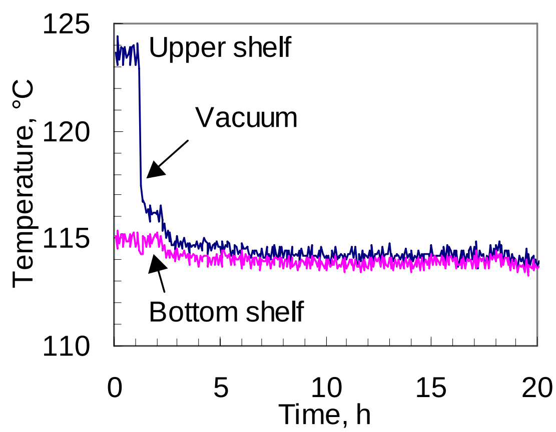


Figure 1. Typical change in temperature over time at a top and bottom shelf of an a vacuum oven before and after evacuation.

9 Preparation

9.1 Apply vacuum and pre-heat the vacuum oven to 115°C ± 1°C. This typically takes 2h to 4h.

9.2 Preheat the laboratory oven to 180°C.

9.3 Sample 400 g to 500 g of material according to ASTM D5176. Pour 30 g ± 0.5 g of hot material into a stainless steel pan. This provides a sealant film about 2 mm thick. See note 3.

Note 3. At the same time, samples can be poured for other tests, including the tracking test (SC-2), the low temperature tests (SC-4 and SC-5), and the adhesion test (SC-6).

10 Procedure

10.1 Close the vacuum valve on the vacuum oven and slowly open the bleed valve. Once atmospheric pressure is reached, open the door and place the sealant pan in the oven. The oven door must be left opened for less than 1 min. Re-apply vacuum. See note 4.

Note 4: During this step, the vacuum pump must be left running.

10.2 Start timing once the vacuum has reached 29 inches of mercury. Maintain a vacuum better than 29 inches of mercury, and the temperature of 115 ± 1°C for 16 hours ± 10 minutes (see note 1).

10.3 After 16 hours, slowly release the vacuum with the bleed valve and transfer the pan to the oven preheated to 180°C. Heat the sealant for 5 minutes or until it is sufficiently fluid to pour into shape for the tests according to SC-2, SC-4, SC-5 or SC-6. Pans may be scraped to collect maximum amount of sealant. See note 5.

Note 5. Allow 24 h at room temperature before an evaluation of the properties according to SC-4, SC-5 or SC-6.

11 Report

11.1 Report the following information: sealant name and supplier, lot number, date received, date aged, aging temperature and vacuum, total aging time in hours and minutes, any deviations from test temperature and vacuum.

12 Precision and bias

12.1 The precision and the bias have not been measured.

13 Keywords

13.1 Pavement, roadways, maintenance, sealant, cracks, joints, aging, weathering, specification, guidelines, practice.

Appendix

The vacuum reading on the oven gauge depends on the atmospheric pressure outside the oven, which depends on elevation (excluding the effect of weather on pressure). The maximum achievable vacuum reading (P) at an elevation h is given by

$$P(h) = Po \exp(-mgh/kT)$$

where Po is the pressure at sea level, m is the average molar mass of dry air, g is the acceleration due to gravity, k is the Boltzmann constant and T is the temperature in Kelvin.

Considering a laboratory temperature of 22°C and vacuum readings in inches of mercury, the above equation can be simplified to

$$P(h) = 29.92 \exp(-hc)$$

where h is the elevation in ft and c is 0.000351 ft^{-1} . If the elevation is taken in meters, c is 0.000115 m^{-1}. As examples, in Denver, CO, the elevation is 5433 ft and the maximum attainable vacuum is 24.7 in Hg. In Edmonton, AB, with an elevation of 650 m, the achievable vacuum is 27.8 in Hg, and in Ottawa, ON, the elevation is 188 m so the achievable vacuum is 29.3 in Hg.

National Cooperative Highway Research Program
Projects in the Fiscal Year 2010 Program

PROJECTS CONTINGENT ON THE AVAILABILITY OF FUNDS

♦ Project 01-48

Integrating Pavement Preservation into the Design Process

Research Field: Design
Source: California
Allocation: $500,000
NCHRP Staff: Amir N. Hanna

State highway agencies increasingly focus their attention on a pavement preservation approach for keeping sound pavement assets longer and effectively addressing poor pavements. Pavement preservation is a proactive approach, and it should be considered in the beginning of the design process because preservation activity will have an impact on the pavement performance and service life. Currently the mechanistic-empirical pavement design guide only considers the process for designing new pavements or rehabilitation. There is no consideration for intermediate treatments to preserve the functional characteristics of the pavement surface prior to rehabilitation. Research is needed to develop a preservation module for the mechanistic-empirical pavement design guide to properly incorporate any planned pavement preservation activities into the design process.

The objective of this project is to develop information that can be incorporated into further enhancements of the mechanistic-empirical pavement design process. The research will develop prediction models for the performance of pavement preservation treatments that could be incorporated in the design process. This objective will be accomplished through the following tasks:

1. Collect all available data and materials associated with pavement preservation treatments.
2. Identify the elements in pavement preservation treatments that need to be incorporated into the mechanistic-empirical design process.
3. Prepare a draft preservation module for the design process and beta test with user agencies.
4. Develop a methodology to evaluate new and innovative treatments.
5. Finalize the preservation module based on feedback from user agencies.

Note: The research being performed under SHRP 2 Project R-26, *Preservation Approaches on High Traffic Volume Roadways* should be reviewed as part of this research.

♦ **Project 01-49**
Guidelines for Forensic Evaluation of Highway Pavements

Research Field: Design
Source: AASHTO Highway Subcommittee on Design
Allocation: $400,000
NCHRP Staff: Amir N. Hanna

Careful, efficient, and well planned forensic evaluation of highway pavements is essential to cost-effective management of pavement assets. Such investigation is appropriately undertaken under several circumstances, such as (1) in the event of premature pavement failure for the purpose of understanding the underlying cause or causes of failure; (2) as part of data collection undertaken to support development and/or calibration of performance prediction models, including local calibration of the Mechanistic Empirical Pavement Design Guide (MEPDG); (3) as a part of more general pavement research efforts to obtain the information needed to fully document and understand observed performance; and (4) in the event of pavements that have performed exceptionally well, for the purpose of understanding the factors contributing to their longevity.

Guidance is needed to assist agencies in planning and conducting appropriate and cost-effective forensic investigations that provide the data needed to support well founded conclusions with minimum disruption to traffic and minimum risk to agency personnel, contractors, and the motoring public. This Guidance must identify the procedures and means for adequately planning and conducting a thorough post construction investigation. Guidance for forensic investigation of Long Term Pavement Performance Program test sections has been developed and documented in *Framework for LTPP Forensic Investigations*. The need exists to build upon this and other available information to develop generally applicable guidance that highway agencies can use in conducting forensic investigation to meet individual agency needs.

The objective of this research is to develop generally applicable guidelines for forensic evaluation of highway pavements. These guidelines will be based on the currently available information and refinements as a result of trial application in several forensic investigations.

♦ **Project 08-77**
Developing Regional Historic Contexts for Post-World War II Housing

Research Field: Transportation Planning
Source: AASHTO Standing Committee on the Environment
Allocation: $250,000
NCHRP Staff: Lori S. Sundstrom

The purpose of this research is to conduct a demonstration project for developing a model regional or state historic context that determines National Register eligibility and non-eligibility of post-World War II housing in order to avoid delays in project delivery and increased project costs associated with potentially eligible houses affected by transportation projects. This context would be used for evaluating the eligibility of properties located within transportation project areas and replace current piecemeal and project-by-project National Register evaluations.

Huge numbers of post-World War II houses, located in every city, town, suburb, and rural area, are either currently historic (i.e., more than 50 years old) or will soon become historic, and thus potentially eligible for listing in the National Register of Historic Places. If these properties are eligible for listing in the National Register, FHWA and state DOTs will be required to take into account the effects of their projects on these properties, pursuant to Section 106 of the National Historic Preservation Act. The houses eligible for listing in the National Register would also be protected under Section 4(f) of the Department of Transportation Act.

Developing an effective national framework for determining National Register eligibility and non-eligibility of post-World War II housing is critical. The proposed approach is very similar to that taken when the Interstate Highway System turned 50 years old in 2006 and became eligible for listing in the National Register. The FHWA and the Advisory Council on Historic Preservation (ACHP) proactively implemented a national solution, and the FHWA worked with the ACHP on an administrative approach to addressing the Interstate System and Section 106 compliance.

A demonstration project for developing regional or state historic contexts will provide a standard framework for state DOTs to effectively evaluate the National Register eligibility of post-World War II housing. Further, the use of these contexts will result in lower future project costs and expedited project schedules and will result in fewer interagency conflicts when compared to current, standard property-by-property and project-by-project National Register evaluations.

As postwar suburbs approach 50 years of age, they are being included in local surveys and are being evaluated according to National Register criteria. Several houses having exceptional significance are already listed in the National Register of Historic Places. Because of the passage of time, the number of houses eligible for listing in the National Register will increase dramatically in the next decade, presenting a major challenge to decision makers and preservation planners. Post-World War II housing is ubiquitous across the country, consisting of thousands of properties. This is especially the case for western cities. For example, Tucson, Arizona, has over 40,000 post-World War II houses (Tucson's Post World War II Residential Suburban Development, 1945-1973). The smaller community of Scottsdale, Arizona, has around 14,000 homes from this era (Scottsdale, Arizona Neighborhood Historic Districts Context). The number of post-World War II houses in more rural locations is also substantial. In James City County, Virginia, for example, the pre-1958 housing stock was about 1,400 units. After 1958, this number rose to over 24,000 homes (Tony Opperman, Virginia DOT, personal communication).

The looming Section 106 and Section 4(f) administrative burden associated with these properties will be tremendous. If state DOTs follow standard approaches for identifying and evaluating the National Register eligibility of these post-World War II properties, state DOTs will be evaluating thousands upon thousands of individual post-World War II houses over the next several years, increasing current Section 106 and Section 4(f) administrative burdens, increasing project costs, and delaying project delivery. State DOT cultural resource management staff workloads could double. State Historic Preservation Office staff will be similarly affected.

Unless post-World War II housing is addressed in a programmatic fashion, each instance of a potentially National Register eligible house will be the subject of negotiation between every state's DOT and FHWA staff and State Historic Preservation Office (SHPO) staff. In addition to providing a cost-effective and efficient means of addressing this distinct housing type, and given that these properties vary little across the country in terms of structure and design, a national approach will provide national consistency and predictability in the implementation of Section 106 and Section 4(f) requirements. At a minimum, the following questions which must be addressed include: (1) the determination of the characteristics that make a post-World War II house eligible, (2) how many modifications and alterations would make a house ineligible, (3) whether individual houses are eligible, (4) whether an otherwise eligible house must be situated in a neighborhood of other post-World War II houses, (5) and how much of a neighborhood needs to retain its original character to be considered a district eligible for listing in the National Register.

After the completion of the demonstration project, the resulting model historic context and associated guidance should be disseminated among all state DOTs and SHPOs. It may also be appropriate to use the results of the demonstration project to develop a national strategy for dealing with this property type in the context of Section 106 and Section 4(f) compliance. The latter would require the involvement of the FHWA, the ACHP and organizations such as the National Council of SHPOs, and possibly the National Trust for Historic Preservation.

Note: The Federal Highway Administration will set aside $100,000 to assist in the implementation of results.

♦ **Project 08-78**

Develop Bicycle/Pedestrian Demand Model to Measure Bicycle/Pedestrian Activity and Relationship to Land Use

Research Field: Transportation Planning
Source: California
Allocation: $300,000
NCHRP Staff: Nanda Srinivasan

Nationally, there is a substantial lack of credible bicycle/pedestrian demand model data. There are existing data sources such as the U.S. Census Journey-to-Work and the National Household Travel Survey that document bicycle/pedestrian activity, but these sources are for work commute trips and do not indicate how many bicyclists/pedestrians there are at specific locations.

Regional/local counts and surveys are not utilizing consistent methodologies that provide understandable bicycle/pedestrian activity trends and relationships to demographic, social, and physical factors. Inconsistent methodologies and minimal credible counts inhibit transportation professionals in local and regional planning and invest in bicycle/pedestrian infrastructure. Adequate bicycle/pedestrian infrastructure provides safety, economic, and health benefits locally, regionally, and nationally. Land use and infrastructure improvements will potentially increase bicycling and walking activities.

This project will produce a national bicycle/pedestrian demand model and database to measure bicycle/pedestrian activity and land use. The need for a national demand model that can accurately measure bicycle/pedestrian activity trends and provide credible data for smart growth land-use projects has long been recognized. The aim to produce a national bicycle/pedestrian database of count information that can be used to develop a national database/demand model will provide local, regional, and national transportation planners and officials with the information and tools needed to better understand walking and bicycling rates, patterns, relationships, and trends for various areas of the country. This demand model tool can provide improved data analysis of proposed land-use projects that have smart growth attributes such as urban infill, pedestrian, transit-oriented, and mixed land-use developments.

♦ **Project 08-79**
Identifying Credible Alternatives for Producing 5-year CTPP Data Products from the ACS

Research Field: Transportation Planning
Source: AASHTO Standing Committee on Planning
Allocation: $550,000
NCHRP Staff: Nanda Srinivasan

In 2006, AASHTO approved a new Census Transportation Planning Products (CTPP) program to provide vital home, work place, and journey-to-work data for effective transportation planning and policy analysis. The CTPP will use data from the Census Bureau's new American Community Survey (ACS) to produce 3-year and 5-year data tabulations to support a host of state and local transportation planning efforts, including air quality and environmental analyses, transit studies, policy and investment scenarios, and travel demand modeling. For the past 4 decades the transportation planning community has relied on CTPP data products developed from the decennial Census "long form" for travel demand forecasting, policy analysis and project planning. The CTPP data products were designed by the states and metropolitan planning organizations (MPOs) and represent one of the most used and recognized data sources. In 2005, when the U.S. Census Bureau decided to eliminate the "long form" and replace it with the ACS, the states and MPOs responded with NCHRP Report 588 in an attempt to understand how to use this new data. The research proposed in this problem statement is an outgrowth of that work.

Over the years, transportation planning mandates and requirements have increasingly called for census data at finer levels of granularity for smaller and smaller areas of geography. For example, in travel demand modeling, data is typically required for smaller geographic units defined as Traffic Analysis Zones (TAZs). However, because TAZs tend to be small, data for many geographic areas will be suppressed under new Census Bureau disclosure rules aimed at protecting an individual's confidentiality. Therefore, to meet the critical transportation planning needs for data, credible alternative methods must be identified for producing 5-year small area, TAZ-level data using the ACS.

The objectives of this research are to:

1. Refine and clarify transportation community acceptance, requirements, and needs for synthetic data.
2. Conduct research and identify credible synthetic data techniques.

♦ **Project 09-49**
Long Term Field Performance of Warm Mix Asphalt Technologies

Research Field: Materials and Construction
Source: AASHTO Highway Subcommittee on Materials
Allocation: $1,500,000
NCHRP Staff: Edward T. Harrigan

The hot mix asphalt (HMA) industry has embarked on a program to substantially reduce mix production temperatures. Reduced mix production and paving temperatures can (1) decrease the energy required to make HMA, (2) reduce emissions and odors from plants, and (3) improve the working conditions at the plant and paving site.

The term warm mix asphalt (WMA) refers to technologies, including various proprietary products and processes, that allow substantially reduced HMA mix production temperatures. Because these technologies were often intended originally to enhance compaction, they may also have positive impacts on HMA performance. Such technologies should make in-place density easier to achieve because they improve the workability of the mix. The majority of aging in an asphalt mixture takes place during mix production when it is exposed to elevated temperatures. By reducing mix production temperature, less oxidative hardening will take place, which should reduce the asphalt mixture's susceptibility to cracking.

WMA technology poses some potential engineering challenges. Reduced hardening during WMA production can increase its susceptibility to permanent deformation. In addition, traffic may not be allowed on the pavement at the conclusion of the compaction process until the mixture cools beyond what is normally required for conventional HMA. Strategies such as the use of higher asphalt binder performance grades and stone matrix mixes can address this issue.

Furthermore, since binders in WMA mixtures may be softer than expected and since some WMA technologies use water as a workability aid, WMA mixtures may be susceptible to moisture damage. This is an issue with some HMA mixtures, of course, but with WMA mixtures there is the possibility of inadequately dried aggregates at the lower production temperatures and/or the introduction of additional moisture to the mix from the WMA technology/process, and this may affect the binder to aggregate adhesion, moisture susceptibility, and performance. The extent to which each of the different types of WMA technologies will impact moisture sensitivity needs to be established in order to provide unbiased moisture performance data and WMA usage guidance to state DOTs, contractors, and asphalt pavement producers.

Acceptance of new technologies depends on their long-term performance and associated life cycle cost analysis as compared to conventional technologies. NCHRP Project 9-47A is identifying new and existing WMA projects from which materials and short-term performance data can be obtained for comparison with HMA. This project will continue Project 9-47A with emphasis on (1) the long-term performance, including moisture susceptibility, of these WMA projects; and (2) methods to assess and remediate any potential WMA moisture susceptibility issues.

The objectives of this research are to (1) develop relationships among engineering properties of WMA mixtures and the long term field performance of pavements constructed with them, (2) provide relative performance measures of pavements constructed with WMA and conventional HMA technologies, and (3) investigate the moisture susceptibility of WMA technologies compared to HMA. Active, close coordination with NCHRP Project 9-47A will be required at all stages of this project.

The following phases (conducted concurrently) and tasks are anticipated to accomplish these objectives:

<u>Phase I, Moisture Susceptibility of WMA</u>
(1) Critically review the literature to identify (i) existing WMA moisture susceptibility studies, especially those which compare laboratory data to field data; (ii) available moisture susceptibility tests; and (iii) existing WMA pavement projects within the U.S. with available moisture susceptibility test results; (2) define a plan for evaluating WMA moisture susceptibility in the laboratory; (3) evaluate test devices and test procedures (includ-

ing AASHTO T283, Standard Method of Test for Resistance of Compacted HMA to Moisture-Induced Damage, AASHTO Water Boil Test for loose coated aggregates, Georgia DOT Triple Freeze Thaw Testing, Hamburg Wheel Track Testing, and Stripping Inflection Point) for use in the laboratory study and provide justification for those recommended for use; (4) define a plan for comparing and relating moisture susceptibility laboratory results to field performance; (5) prepare an interim report for Tasks 1 through 5; (6) conduct the Task 4 work plan; (7) evaluate selected WMA field projects for moisture susceptibility and relate laboratory moisture susceptibility test results to field performance; (8) prepare a recommended practice for reducing the occurrence of moisture susceptibility issues with WMA; (9) based on the results of Tasks 6 and 7, recommend (i) changes to existing AASHTO test methods and specifications and, as needed, (ii) new test methods in AASHTO standard format for evaluating the moisture susceptibility of WMA mixtures and pavements; and (10) prepare a final report for Phase I.

Phase II, Long-Term Performance of WMA

(1) Conduct a literature review and survey of state DOTs, as needed, to determine (i) availability of field trial/test sections, both existing and planned; (ii) data available from existing field trial/test sections (with emphasis on those field trials/sections utilized in Project 9-47A); and (iii) information that will allow the selection of laboratory tests to relate laboratory test properties to field performance for rutting, fatigue cracking, thermal cracking, aging, and water sensitivity; (2) identify and select field trial/test sections for inclusion in the study; (3) prepare a detailed sampling and testing plan for determining the long-term performance of the field trial/test sections identified in Task 2; (4) prepare an interim report for Tasks 1 to 3; (5) perform sampling and testing associated with the approved work plan and perform the analysis associated with meeting the objectives of the project; (6) revise structural design and mixture design methods associated with WMA technologies based on the results from Task 5; and (7) prepare a final report for Phase II, including a plan for extended monitoring of the performance of field trial/test sections.

Note: The AASHTO Standing Committee on Research combined Problem 2010-D-08, Moisture Sensitivity of Warm Mix Asphalt Technologies with this study and increased the funding request to $1,500,000.

♦ **Project 10-80**

Conversion of the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals" to the Load and Resistance Factor Design (LRFD) Methodology

Research Field:	Materials and Construction
Source:	AASHTO Highway Subcommittee on Bridges and Structures
Allocation:	$500,000
NCHRP Staff:	Waseem Dekelbab

In June 2000, AASHTO and the Federal Highway Administration agreed on an implementation plan for the design of highway structures utilizing the Load and Resistance Factor Design Methodology (LRFD). As part of that agreement, all new culverts, retaining walls, and other standard structures on which states initiate preliminary engineering after October 1, 2010 shall be designed by the LRFD Specifications. The current edition of the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals" contains small sections that implement the LRFD approach, but the Specification is generally based on the Working Stress design methods. Additionally, design, construction, and inspection language is intertwined in the specification and commentary resulting in a document that is cumbersome and difficult to follow.

The entire Specification needs to be converted to the LRFD design approach and reorganized to provide design engineers with a specification that implements the state-of-the-art design approach; separates the design, construction, and inspection criteria into three distinct sections; is consistent with other AASHTO documents; and allows states to meet the above implementation plan.

The goals of the proposed research are closely aligned with the grand challenges of optimizing structural systems, advancing the AASHTO specifications and managing knowledge. These were identified in the AASHTO Subcommittee of Bridges and Structures report "Grand Challenges: A Strategic Plan for Bridge Engineering" published in June, 2005.

The objective is to develop a new edition of the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals" based on the LRFD methodologies. The resulting Specification would also be logically arranged with distinct sections for design, construction, and inspection/maintenance. Inspection of these structures has not been codified in the past; this is an excellent opportunity to address this issue.

The successful completion of this research is expected to improve the safety and reliability of structural supports nationwide. Agencies will be in a better position to meet the LRFD implementation plan, and the provisions will facilitate the design, construction, inspection, and maintenance of their structural supports for highway signs, luminaries and traffic signals. The probability-based specification will result in structures that are based upon a more uniform set of design criteria. Some structures may be more expensive; however, some may be less. The specification will promote quality construction/fabrication practices and it will also address the current shortcoming of inspection and maintenance or these non-redundant ancillary structures. The combination of these efforts will allow agencies to better assess, manage, and maintain these transportation assets.

♦ **Project 10-81**
Evaluation of Fuel Usage Factors in Highway Construction

Research Field:	Materials and Construction
Source:	AASHTO Highway Subcommittee on Construction, Virginia
Allocation:	$300,000
NCHRP Staff:	Edward T. Harrigan

Price adjustments of selected commodities in highway construction are used in construction contracting as a way of reducing risks to the contractor related to price fluctuations over the life of a contract. The benefits to contracting agencies are bids that better reflect real costs. Fuel is a commodity for which price adjustments are allowed. Fuel usage factors are commonly applied by state and local agencies in calculating the amount of fuel for an escalation/de-escalation contract specification.

The current fuel usage factors were first published in Highway Research Circular Number 158 by the Highway Research Board in July 1974. They were later incorporated into FHWA Technical Advisory T 5080.3, released in 1980, to provide direction on the use of price adjustment contract provisions. These factors have remained unchanged over the past 35 years, despite changes in the purchasing power of construction dollars, construction methods, industry processes, efficiency of equipment, and fuels used. Thus, it is unlikely that fuel usage factors are accurate or effective in addressing the current risk of fuel price fluctuations.

Gasoline and diesel fuel usage factors exist for excavation (gallons per cubic yard), aggregate, asphalt production and hauling (gallons per ton), and Portland cement concrete (PCC) production and hauling (gallons per cubic yard). Of even greater concern, fuel usage factors for structures and miscellaneous construction are expressed in gallons per $1,000 in construction. Current fuel factors are required, in addition, to consider environmental impacts of construction methods related to lower fuel consumption and emissions, urban heat island mitigation, smog reduction, and lower energy footprint.

The fuel usage factors in FHWA Technical Advisory T 5080.3 are subject to at least three analytically separable sources of error. First, the effects of inflation on construction costs over three decades is primarily of concern for the usage factors for structures and miscellaneous construction because these fuel usage factors were established in gallons per thousand dollars, and the dollar amounts were established in 1980 and have never been revisited. Second, the relationship of fuel consumption to production and hauling of specified quantities of aggregate, asphalt, and PCC have likely been affected by changes in construction practice, use of new and prefabricated materials, improved equipment, and improved fuel efficiency. Third, and last, there have been changes in fuel preference, particularly in the substitution of natural gas for diesel in asphalt plant operations. While an examination of inflationary trends is a relatively simple analysis, addressing the other impacts is far more complex and challenging.

The objectives of this research are to (1) analyze the effects of inflation in relevant areas of construction, (2) develop a revised table of fuel usage factors for the major categories of highway construction addressed in FHWA Technical Advisory T 5080.3, and (3) develop a recommended method and schedule for future updates to the fuel usage factors. The research findings will be of immediate use to FHWA in updating the information in Technical Advisory T 5080.3.

The following tasks are anticipated to accomplish these objectives: (1) review existing research, including (i) the original study compiled by FHWA and published in Highway Research Circular Number 158, July 1974, (ii) the questionnaire sent to more than 3,000 highway contractors in the United States in 1974 with 400 responses, and (iii) the analysis performed by the Federal Highway Administration's Region 8 office on the data acquired in 1974, to the extent that relevant information is still available; (2) survey the state DOTs to develop a synthesis of current practices by state DOT agencies and document what methods they have developed to address costs related to fuel usage factors issues; (3) analyze inflation effects to develop a construction inflation index that will provide estimates of the present and expected future value of construction, based on the categories in the 1980 FHWA Technical Advisory T 5080.3; (4) identify changes in construction practices since 1980 in the major categories of highway construction addressed in FHWA Technical Advisory T 5080.3 (excavation, aggregates, asphalt concrete, PCC pavement, structures, miscellaneous); (5) based upon the results of the previous tasks, develop fuel usage factors that apply to current construction practices; (6) develop a method and schedule for future updates of fuel usage factors, including identification of data sources and recommended analytical procedures; and (7) prepare a final report and recommendations that provide (i) full documentation of the research methods and findings and (ii) recommendations for the updated fuel usage factors in highway construction.

♦ **Project 10-82**
Performance Related Specifications (PRS) for Pavement Preservation Treatments

Research Field: Materials and Construction
Source: California
Allocation: $600,000
NCHRP Staff: Amir N. Hanna

Pavement preservation treatments, by their diverse nature, do not lend themselves to traditional methods based specifications. Many states have tried to develop warranty specifications to address this gap. However, warranties still do not necessarily provide the correct strategy for pavement preservation treatments, so construction specifications are moving into the PRS arena. Quality of pavement preservation treatments depends on the contractor operations, personnel, equipment, and methods. With the current specifications and the low bid process, there is no incentive for a contractor to make the extra effort to insure quality. In order to overcome this problem, research is needed to develop and implement PRS that allow contractors to design and construct pavement preservation treatments using conventional materials, rubber asphalt, or other modified materials.

The need for research for the development of PRS for pavement preservation treatments has been cited in many preservation workshops and expert group meetings subsequent to the FHWA's National Pavement Preservation Forum held in San Diego, California in 2001. A recent NCHRP project (Project 20-07/Task 184) supported the need for PRS for pavement preservation but was rated as a low priority need because the major focus in most agencies was to determine what treatments to apply and what type of performances could be expected from those treatments. In response to these research needs, some state highway agencies have developed project-specific PRS for limited preservation treatments. Yet systematic research for the development of more generally applicable PRS is needed. In January 2008, FHWA developed (with state and local agencies, industry and academia) the Transportation System Preservation (TSP) Research, Development, and Implementation Roadmap to fill the current gaps in the understanding of pavement preservation. The Roadmap contains 38 items of immediate research needs under 6 areas in pavement preservation; the PRS for pavement preservation was identified as the highest priority topic among them. Additional information supporting the need of PRS for pavement preservation treatments is cited in TRB Needs Statement entitled Warranties for Concrete Pavements.

The objectives of this project are to (1) determine which engineering properties need to be measured for performance/acceptance; (2) determine how performance parameters should be measured; (3) determine which pavement preservation treatments lend themselves to incentives/ disincentives clauses, and recommend limits for incentives/disincentives; (4) provide a template for incorporating PRS into treatment-specific projects; and (5) develop a draft manual that provides guidance and measurement techniques for the identified engineering properties. The following tasks are identified for accomplishing these objectives:

1. Conduct an international literature search of current pavement preservation performance-related specifications as applied to asphalt and concrete pavements, and provide a report that describes the benefits of using PRS vs. traditional method-based specifications.
2. Determine the desired attributes that can be measured and the quality of relevant pavement preservation treatments.
3. Determine treatment-specific performance measures and acceptance criteria
4. Determine criteria for the appropriate use of incentives/disincentives.
5. Develop draft provisional standards for PRS, as they apply to pavement preservation treatments, for AASHTO consideration.

Note: The AASHTO Standing Committee on Research requested that this project be combined with Problem 2010-F-09. The research shall include additional tasks to identify and validate testing protocols and acceptance criteria for predicting long-term treatment performance, and consider SHRP 2 related projects.

♦ **Project 10-83**
Alternative Quality Systems for Application in Highway Construction

Research Field:	Materials and Construction
Source:	AASHTO Highway Subcommittee on Construction, AASHTO Highway Subcommittee on Materials
Allocation:	$500,000
NCHRP Staff:	David A. Reynaud

This research proposal addresses the quality management challenges associated with achieving two of AASHTO's goals. Alternative and improved quality management practices are required to maintain appropriate results while accelerating project delivery (objective 1C), and must be part of a comprehensive framework for improved project delivery of all transportation projects (objective 4B).

Evolving industry roles and the adoption of alternative project delivery methods have already changed the conventional construction management practices that public agencies use to ensure appropriate project delivery, contract compliance, and quality assurance. However, many state agencies are applying traditional quality management practices to all of the available project delivery methods. Critical components of these new project delivery methods include the changing relationships among public agencies, contractors, and private engineering firms, including risk allocation processes, general contract administration procedures, and quality control/quality assurance. This research would provide recommendations for alternative quality systems to address the various project delivery methods. The results would include a web-based decision support system for use by the state agencies.

More efficient and effective quality control/quality assurance systems, driven by both contractors and owners, are required due to increasingly higher construction costs and reduced workforce availability and experience levels. Owners are challenged to provide quality and timely results with fewer staff than are inherent in the traditional public agency inspector role. In order to achieve a successful project or operation, quality must be built into all aspects of project delivery and ranges far beyond matters of materials testing. Research and guidance is required to assist owners with the management of programs that will increasingly be delivered by methods that are still relatively new, and will remain significantly different than the traditional design.

Quality control includes all aspects of project work. As agencies work to accelerate project delivery, in conjunction with AASHTO partners and federal agencies, they must also work to improve and innovate all aspects of project delivery for all kinds of projects (from ITS to new facilities) including quality management functions related to planning and environmental processes, right-of-way acquisition, utility relocation, design, construction techniques, and construction management. Specific tasks requiring oversight or approval include preparatory work activities, design development and construction plans, traffic control plans, transparency of operations, rapid communication of test results and resolution of non-conforming items, and finally auditing practices of contractor quality control and assurance for interim and final deliverables. In a time of diminishing resources and escalating costs better quality process tools can increase our oversight effectiveness. State agencies need assistance to implement new systems that emphasize contractor quality control and assurance. Such assistance will allow the owner to leverage its resources to more systematically have assurance of quality levels thru a verification of contractor quality system processes.

The traditional design-bid-build delivery system in the United States uses construction specifications that result in detailed contractor quality control requirements. Acceptance programs, including verification tests historically have been performed by State highway agency staff. Only recently have contractors been required or encouraged to develop their own quality control plans. However, Federal regulations do not allow the use of the contractor quality control test results in the acceptance decision, unless verified by the owner.

Many international scan teams, most recently the Construction Management Scan team has noted that more formal quality management systems are used by contractors abroad than in the United States. With the exception of Germany, there is a heavy reliance on International Organization for Standardization (ISO) 9001 series methods. Contractors can be rated on their quality management plans before project award, held to these requirements during construction and be evaluated in post-project reviews.

European countries require ISO 9001 certification, and contractors must develop the project quality plans in conjunction with their companies' certified quality procedures. Typically, contractors are primarily responsible for ensuring that materials and all other contract deliverables meet the required quality levels, and they provide certifications and test results. Tests are conducted as work progresses. All work is subject to owner review, but separate tests normally are not conducted. The set of checks by the owner consists of a mixture of system checks, process checks, and product checks. Most tests are standardized and the contractor has to use the standards. Most standards are legally enforceable. Other items of work that are not materials based such as traffic control devise layout and placement or final subgrade elevation tolerances also are included in the quality system criteria.

The research objectives are summarized as follows:

- To define the objectives and benefits of the traditional quality control and quality assurance approach that is part of the FHWA's regulatory requirements. To define the objectives and benefits of alternate quality system requirements included in ISO 9001 (those used in Europe and the United States), Corps of Engineers (COE), Federal Aviation Administration (FAA), Federal Transit Authority (FTA) and other systems, as they are being applied in United States and internationally. Document the advantages and disadvantages of these alternate systems in relationship to the traditional quality control and quality assurance approach.
- To describe how these various systems work to achieve quality assurance objectives.
- To assess their potential impact on current state DOT and industry practices if alternative quality systems were applied to innovative contracting projects let in the best value environment as well as conventional design-bid-build projects let in the low bid competitive environment.
- To develop a plan that assists states in evaluating and deploying these quality systems. It is understood that there may not be a single, preferred solution but instead, options. The integration and compatibility of the contractor's and owner's programs need to be considered.
- To obtain a consistent definition of quality terminology among FHWA, the state DOTs and the highway construction industry. AASHTO definitions in R10 and ISO definitions need to be harmonized for clear communication.

Tasks

1. Identify and describe quality management systems that are used in the construction industry, with emphasis on the highway construction industry. Examine as a minimum ISO 9001 and the COE approaches. There will be a survey and literature search to include international information as well.
2. Describe and contrast current applications of quality management systems in the U.S., Canada, and Europe, with focus on alternative project delivery strategies, such as: design-bid-build, best value, design-build, and others. This includes those that are in place by contractors and owners.
3. Contrast these quality systems with conventional state DOT and industry practices. Two workshops should be held to allow for group synergy and participation to allow for a comprehensive evaluation.
4. Identify the benefits the contractor might accrue from the adoption of such a system. Consider the paybacks such as consistency, productivity, costs, risk management, employee awareness, on-time delivery, staffing levels, timely completion of testing, improved product performance, better risk alignment, strengthened business capabilities, more consistent management structure across jurisdictional lines, reduced claims, etc. This is a philosophy of "doing it right the first time" approach to construction.
5. Identify the benefits that a DOT might accrue from the adoption of an appropriate system including improvement of products, reduced inspections by optimizing use of staff by reducing duplication of work, reward quality contractors, etc.
6. There may be more than one solution for effective quality systems. If so, then project selection guidelines need to be developed to tie the appropriate quality system to the appropriate type of project and alternative

project delivery methodology. Based on the information gathered, recommend improvements to the traditional quality system that is used.

7. Present detailed case studies of application on various U.S. projects.
8. For each recommended quality system and the appropriate project selection guidelines, describe the potential implications if adopted as a standard practice by state DOT organizations and the highway construction industry. The measurement of success of the quality management system, particularly in the low-bid environment, is one of the potential implications. Recommend ways the traditional quality system could be incrementally improved by incorporating only select portions of the recommended quality system. Recommend adjustments to the quality system to accommodate traditional low-bid contracting, as well as design-build, and best value. Any adjustments needed to accommodate public-private partnership type projects should be addressed.
9. Develop a plan that assists states in evaluating and deploying the appropriate quality system for the alternative project delivery method selected. Identify the barriers to implementation and ways to overcome them.
10. Provide a web-based decision support tool that captures institutional knowledge gained by leading practitioners, and enables state agencies to access past experience and obtain guidance for system development or implementation.

Research in the area of innovation of procurement of construction is urgently needed to find techniques that can ensure quality. With limited resources at their disposal, the research will allow owners to optimize their efforts to ensure quality when these innovative methods of project delivery are selected. Research in this area should have a high benefit/cost payoff due to the large potential cost savings when projects are constructed with effective quality systems.

The payoff of this research will be the establishment of a knowledge base for best practices of quality systems available for a variety of project delivery techniques to supplement existing state DOT knowledge bases. This is a time of great innovation while many owners are experiencing significant turnover among their senior design and construction professionals. Establishing a knowledge base will be critically useful for the new generation entering into the work force. The research will identify critical requirements to implement various quality systems (handbook, training, guides, etc.) to disseminate the findings to the highway construction industry. The implementation should include the recommendations in a standard format for AASHTO practice.

♦ **Project 12-85**
Roadway Bridges Fire Hazard Assessment

Research Field: Design
Source: AASHTO Highway Subcommittee on Bridges and Structures
Allocation: $350,000
NCHRP Staff: Waseem Dekelbab

Bridge fires can cause major disruptions in highway operations. Although major fires are infrequent, there are an undocumented number of smaller fires that occur on highway bridges throughout the United States each year which cause varying degrees of disruption and repair and maintenance costs. These incidents stem primarily from vehicle (often truck) fires, but bridges can also be affected by fires in adjacent structures. The recent San Francisco bridge fire has highlighted the need to better understand the frequency and hazard potential of these incidents as well as review available information on potential mitigation strategies and damage assessment and repair techniques.

The objectives of the proposed study are to:

1. Survey fire incidents on bridges in the U.S. in recent years, documenting trends.
2. Quantify design fires (in terms of fuel load probability distribution) for vehicle fires and carry out the resulting hazard assessment.
3. Develop suggested design guidance—spatial separation, protection methods (include reference to combustible elements such as FRP), and detailing to minimize risk (for example, how down spouting can promote damage in fuel spills).
4. Consider impact of truck design (for example, location and protection of fuel tanks).
5. Assemble information on post fire structural assessment and repair techniques.
6. Identify areas where additional data is needed.
7. Develop bridge management practices for use of bridge and highway right-of-way.

The study as planned is a literature review and hazard assessment; no fire testing is contemplated. Reference will be made to the National Fire Protection Association Standard 502, Standard for Road Tunnels, Bridges, Limited Access Highways®. Hazard assessment techniques which have been applied to structures other than bridges can be adapted for use in this study. Actual risk, target risk, and implementation procedures will be assessed.

The recent major fire in San Francisco highlights a major area of concern for State Highway Departments where little or no design guidance is available. Assembly of readily available information for use by the highway community for these incidents will permit the implementation of some potentially straightforward mitigation strategies as well as assessment and repair techniques. The technical data from the research can be used for developing AASHTO guidelines for planning, designing, constructing, maintaining, and inspecting highway bridges; developing guidelines for emergency management; and contributing to a risk-management approach to bridge safety inspection and maintenance.

This proposed research addresses the *Grand Challenges*—Advancing the AASHTO Specifications, Extending Service Life, and Optimizing Structural Systems, among others by developing technical data which can be used to provide performance standards for design and management of highway bridges.

Note: The AASHTO Standing Committee on Research increased the funding request from $175,000 to $350,000.

♦ **Project 12-86**
Bridge System Safety and Redundancy

Research Field: Design
Source: AASHTO Highway Subcommittee on Bridges and Structures
Allocation: $500,000
NCHRP Staff: Waseem Dekelbab

A good quantification of redundancy is not currently available to bridge engineers. Construction costs may be needlessly high on some bridges due to member redundancy, and less redundant existing structures may go unidentified. Redundancy, operational importance, and ductility can be considered during design by using load modifiers from the *AASHTO LRFD Bridge Design Specifications* ranging from 0.95 to 1.05 each. However, such factors should be better specified according to system type, and should be on the resistance side of the equation.

A framework for evaluating redundancy in highway bridges is described for superstructures in *NCHRP Report 406*. For superstructures, a "system factor" between 0.8 and 1.2 is applied based on the girder spacing and number of girders in the system. However, the charts are limited to steel and pretensioned I-beam-slab bridges, and only a small general bonus is given for diaphragms. An alternative methodology is provided to generate the system factor using nonlinear analysis and incrementally increasing the HS20 load, but ability to transmit load longitudinally is still not addressed.

Similarly, substructure redundancy is addressed in *NCHRP Report 458*. System factors are provided for confined and unconfined 2- and 4-column piers; spread footings, drilled shafts, and piles in various soil types. A direct redundancy analysis procedure is also provided. Any ability of the superstructure to enhance the substructure redundancy is ignored.

The quantification of redundancy in highway bridge structures is necessary before reliability-based non-collapse criteria can be developed for blast loads, ship-impact, storm surges, or seismic-force effects. A fresh approach is needed to merge past efforts that developed super- and substructure redundancy independently, to quantify the transfer of lateral loads in the superstructures inclusive of multi-cell cross-sections, and to consider the affect of superstructures on substructure redundancy especially in the case of framed structures and super-substructure connections made fixed for live load.

The objective of the research is to combine the techniques presented in *NCHRP Reports 406* and *458*; expand the work to include lateral loads on superstructures, framed systems, single- and multi-cell boxes; and demonstrate the suitability of the proposal within the framework of the previously defined ultimate (strength), functionality (service), and damaged ultimate (collapse) limit states. The following tasks are envisioned:

1. Review *NCHRP Reports 406* and *458*, as well as Interims to the LRFR Manual for System Factors in Segmental Bridges. Do a literature search to see if any other related studies have since been done.
2. Expand on how the quantity and quality of intermediate and end diaphragms in the superstructure play a role in redundancy.
3. Expand to include single- and multi-cell cross-sections where member resistance may or may not be relevant since a "whole-width" design is often done.
4. Make any necessary updates to the functionality limit state for the recent displacement-based "LRFD Guidelines for Seismic Design of Bridges."
5. Develop a methodology for accessing the redundancy in both the super- and substructure when the superstructure contributes to the substructure's lateral load resistance.
6. Perform parameter studies to show that results are reasonable for all structure types, regardless of girder spacing, number of girder webs, number of substructure units, etc.
7. Develop a list of design examples to illustrate the proposed methodology for three limit states and structure types. List to be approved by project panel.
8. Prepare draft specification changes for both design and rating.

9. Present findings to the AASHTO SCOBS T5 Loads and Load Distribution Committee.

The ability to quantify "additional" capacity in bridges due to system redundancy is crucial in understanding bridge performance when subjected to malicious attack, vessel collision, earthquake, or storm surges. Bridge Owners need this information to help determine which existing bridges are most vulnerable due to lack of redundancy, and how to provide adequate redundancy in new bridges.

♦ **Project 14-20**
Quantifying the Costs and Risks of Delayed Maintenance

Research Field:	Maintenance
Source:	AASHTO Planning Subcommittee on Asset Management, AASHTO Highway Subcommittee on Maintenance
Allocation:	$700,000
NCHRP Staff:	Amir N. Hanna

The cost of delayed maintenance/preservation is often not well quantified or readily apparent. When making budgeting decisions one may be tempted to choose spending less on preventive maintenance strategies in the near-term because the long-term implications of such a decision are not easily understood. However, delaying maintenance/preservation projects (pavements and bridges) could push projects into total reconstruction and substantially increase the total life-cycle costs of providing a structurally sufficient pavement or structure. Also, there are associated risks with delaying/postponing maintenance/preservation activities. This research seeks to develop information to better understand and quantify the risks associated with delaying preventive maintenance activities.

The objectives of the research are to (1) describe the impact and savings of undertaking proactive maintenance/preservation activities early in the life of a pavement and bridge structure as compared to delaying these activities; (2) perform an analysis and recommend when it is best to undertake pavement and bridge preservation actions; and (3) describe and outline the process that can be undertaken by state DOTs to conduct a risk analysis of postponing maintenance/preservation activities on pavements and bridges.

Note: The AASHTO Standing Committee on Research requested that this project be combined with Problem 2010-F-04. The research shall include additional tasks to establish the relationship between the level of service (LOS) of a roadway and the cost of maintenance required to achieve it as well as a method to quantify the cost associated with obtaining an improved level of service. These tasks will include conceptual development, data gathering, analytical modeling, and trial use of the developed procedures in conjunction with maintenance programs.

♦ **Project 14-21**
Optimization of Resource Allocation for Highway Preservation Needs

Research Field: Maintenance
Source: AASHTO Planning Subcommittee on Maintenance
Allocation: $350,000
NCHRP Staff: Andrew C. Lemer

State DOTs and the federal government have invested significant resources in building our nation's highway system. All of the asset categories that comprise our highway system, e. g., pavements, bridges, traffic signals, drainage pipes, signs, lights, and more, must be maintained, rehabilitated, and ultimately replaced. As our transportation system has aged, system preservation needs have moved to the forefront in the funding priorities of many DOTs.

However, resources for highway preservation typically have not kept pace with preservation needs. Most transportation agencies lack sufficient resources to attain and maintain desired service levels for all of their highway assets and must try to optimize the allocation of their limited resources to preserve a diverse portfolio of assets. Given large variations in service lives—e. g., the life of a bridge versus pavement markings—variations in asset class management systems, and the breadth of competing funding needs, the problem of optimizing resource allocations to preservation needs is extremely difficult.

A fundamental and practical issue with optimizing resource allocations to the preservation of various asset categories is the identification of common performance objectives that can transcend all of the asset categories that comprise our highway system, and that could serve as basis for optimization. Such common objectives may include, for example, maximizing remaining service life or minimizing long-term costs. A number of optimization models have been developed to assess and help select the best strategy of preservation or replacement alternatives for a given investment level within an asset category, e.g., in pavement management and bridge management systems, but optimization criteria and practical models to allocate resources across a broad array of highway asset categories do not exist.

The objectives of this research will be to develop objectives and measures of effectiveness that may be used to optimize resource allocation for preservation of assets across the entire range of highway assets for which a DOT is responsible.

The research to accomplish this objective might include the following tasks: (1) preparation of an annotated literature review on optimization criteria and objectives to allocate resources across various transportation asset categories; (2) identification of optimization objectives and criteria that may be suitable to allocate preservation resources across a broad portfolio of highway asset categories; (3) assessment of the potential advantages and disadvantages of the optimization objectives and criteria for use in the intended context; (4) assessment of potential issues associated with implementing the most advantageous optimization objectives and criteria in a practical optimization model within state DOTs; (5) demonstration and documentation of the use of the recommended optimization objectives and criteria in the allocation of resources across highway asset categories through realistic case study examples; (6) identification of specific future research needed to achieve the implementation of allocation optimization models for the preservation of a broad portfolio of highway asset categories within state DOTs.

♦ **Project 14-22**
Effective Removal of Pavement Markings

Research Field: Maintenance
Source: Rhode Island
Allocation: $200,000
NCHRP Staff: David A. Reynaud

To find an environmentally safe means of removing line striping and other markings without damaging the underlying pavement and altering the visible character of the surface course, or to develop an alternative to paint which will perform adequately under interstate conditions and can be removed without damaging mechanical or chemical action.

During construction projects, it is often necessary to implement lane shifts in order to detour traffic around work zones. Shifting lanes requires obscuring or removing the existing pavement markings and applying new markings along the new alignment. The Manual on Uniform Traffic Devices (MUTCD) requires that all visible traces of the existing marking be removed or obliterated in order to provide a clear line of travel for the motorists. It does not allow for removal methods that will scar the pavement, with no specification for a level of scarring that is acceptable.

Among the primary requirements of permanent marking systems is to create a durable, strongly bonded material. It has to be capable of standing up to several years of wear due to heavy traffic at highway speeds and resist the environment (UV exposure, freeze/thaw, chemicals, etc.). Many of the new systems are epoxy-based and adhere adamantly to the pavement. Black tapes that are applied over the existing markings to hide them tend not to last long enough and/or have different reflective indices than the pavement and so may confuse drivers as to the correct lane to follow. The problem may be exacerbated in wet weather. Chemical systems that are aggressive enough to remove epoxies and other products would raise safety and environmental concerns. As a result, removal generally requires grinding of the markings, which leaves undesirable scarring often mistaken for actual pavement markings under low-light or wet conditions, so the owners of public highways are faced with a very difficult problem.

Research Questions: The problem at hand would need to be addressed for two different circumstances, although there would be some overlap.

Phase I—Investigate how to remove or fully obscure existing markings without damaging the pavement:

1. Are there systems (perhaps organic, like citrus based bituminous solvents) that would be more environmentally friendly?
2. Is there a mechanical process such as a combination of heat and power tools that could effectively remove a sufficient amount of the markings?
3. Is there a method of applying a durable coating over the existing pavement marking that will blend in to the appearance of the pavement, perhaps by using color matching technology? Or could the full width of the pavement be covered completely without losing friction characteristics, in a cost-effective manner?
4. How much is adequate to meet the MUTCD requirements? How much tolerance is there for altering the pavement surface?
5. Would the system developed be of a reasonable cost (relative to existing systems and methods) for materials, equipment, and labor?

Phase II—Develop a pavement marking system that meets durability and visibility standards, but with a designed means for full removal:

1. Can a coating that is durable in a heavy traffic environment still be created such that it can be removed cleanly? Are there existing systems in use for other applications that can be modified to meet this requirement?

2. What would the environmental constraints be for the system?
3. How much is adequate to meet the MUTCD requirements? How much tolerance is there for altering the pavement surface?
4. Can such a coating be designed to be cost effective (relative to existing systems and methods) in both application and removal?
5. If the system is significantly more expensive than current systems, could locations where line shifts are more likely be identified, so that the use could be restricted to an as-needed basis?

Both systems, after a thorough laboratory evaluation, could be used on a demonstration basis in highway construction projects for testing in field conditions. The use would be for limited runs to minimize the effects on traffic flow in the event that the tests are not successful.

Owners of public highways are faced with MUTCD requirements for pavement markings that cannot be properly met by existing methods. A new system for removal of existing markings and a new one with a controlled means of complete removal would help owners meet the MUTCD regulations and provide safer traffic management. This would be especially true in work zones where traffic control is critical.

Note: The AASHTO Standing Committee on Research directed that the scope address the development of best practices and a recommended test for MUTCD compliance. Phase II is not envisioned.

♦ **Project 15-39**

Superelevation Criteria for Sharp Horizontal Curves on Steep Grades

Research Field: Design
Source: AASHTO Highway Subcommittee on Design
Allocation: $500,000
NCHRP Staff: David A. Reynaud

Sharp horizontal curves on steep grades represent a particularly dangerous situation for vehicle operators, especially heavy vehicle operators. Examples where this combination may occur are high-speed interchange movements, switchback curves on mountainous two-lane, two-way roads or high-speed downgrade curves on limited access roadways. At these locations, the complicating factors of vehicle off-tracking, pavement slope, and pavement friction fully tax the driver's ability to provide correct vehicle positioning without compromising control of the vehicle. Accident problems have arisen where, as a result of reconstruction, older highways with 12% to 17% superelevation have been rebuilt using 8% and 10% superelevation in accordance with current standards. Superelevation criteria, and other associated horizontal curve criteria, for situations where steep grades are located on sharp horizontal curves have not been developed.

NCHRP Projects 15-16 and 15-16A, documented in *NCHRP Report 439: Superelevation Distribution Methods and Transition Designs*, evaluated and recommended revisions to the horizontal curve guidance presented in the 1994 AASHTO publication, *A Policy on Geometric Design of Highways and Streets (Green Book)*. The two principal design elements evaluated were the use of superelevation and the transition from a tangent to a curve. The transition recommendations were incorporated into the 2001 edition of the *Green Book* and the superelevation recommendations were included in the 2004 edition of the *Green Book*.

NCHRP Report 439 noted that significant roadway downgrades deplete the friction supply available for cornering. This depletion results from the use of a portion of the friction supply to provide the necessary braking force required to maintain speed on the downgrade. The report found that both upgrades and downgrades yield an increase in side friction demand and a decrease in side friction supply. This undesirable combination results in a significant decrease in the margin of safety resulting from roadway grade, especially for heavy vehicles. Superelevation criteria and horizontal curve criteria for this situation were not developed.

The 2004 *Green Book* contains the following: "On long or fairly steep grades, drivers tend to travel faster in the downgrade than in the upgrade direction. Additionally, research has shown that the side friction demand is greater on both downgrades (due to braking forces) and steep upgrades (due to the tractive forces). Some adjustment in superelevation rates should be considered for grades steeper than 5%. This adjustment is particularly important on facilities with high truck volumes and on low-speed facilities with intermediate curves using high levels of side friction demand."

The 2004 *Green Book* further states that this adjustment for grade can be made by assuming a slightly higher design speed for the downgrade and applying it to the whole traveled way. There are no guidelines as to how this adjustment should be made for two-lane or multilane undivided roadways. More definitive guidance on this adjustment, as well as adjustment for other elements of the horizontal curve, is needed.

The objective of this research is to develop superelevation criteria for horizontal curves on steep grades. Other criteria associated with design of horizontal curves such as tangent-to curve transitions, spiral transitions, lateral shift of vehicles traversing the curve, need for pavement widening, and minimum curve radii should also be considered in the development of the criteria. The criteria may be based on quantitative evidence obtained from theoretic considerations and simulations but should be supported by actual field observation.

The research should include a review of current practice, development of a work plan to achieve the research objectives, collection of data and other information, evaluation of effects of various alternatives and candidate criteria, and preparation of final criteria. The recommended criteria should be documented in the final report and also presented in a form that could be used by the AASHTO Technical Committee on Geometric Design in a future edition of the Green Book.

This research topic was selected by the TRB Committee on Geometric Design, TRB Committee on Operational Effects of Geometrics, and the AASHTO Technical Committee on Geometric Design at their combined meeting in June, 2004 as one of the five highest priorities for research. The research is needed immediately to fill a gap in current superelevation design policy. The superelevation guidance will apply to high speed interchange ramp alignments on descending grades. As such, the research findings will have applications in every State and not just to those with mountainous terrain. Considering the research will apply to interchange movements, this research topic will be of use in the design of highways nationwide.

♦ **Project 15-40**
Designing the Roadway Transition from Rural Highways to Urban/Suburban Highways or Streets

Research Field: Design
Source: AASHTO Highway Subcommittee on Design
Allocation: $300,000
NCHRP Staff: B. Ray Derr

There is a safety problem in locations where drivers operating in a high-speed rural environment must transition to operating in a low-speed urban/suburban environment as they approach a small community. There are numerous locations where traffic at speeds of 50 mph and higher on a rural highway must slow to 35 mph or less as a driver approaches a small urban area. Roadway features that present visual information that assist in reducing travel speeds are typically minimal or non-existent. Many times this results in inappropriate speeds for a significant distance into the urban area. Some of the AASHTO design criteria are unclear for intermediate speeds (between 30 and 50 mph) in these areas. For example AASHTO recommends that vertical curbs only be used in low-speed areas, however, it's often used in transition zones posted at 50 mph. Also the need for and/or rate of superelevation can be very different for the same design speed depending on the method chosen to apply any required superelevation. Numerous localities are desirous of developing the transition area into gateways for their communities demonstrating the need for clear and appropriate criteria.

The transition from rural high-speed operations on a highway to low-speed operating conditions in urban/suburban conditions is a safety problem for all roadway users. In this context, the vehicle operator does not recognize the changed environment and adjust the vehicle's speed to a more appropriate level, thus creating the potential for safety problems on the urban street because of the typical high speed at the entry to the urban/suburban zone and for a significant distance into the area. Conveying the need for action on the part of the driver to restrain speed is a challenge for designers and enforcement officials. Some of these roads remain arterials that facilitate longer commutes, frequent local access, bicycle/pedestrian access, and in some cases, on-street parking.

Techniques are not available to assist the designer and enforcement officials in developing and providing for an appropriate design in these intermediate speed transition areas. There is a need to develop various techniques and tools to assist practitioners to provide a safe transition in operating conditions as a driver moves from rural to urban areas. The tools and techniques may involve application of existing tools and techniques, unique designs, signs and markings, as well as treatments adjacent to the roadway. The recent emphasis on context sensitive design/solutions may provide additional insight to potential solutions and their effectiveness relating to the transition area of concern.

The objective of this research project is to develop additional techniques and tools for designers and traffic engineers for application in developing effective designs for urban/suburban projects with intermediate speeds or transition zones from rural to urban/suburban environments.

♦ **Project 15-41**
Updated Headlamp Design Criteria for Sag Vertical Curves

Research Field: Design
Source: AASHTO Highway Subcommittee on Design, Kansas
Allocation: $250,000
NCHRP Staff: B. Ray Derr

Headlamp sight distance is one of four design criteria for sag vertical curves and is the most often used of the four criteria. The current criterion bases the length of a sag curve on the distance illuminated by a headlamp beam that diverges at 1 degree above the horizontal. This criterion was developed in the late 1930s and has remained unchanged since except for a decrease in the headlamp height (from 2.5 to 2.0 ft in the 1965 Blue Book). At the time the criterion was developed, the sealed beam headlamp was established as the standard headlamp system for U.S. vehicles and the sealed beam headlamp continued to be the standard headlamp into the mid-1980s. However, starting in the mid-1980s, vehicle manufacturers began introducing changes in headlamp design with varying headlamp performance. A more detailed research study is needed to update the sag curve sight distance criteria so that it reflects the performance of the modern vehicle fleet.

The design criteria for sag curves have not been investigated in detail in over 60 years. There have been significant changes in vehicle design/performance and in driver perception during the ensuing years. It is appropriate to evaluate the current design criteria to determine whether the basis for design is still valid and whether improvements can be realized through revised design criteria. Revised criteria could improve safety, operations, and/or reduce construction costs. Based on the exploratory research conducted to date, it is expected that this research will produce recommendations for changes in the design criteria for sag curves. The changes could be as simple as a reduction in the 1.0 degree α angle currently used to a more extensive change such as a new criteria based on a different concept for sag curves.

The objective of the proposed research is to evaluate the issues associated with sight distance on sag vertical curves and develop updated criteria that reflect the conditions associated with modern highways and vehicles. Among the critical factors to be addressed are the performance characteristics of modern headlamps, appropriate headlamp illumination levels for sag vertical curves, and the relation between revised sag curve sight distance criteria and other sag curve design criteria. Other sag curve design criteria also should be considered, particularly with respect to comparisons between the various criteria that may affect sag curve design. The research results should provide specific recommendations for revisions to sag curve design criteria found in *A Policy On Geometric Design of Highways and Streets,* AASHTO.

♦ **Project 15-42**
Use of Bicycle Lanes for Various Roadway Characteristics

Research Field: Design
Source: AASHTO Joint Technical Committee on Nonmotorized Transportation
Allocation: $300,000
NCHRP Staff: Christopher J. Hedges

U.S. practitioners have minimal nationally recognized guidance regarding the roadway characteristics under which bicycle lanes should be provided or, at least, considered. The current (1999) edition of the *AASHTO Guide for the Development of Bicycle Facilities* describes design of bicycle lanes, but presents virtually no guidance about roadway conditions under which they should be provided, considered, or omitted. On busier urban roadways with operating speeds above 40 mph, usage of bicycle lanes often is observed to be modest; the *Guide* simply observes that "additional widths (more than 5 ft.) are desirable" where speeds exceed 50 mph or truck volume is heavy. It is sometimes suggested that, at some threshold, designation of nearby bike routes should be considered in lieu of bicycle lanes or perhaps on thoroughfares with relatively low speeds or truck volumes, or where on-street parking is allowed, wide curb lanes or shared roadway treatments may be as or more effective than a bicycle lane.

Some state DOTs have adopted policies of (generally) routine provision of bicycle lanes in urban projects, some consider whether the road is included in the local bicycle plan, and some consult criteria tables in a 1994 study published by FHWA, "Selecting Roadway Design Treatments to Accommodate Bicycles." Selection factors proposed in this report are traffic volume, average traffic operating speed, "traffic mix" (presence of heavy vehicles), on-street parking, sight distance, and intersection spacing. For a given combination, tables identify a "desirable" treatment (wide curb lane, shared lane, paved shoulder, or bike lane) of recommended minimum width (at least as great as AASHTO's). The authors described their recommendations as "preliminary" and anticipated that the tables would be refined as the state of the practice evolved, but no revision has ever been developed.

The objective is to develop design criteria for bicycle lanes based on roadway characteristics including, but not limited to, classification, speed, ADT, number of trucks, the grade of the roadway, and parking. The design criteria will help determine if bicycle lanes should be installed and if so, what would be the recommended width of the bicycle lane, the adjacent travel lane, and, if applicable, parking.

♦ **Project 16-05**

Development of Cost Effective Treatments of Roadside Ditches to Reduce the Number and Severity of Roadside Crashes

Research Field: Design
Source: AASHTO Highway Subcommittee on Design
Allocation: $400,000
NCHRP Staff: Charles W. Niessner

Roadside ditches or swales are an integral feature of highways, especially two lane rural highways. They are critical for control of storm water runoff on highways. Where space allows, shallow swales are used, but when right-of-way is limited, ditches with deeper and sharper drops are used. These features can be obstacles to errant motorists that leave the roadway. The Fatality Analysis Reporting System (FARS) indicated in 2006 1,260 fatal crashes occurred where a ditch was the first harmful event. It is not possible to differentiate between ditches and swales in the data. There has been a trend over the past 15 years that over 1,000 fatalities annually can be attributed to ditches.

The *AASHTO Roadside Design Guide* provides some guidance on preferred configurations for ditches. This guidance is based on the results of limited testing and simulations conducted in the 1970s. There is variation in the practices across the states for designing and maintaining ditches and, for many miles of roads, the ditches are a remnant of older highways that have never been updated to current standards.

The limited right-of-way often dictates the configuration of ditches and in many cases the preferred configurations are not practical. Enclosed drainage systems are expensive and result in additional requirements for treatment and discharge of the runoff. Installing a barrier between the travelled way and the ditch reduces the available clear zone, is impractical in respect to cost in many cases, and presents additional problems, such as terminal design and sight distance, when driveways are allowed. Since ditches are part of a drainage system, other elements such as culverts, inlets, and holding basins require structures that become roadside obstacles (e.g., headwalls, riprap, and curbs).

An urgent need exists to reduce the number and severity of crashes involving roadside ditches, through a deeper understanding of the factors involved in crash events, the evaluation of vehicle dynamics, and the identification and cost-benefit assessment of treatment options. With this information, cost effective countermeasures can be identified and implemented to mitigate ditch crashes.

The objectives of this research are to:

- Develop deeper insights into the interaction of factors that influence the nature of crashes involving ditches.
- Analyze the influence of varying ditch configurations on vehicle dynamics and their role in the severity of crashes.
- Identify cost effective treatments for roadside ditches that will reduce the number and severity of crashes.
- Develop improved guidance for ditch design and maintenance for inclusion in the Roadside Design Guide.

This effort should focus on identifying treatments for ditch design and maintenance as other efforts are already focusing on the related topic of keeping vehicles on the roadway.

To meet the project objectives the following tasks would be performed:

- Review domestic and international literature with a focus on ditch design and countermeasures that have been tried and evaluated. Consider undertaking a review of agency standards (i.e., on-line) and conducting a survey to identify innovative treatments that may not have been documented.
- Analyze collisions involving ditches to give context to the types of collisions involved (e.g., rollover, curve related, pavement edge scuffing) so that the counter measures can be focused. Attempt to get needed insights on these crashes from existing sources of data.

- Model dynamics of vehicles traversing ditches to evaluate the vehicle reactions to different cross sections and treatments. These efforts should build upon the results of current work.
- Develop a range of alternative treatments for ditches based upon knowledge gained in the literature review, contacts, and crash analyses. Organize a "brainstorming session" with knowledgeable professionals to identify other potential treatments.
- Formulate guidelines for the deployment of ditch treatments that consider the risk factors, costs, feasibility, road geometry, and traffic.
- Undertake a cost effectiveness analysis for high- to low-cost alternatives to enhance guidance relative to available budgets. Identify the expected benefits of these treatments to allow rational selection of alternatives.
- Draft new guidelines for the design and treatment of ditches in priority locations. Review these guidelines and the rationale for them with the panel and a select group of knowledgeable engineers.
- Prepare a final report that documents the efforts undertaken and thought processes that led to the guidelines.

♦ **Project 17-46**
Comprehensive Analysis Framework for Safety Investment Decisions

Research Field:	Traffic
Source:	AASHTO Standing Committee on Highway Traffic Safety
Allocation:	$600,000 (Additional $150,000 from Federal Highway Administration)
NCHRP Staff:	Charles W. Niessner

The diverse safety community in the United States continues to make substantial, incremental progress in developing and implementing cost-effective approaches. AASHTO and FHWA have provided national leadership with work such as Model Minimum Inventory of Roadway Elements (MMIRE), the Digital Highway Measurement System, the Interactive Highway Safety Design Model, SafetyAnalyst, etc.; and towards critical upcoming milestone products such as the Highway Safety Manual and SHRP2 results (especially the crash causation database which will be created). NHTSA and FMCSA, working with AASHTO/FHWA and other partners, have advanced similar improvements focusing on behavioral and heavy vehicle issues.

While the range of current efforts is impressive, we are just on the cusp of creating a truly comprehensive analysis and decision-support system with the capability to compare the effectiveness of investment and policy opportunities across the 4 Es of safety (i.e., the contributions of engineering, education, enforcement, and emergency medical services). This project would create and sustain a nationally-coordinated, multi-year initiative to integrate efforts like those noted above into a Comprehensive Safety Analysis Framework. This Framework is envisioned as a 'blue print' which the full safety community will contribute to and which will provide for objective, data driven evaluation of safety programs, policies, and investments across Federal, state, and local levels.

Objectives and tasks to create and sustain the Safety Analysis Framework include: (1) develop, pilot test, evaluate, fine tune, and update the model framework for estimating the effectiveness of behavioral countermeasures; (2) in-depth evaluation of existing and soon-to-be-released tools; (3) assessment of critical deficiencies in data and tools to support better comprehensive decision making; (4) development of a comprehensive, consensus strategic plan for further development and support of data systems and analytical tools to address critical deficiencies, coordination among ongoing activities, professional capacity needs, and support for investment decision making and policy analysis and development; and (5) implementation support for the multi-year program, to include tasks such as coordination of data needs across all elements, development of '4 E' policy analysis tools not currently available, quality assurance of analysis algorithms, software integration efforts as needed, communication, training, and technical assistance for at least the first several years.

The intended outputs are: (1) a strategic development and deployment program coordinated across partners in the 4 Es; and (2) an initial version of a next generation of tools that permits objective analysis of investment decisions across the 4 Es. The expected benefit/outcomes are significantly more effective investment decisions and, as a result, steeper reductions in motor vehicle fatalities and serious injuries.

♦ **Project 17-47**
Human Factors Guidelines for Road Systems-Phase IV

Research Field: Traffic
Source: AASHTO Standing Committee on Highway Traffic Safety
Allocation: $500,000
NCHRP Staff: Charles W. Niessner

The TRB Joint Subcommittee on "International Human Factors Guideline for Road Systems," AND10(2), was created to help plan the development of a human factors guideline for road systems that highway designers and traffic engineers could readily use in their work. NCHRP 17-18(8), *Comprehensive Human Factors Guidelines for Road Systems* was initiated in 2001 and provided the framework for the guideline and two chapters. The project was completed on January 31, 2005.

NCHRP Project 17-31 began in August 2005 to develop additional chapters and integrate them with the work completed under Project 17-18(8). The Project 17-31 contractor developed a style guide for the Guideline, refined Chapters 1 through 5 from NCHRP Project 17-8(8), and prepared four new chapters: signalized intersections, unsignalized intersections, work zones, and horizontal curves. Project 17-31 was completed in August 2008. The work completed under project 17-31 has been published as NCHRP Reports 600A and 600B.

NCHRP Project 17-41 started in March 2008 and is developing 5 additional chapters. The completion date for this project is March 2010. With the completion of Project 17-41 there are 7 chapters remaining to be completed.

The Human Factors Guide (HFG) is intended to be a resource document for highway designers, traffic engineers, and other practitioners. The purpose of the HFG is to provide the best factual information and insight on road users' characteristics, in a useful format, to facilitate safe roadway design and operational decisions. The impetus behind this effort was the recognition that current design references have limitations in providing the practitioner with adequate guidance for incorporating road user needs and capabilities when dealing with design and operational issues.

The work of this Joint Subcommittee is being coordinated with the development of the Highway Safety Manual being overseen by the TRB Task Force to Develop a Highway Safety Manual (HSM). The first edition of the HSM is expected to be produced after the completion of NCHRP Project 17-36 in January 2009. While the HSM includes one section of a chapter on human factors, it will provide only a broad scope and not include guidelines.

The objective of this project is to complete the development of the Human Factors Guide.

The following tasks would be conducted: (1) literature review, (2) develop a list of topics under each chapter, (3) prepare annotated outline for each guideline, (4) develop draft guidelines, and (5) develop final guidelines.

♦ **Project 17-48**
Development of a Strategic National Highway Infrastructure Safety Research Agenda

Research Field: Traffic
Source: AASHTO Standing Committee on Highway Traffic Safety
Allocation: $400,000 (Additional $100,000 from Federal Highway Administration)
NCHRP Staff: Christopher J. Hedges

Reducing the number of fatalities and injuries from highway traffic crashes is a high-priority goal shared by AASHTO, FHWA, and the States. While fatality rates have steadily decreased over time, the number of fatalities has remained unacceptably high. While increased safety funding under SAFETEA-LU and States' development and implementation of strategic highway safety plans can be expected to significantly improve highway safety, research is needed to develop innovations that will be needed to achieve AASHTO's goal of halving fatalities in 20 years. Achieving the greatest benefits from research will require well-targeted and coordinated research investment.

At the request of FHWA and AASHTO, TRB convened an expert committee to provide an independent review of current processes for establishing research priorities and coordinating highway safety research activities. In TRB Special Report 292, the committee presented its findings and recommended that "an independent scientific advisory committee should be established and charged with (1) developing a transparent process for identifying and prioritizing research needs and opportunities in highway safety, with emphasis on infrastructure and operations; and (2) using the process developed to recommend a national research agenda focused on highway infrastructure and operations safety."

This problem statement proposes an NCHRP project to implement expert committee's recommendation. Development of a national research agenda would support the Safety Management Subcommittee of the AASHTO Standing Committee on Highway Traffic Safety in carrying out its highway safety research oversight and advocacy responsibilities.

The objectives of this project are to: (1) develop a transparent process for identifying and prioritizing research needs and opportunities in highway safety; and (2) using the process developed to recommend a national research agenda focused on highway infrastructure and operations safety. Specific tasks necessary to achieve these objectives include:

1. Develop a process for identifying and prioritizing safety research needs that includes the following features: (a) a quantitative analytical approach that examines clearly defined criteria to determine the value of a research project or topic, and (b) the involvement of a mix of experts to formulate an agenda that is informed by the quantitative analysis results.
2. Develop research priorities by applying the process to identify critical safety problems, identify potential research issues, assess the status of data and methodologies to conduct research that addresses the problems, estimate the costs and timeframes for research, and assess the likely outcome of alternative research topics.

♦ **Project 20-83(06), FY 2010**
Effects of Socio-Demographics on Travel Demand

Research Field: Special Projects
Source: AASHTO Standing Committee on Research
Allocation: $1,000,000
NCHRP Staff: Christopher J. Hedges

The transportation industry will face new and emerging challenges in the future that will dramatically reshape transportation priorities and needs. AASHTO recognizes that research can help ensure that transportation practitioners are equipped to deal with future challenges facing the industry over the next 30 to 50 years. For the fiscal year 2009/2010 programs, AASHTO allocated $8,000,000 to examine longer term strategic issues, both global and domestic, that will likely affect state departments of transportation (DOTs). This is one of seven projects selected and deals with the effects of socio-demographics on travel demand.

The profile of America is expected to change substantially over the coming 40 years. According to the U.S. Census Bureau, the U.S. population is projected to increase to 438 million by 2050, more than a 40% increase from the 2008 population of 304 million. This population will be more ethnically diverse; over 80% of the projected population increase is attributed to immigrants and their descendents. The population also will be substantially older; it is estimated that more than 20% of the U.S. population will be 65 years or older by 2050, compared to 12.6% currently. The sizeable increase in population will create the need for more housing, employment, and services, which may lead to substantial impacts on travel patterns and demands. It is estimated that majority of the U.S. population will live in mega-regions, with more than 80% of the population in urban/metro areas, including suburbs. Baby boomers are expected to choose a 'soft retirement' and continue to work part-time beyond retirement age. Young people coming out of full-time education may increasingly choose to enter what they consider temporary, short-term jobs, which they use to finance international travel, volunteering in nonprofit or arts-related careers, and/or continued education. These nontraditional workers are sometimes referred to as "Moofers" (Mobile Out of Office Workers). Potential changes in family structure, incomes, lifestyles, and social expectations also may occur.

Transportation demand is a function of the number and types of people in an area, their lifestyles, and economic structure and activity. Many issues over the next 30 to 40 years will change the population's transportation needs, travel patterns, and expectations regarding mobility. These include changes in demographics (e.g. population size, affluence, ethnicity, age, etc.) and technologies that substitute or alter travel behaviors (e.g. telecommuting opportunities or mode shifts). In addition to passenger travel, changes in global and national economic activity, fuel prices, and policy will influence freight travel demand. The interplay between these issues is important as well. The effects of some trends, such as population growth, may mitigate or amplify the effects of others, such as the aging population or migration. In 2050, the U.S. population will be significantly larger, older, and more ethnically diverse than today. Clearly, these trends may dramatically influence transportation demands and patterns. Some of these trends suggest a dramatic increase in mobility needs – the addition of over 130 million more Americans in the next 40 years, medical advances that enable older Americans to have ncreasingly active lifestyles, and shifts in the growth areas within the U.S. suggest surging travel demands. At the same time, it is plausible that travel demands will not increase substantially, due to enhancements in information and communication technologies, changes in land use patterns (e.g., movement to urban, pedestrian-oriented areas that minimize vehicle travel demands), increases in fuel prices, and changes in attitudes toward transit and alternatives to driving. Furthermore, if a majority of the population increase is from immigrants, then their transportation habits may differ. The patterns of travel also could change substantially, with travel increasing for different types of trips, in different locations, and at different times than currently.

Long-range transportation planning being conducted by States and Metropolitan Planning Organizations takes an outlook of 20+ years into the future, but is largely based on the current relationships between demographics, land use patterns, and travel activities. A wide range of demographic, social, technological, and economic changes are likely to affect travel demands and patterns in the future. These changes, and their fundamental relationships to travel demand, are currently not well understood.

The objective of this project is to determine how socio-demographic factors are likely to affect travel demand over the next 50 years, and to identify strategies and actions that can be used by state DOTs to plan and prepare for plausible future scenarios.

The research should focus on understanding the fundamental relationships between social, demographic, and economic factors and travel demands. These include effects such as increasing diversity, aging and retirement patterns, personal wealth, increasing immigration and its impact, increasing mega-regions, changing regional migration patterns, and the decreasing size of households and changing family structures. It also would help to develop more accurate tools and approaches for forecasting travel demand and behavior.

Note: At its March 2009 meeting, the AASHTO Standing Committee on Research recommended continuing the series of projects under Project 20-83, *Long-Range Strategic Issues Facing the Transportation Industry,* and allocated $3,000,000 for FY 2010. From this amount, they selected two additional projects from a list of previously suggested topics and allocated $1,000,000 to each one: (1) Effects of Socio-Demographics on Travel Demand, and (2) Sustainable Transportation System and Sustainability as an Organizing Principle. These two projects are now being listed as “New Projects.” The remaining $1,000,000 is discussed under “Continuations” for Project 20-83.

♦ **Project 20-83(07), FY 2010**

Sustainable Transportation Systems and Sustainability as an Organizing Principle for Transportation Agencies

Research Field: Special Projects
Source: AASHTO Standing Committee on Research
Allocation: $1,000,000
NCHRP Staff: Lori L. Sundstrom

The transportation industry will face new and emerging challenges in the future that will dramatically reshape transportation priorities and needs. AASHTO recognizes that research can help ensure that transportation practitioners are equipped to deal with future challenges facing the industry over the next 30 to 50 years. For the fiscal year 2009/2010 programs, AASHTO allocated $8,000,000 to examine longer term strategic issues, both global and domestic, that will likely affect state departments of transportation (DOTs). This is one of seven projects selected and deals with the integration of sustainability as a core principle in a DOT's investment and operations decisionmaking processes.

Building on related ongoing and completed research, this research will refine the definition of what constitutes a sustainable transportation system and demonstrate how DOTs can integrate sustainability principles into their day-to-day operations and investment decision making, including long-range planning, scenario development, and forecasting activities. The research also should address internal processes and organizational schemes that would assist DOTs in institutionalizing sustainability.

Sustainability is likely to become a core focus for transportation agencies in the future, both because citizens and state legislatures are requiring it as a matter of public policy and because it has the potential to produce more cost-effective transportation solutions over the life span of a transportation facility. DOTs are challenged, however, to identify what a sustainable transportation system is in a manner that they find useful for purposes of making decisions that will balance short-term cost effectiveness with long-term sustainability. Guidance is needed on how to achieve a sustainable transportation system that is economically feasible and sound, environmentally friendly or at least environmentally benign, and supports commerce and facilitates mobility. DOTs also are interested in how their internal operations can be made to operate in a sustainable manner and how they can monitor their progress and measure success.

This research should construct one or more scenarios in which DOTs will be asked to achieve sustainability goals that reflect likely conditions 30-50 years in the future. The research should place key DOT activities such as forecasting, planning, project development, maintenance, and operations, in that future context and should at a minimum also consider basic relationships between transportation, public health, environmental justice, and quality of life in developing a framework for sustainability that will serve DOTs. The research will identify key elements of a long-term sustainable transportation system and strategy.

Increasing societal awareness on the environmental affects of the surface transportation system has already led to new demands on DOTs to provide environmentally and socially responsive infrastructure and transportation services. To meet these expectations, DOTs may require data and models that can calculate marginal environmental, societal, and economic values of transportation system performance and compare with marginal costs. In addition, the role of federal agencies and federal regulation and how they help or hinder the DOTs pursuit of sustainability should be considered. Building on research currently under way in the Strategic Highway Research Program 2, this project also may project how different and more comprehensive approaches to environmental protection and enhancement will enable DOTs to achieve a sustainable transportation system.

Note: At its March 2009 meeting, the AASHTO Standing Committee on Research recommended continuing the series of projects under Project 20-83, *Long-Range Strategic Issues Facing the Transportation Industry,* and allocated $3,000,000 for FY 2010. From this amount, they selected two additional projects from a list of previously suggested topics and allocated $1,000,000 to each one: (1) Effects of Socio-Demographics on Travel

Demand, and (2) Sustainable Transportation System and Sustainability as an Organizing Principle. These two projects are now being listed as "New Projects." The remaining $1,000,000 is discussed under "Continuations" for Project 20-83.

♦ **Project 20-84**
Streamline and Simplify Right-of-Way Procedures and Business Practice

Research Field: Special Projects
Source: Federal Highway Administration, California, Oklahoma, Washington
Allocation: $500,000
NCHRP Staff: David A. Reynaud

Several State Departments of Transportation (DOTs) are considering revising their right-of-way business practices with the goal of simplifying and streamlining processes. Current right-of-way practice and procedure manuals are the products of 40 years of statutes, case law, regulations, management styles and best practices. The procedural manuals have chapters to cover elements such as: a) appraisal; b) appraisal review; c) relocation planning and assistance; d) relocation eligibility and supplemental payments; e) nonresidential relocations; f) acquisition and negotiations; g) legal settlements; h) eminent domain; i) titles and closing; j) property management; k) leasing; l) sale of excess property; m) mapping and geographic information systems (GIS); n) encroachments; o) contracting for services; and p) administrative costs.

Procedures and guidelines are often an accumulation of historical practice or those adopted from other agencies. State procedures vary widely because of differences in State laws. Local agencies are required to follow State DOT procedural manuals when they use State or Federal funding. Questions arise as new staff try to understand the reason or underlying basis for requirements. Contractors and consultants face a wide array of requirements and forms among the various States.

This research is in support of the AASHTO Highway Subcommittee Right-of-Way and Utility strategic plan to provide leadership and support to member agency right-of-way staff. This research will provide new direction and lead to immediate cost savings by reducing the hours required to accomplish certain functions. This research will result in streamlined business practices that are easier to maintain, cost effective and result in delivery of projects sooner.

Research is needed to provide information to State DOTs and local agencies to rationally evaluate current right-of-way procedures and business practices; to determine what function is served by each procedure; to determine the need for each procedure, i.e. statute or practice; to document the benefits and operational logic for continuing a procedure, modifying, or eliminating it, evaluate the cost of maintaining current procedures and to quantify the benefits from them. This includes, but is not limited to, the current cost of agents, training new agents and administrative costs on a parcel or tract basis. Determine what processes are essential to providing a consistent product and comply with statutory requirements, such as the Uniform Relocation Assistance and Real Property Acquisition Policies Act of 1970 (Uniform Act), as well as the most common elements of State eminent domain laws, identify institutional, political, and economic barriers to the adoption of procedures that will be easier to maintain for the next 20 years, and examine and compare several common types of existing FHWA approved right-of-way manuals used by State DOTs and local agencies and common State regulations.

Agencies will be contacted and staff will be interviewed to ascertain what are the origins, purpose and authorities for the existing procedures; what criteria and procedures are needed, as a minimum, to protect owner and tenant rights; what procedures would work if the agency could start anew; how and/or whether procedures might be modified for local agency use, i.e., a stand alone manual for local agencies; what are the issues in administering procedures that need to be addressed to assure consistent application; and what are the institutional, political, and economic barriers to implementation?

A major objective of the research will be to develop a rationale or basis for a new or modified approach. This will include an objective analysis of all key elements mentioned above, i.e., appraisal, appraisal review, relocation, etc.

This research would culminate by analyzing the typical right-of-way business model for the four major elements of appraisal, acquisition, relocation, and property management, and developing a revised model that is less costly to maintain.

It would outline a sample procedural manual with forms that could be used to administer a simplified and cost-effective right-of-way program that is responsive to national statutes and the Uniform Act. The resulting business model would be accompanied by a cost/benefit analysis and recommended roll-out implementation plan that could be readily adopted and applied by State DOTs and local agencies for national consistency.

One of the initial goals of the Uniform Act was to create a fair and consistent process for the acquisition of real property by public agencies. This research would help us assure the continued uniformity of the process.

This research will be a direct follow-on to the 2008 International Scan for ROW and Utilities called "Integrating & Streamlining Right of Way and Utility Processes with Planning, Environment, and Design." Ideas and strategies derived from the 2008 International Scan will feed directly into revised business practices. State DOTs who undertake pilot projects in 2009 will be able to use lessons learned and provide input to this research effort. The timing is beneficial for all parties in that this research product will bring about full implementation of the 2008 streamlining strategies.

This research will provide new direction and lead to immediate cost savings by reducing the hours required to accomplish certain functions. This research will result in streamlined business practices that are easier to maintain, cost effective and result in delivery of projects sooner. The effort devoted to training new right-of-way agents, who may or may not stay with the agency, is becoming cost prohibitive and is time consuming.

It is anticipated that there will be many institutional barriers to overcome. Many right-of-way agents have adapted to the current procedures and will be resistant to change. State DOT legal staff also may resist changes, thinking that revised procedures may affect property owner rights. In order to address these barriers, the final research report should contain an outline of a revised procedural manual that would be sufficient to meet Federal regulations and laws, allowing each State to augment this information with specifics to address that particular State's laws.

♦ Project 20-85
Wind, Solar, and Ground-Source Energy for Maintenance Area Facilities

Research Field: Special Projects
Source: Minnesota
Allocation: $500,000
NCHRP Staff: Edward T. Harrigan

In the face of rising energy costs, state DOTs are striving to reduce the energy requirements for heating, cooling, and maintaining their facilities. In addition to the customary options of oil, gas, and electrical energy, alternative sources of energy, including wind power, solar power, and ground-source heating and cooling, are being evaluated for their cost effectiveness and reliability. While none of these alternatives to conventional energy sources may be effective alone in all situations, together they may provide appreciable savings in energy use for some facilities. In the future, energy conservation guidance for state agency facilities may mandate increased efficiencies in facility energy consumption. This research will evaluate active energy options including wind power, solar power and ground-source heating and cooling, as well as passive building and site modifications to reduce energy use.

The objectives of this research are to identify and evaluate effective, implementable means of energy conservation for maintenance and other state DOT facilities.

The following tasks are anticipated to accomplish these objectives: (1) assess alternative energy options currently employed at DOT-managed facilities; (2) assess potential areas/sites where alternative sources of energy such as wind, solar, and ground-source heating/cooling may be effective; (3) assess maintenance area sites where passive site and building modifications have the most probability of reducing energy use; (4) prioritize a short list of sites where alternative energy sources and passive energy reduction measures would have the most potential for being effectively employed in the near-term (within 10 years); (5) using currently available technology for the three alternative energy options and passive site and building measures, develop preliminary, detailed plans for installation of one option at each of four selected sites reflecting differing climatic and physiographic regions within one or more states; (6) develop cost performance measures for evaluation of reduced energy use and related cost effectiveness for each of the four options employed; and (7) in partnership with a DOT, construct and maintain applicable structures and measures for the four options and monitor effectiveness for two years in accordance with the established performance measures.

♦ **Project 20-86**

Skill Set Requirements and Career Opportunity Awareness for Transportation Systems Operation and Management Needs at High School and College Levels

Research Field: Special Projects
Source: California
Allocation: $375,000
NCHRP Staff: Andrew C. Lemer

There is an accelerating shift in state Departments of Transportation (DOTs) away from the more traditional design and construction function of civil engineering to a systems operation and management (SOM) function. Fewer engineering students, coupled with the imminent departure of the over-represented older groups of engineers (over 40 years old) will soon deplete the ranks of qualified transportation professionals. A study done by the Texas Transportation Institute in 1998, for example, noted that the "…increased emphasis on transportation systems operation and management requires a skill set not available in traditionally trained students."

Research has confirmed the need to provide outreach to students at all educational levels, particularly middle school and high school, to positively influence students' interest in transportation-related engineering careers. Programs have been implemented in a number of regions with this goal. However, there is little information on how much emphasis is placed on highlighting career opportunities for transportation engineering SOM functions. Research is needed to address the shift in emphasis from capital programs to operations and management and how to motivate students to be interested in this aspect of a career in transportation.

The objective of this research will be to define a set of knowledge, skill, and ability requirements needed by the DOTs for their current and future engineers relative to the increased emphasis on SOM responsibilities. The research will include proposals for high school and college curriculum enhancements that will better meet the future workforce needs of the DOT. The research also will propose outreach programs for grades 6-12 and undergraduate students that emphasize the expanded career opportunities of different engineering skills beyond those of design/build, to include SOM activities within the DOT.

♦ **Project 24-34**
Risk-Based Approach to Bridge Scour Prediction

Research Field:	Soils and Geology
Source:	AASHTO Highway Subcommittee on Design
Allocation:	$500,000
NCHRP Staff:	Waseem Dekelbab

Current practice for the prediction of scour depth at bridge piers and abutments uses empirical equations developed primarily from laboratory-scale studies, supplemented by limited data from field measurements. Equations for contraction scour (both clear-water and live-bed conditions) are based on an approach that combines both empirical and deterministic relationships. Additionally, the statistical analysis that was performed on the data collected from the laboratory studies and was used to create these relationships employs various statistical approaches that possibly provide more conservative results than necessary. When you also take into account the uncertainty associated with the development of key parameters used in the empirical relationships, the room for error is significant. In contrast, because of numerous advantages, bridge structural engineers, and more recently geotechnical engineers, have adopted Load and Resistance Factor Design (LRFD) which is a probabilistic approach to design. LRFD considers a probabilistic approach and allows for the possibility of assessing the level of risk associated with a given design. There is a need for the bridge scour engineer to have the option of performing scour calculations using probabilistic methods so that risk can be more appropriately assessed and the option of something other than the most conservative design considered.

Current practice for determining the total scour prism at a bridge crossing involves the calculation of various scour components (e.g., pier scour, abutment scour, contraction scour, and long-term channel changes). Using the principle of superposition, the components are considered additive and the scour prism then is drawn as a single line for each frequency flood event (e.g., 50-year, 100-year and 500-year flood events). This approach does not provide an indication of the uncertainty involved in the computation of any of the additive components. Uncertainties in hydrologic and hydraulic models and the resulting uncertainty of relevant inputs (e.g., design discharge, flow duration, velocity, depth, flow direction, etc.) to the scour calculations will have a significant influence on scour prediction.

To develop an overall estimate of confidence in the estimated scour magnitude, one must examine the level of confidence associated with the results of the hydrologic analysis (design discharges, flow duration, etc.), the level of confidence associated with the hydraulic analysis (depths, velocities, flow direction, etc.), and the level of confidence associated with the scour estimates (pier, abutment, contraction, long-term channel changes, etc.). Scour reliability analysis involves quantification of the uncertainties in each of these steps and then combines them in such a way that the overall estimate of confidence is known for the final prediction of scour.

For the hydrologic analysis component, the desired end product could result in a probability density function (PDF) of the peak discharge. This can be done by examination of the flood flow frequency curve developed from gage records. If no gage records are available and regional regression equations are used, levels of confidence based on the results of the statistical analysis used to develop the regression equations can be used. If a single or lumped-parameter hydrologic model is used, important parameters could be identified, a PDF developed for these parameters, and a Monte Carlo simulation of these parameters could be performed to obtain the PDF of the peak discharge. The same can be performed for the hydraulic model except that the PDFs of the relevant hydraulic parameters would be developed using Monte Carlo simulations.

Current practice provides an estimate of scour based on the hydrologic and hydraulic conditions associated with a specified design event (a 100- or 500-year flood, for example). The scour equations are generally understood to be conservative in nature, and have been developed as “envelope” curves for use in design. The research objective is to develop a methodology that can be used in calculating bridge scour so that the scour estimate can be linked to a probability; for example, there is a 95.0% probability that the maximum scour will be 8.3 feet or less over the life of the bridge.

To achieve this objective, at a minimum the following tasks must be performed:

1. *Review of existing knowledge*: Some work along these lines has already been done in the area of hydrologic and hydraulic analysis. Relating the uncertainty associated with the hydrologic and hydraulic analysis to the uncertainty associated with the scour estimation techniques needs to be performed. Other disciplines where risk and reliability approaches are being integrated into engineering design also should be explored and documented by the research team.
2. *Identify uncertainties*: This task will consist of identifying and evaluating the parameters associated with each of the various components (hydrology, hydraulics, and scour).
3. *Formulate the methodology:* This task will consist of combining the uncertainty associated with each of the various components (hydrology, hydraulics, and scour) into a procedure to use for scour prediction. The results of this task will ultimately lead to a probabilistic method to compute and evaluate bridge scour that will be consistent with LRFD approaches used by structural and geotechnical engineers.
4. *Proof of concept*: This task will consist of validating the methodology against data sets where variability in measured scour has been quantified. The new methodology must be demonstrated to be consistent with probabilistic approaches currently used by bridge structural and geotechnical engineers.
5. *Final Report*: The final report will be written in two parts. The first part will document the research performed to arrive at the methodology. The second part will be written in the form of a manual that provides design guidelines for practitioners in the field of bridge scour calculation.

Currently scour estimates at bridge foundations use the best available technology, but are still roundly criticized as being overly conservative. The most common complaint is that the equations that were developed under laboratory conditions don't fit conditions at the site. Often this results in deeper foundations than necessary which leads to more costly bridge designs, which can stress already overloaded state department of Transportation budgets for bridge replacement and repair. Bridge designers and engineers are in need of a tool to make cost versus reliability tradeoff decisions with respect to scour and foundation design. A reliability-based design procedure for estimating scour at bridges will provide a consistent methodology for making decisions on design scour depth based on calculated risk instead of estimates which can be overly conservative.

The pay-off is a scour estimate that will be more reliable in that it will be tied to a selected level of reliability that can be effectively communicated to the public. This type of approach will help alleviate over-conservatism in bridge design inconsistent with accepted target risk levels.

♦ **Project 25-33**
Evaluation of the Methodologies for Visual Impact Assessments

Research Field:	Transportation Planning
Source:	California, Rhode Island
Allocation:	$100,000 (Additional $100,000 from Federal Highway Administration)
NCHRP Staff:	Nanda Srinivasan

National Environmental Protection Act (NEPA) requires that visual impacts be considered for highway improvement projects. To assist State Departments of Transportation (DOTs), the FHWA developed *Visual Impact Assessment for Highway Projects* in 1981 to provide guidance in analyzing and quantifying visual impacts for highway proposals. This is the standard methodology used throughout the country to identify visual impacts for highway improvements. The guidance is over 27 years old and has not been evaluated for effectiveness, nor have any substantial updates been made. As a result, some DOTs have modified this methodology to meet their needs. This implies that the FHWA methodology may no longer be effective in meeting NEPA requirements for the protection of scenic quality. To understand the value of the current FHWA guidance and to justify the substantial resources needed to prepare visual impact assessments, there is a critical need to understand the usefulness of the guidance as an effective environmental assessment tool for evaluating scenic impacts of highway design. Research on this topic could provide recommendations for updating the current FHWA visual analysis guidance, benefiting DOTs nationwide by streamlining the methodology.

Using accepted research methods, a consultant would survey DOTs to determine applicability and effectiveness of the FHWA visual analysis process in assessing visual impacts for highway projects. Research would determine if the FHWA methodology is still being used, has been modified, or if the DOTs have adopted their own approach. The project would include an examination of 50 to 75 highway projects of different sizes across the country to determine if the visual impact studies were instrumental in protecting scenic resources identified in the environmental reports.

♦ **Problem No. 2010-D-20**

Modulus Based Construction Specifications and Issues for Highway Earthwork and Unbound Base Materials

Research Field: Materials and Construction
Source: Louisiana
Allocation: $500,000
NCHRP Staff: David A. Reynaud

Earthwork and unbound bases are a significant portion of highway construction and are important to the performance of highway infrastructures. Due to their accumulated experience over the years, highway engineers and practitioners feel comfortable in specifying construction compaction quality control in terms of dry unit weight and moisture content. However, there is a lack of direct connection between design and construction, in the sense that the dry unit weight and moisture content of materials cannot be used directly in design. Instead, the mechanical properties of materials, such as strengths and moduli, are required. In the case of pavement engineering, both the 1993 AASHTO Pavement Design Guide and the new Mechanistic-Empirical Pavement Design Guide (M-EPDG), which is newly adopted by AASHTO, require the resilient moduli of bases and subgrade as major input for highway pavement structural design.

Due to the limitations of current practice in the quality control and quality assurance for earthwork and unbound base construction, the technology of intelligent compaction has been developed. The stiffness, or modulus, of compacted materials is measured during the compaction process and used as feedback to automatically adjust the compaction effort to be applied. The question, not just with the intelligent compaction but with all construction techniques, is whether the field determined stiffness or modulus can be used as an acceptance criterion for compaction quality control. The doubt and reluctance to accept this new approach lie in the concerns regarding the long term performance of compacted materials. Therefore, the modulus-based construction specifications should address issues with a perspective of long-term performance.

The fact that modulus is strongly influenced by the variation of moisture content for earth and unbound materials is well understood. The variation of moisture content, in turn, depends on the materials' capability, which is controlled by the materials' compositions and physical conditions, to absorb available free moisture, which is controlled by the local climatic environment and the distance to the ground water table. All of these should be reexamined on the basis of the principle of unsaturated soil mechanics with respect to highway engineering and construction. If the Enhanced Integrated Weather Model can be developed and implemented in the new M-EPDG endorsed by AASHTO, a similar procedure with a more flexible format also should be able to be developed and be tailored to fit in various local environment and climatic conditions.

The objective of the research is to provide state highway agencies with a guideline that includes procedures to develop a local modulus- or stiffness-based construction specifications to be utilized in the compaction of earth and unbound base materials. The procedures should be based on an extensive study of the engineering properties of various material types under different environmental and climatic conditions based on the principle of unsaturated soil mechanics. The study also should evaluate and compare various in-situ testing devices available for moduli at the national level. The study should seek the participation of state highway agencies and use states from different regions as examples to demonstrate the feasibility of the recommended guideline. The research team should include members from academia, industry, and state highway agencies. The study will require, but will not be limited to, perform literature review, solicit and select states for participation; evaluate the Enhanced Integrated weather model used in the new M-EPDG or other models to predict the long term variation of field modulus or stiffness and select the best model; collect additional lab and field data to calibrate/validate the selected model; run the model to analyze and generate charts and diagrams for the various combinations of material types, engineering properties, and environmental and climatic conditions for individual state highway agencies to use as a reference; implement the developed protocol in the participating state highway agencies; and write a final report to document the research effort and final results.

Improving the construction qualities of earth and unbound base materials in highway construction and linking the construction with pavement design procedure will have a fundamental impact on highway engineering in the United States. The guideline developed in this study will help state highway agencies to develop their local modulus- or stiffness-based construction specifications for earth and unbound base materials through demonstration and technical guidance considering local materials, environment, and climatic conditions. The results of this study, if implemented properly, will greatly promote the improvement of both design and construction of pavement structures and a more cost-effective use of highway construction budget due to improvement in predicting pavement performance. The successful execution of this study will promote and expedite the implementation of intelligent compaction technology in highway construction so a better construction quality of highways can be achieved. It also will assist in the implementation of the new M-EPDG, in the sense that the results from construction quality control and assurance will be secured to meet the requirement of pavement structure design and the environmental and climatic impact on pavement performance will be better understood. Data accumulated from this study and its implementation also will lay the foundation for future improvement of the M-EPDG. Therefore, the potential for payoff from the achievement of project objectives is significant and cannot be overestimated.

Attachment 5

Oklahoma Transportation Center Research Project OTCREOS7.1-16
"Quantifying the Costs and Benefits of Pavement Retexturing
as a Pavement Preservation Tool"

Principle Investigator: Douglas D. Gransberg, PhD, PE, University of Oklahoma
Co-Principle Investigator: Musharraf Zaman, PhD, PE, University of Oklahoma
Doctoral Research Assistant: Caleb J. Riemer, EI, Oklahoma Department of Transportation
Masters Research Assistant: Dominique Pittinger, Broce Construction Co., Norman, Oklahoma.

Abstract: With the decline in the condition of the nation's transportation infrastructure, pavement preservation has become an essential component to every state Department of Transportation's (DOT) program. Oklahoma's annual construction budget is far less than many other states in the region and as a result, preserving the state's infrastructure is doubly important in this state. Currently, pavement maintenance/preservation research has been mostly limited to investigatory material science across the US, which while valuable, does not usually provide the technical and more importantly, financial information that pavement managers need to make informed decisions. In fact, the majority of the research is done by the commercial entities that manufacture and sell the various pavement preservation products, leaving DOTs with no choice but to experiment with different means and methods by trial and error. This proposed research builds on past research by conducting a long-term study of various methods to restore pavement skid resistance by retexturing the existing surface with either a surface treatment, chemical treatment, or a mechanical process and furnish the Oklahoma DOT with the technical engineering data for each treatment coupled with an economic analysis of the costs and benefits associated with each treatment. This will furnish ODOT pavement managers the required information to make rational engineering decisions based on physical and financial data for the use of potential pavement preservation tools, evaluated under the same conditions over the same period by an impartial investigator.

The essence of the project is a series of 17 asphalt and 3 concrete pavement test sections on State Highway 77H (Sooner Road/25K ADT) between Norman and Oklahoma City. Each test section is ¼ mile long and one lane wide. Details for each section are shown in the table on the next page. Surface friction and pavement macrotexture was measured on each test section before the treatments and on a monthly basis for at least two years after application. Thus, changes in both skid resistance and pavement macrotexture are being recorded over time, and each treatment's performance can then be compared to all other treatments in the same traffic, environment, and time period. The project's major deliverable will be a pavement surface texture maintenance guide that can be used by ODOT pavement managers to restore surface texture and skid resistance to various types of pavements throughout the state. This will constitute a surface retexturing "toolbox" that contains both the technical engineering information as well as the economic analysis of each treatment's efficacy. The plan is not to identify the "best" method but rather to quantify the benefits of all the treatments in a manner that then allows a pavement maintenance engineer to select the right pavement preservation "tool" for the specific issue that they need to address and satisfy the fundamental definition of pavement preservation: "put the right treatment, on the right road, at the right time."

The project features a partnership between Oklahoma Transportation Center, the University of Oklahoma, ODOT, and members of the pavement preservation industry from Oklahoma, Texas, Louisiana, and Mississippi. The project demonstrates the benefits of each pavement preservation materials, means and methods in a manner that will not only be of value to ODOT and other Oklahoma public agencies but also to the rest of the nation.

Test Section Details

Source	Total Amount	Project Contribution
ODOT* Phase 1	$172,567	Asphalt Test Section 1: Standard 3/8” chip seal Asphalt Test Section 2: Standard 5/8” chip seal Asphalt Test Section 3: Standard 5/8” chip seal with a fog seal Asphalt Test Section 4: Open Graded Friction Course Asphalt Test Section 5: Open Graded Friction Course w/fog seal Asphalt Test Section 6: Permeable friction course Asphalt Test Section 7: Microsurfacing Asphalt Test Section 13:Fog seal Asphalt Test Section 14: 1” Mill and Inlay
ODOT* Phase 2	$80,849	Asphalt Test Section 15: Single size chip seal
Blastrac, Inc. Edmond, OK	$28,320	Asphalt Test Section 8: Pavement retexturing using shotblasting Concrete Test Section 1: Pavement retexturing using shotblasting Concrete Test Section 3: Pavement retexturing using shotblasting treated with Nanolithium densifier
Skidabrader Inc., Ruston, LA	$10,200	Asphalt Test Section 9: Pavement retexturing using shotblasting Asphalt Test Section 10: Pavement shotblasting w/fog seal Concrete Test Section 2: Pavement retexturing using shotblasting
JLT Corp., Cushing, OK	$25,594	Asphalt Test Section 11: Pavement retexturing using a flat headed planing (milling) technique with conditioner Asphalt Test Section 12: Conditioner/rejuvenator with crack seal
Ergon Inc. Austin, TX	$23,500	HFRS-2P emulsion binder for chip seal test sections
Haskell Lemon OKC, OK	$14,960	Asphalt Test Section 16: Nova Chip.
Polycon Manf. Inc. Madison, MS	$7,500	Apshalt Test Section 17: E-Krete pavement surface stabilizer
Pathway Services, Inc. Tulsa, OK	$39,750	Quarterly testing with vehicle-mounted digital imaging device.
OU	$10,846	OU Cost share for GRA tuition
Total Match	**$414,086**	
OTC Phase 1	$195,502	
OTC Phase 2	$118,602	
Total Project	**$728,190**	
*Includes in-kind costs for traffic control, skid testing, engineer/technician time during testing, project signage, traffic counter, and Transtec 3-D imaging.		

Quantifying the Costs and Benefits of Pavement Retexturing as a Pavement Preservation Tool

The University of Oklahoma

PI: Doug Gransberg PhD, PE

Co-PI: Musharraf Zaman, PhD, PE

PhD RA: Caleb Riemer, EI (ODOT)

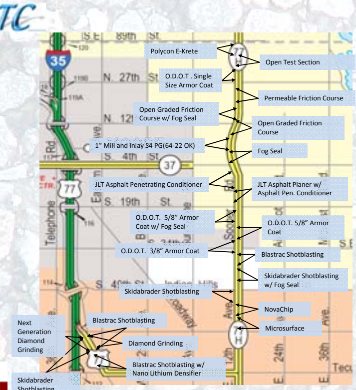
MS RA: Dominique Pittenger

Project Objective

- Leverage the Austroads pavement retexturing research to Oklahoma's benefit.
- Conduct a comparative field evaluation of common methods to restore skid resistance
  - Surface treatments
  - Mechanical methods
- Complete a life cycle cost analysis of each method in the study
- Produce a pavement preservation management tool for ODOT maintenance engineers to make rational decisions.

Project Objective

- The project will produce a document that allows a maintenance engineer to select the proper treatment for specific problem within a given context.
  - Example: skid lost in winter – use non-asphalt based retexturing – shotblasting
  - Example: Aggregate polished on long stretch of road – microsurfacing an option. If high traffic, need to minimize traffic control – shotblasting is an option
  - Example: Severe loss of skid on freeway exit ramp – Pick treatment that generates maximum skid increase.
- The objective is **NOT** to rank order the alternatives or identify the "best" one.
- It is to fill the engineer's toolbox with tested tools.



In total there are 23 test sections. Each section is 0.25 miles long.

Asphalt test sections are along state highway 77H and are all in the outside southbound lane.

Concrete test sections are located on U.S. highway 77 and encompass the full width of the driving surface. Plus an additional one in the southbound outside lane.

Test Section Sponsors

- Oklahoma Department of Transportation
- Blastrac, Inc. Edmond, OK
- Penhall Diamond Grinding, Anaheim, CA
- JLT Corp. Cushing, OK
- Ergon Emulsions and Materials, Austin, TX
- Skidabrader, Inc. Ruston, LA
- Polycon, Madison, MS
- Haskell Lemon & Hall Brothers, OKC, OK
- Pathway Services, Tulsa, OK

J.L.T. Corporation

Flat Headed Mill with Asphalt Penetrating Conditioner

Polycon

E-Krete

Blastrac

Asphalt & Concrete Shot Blasting

Skidabrader

Concrete & Asphalt Shot Blasting with Fog Seal

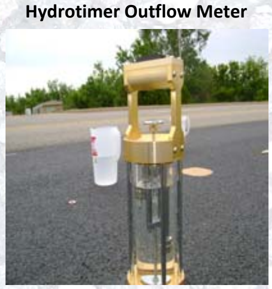
Testing Protocol

- Follow change in macrotexture & skid resistance over time.
- Macrotexture tests
  - ASTM STP 583 Outflow meter
  - TNZ T/3 Sand circle
  - TNZ P/17 Performance Spec for chip seal texture
  - RoboTex (Transtec)
  - High Speed Truck Mounted Laser (Pathway)
- Skid resistance measured by ODOT skid tester
  - Ribbed tire (40 mph) & Smooth tire (40 mph and 60 mph)
- Test conducted monthly for 3 years
- Plans to extend period to surface failure, but will reduce frequency to quarterly tests

Macrotexture Testing

TNZ T3 Sand Circle

Hydrotimer Outflow Meter



Economic Analysis

- Tracking change in engineering properties must be correlated with an analysis of the cost of supplying those properties
- Life cycle cost analysis for each tested alternative.
- Develop life cycle cost model for pavement texture over time.
- Use of Cost Index Number Theory to quantify "Bang for the Buck"

Pavement Retexturing

Questions????

For further information contact:

Doug Gransberg or Caleb Riemer

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405-325-6092 580-332-1526

Texas Pavement Preservation Center

Dr. Yetkin Yildirim, P.E.
Director
Texas Pavement Preservation Center
http://www.utexas.edu/research/tppc

Summary

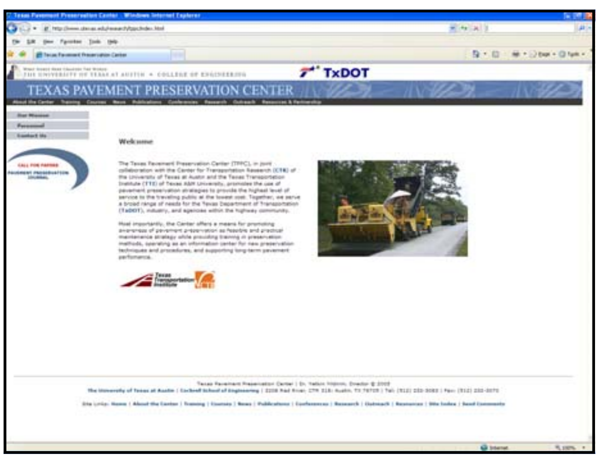
- TPPC Web Page
- Courses
- Newsletters
- Annual Pavement Preservation Seminars
- Research Work
- Publications

TPPC Personnel

- Center for Transportation Research (CTR)
  - Dr. Yetkin Yildirim, P.E. (Director)
  - Dr. Kenneth H. Stokoe, P.E.
- Texas Transportation Institute (TTI)
  - Joe W. Button, P.E.
  - Cindy Estakhri, P.E.

TPPC Web Page

- User Friendly Training Materials
  - People who learn by reading
    - » Newsletters
    - » Research Reports
  - People who learn by seeing
    - » On-line courses
    - » Presentations and Powerpoint files



TPPC Web Page

- Summary of the web page
  - Mission
  - Personnel
  - On-line Courses
  - Publications
  - Newsletters
  - Pavement Preservation Journal
  - Resources and Partnership
  - Contact Information

TPPC Courses

- On-Line Courses
  - www.utexas.edu/research/tppc/index.html
- District Level Courses
  - Have been provided in Ft Worth, Austin, Lubbock, San Angelo, Lufkin, Bryan

Online Courses

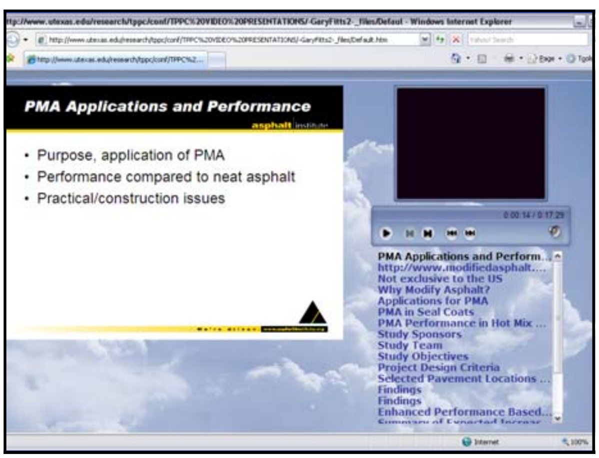
- Available at the TPPC Web Page
- Free and Open to the Public
- Highly ranked by the UT Austin CLEE
- 56 Lectures from Conferences and Seminars

Online Course Topics

- "Pavement Preservation"
- "Asphalt Chip Seal Binders"
- "Microsurfacing"
- "Binder Selection"
- "Performance-Based Maintenance Contracting"
- "Project Selection and Funding"
- "Developing and Testing Performance Measures"

Online Course Topics

- "The Prime Coat"
- "Chip Seal Asphalt Providers"
- "Successful Contracts and Reduced Claims"
- "Defining a Performance Measurement Methodology"
- "Contract Start-Up and Managing the Contract"
- "Seal Coat Preparation"
- "Seal Coat Aggregates"



Advantages

- Interactive Options
  - Pause and discuss, rewind, look ahead
- Community Viewing
  - Watch and participate with colleagues in your own environment
- User Friendly
  - Easy to operate directly off website
- Accessibility
  - Log on from any computer with internet access

Advantages

- No Additional Software Needed
  - Internet Explorer
- No Travel Expense
  - Save money on gas, lodging, and meeting rooms
- Great Resource
  - Return to conferences for reference at any time
- Convenient
  - Can be accessed by user at personal convenience

District Level Courses

- Seal Coat Courses
- MNT 702 Seal Coat Inspection and Applications
- MNT 703 Seal Coat Planning and Design
- Micro-surfacing Course
- 255 Certificates and CEU letters were provided

Training Classes

- *MNT 702 Seal Coat Inspection and Applications – 184*
  - Fort Worth District
    - » 35 Certificates
  - Austin District
    - » 35 Certificates
  - Lubbock District
    - » 35 Certificates
  - San Angelo
    - » 15 Certificates
  - Lufkin
    - » 22 Certificates
  - Bryan
    - » 42 Certificates
- *MNT 703 Seal Coat Planning and Design – 71*
  - Fort Worth District
    - » 18 Certificates
  - Austin District
    - » 6 Certificates
  - Lubbock District
    - » 10 Certificates
  - San Angelo
    - » 14 Certificates
  - Lufkin
    - » 4 Certificates
  - Bryan
    - » 19 Certificates

TPPC Newsletter

- Available on the TPPC Webpage
- PDF files
- Total of 12 issues have been edited and published
- Issue number 13 and Issue number 14 will be available in the Summer

Pavement Preservation Seminars

- Dates
  - Fall 2005, 2006 and 2007
- Place
  - Austin Convention Center
  - AGC and FP2
- Program
  - Technical Papers
  - Q&A sections
  - Expert Panels – TxDOT, Material Producers and Contractors

Research Work

- Crack Sealing
- Thin Asphalt Overlays
- Fog Seals
- OGFC – mix design, performance, winter maintenance
- Warm Mixes
- Seal Coats

Summary of Publications 2005-2009

- 22 Technical Papers
- 9 Research Reports
- Pavement Preservation Journal: 7 Articles
- 11 National and International Presentations

Attachment 6

TRANSPORTATION SYSTEM PRESERVATION TSP•2 TECHNICAL SERVICES PROGRAM • WWW.TSP2.ORG

AASHTO TSP•2 PROGRAM
FACT SHEET

Introduction

This fact sheet has been prepared to provide the AASHTO Standing Committee on Highways (SCOH) and its Subcommittees on Maintenance; Bridges and Structures; Materials; and Design, Joint Technical Committee on Pavements with information on recent changes to the Transportation System Preservation Technical Services Program (TSP•2).

Established in 2006, the TSP•2 Program was created to serve as a clearinghouse for transportation system preservation technical information and expertise and to facilitate communications and exchange between transportation practitioners. As part of this effort, the TSP•2 incorporates a preservation Help Desk, on-line website, regional preservation partnership groups, as well as specific technical support and presentations when requested to accomplish its mission. An oversight panel comprised of representatives from the subcommittees identified above, with the addition of the Standing Committee on Planning, Asset Management Subcommittee, and the AASHTO regions provides direction for the program.

From its inception, implementation of the TSP•2 was intended to be achieved through a series of phased steps. Phase I included the development of the TSP•2 website and Help Desk with a primary focus on pavements through a voluntary contribution of $6,000 per state. Phase II was approved by AASHTO in 2007 and encompassed the establishment and operation of Regional Pavement Preservation Partnerships, bringing the total annual contribution for the program to $9,500. Thus far, about 35 states have been participating in the program. At its December 2007 meeting, the TSP•2 Oversight Panel voted to recommend that the AASHTO SCOH move forward with the implementation of Phase III (Bridge Preservation) based on the level of interest expressed by state bridge practitioners. At its annual meeting last October in Hartford, Connecticut and upon the recommendation of SCOH, the AASHTO Board of Directors voted in favor of expanding the program to include Bridge Preservation. Once implemented, Phase III will incorporate the Bridge Preservation component into a program website, regional partnerships, and Help Desk similar to the Pavement Program. This Fact Sheet outlines the goals, implementation plan, benefits, and costs associated with this final Phase of the TSP•2 Program.

Goals of Phase III

The goals of Phase III as envisioned by the Oversight Panel are to incorporate a comprehensive Bridge Preservation component into the TSP•2 Program. This means that Bridge Preservation will be given full and equal representation within the website, Help Desk, and Partnership Program components. More specifically:

- The TSP•2 website will require a redesign to clearly divide bridge and pavement preservation resources into distinct areas. Layout of the website will be optimized to allow respective bridge and pavement practitioners ready access to information that is relevant to them.
- Regional Bridge Preservation Partnership groups will be created or identified to run independently and in parallel to groups already established for pavements.
- Bridge engineering and maintenance specialists will be available to provide bridge preservation practitioners with relevant and timely technical assistance through a Help Desk.

AASHTO THE VOICE OF TRANSPORTATION

- The organizational template that has been developed for pavements should be utilized for bridges to maintain operational efficiency.

- The expanded program will focus on the promotion of bridge preservation and facilitate the exchange of related technical information. While bridge management, design, rehabilitation and construction interests are encouraged to participate, the TSP•2 will not concentrate on these areas of expertise.

Costs

AASHTO has approved an annual voluntary contribution of $20,000 per member agency to operate the complete TSP•2 program. This contribution amount was based upon over two years of experience on the part of AASHTO and the National Center for Pavement Preservation (NCPP) in running similar partnership and Help Desk efforts for pavements. Partnership travel and labor budgets have been realistically determined based upon these experiences, and accurately represent the rapidly escalating costs of travel and lodging.

This $20,000 will be inclusive of all services (i.e. bridge and pavement Help Desks, bridge and pavement partnerships, bridge and pavement on-line technical resources, as well as specific technical support and presentations when requested).

These funds would be placed in an AASHTO voluntary fund account and will be controlled by the TSP•2 Oversight Panel. All expenditures are cost reimbursable and subject to audit. The $20,000 is a wise investment in the preservation program and its techniques. Addressing both pavement and bridge interests through one AASHTO system preservation program is efficient and avoids confusion for states with respect to participation and billing.

In addition, FHWA has given approval to a request from AASHTO to waive the state match, thereby allowing the use of 100% State Planning and Research (SP&R) funds to be utilized for the voluntary contribution. (See attached FHWA letter)

Next Steps

A letter dated January 15, 2009 was sent to each of the state CEO's from AASHTO Executive Director John Horsley (See attached Horsley letter) along with an invoice encouraging them to contribute to the TSP•2 as well as two other technical services programs which were recently approved. AASHTO is seeking to generate a sufficient level of funding in the program account before starting the Bridge Preservation initiative. AASHTO expects to advertise a Request for Proposals for the TSP•2 Bridge Preservation Technical Services Program contract in the near future with the selection being made by the Oversight Panel. AASHTO is also seeking a chief engineer replacement for its recently retired Oversight Panel Chair.

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AASHTO
THE VOICE OF TRANSPORTATION

Allen Biehler, President
Secretary, Pennsylvania Department of Transportation

John Horsley, Executive Director

444 North Capitol Street NW, Suite 249, Washington, DC 20001
(202) 624-5800 Fax: (202) 624-5806 • www.transportation.org

January 15, 2009

Mr. Joe McInnes
Transportation Director
Alabama Department of Transportation
P.O. Box 303050
Montgomery, AL 36130 3050

Dear Mr. McInnes:

During this year's annual meeting held in Hartford, Connecticut, October 16-20, 2008, the Standing Committee on Highways and the Board of Directors approved the following Technical Service Programs as summarized below:

- **Expansion of the Transportation System Preservation** known as TSP-2 which will support the research, technical, and program needs of the member states in their development and implementation of their own preservation programs. TSP-2 had proven to be a successful program for pavement preservation and with the expansion of the program; bridges will be incorporated into the program. With reduced highway funds, DOTs will be able to preserve not only their pavements but their bridges as well. Voluntary contribution from each participating state: $20,000 annually. (Please see attachment A for a list of state contributions)

- **Advance Equipment Technology** – The AASHTO Subcommittee on Maintenance will oversee the Equipment Focus Group, which will keep current data pertaining to new types of equipment along with all advancing innovation and technology directly related to equipment fleet. This information will be disseminated throughout the state DOTs to reduce costs of maintenance operations. A voluntary assessment of $3,000 per participating DOT will fund the initial cost of the program as well as ongoing activities.

- **Safe, Reliable and Secure Transportation Operations** will share the increasing technologies and best practices used in the areas of safety, reliability, and security to improve the effectiveness and efficiency of the state DOTs. The program will expand the support to three AASHTO committees: Special Committee on Transportation Security; Standing Committee on Highways' Subcommittee on Systems Operations and Management; and Standing Committee on Highway Traffic Safety's Subcommittee on Safety Management. The program will also be used for leverage to receive a federal contract and cooperative agreement funding. The Board of Directors has approved a voluntary assessment of $10,000 per member department annually to fund the establishment of the program along with all ongoing activities.

With the development/expansion of each of the listed Technical Programs, AASHTO kindly requests your participation in funding these important programs. Individual invoices are enclosed seeking FY 2009 voluntary contributions from your member Department to support the development of each of these critical programs. Similar invoices will be rendered each fiscal year for the continued support of these technical programs and, as always, AASHTO appreciates your continued support and thanks you for your interest in these programs.

Sincerely,

John Horsley
Executive Director

Enclosure

U.S. Department
of Transportation
Federal Highway Administration

Research, Development, and Technology

Turner-Fairbank Highway
Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

OCT 15 2008

Refer to: HRTM-02

Mr. John Horsley
Executive Director, American Association of
State Highway and Transportation Officials
Suite 249
444 N Capitol Street, NW.
Washington, DC 20001

Dear Mr. Horsley:

This is in response to the October 10 e-mail from Ken Kobetsky of the American Association of State Highway and Transportation Officials (AASHTO), describing a proposal to create programs to serve as a clearinghouse for transportation system preservation technical information and expertise, and to facilitate communications and exchange between transportation practitioners; the AASHTO Subcommittee on Maintenance, Standing Committee on Highways and the AASHTO Board of Directors has endorsed the proposal. The estimated project cost is $600,000 and AASHTO is seeking a minimum of 30 member states to participate in funding the project at $20,000 each. AASHTO has asked FHWA for approval of a waiver of the non-Federal match for the use of State Planning and Research (SP&R) funds for the AASHTO-sponsored project proposal.

We have determined that the proposed study meets the criteria for use of SP&R funds used for Research and Development studies without non-Federal funding match. States are authorized to proceed with the study using 100 percent State Planning and Research (SP&R) funding.

We understand that you will finance this project by pooling funds from the States, but the project will not be included in the FHWA's Transportation Pooled Fund Program and no number will be assigned. Participating States will provide their funds directly to AASHTO for the project.

You may contact Lisa Williams (lisa.a.williams@dot.gov, 202-493-3375) if you have any questions about this waiver.

Please let us know if we can be of any further assistance in conducting this study.

Sincerely yours,

Michael F. Trentacoste
Acting Associate Administrator

cc:
Ken Kobetsky, AASHTO

Informational Briefing on Pavement Preservation

Our nation's highways represent the largest single public infrastructure investment in the history of mankind. We must practice good stewardship of this investment or future generations face horrendous consequences. Fortunately a system of pavement preservation exists to safeguard our highway assets.

Our every dollar invested in pavement preservation significant benefits accrue for the taxpayer and the highway user. These include:

- **Employment**. The Federal Aid Highway Program supported approximately 38,000 full-time jobs nationally per $1 billion of investment. Pavement preservation projects are uncomplicated and ready to implement and they are labor intensive and put Americans back to work immediately. On average, these projects support about 25% more jobs compared with new construction projects.

- **Cost savings for the taxpayer.** Expenditures on preservation will extend pavement lives and defer the need for costly rehabilitation / reconstruction. Each $1.00 spent on preservation will save approximately $6.00 to $10.00 or more in rehabilitation / reconstruction.

- **Impact on the Motorist.** Pavement preservation can reduce traffic delays by using faster application of preservation techniques and reduce user costs by maintaining pavement networks in better condition.

- **Environmental sustainability.** Pavement preservation is socially responsible and eco-friendly. It uses less of the earth's resources than highway rehabilitation and reconstruction programs and advances a "green" environment.

Adopting a pavement preservation approach will enable our nation to increase and sustain our highway system and national security. It will accelerate job creation, stimulate the economy, and safeguard our environment for future generations.

The attached informational packet contains:

1. A white paper entitled, "A National Initiative for Infrastructure Pavement Preservation"
2. A guide entitled, "A Quick Check of Your Highway Network Health"
3. A booklet entitled, "At the Crossroads: Preserving our Highway Investment"

Pavement Preservation CTF
Draft April 16, 2009

Congress Should Embrace and Support Pavement Preservation

- Pavement Preservation: "A program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extend pavement life, improve safety and meet motorist expectations."[1] This concept is characterized as using "The right treatment for the right road at the right time."[2]

- Congress should specifically recognize Pavement Preservation as a tool for state governments to stretch available federal funding, and establish a dedicated source of funding for Pavement Preservation separate from 3R.

- Congress needs to act because states and localities have been reluctant to perform inexpensive proactive maintenance absent clear direction from Congress that proactive preservation activities are eligible for use of federal highway funds.

- Congress should establish a separate funding category for Pavement Preservation under provisions for Federal Highways, to encourage proactive measures to preserve the investment in our transportation network, and extend the reach and benefit of federal highway funding.

- Funding for proactive Preservation strategies must remain firewalled from funds allocated for Rehabilitation (3R) and Reconstruction, in order to allow states to cost-effectively surface-treat and maintain existing roads without triggering federal requirements that would render preservation cost-prohibitive.

- Benefits of Pavement Preservation:

    - ROI - Every $1.00 spent on Pavement Preservation will save from $6.00 to $10.00 or more in rehabilitation / reconstruction costs.

    - On average, pavement preservation projects support approximately 25% more jobs on a dollar for dollar basis compared with new construction or rehabilitation projects.

    - Pavement preservation is socially responsible and eco-friendly. It utilizes up to 80% less of the earth's non-renewable resources than highway rehabilitation and reconstruction programs.

    - Pavement preservation improves efficiency and safety, reducing motorist delays by using techniques that can be completed faster with less traffic disruptions.

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[1] Federal Highway Administration Memorandum, "ACTION Pavement Preservation Definitions," September 12, 2005. http://www.fhwa.dot.gov/pavement/preservation/091205.cfm

[2] National Center for Pavement Preservation at Michigan State University, http://www.pavementpreservation.org/

Pavement Preservation CTF
Draft April 16, 2009

The Pavement Preservation Congressional Task Force is a collaborative group of independent contractors and supplier members of the Asphalt Emulsion Manufacturers Association (AEMA), the Asphalt Recycling & Reclaiming Association (ARRA), and the International Slurry Surfacing Association (ISSA). Our member firms are innovators in the areas of Pavement Preservation and have decades of demonstrated experience in achieving cost savings and environmental benefits through the use of our products and techniques.

<u>Pavement Preservation Principles for SAFETEA-LU Reauthorization</u>

<u>OVERVIEW</u>

Adopting the Pavement Preservation approach of using the *right* treatment for the *right* road at the *right* time will yield significant environmental and economic benefits. By explicitly recognizing the benefits of Pavement Preservation and by directing recipients of federal highway funds to give full consideration to utilizing Pavement Preservation techniques, Congress can in the upcoming SAFETEA-LU reauthorization encourage recipients of federal transportation dollars to treat more lane miles, at a lower cost, and put more people to work, compared with implementing traditional construction methods exclusively. Pavement Preservation practices will, over the long term, extend the useful life of roadways, reduce work zone congestion, be more environmentally responsible, and stretch funding further across the road network.

1. What is Pavement Preservation?

Pavement Preservation is defined by the Federal Highway Administration (FHWA) as "a program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extend pavement life, improve safety and meet motorist expectations."[1] As defined by FHWA, Pavement Preservation practices include both preventive and corrective non-structural actions to provide cost-effective alternatives which address the highway infrastructure needs. As defined by the National Center for Pavement Preservation, the definition advises employing "the *right* treatment for the *right* road, at the *right* time."[2]

2. Pavement Preservation: A New Way to Address Infrastructure Needs

There is merit in revisiting the mindset of previous generations and applying their principles to how we approach infrastructure construction, maintenance and repair. As a result of the recent trying economic times, people are increasingly returning to the traditional mindset of repairing and preserving what they already have in an effort to stretch tight economic resources. That same approach would serve policymakers well when it comes to addressing spending needs for infrastructure.

Traditionally, road preservation has not been considered a priority by highway users, primarily because roadway deterioration occurs almost imperceptibly over time – until it is too late. While most people practice preventive maintenance in an effort to preserve the value of such major assets as their homes and automobiles, motorists tend to think of maintaining the infrastructure only after a tragic road or bridge failure occurs, producing disruptions to service and shocked reactions of disbelief.

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[1] Federal Highway Administration Memorandum, "ACTION Pavement Preservation Definitions," September 12, 2005. http://www.fhwa.dot.gov/pavement/preservation/091205.cfm

[2] National Center for Pavement Preservation at Michigan State University, http://www.pavementpreservation.org/

Most states and localities have historically dedicated maintenance resources to the most deteriorated roads, usually devoting costly repairs only to pavements that have suffered distress. Prioritizing maintenance in a "worst first" approach does nothing to extend the capital lives of the Nation's investment. Under a Pavement Preservation approach, however, roadways could be proactively preserved and roadway life-spans extended well beyond design-life expectations – leading to lower capital construction costs over the long term and more efficient use of the federal funds.

3. The Importance of Improving and Maintaining Highway Infrastructure

The United States system of roads and highways -- valued at over $1.75 trillion – has been steadily deteriorating. The American Society of Civil Engineers (ASCE) 2009 Report Card for America's Infrastructure assigns a grade of D- for the nation's roads.[3] ASCE estimates the total cost of repairs and needed upgrades at $2.2 trillion – an increase of $600 million over the 2005 cost.[4]

The nation's economic vitality depends on its highways to move people, goods, and services 24 hours a day, 7 days a week. In fact, a healthy and well-connected highway system has been the primary infrastructure investment that has driven our strong national economy.

The average formally planned life--known as the "design life"--of U.S. roads is approximately 20 years, concrete pavements is about 40 years and asphalt pavements about 15 years. In practice, the effective service life of roadways can be extended even longer with effective pavement preservation programs.

4. Economic and Environmental Benefits to Including Pavement Preservation Efforts in the SAFETEA-LU Reauthorization.

Adopting a Pavement Preservation approach will enable America to sustain its highway system and to increase our national security. Pavement Preservation will accelerate job creation, stimulate economic growth, and safeguard our environment for future generations.

For every dollar invested in pavement preservation, significant benefits accrue to the taxpayer and the highway user. These benefits include:

- **Stretching Tax Dollars.** Pavement Preservation expenditures will extend pavement life and defer the need for costly rehabilitation / reconstruction. Every $1.00 spent on preservation will save from $6.00 to $10.00 or more in rehabilitation / reconstruction costs. Pavement Preservation forestalls the

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[3] http://www.asce.org/reportcard/2009/
[4] Ibid.

ultimately more expensive, time consuming and disruptive need for reconstruction.

- **Jobs. Jobs. Jobs.** On average, pavement preservation projects support approximately 25% more jobs on a dollar for dollar basis compared with new construction or rehabilitation projects. Pavement preservation projects are uncomplicated, ready to implement, labor-intensive, and can put Americans back to work immediately.

- **Lower Long-term Cost.** Pavement Preservation is a more economical and prudent approach to maintaining the entire road network, based on a Life-Cycle Cost Analysis of the Net Present Value per Square Yard or Lane Mile.

- **Environmental Sustainability.** Pavement preservation is socially responsible and eco-friendly. It utilizes 80%[5] fewer of the earth's non-renewable resources than highway rehabilitation and reconstruction programs. Pavement preservation advances a "green" environment by minimizing transportation's environmental impact. The cumulative environmental expense of not only demolishing, hauling, and disposing existing pavement, but also of manufacturing new pavement is substantial. A Pavement Preservation approach diminishes the demand on natural resources and reduces the production of greenhouse gases by emphasizing a selective philosophy of using the "right treatment for the right road at the right time."

- **Reducing the Impact on Motorists.** Pavement preservation reduces traffic delays by using techniques that can be completed faster with less traffic disruptions. It also offers reduced user costs by maintaining entire pavement networks in better overall condition. Reducing the time that motorists spend in traffic delays due to roadway construction reduces overall emissions from motor vehicles. Pavement Preservation improves the surface characteristics of the roadway, thereby improving user safety.

A shift from the prevalence of "worst-first" road maintenance practices to a pavement preservation mindset will take a very special commitment and a real understanding of the vast potential benefits to be gained for our nation's economy and environment. The expenditure of limited maintenance funds on carefully chosen and timed preservation projects will yield reconstruction savings substantially in excess of the preservation expenses.[6]

However, federal funding eligibility in the SAFETEA-LU legislation has been interpreted to preclude using federal funds for pavement preservation practices.

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[5] Comparison of emulsion and slurry surface treatment v. asphalt milling and disposal and new paving with asphalt overlay.

[6] Larry Galehouse, James S Moulthrop, R. Gary Hicks, "Principles of Pavement Preservation – Definitions, Benefits, Issues, and Barriers," *TR News*, September-October 2003, page 8, figure 2.

The current law addresses the existing highway needs only through resurfacing, rehabilitation and restoration (3R). The 3R program merely addresses costly "major work." The SAFETEA-LU reauthorization legislation should dedicate a share of funding to Pavement Preservation. This allows highway agencies to extend the highway service life in a more cost effective and environmentally responsible manner.

# # # # #

Who We Are

The Pavement Preservation Task Force is a collaborative effort of independent contractors and supplier members of the Asphalt Emulsion Manufacturers Association (AEMA), the Asphalt Recycling & Reclaiming Association (ARRA), and the International Slurry Surfacing Association (ISSA). We are committed to working together to conduct nationwide education and outreach on Pavement Preservation activities and to ensure that SAFETEA-LU reauthorization legislation facilitates a Pavement Preservation approach to help recipients effectively maximize federal highway funding**.**

# # # # #

Transportation System Preservation

Newer is better. Out with the old, and in with the new. Innovation, a term derived from new, is equated with progress. Progress in our fast-paced society has evolved into the "new is best" mentality, somewhat driven by clever marketing gurus, but also preached by our predecessors who wanted a better life for themselves and for future generations.

But not everything new is indeed more favorable, as in the case of an entire network of highways and bridges. When expansion is the focus for that complex, interconnected system—so as to improve safety, mobility, or efficiency—new construction is indeed a viable option. But the costly days of building and rebuilding for the sake of satisfying some naïve quest for novelty cannot be the primary focus for our Nation's continued investment in the future.

Our depression-era predecessors also forwarded the notion of sustainability, of preserving and maintaining our belongings in good condition to reap lasting benefits. Sustainable investment in America's transportation infrastructure must be dependent upon the long term strategy of Transportation System Preservation.

A well maintained, sound highway system is a critical component for a healthy economy. Our country's economic vitality depends on its highways to move people, goods, and services, 24 hours a day, 7 days a week. To serve its purpose, our highway system must be in good physical condition and provide a high degree of connectivity and efficiency. The economy of individual states also depends on highways.

In 1997, about 75 percent of our nation's products (by value) were shipped by truck; 42 percent were shipped out of the originating state, and at least 15 states shipped at least 80 percent of their products by highway.[1] The nation's highway system is important to both our economic vitality and national defense. After the September 11, 2001 attack on New York's World Trade Center, all modes of moving goods and services in this country suffered short-term disruptions,[2] with the single exception of our Nation's highways.

Americans take for granted our nation's nearly 4 million miles of paved public roads, highways and bridges. The public expects and deserves a safe, smooth ride. Generation after generation of Americans have contributed toward expanding this complicated transportation network, at the same time believing a new roadway equals permanence. In reality, the United States highway system—valued at over $1.75 trillion—has been steadily deteriorating, forcing a growing need for additional investment in this valuable infrastructure asset. Allocating resources to build and rebuild roadways and bridges is not the solution, however, unless we are also serious about preserving and maintaining this fundamental investment. Transportation System Preservation must be incorporated into the current infrastructure investment strategy to sustain this vital resource for future generations.

Roadway Predictions

The challenge of maintaining and improving the condition and performance of our highways and bridges is compounded by the following future projections for 2020:

- Traffic congestion will increase significantly.

- Total highway freight traffic will increase by 65 percent.[3]
- Sustaining the average condition and performance of our roads and bridges is estimated to require more than $75 billion in capital outlay each year—18 percent more than in 2000.[4]
- Maintenance ($36.33 billion,) administration ($32.88 billion,) and debt retirement ($8.01 billion) require an additional $77 billion.[5]
- By 2020 improving the system effectively will cost about $107 billion—65 percent more than the capital funding in 2000.[6]
- Roads will deteriorate and road roughness[7] will increase by more than 25 percent, and the amount of pavement with acceptable ride quality will decrease by more than 12 percent.[8]
- Escalating road work will increasingly impede our ability to move freely on our highway system. Even now, "the majority of our nation's population travels through a work zone at least once every day. It is also estimated that over 80 percent of Federal-aid funds go into products that the public sees and experiences in work zones."[9]

Shift from "New is Better" and "Worst First" Toward the Preservation Mindset

The result of Transportation System Preservation practices made today may not become apparent for a period of time, perhaps as long as 5, 10, or even 20 years. Highways and bridges are long-term investments that deteriorate slowly, while our fast-paced society has a marked preference for actions that yield short-term benefits. Many people regularly practice preventive maintenance to preserve the value of their personal assets such as homes, furnaces, and automobiles. But when it comes to public infrastructure such as roads and bridges, users tend to be blind to the concept of preventive maintenance, believing our highways will last forever. Only tragic failures, such as the Minneapolis bridge collapse, or the disruption of daily commuting due to lengthy reconstruction or rehabilitation delays gets immediate attention.

Both road builders and road users generally associate construction or reconstruction with the idea of progress, whereas innovative preservation techniques contribute to a cleaner, greener, more progressive advancement of system sustainability. Paradoxically, general interest in system preservation appears only when roads and bridges have reached a state of dire decay. After many years of use without preservation, roads are often in such serious condition that they can no longer be maintained cost-effectively and must be reconstructed. Rebuilding is costly in terms of scarce capital, user inconvenience, and detrimental environmental impact.

For every dollar invested in pavement preservation significant economic, environmental, and safety benefits accrue for the taxpayer and the highway user. These benefits include:

- **Employment**. The Federal Aid Highway Program supported approximately 38,000 full-time jobs nationally per $1 billion of investment. Preservation projects are uncomplicated and ready to implement; they are labor intensive and put Americans back to work immediately. On average, preservation projects support about 25% more jobs compared on a dollar per dollar basis with new construction or rehabilitation projects.
- **Cost savings for the taxpayer.** Preservation expenditures will extend pavement lives and defer the need for costly rehabilitation / reconstruction. Each $1.00 spent on preservation will save approximately $6.00 to $10.00 or more in rehabilitation / reconstruction. Preservation also defers or forestalls the ultimately more expensive, time consuming and traffic disruptive reconstruction.
- **Impact on the motorist.** Transportation System Preservation can reduce traffic delays by utilizing preservation strategies that can be completed faster, with less traffic disruption, at reduced user costs by maintaining the network in better overall condition. Pavement Preservation improves the surface characteristics of the roadway, thereby improving user

safety. Slower deterioration of roadway surfaces provides motorists with substantially safer driving conditions.

- **Environmental sustainability.** Preservation is socially responsible, eco-friendly, and is consistent with reducing greenhouse gas emissions and slowing the pace of climate change. The environmental effect of minimal and efficient preservation practice is dwarfed by the colossal environmental costs of manufacturing nonrenewable construction materials, conducting construction site operations, and increasing traffic emissions due to lengthy construction delays. The preservation model utilizes considerably less of the earth's non-renewable resources than highway rehabilitation and reconstruction programs and advances a "green" environment by minimizing transportation's environmental impact.
- **Budget friendly.** Preservation techniques, combined with reconstruction and rehabilitation strategies, will allow the owner agencies to level out their annual budget needs by balancing expenditures across network needs. The concept of Remaining Service Life allows owner agencies to balance the level of service, adding value to precious funds.

Adopting a strategy of preservation will enable our Nation to increase and sustain our transportation system and national security. Transportation System Preservation will accelerate job creation, stimulate economic growth, and safeguard our environment for future generations.

Planning for the Future

By adopting a Transportation System Preservation model that preserves and proactively corrects minor deficiencies early, our bridge and roadway lives can be substantially extended at comparatively lower cost. An economical way of maintaining infrastructure is to lengthen the time between initial design and reconstruction. Whereas some strategies entail design changes, other methods advocate preservation techniques, thereby slowing the rate of deterioration. Clearly, some resources will always be required for building and reconstruction; but if we can significantly extend the time between construction and reconstruction through innovative preservation practices, we will have more resources to devote to other network needs. The expenditure of limited funds on carefully chosen and timed preservation will inevitably yield reconstruction savings substantially in excess of preservation expenses.[10]

Primary benefits of Transportation System Preservation include the potential for reducing traffic delays by using faster application of preservation techniques and for reducing user costs by maintaining transportation networks in better condition. Because preservation is less costly than construction and can be applied more quickly, more people can be employed through a preservation program. Fewer nonrenewable resources are purchased so that precious monetary resources can be director toward personnel, rather than product.

We can arrest and reverse the deterioration of our transportation system assets in two principal ways:

1. We can continue to allocate ever-increasing amounts of scarce tax dollars and try to spend our way out of the problem. By spending enough resources, we can stay ahead of deterioration while improving system quality. To be sustainable, this method requires continuously high replacement expenditures; or

2. We can utilize our precious resources in a more cost-effective way—by changing from a build-and-reconstruct process to a build, "maintain and preserve," and reconstruct process, thus extending substantially the lifespan of our transportation system. Such a preservation approach will enable us to increase and sustain our highway system quality within the constraints of our valuable, but limited resources.

A healthy and well-connected highway and bridge system is the primary transportation infrastructure and sustainable investment that drives our national economy. The Nation's precious roadways are dependent upon innovative, green, and fiscally responsible preservation techniques that extend the lifespan of our valuable roads and bridges well beyond yesterday's antiquated lifecycle expectations. As the caretakers of this vital infrastructure, we must focus on conserving our investment by making wise asset management decisions. Let us force back the creeping erosion of our roadways that drives costly reconstruction and ecologically devastating waste. We must thwart the outdated mentality of planned obsolescence and embrace preservation strategies that will cost-effectively maintain our nation's transportation infrastructure.

Our Nation's highways and bridges represent the largest single public infrastructure investment in the history of mankind. We must practice good stewardship of our transportation assets and this investment or future generations will face horrendous consequences. Transportation System Preservation serves as the cornerstone for sound asset management which is needed to safeguard our transportation investments and to provide the service demanded by the Nation's taxpayers.

Endnotes

[1] "The Road to Prosperity: The Importance of the Federal Highway Program to the Economic Prosperity of Individual States," prepared by William Buechner, Ph.D., Director of Economics and Research, American Road and Transportation Builders Association, September 1997.

[2] Disruptions lasted for several days.

[3] Government Accounting Office (GAO) Report GAO-03-744R, "Trends in Federal and State Capital Investment in Highways," 18 June 2003.

[4] Year 2000 dollars

[5] http://www.fhwa.dot.gov/policy/ohim/hs04/hf.htm (discht.xls spreadsheet)

[6] http://www.fhwa.dot.gov/policy/2002cpr/Ch7b.htm

[7] Based on the International Roughness Index scale

[8] U.S. Department of Transportation, "2002 Conditions and Performance Report," Executive Summary and Exhibit 9-1.

[9] Federal Highway Administration (FHWA): http://www.fhwa.dot.gov/reports/bestprac.pdf "Meeting the Customer's Needs for Mobility and Safety During Construction and Maintenance Operations." Office of Program Quality Coordination, September 1998, page 47.

[10] Larry Galehouse, James S Moulthrop, R. Gary Hicks, "Principles of Pavement Preservation – Definitions, Benefits, Issues, and Barriers," *TR News*, September-October 2003, page 8, figure 2.

A National Initiative for Infrastructure Pavement Preservation

Developed by:
The National Center for Pavement Preservation at Michigan State University

Many of us in the United States take our country's nearly 4 million miles of paved public roads and highways for granted. The public expects and deserves a safe, smooth ride. Our country's economic vitality depends on its highways to move people, goods, and services 24 hours a day, 7 days a week. In fact, a healthy and well-connected highway system has been the primary infrastructure and national investment that has driven our strong national economy. But the United States highway system—valued at over $1.75 trillion-- is steadily deteriorating. Allocating resources to rebuild roadways and bridges and to add capacity faster is not the solution unless we are serious about preserving our current investment and plan for the necessary investment to sustain both existing and expanded transportation infrastructure.

Through out history, the success of entire civilizations--ancient Rome for example—was based in part on their road systems. In our culture, road construction is often heralded with naming ceremonies, ribbon cuttings and other forms of recognition honoring promoters and contractors for the transportation improvement and the future service it brings to the highway user and the economy. Most of us think of a road as something that will last forever. In the public view a new road equals permanence.

Although the average formally planned life--known as the "design life"--of U.S. roads is approximately 20 years, concrete pavements is about 40 years and asphalt pavements about 15 years. In practice, their effective service life can be extended even longer with effective pavement preservation programs.

Roadway Predictions

Transportation officials are very concerned about the challenge of maintaining and improving the condition and performance of our roads and highways in view of the following future projections for 2020:

- Traffic congestion will increase significantly.
- Total highway freight traffic will increase 65 percent.[1]
- Sustaining the average condition and performance of our roads and bridges is estimated to require more than $75 billion in capital outlay each year--18 percent more than the 2000.[2] Maintenance ($36.33 billion,) administration ($32.88 billion,) and debt retirement ($8.01 billion) require an additional $77 billion.[3]
- By 2020 improving the system effectively will cost about $107 billion--65 percent more than the capital funding in 2000.[4]
- Maintaining the status quo annual highway investment will increase user, travel time, and vehicle operating costs (see Table 1). Roads will deteriorate and road roughness[5] will increase by more than 25 percent, and the amount of pavement with acceptable ride quality will decrease by more than 12 percent.[6]

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[1] Government Accounting Office (GAO) Report GAO-03-744R, "Trends in Federal and State Capital Investment in Highways," 18 June 2003.

[2] Year 2000 dollars

[3] http://www.fhwa.dot.gov/policy/ohim/hs04/hf.htm (discht.xls spreadsheet)

[4] http://www.fhwa.dot.gov/policy/2002cpr/Ch7b.htm

[5] Based on the International Roughness Index scale

[6] U.S. Department of Transportation, "2002 Conditions and Performance Report," Executive Summary and Exhibit 9-1.

Furthermore, escalating road work will increasingly impede our ability to move freely on our highway system. Even now, "the majority of our nation's population travels through a work zone at least once every day. It is also estimated that over 80 percent of Federal-aid funds go into products that the public sees and experiences in work zones."[7]

The Value Added by Our Roadways

A good highway system is a critical component of a healthy economy. To serve its purpose, our highway system must be in good physical condition and provide a high degree of connectivity and efficiency. Our Nation's highway system is important to both our economic vitality and national defense. After the September 11, 2001 attack on New York's World Trade Center, all modes of moving goods and services in this country suffered short-term disruptions[8] except highways.

The economy of individual states also depends on highways. In fact, the economic prosperity of most states depends more on out-of-state highways than in-state highways. In 1997, about 75 percent of our nation's products (by value) were shipped by truck; 42 percent were shipped out of the originating state, and at least 15 states shipped at least 80 percent of their products by highway.[9]

Investments in our highways have a significant effect on productivity:[10]

- **Employment**. The Federal Aid Highway Program supported approximately 42,100 full-time jobs per $1 billion of investment in 1996.

- **Production cost savings.** Industries realize as much as 24 cents in production cost savings for each dollar invested in highways (1950-89 figures).

- **Productivity growth.** Our highway network contributes an average of one quarter of the nation's annual productivity growth (1950-89 figures).

- **Social rate of return.** The net social rate of return from our nation's highway network equals or exceeds the 10 percent rate of return on private capital and long-term interest rates (1980-89 figures).[11]

Our Road Repair Mindsets: "New is Better" and "Worst First"

Many people regularly practice preventive maintenance to preserve the value of assets such as their homes, furnaces, and automobiles. But when it comes to public infrastructures such as roads and bridges, users tend to be blind to the concept. Road agencies are increasingly coming to view their highway systems as assets worthy of preservation in the same sense as equipment or buildings. These road agencies correctly perceive the negative consequences, such as higher costs, that result from poor maintenance policies and practices. While road construction skills are often plentiful, preservation skills

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7 Federal Highway Administration (FHWA): http://www.fhwa.dot.gov/reports/bestprac.pdf "Meeting the Customer's Needs for Mobility and Safety During Construction and Maintenance Operations." Office of Program Quality Coordination, September 1998, page 47.

8 Disruptions lasted for several days.

9 "The Road to Prosperity: The Importance of the Federal Highway Program to the Economic Prosperity of Individual States," prepared by William Buechner, Ph.D., Director of Economics and Research, American Road and Transportation Builders Association, September 1997.

10 FHWA: http://www.fhwa.dot.gov/policy/empl.htm

11 Net Social Rate of Return represents net benefits, excluding depreciation.

are in short supply, and some managers find it easier to build new roads than keep existing roads in good operating condition.

Many road users, keenly aware of their payment of fuel taxes at the pump, view roads as service the government should provide. Road preservation does not appear to be a pressing issue to the highway user because road deterioration occurs over time and is almost imperceptible to the average person. Only tragic failures or disruption of service gets their immediate attention. Finally, both road builders and road users generally associate construction or reconstruction with the idea of progress. Paradoxically, general interest in road preservation appears only when roads have deteriorated to such a degree that serious traffic problems arise.

After many years of use, roads are often in such bad condition that they can no longer be maintained cost-effectively. Rebuilding them is costly in terms of use of scarce capital and user inconvenience. States, counties, and municipalities have considerable experience in designing and executing new road construction and re-construction, but many are not adequately prepared for preserving the existing road infrastructure to meet the needs of users and the economy in general.

Planning for the Future

By adopting a preservation model that proactively corrects minor road deficiencies early, our roadway lives can be substantially extended at comparatively lower cost.

Public road agencies must have access to the financial resources necessary to adequately preserve and generally manage road networks. In the traditional "business as usual" model, agencies assume that sufficient funding will always be available to rehabilitate and reconstruct roads when the need arises. From time to time, they use funds allocated for maintenance to perform repairs that become necessary as a result of traffic and climatic factors. Usually these repairs are made after pavements have suffered distress; they do not extend the capital lives of such pavements. Meanwhile, population increases and economic activity continue to grow faster than the resources needed to sustain the system using existing appropriations. Consequently, Congress has had to appropriate ever-increasing funds to maintain the nation's percentage of good roads.

A more economical way of maintaining good roads is to lengthen the time between when they are built and when they need to be reconstructed. Many strategies can be applied. Some involve design changes while others involve slowing down their rates of deterioration. Clearly, some resources will always need to be devoted to building and reconstructing our roads, but if we can expand the time between construction and reconstruction, we will have more resources to devote to other network needs. The expenditure of limited maintenance funds on carefully chosen and timed preservation will eventually yield reconstruction savings substantially in excess of the preservation expenses.[12]

Political Lobbying to Prevent Pavement Preservation Support

In some cases, a few industry interests will rely on political lobbying to prevent funding and support of pavement preservation. Although they may offer technical reasons, industry groups are motivated by the effect preservation could have on the capital investments on which that industry has grown accustomed. If an agency adopts the new approach to managing their highway investments, using more preservation technology to extend the performance of the capital expenditure and level out the need for capital growth, these industry interests fear loss of market share.

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[12] Larry Galehouse, James S Moulthrop, R. Gary Hicks, "Principles of Pavement Preservation – Definitions, Benefits, Issues, and Barriers," *TR News*, September-October 2003, page 8, figure 2.

Primary benefits of pavement preservation include the potential for reducing traffic delays by using faster application of preservation techniques and for reducing user costs by maintaining pavement networks in better condition. Because pavement preservation cost less and are applied more quickly, more people can be employed through a preservation program. Although widely acclaimed, these benefits still need stronger documentation in national research studies.

Next Steps

Roads and highways are long-term investments that deteriorate slowly. The results of changes made today may not become apparent for a period of time, perhaps as long as 10 or 20 years. People have a marked preference for actions and changes that will yield short-term benefits. Therefore, changes of the type required for pavement preservation will take a very special commitment and a real understanding of the vast potential benefits to be gained. Funding must be available by:

- Ensuring that language is included in re-authorization bill to fund pavement preservation and corresponding education and research efforts
- Requiring the concept of pavement preservation to be implemented by state highway agencies and local agencies
- Designating a university transportation center as resources to advance pavement preservation.

Summary

We can arrest and reverse the deterioration of our highway system in two principal ways:

1. We can continue to allocate ever-increasing amounts of scarce tax dollars and try to spend our way out of the problem. By spending enough resources, we can stay ahead of deterioration while improving system quality. To be sustainable, this method requires continuously high expenditures. These are funds we do not have today at the Federal, State or local levels, or
2. We can use our resources in a smarter way: by changing from a build-and- reconstruct process to a build, preserve, and reconstruct process. Such a preservation approach will enable us to increase and sustain our highway system quality within the limits of our present resources.

Pavement preservation is the clear choice for sustaining a strong national economy.

Preservation of Pavement
Texas Underseal

Tammy B. Sims, P. E.
Special Projects Engineer
Maintenance Division
Texas Department of Transportation

1

Texas Underseal

The Practice:
- unsound surface mix
- maintain grade
- reflection cracking
- surface delamination

2

Texas Underseal

Objective:
- remove top surface layer
- residual reflection
- create better bond

3

Texas Underseal

Purposes:
- moisture barrier
- reduces slippage
- consistent application
- temporary surface

4

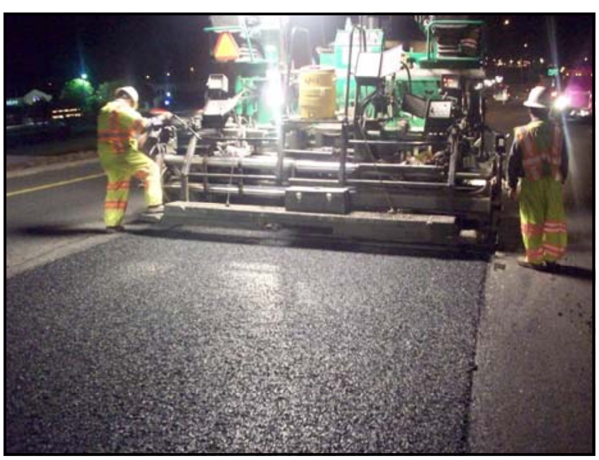
Texas Underseal

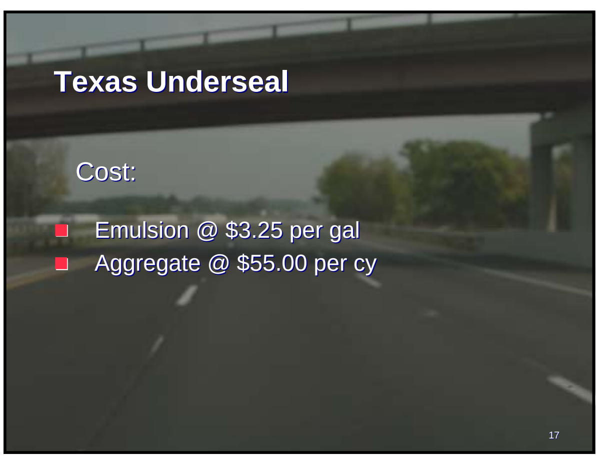
Materials:

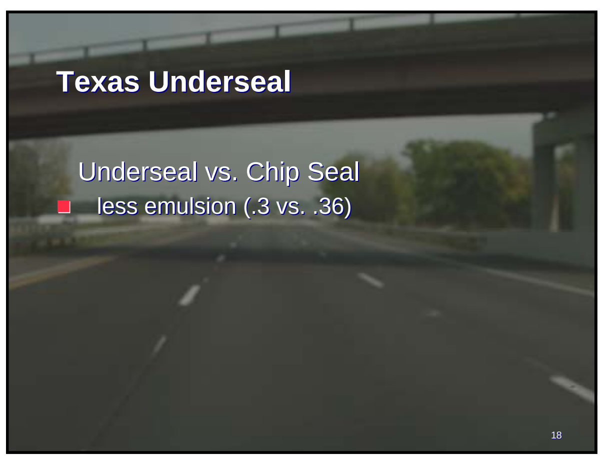
- Emulsion – CHFRS-2P (0.3 gal/ SY)
- Aggregate – Grade 4 (1 CY/140SY)

5


6135
INGERSOLL-RAND
PT-125R




Texas Underseal
Cost:
Emulsion @ $3.25 per gal
Aggregate @ $55.00 per cy
17


Texas Underseal
Underseal vs. Chip Seal
less emulsion (.3 vs. .36)
18

Advantages

- Longer pavement life
- Better performance
- Improved pavement condition
- Higher public satisfaction
- Cost savings
- Increased safety

19

End of the Road

tsims@dot.state.tx.us

National Transportation Product Evaluation Program
NTPEP -- AASHTO's Win-Win Solution for Government and Industry

Crack Sealers for HMA Pavements

Chairman: Michael Rafalowski, FHWA (Washington, DC)
Vice Chairman: David Iverson, Minnesota DOT
AASHTO Staff Liason: Henry Lacinak

Table of Contents

NTPEP Reports:

NTPEP Report 16002.1 - One-Year Field and Laboratory Evaluation of Hot Mix Asphalt Crack Sealing Materials (2005 Minnesota Test Deck)

NTPEP Report 16002.2 - Two-Year (Twenty-Nine Months) Report of Field and Laboratory Evaluation of Hot Mix Asphalt Crack Sealing Materials (2005 Minnesota US 169 Test Deck) - Part A and Part B

AASHTO is pleased to announce launch of a coordinated evaluation of Crack Sealers for HMA pavements under the auspices of its National Transportation Product Evaluation Program (NTPEP).

Crack Sealers are commonly used to protect and seal Hot Mix Asphalt (HMA) pavement cracks from moisture and other deleteriou material. Under this NTPEP evaluation program the sealers are field tested for three years and laboratory tested according to a variety of ASTM specified test methods.

The NTPEP Technical Committee responsible for authoring the Project Work Plan convenes yearly at the NTPEP National (Annual) Meeting. During this working meeting, the Project Work Plan is discussed, changes made, and shortly thereafter balloted for adoption. Improvements are immediately implemented in the next available testing cycle. When compared to other standards development exercises, the NTPEP process is agile, effective and efficient. -NTPEP-

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Section 1: Crack Sealers Technical Committee

The NTPEP Technical Committee is the working group charged with developing and maintaining the Project Work Plan for testing of a particular product, material or device. The Technical Committee convenes at least annually in conjunction with the NTPEP Annual Meeting. Technical Committee membership consists of AASHTO members; and up to two Industry members assigned by the NTPEP Chairman.

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Section 2: Project Work Plan

NTPEP tests products according to a Project Work Plan, which describes the laboratory and/or field test protocols used to conduct the evaluation. The Project Work Plan is a consensus-based document, and includes peer review and input from industry experts. Each Project Work Plan is adopted after receiving at least two-thirds affirmative support from 52 AASHTO member states. The Project Work Plan is the basis for host states to conduct their testing and evaluation. When implemented by state DOTs, their own state standard specifications may supersede the NTPEP Project Work Plan. Industry is advised to be aware of state-level requirements, which may supersede basic NTPEP testing.

Project Work Plan for Field and Laboratory Evaluation of HMA Crack Sealing Materials

General Notes and Sample Requirements - Pavement Crack Sealant Materials for HMA Pavements

Draft: Project Work Plan for Field and Laboratory Evaluation of Hot Mix Asphalt Crack Sealing Materials (Rev Oct 08)

Section 3: Submit a Product (2009 Test Cycle)

Application Package Coming Soon

Section 4: Technical Committee Minutes

2005 Annual Meeting Mystic, CT

2005 Fall Conference Call

2006 Fall Conference Call

2007 Annual Meeting Boise, ID

2008 Annual Meeting Madison, WI

2008 Fall Conference Call

Section 5: Industry Resources

- AASHTO Subcommittee on Maintenance
- National Center for Pavement Preservation
- Foundation for Pavement Preservation

You can **submit a resource link** yourself.

Vägverket

Performance of flow mixing technology (KGO-III method)

By Mats Wendel
Swedish Road Administration

Vägverket

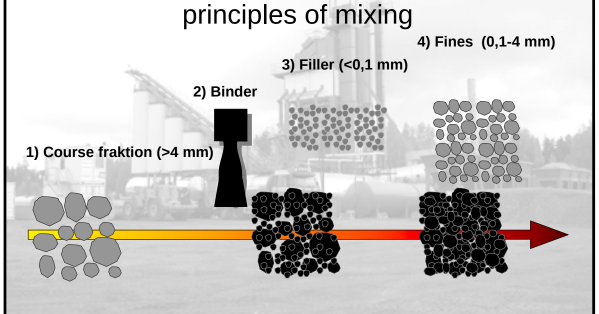
What is flow mixing method?

- A technology to produce asphalt and concrete mixes with same or better quality without use of extra additives.
- Through a controlled way of production reduces valuable sources of raw material, saves energy, and gives long-lasting products.

* KGO-III method is patented by Karl Gunnar Ohlson

Vägverket

Flow mixing method*, principles of mixing



Mats Wendel, Swedish Road Administration

* According to KGO-III method

Normal mixing procedure

Normal mixing procedure

In the video, material is used to illustrate the method. All repeating has same amounts of material (water, filler (<0,075 mm) and sand (0,1-2 mm)

Flow mixing procedure

Flow mixing procedure

In the video, material is used to illustrate the method. All repeating has same amounts of material (water, filler (<0,075 mm) and sand (0,1-2 mm)

Comparison: Normal vs Flow mixing procedure

Comparison

Under same conditions!

Positive effects due to observations and testing

- Production temperature can (and must) be lowered – compactability not effected. (No smoke!)
- In Swedish top layers, the binder content (%AC), decreased by 0,5%-units (equals to approx 8% less bitumen).
- The pavement has a characteristic glow, that indicates thick film coating.
- Better homogeneity! Asphalt mix is more sticky and have less segregation.

Concrete

- Conventional concrete start to crack at 40% of break load whereas the KGOIIc concrete goes to break without developing of cracks.
- The gain is that the concrete do not crack at changes in the temperature.
- Due to toughness in the concrete, the breaking strength is equal or higher than the conventional at 10% reduction in cement.

Ocular comparison (the effect)

Normal AG22 Flow mixed AG22

Environmental effects

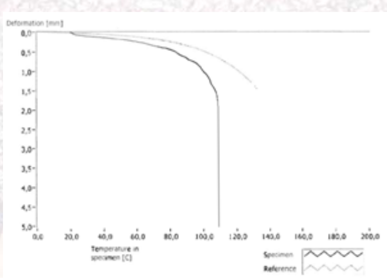
Better homogeneity, lower temperature = less/no smoke

- >500.000 ton produced so far in Sweden

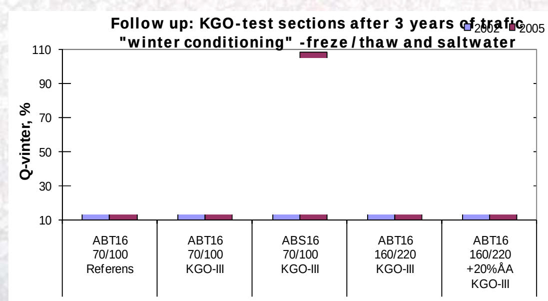
+ Lower temperature
+ Less bitumen
= better quality and better for our environment!

New test method

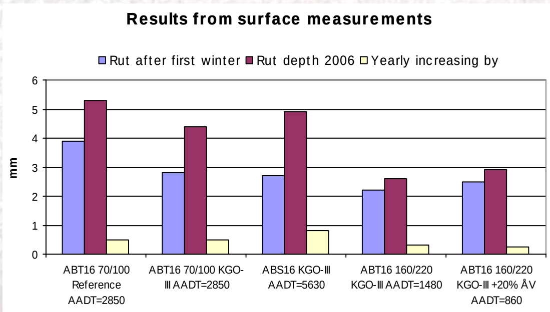
- There is a test method to verify that production works.
- Both Marshall and drilled cores can be tested.
- This is the key to ensure quality.



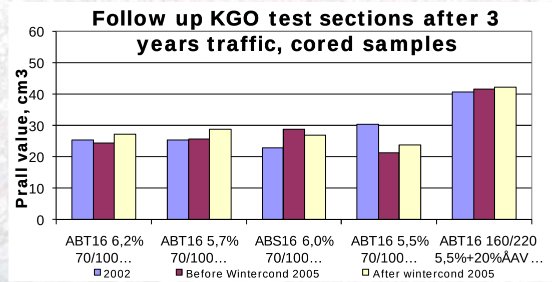
The TOD test provide a measure of viscous deformation in the specimens.
The TODc, concrete is carried out on beams. The test provide a measure of the deformation in the specimen at break.



Rut depth development



Abrehesion test (Prall)



Questions?

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Pictures from production of test sections, observed 5 years.

Other roads manufactured with the KGO-III method

From 2008 is KGO-III method allowed on “State roads” according to SRA spec.

Swedish research take a lead in the technology for production of asphalt pavings and cement concrete

The new production methods are named KGOIII and KGOIIc respectively. The main concept of this announcement concerns the changes in the production of asphalt pavings by the KGOIII method.

The compacted KGOIII hot asphalt mix is a suspension. The suspension asphalt pavings is known by road authority and producers of asphalt mix in Sweden for its remarkable increased quality when compared with other known hot mixes, thus providing the paving with 30% increased durability and a pavement free of common defects during the service time.

Description of the research

The research is described in a thesis which provides a full technical description for production and for the teaching of the new generation of technologists in physics of the production of suspension asphalt paving and cement concrete.

The result of mixing methods

By the known mixing method, the fractions of mineral aggregate particles are unsorted when added and mixed with the bituminous liquid in the mixer of the asphalt plant. This state of admixing unsorted aggregate fractions with the liquid causes the defects in the asphalt mix that reduces the durability of the paving.

By the KGOIII method of mixing the mineral aggregate is sorted in three separate fractions for addmixing with the bituminous liquid in the mixer of the asphalt plant, a coarse, a fine and a filler fraction, each fraction having its own specific function towards the liquid for developing the suspension paving. The hydrostatic function of the suspension provide the aphalt pavings with optimum durability. Also the fact that the paved mix is without defects increases the service life of paving.

Environmental profits

a) A relevant technology with an easy control system shows when the **extended properties of durability** of the asphalt pavings and cement concrete are obtained.

b) The investment value of the product is increased by 30% by an **increased durability**.

c) The **traffic safety** on asphalt pavings **is improved** by an improved surface structure and without the tendency of deforming by viscous flow.

d) The temperature of the production is 130-140°C, which **reduces energy consumption** by 1 litre/ton of oil for heating of the aggregate.
The conventional production is made at the temperature of 160-180°C.

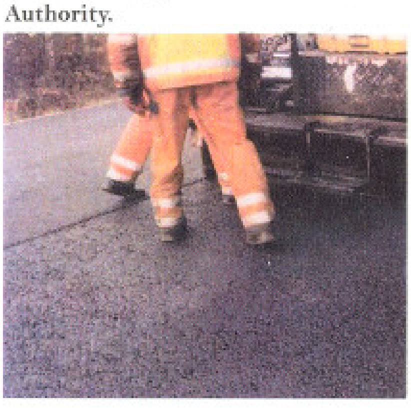
e) At the low mixing temperature there is hardly any smoke or smell from the mix during the production and at the paving site.
Therefore the development **meets with the new requirements of the environmental laws** that govern industrial production of asphalt pavings.

f) The **bitumen content** in KGOIII suspension mix for surface pavings is 5 litre/ton **lower** than for the conventional mix.

Environmental and economical savings

The consumption of asphalt mixes for surface pavings in Sweden is around 6 million tons/year. When producing asphalt mixes with the KGOIII method, several major environmental and economical savings are obtained in form of less bitumen, less heating oil and the less rate of consumption of asphalt paving over time from extended durability. Se below.

The method is utilized, and savings are recorded, by the Swedish Road Authority.



Scope of effects on economy and environment

Economy

The following evaluation is based on the approximate quantity of 6 million tons of asphalt mix produced for surface pavings each year in Sweden.
The high quality of the KGOIII mix increases the durability of pavings by 30% compared to pavings by the conventional method. In other words, matching the durability of 6 million tons of KGOIII asphalt mix require 7.8 million tons of conventional mix. The increased durability saves 1.8 million tons of mix/year. The value of 1.8 million tons of paved mix is around SEK 900 millions. After three years, almost one year of production is saved.

Environment

The raw material of 1 800 000 tons of asphalt mix is made up of:

Bitumen	108 000 tons
Heating oil	1 800 tons
Aggregate material	1 692 000 tons

To this comes the production, transportation of mix to paving site and the paving of mix.

* The 30% of increased durability refers to traffic by studded tyres. In Germany for example, where studs are not allowed, the durability of the paving will double.

Researcher:
Karl Gunnar Ohlson
Fields of research:
Production of asphalt pavings and cement concrete
Strategy of research:
Theoretical science applied on Particle technology and rheology of suspensions.
Extent of research:
A paradigm shift in technology
Patent:
Mixing methods and mechanical devices on the production units
Technical adviser:
Thorbjörn Lindquist, Technovation
Tel +46 451 31950

Long-term performance of flow mixing technique

Manufacturing by KGO-III method

L. VIMAN, M. WENDEL, S. F. SAID

The flow mixing technology has been used in surface layer in many roads in the middle part of Sweden in the last years. The long-term performance evaluation of three roads with 5 test sections has been examined from time of compaction during the last 3 years of traffic.

KGO-III technique, consists of 3 steps:

- Mixing of binder and coarse aggregate fractions.
- Addition of filler fractions in a controlled way
- Addition of fine fractions (0.063 – 4 mm) comes last.

The KGO-III mixing procedure, using 0.5 % less binder content and decreased mixing temperature by 30 °C.

This gives environmental and economical benefits!

Field result

- ✓ Noise
- ✓ Friction
- ✓ Texture (MPD)
- ✓ Rutting

Laboratory result

- ✓ Void content
- ✓ Wear resistance
- ✓ Stiffness
- ✓ Durability

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Field observations

Notice the stiff edge of the layer and the typical shiny surface for KGO-III

Picture of the surface of new and 3 year old KGO-III asphalt concrete

Conclusions

- Good results for KGO-III mixes according to noise, friction, texture and rut depth.
- Laboratory tests indicate good durability against severe winter condition for KGO-III mixes.
- The KGO-III mixes also proved to have good wear resistance.

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New Standards of Asphalt Pavings and Cement Concrete

The research on improving the standards of asphalt pavings and cement concrete was initiated by a phenomenon of advanced asphalt mix observed in trial mixing. (1970).

The phenomenon was eventually explained as an asphalt mix in state of suspension.

1) The difficulty to produce such a mix is that the bituminous liquid of the mix must be continuous through the skeleton of the aggregate. When mixing the liquid with the aggregate of different fractions the liquid is, with the conventional method of mixing, divided in small volumes as coatings on the surface of the coarse, as well as the fine aggregate particles. When reaching this state of mix the remaining problem is to fuse to 100 percent, the small volumes of liquid coatings into a continuous liquid through the skeleton of the aggregate particles. However, by the conventional methods with the division of the liquids, in small volumes on fine fraction aggregates, the liquid fail to fuse. This is observed by the mix tendency to segregate.

2) The solution to the problem was to introduce a method where the surface coating of the fine fraction was avoided. This was obtained by a technique of mixing whereby the dust- and fine fraction particles became dispersed in the liquid coatings on the surface of the coarse fraction (figure 1). With the division of the liquid avoided and with the liquid volume expended by the volume of the dispersed particles, the liquid had the power to fuse to 100 percent, which is required for the mix to become a suspension.

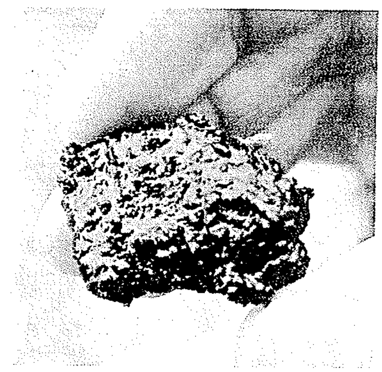
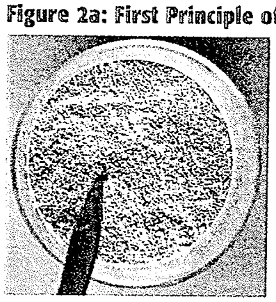


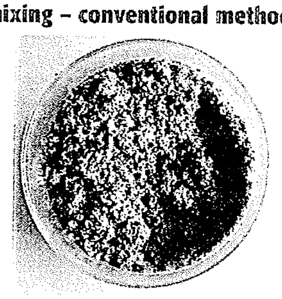
Figure 1

The hypotheses of the research on production of suspension asphalt pavings and cement concrete was deducted from an experimental reproduction of the phenomenon. The result is shown by figure 2a), the liquid of mix is unfused, figure 2b) the liquid of mix is fused into a suspension.

Figure 2a: First Principle of mixing – conventional method

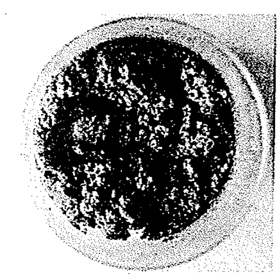


Fines and filler only



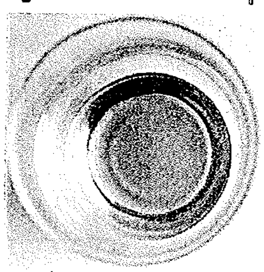
Fines, filler and 40% of the liquid.

Fines, filler and 75% of the liquid.



Fines, filler and 100% of the liquid.

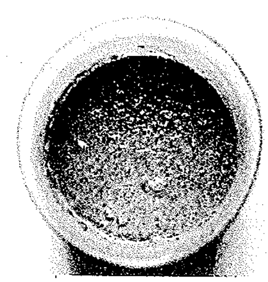
Figure 2b: Second Principle of mixing – KGO III method



Liquid only

The liquid and 40% of the fines and filler.

The liquid and 75% of the fines and filler.



The liquid and 100% of the fines and filler.

Attachment 10

NCHRP FY2010 Project approved
>> >> Congratulations Nick,
>> >>
>> >> The three problem statements that you authored for FY2010
regarding
>> >> pavement have been chosen for funding in the FY2010 program.
>> Problem
>> >> Statements D-16, Performance Related Specifications (PRS) for
>> Pavement
>> >> Preservation Treatments, and F-06, Acceptance Criteria for
pavement
>> >> Surface
>> >> Treatments, were combined. Problem Statement C-15,
Integrating
>> Pavement
>> >> Preservation into the Design Process will be researched on its
own.
>> >>
>> >> You are hereby nominated to be a participant on the panels
that
>> will
>> be
>> >> organized to carry out this research. Your name will be
submitted
>> to
>> >> NCHRP. Please send me your resume by April 10, 2009. Your
>> information
>> >> will be forwarded to the national NCHRP offices.

The Field Guide

Attachment 13

The Polymer Modified Emulsion (PME) Study

- Sponsored by:
  - **Central Federal Highway Lands Division Federal Highway Administration**
    - Mike Voth, James Sorenson
- Investigators:
  - **National Center for Pavement Preservation**
    - Larry Galehouse, John Johnston
    - GHK, Inc.
      - Gayle King, Helen King
    - Industry volunteers
      - PRI, BASF, Paragon, SemMaterials, UW, many others

The Problem

- ✓ **Experience: polymer modification results in better short- and long-term performance**
- ✓ **No definitive guide**
  - For selecting, specifying & using polymer emulsions
- ✓ ***Areas of interest***
  - Use of PMEs vs. conventional emulsions
  - Optimal % polymer
  - Use on non-roadway applications (parking lots, trails, bike paths)

The Project

- Literature review & knowledge gathering sessions
  - **Industry, academic, federal & local government agencies**
  - **On-line user/ producer survey**
  - **Presentations & input: AEMA/ARRA/ISSA, TRB, ETGs, AASHTO**
- Draft performance spec
- Field trials
- Report & Field Guide

The Field Guide

- ✓ **Target Audience**
  - FLH project development & maintenance engineers
  - APWA, NACE, LTAP, and AASHTO
- ✓ ***Goal***
  - Needed info on PME
    - **Definition**
    - **Selection**
    - **Specification**
    - **Use**

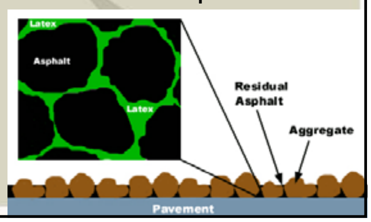
The Field Guide

TABLE OF CONTENTS

Findings – What Are PMEs?

- ✓ Water based, emulsified asphalt & polymer
- ✓ Performance depends on:
  - Type of polymer
  - Compatibility of polymer & asphalt

PME Chip Seal



Findings – When & Why PME?

- ✓ PME recommended for all emulsion applications
  - Improve performance
    - Stiffer at high temperatures (bleeding, rutting)
    - Less brittle at low temperatures (shelling, cracking)
    - More adhesive (early chip loss, raveling, delamination)
    - Less susceptible to moisture damage
    - Less susceptible to oxidative aging (raveling, cracking)
    - More elastic – fatigue resistant (chip loss, cracking)

Findings – When & Why PME?

- ✓ Increase service life
- ✓ Prevent early failures
- ✓ Cost differentials vs. no polymer
  - Mn/DOT: total project cost ≈7% higher
  - 2008 study field projects: 4-11% higher
  - Right Treatment - Right Road - Right Time

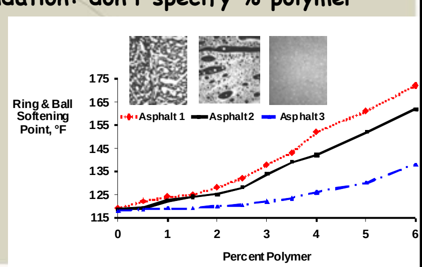
www.pavementpreservation.org/toolbox/guidelines.html

Findings -When & Why PME?

- ✓ Chip seals
  - Early & long term stone retention
  - Quicker traffic return
  - Fewer broken windshields
  - Reduced flushing & bleeding
  - Greater tolerance for quantities & aggregate embedment factor
  - Increased durability
    - Better performance on high volume roads

Findings – How to Specify PME

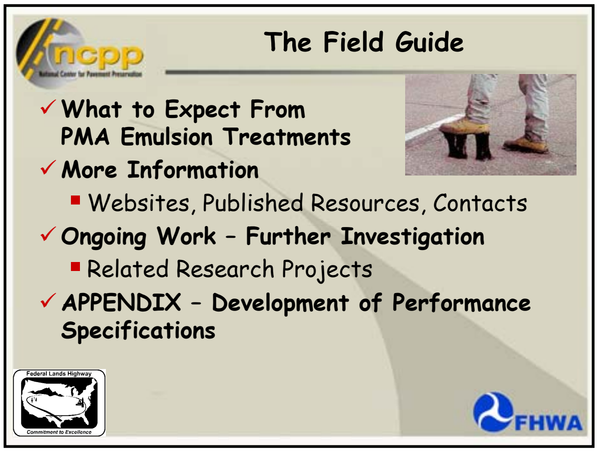
- ✓ Current specs don't correlate with performance
- ✓ Recommendation: don't specify % polymer



Findings – How to Specify PME

Recommendations:

- Update ASTM D-244 with performance-related tests
  - Low temp residue recovery method
  - Adapted Superpave binder tools preferred
  - Aging procedure for residues
  - Revise emulsion viscosity method
    - ❑ Field viscosity test
- Develop Approved Supplier Certification program
  - ❑ To prevent shipping & construction delays


The Field Guide
What to Expect From PMA Emulsion Treatments
More Information
Websites, Published Resources, Contacts
Ongoing Work – Further Investigation
Related Research Projects
APPENDIX – Development of Performance Specifications
FHWA

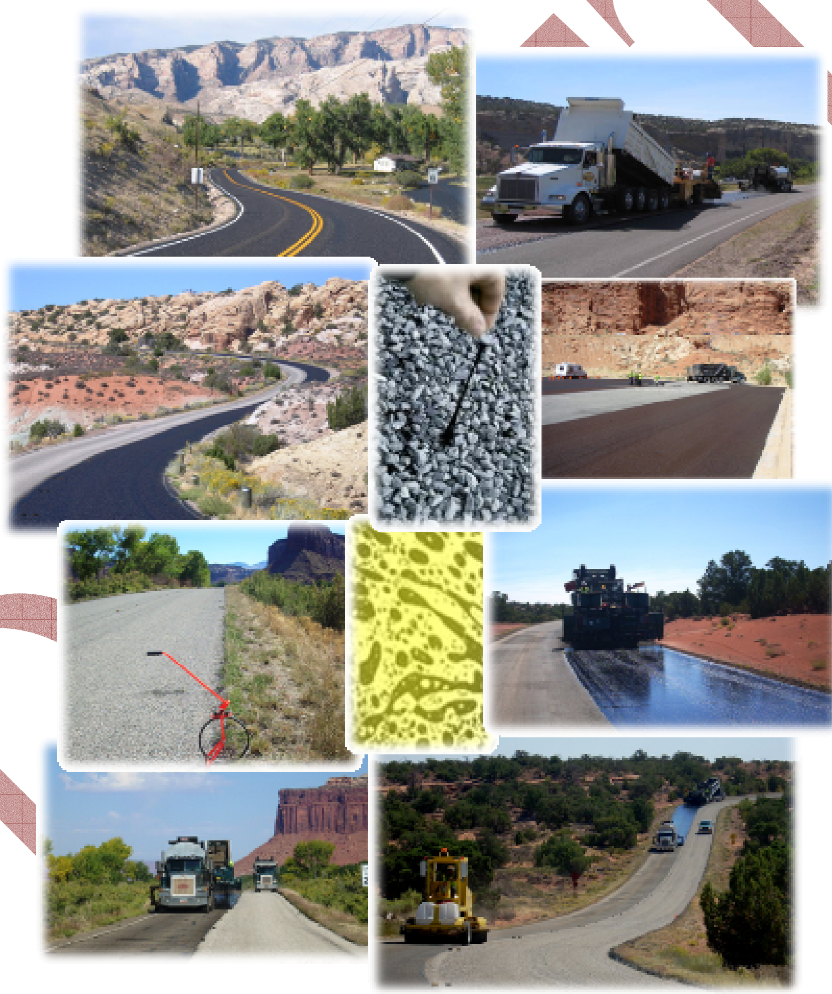

The Field Guide
To be available soon from FHWA
www.pavementpreservation.org
FHWA

Attachment 13A

POLYMER MODIFIED ASPHALT EMULSIONS

Composition, Uses and Specifications for Surface Treatments

Publication No. FHWA-CFL/TD-08-00x **April 2009**



U.S. Department of Transportation

Federal Highway Administration

Federal Lands Highway

Commitment to Excellence

FOREWORD

While guidance, manuals, and specifications exist for conventional asphalt emulsions, the Federal Land Highway (FLH) Division of FHWA desired further guidance for using high performance modifiers (polymers) for asphalt emulsions, including dosing rates, methods of modification, and benefits derived from their use. Polymer modification increases initial costs, but treatment performance is reportedly improved, and life cycle costs will be lowered with appropriate use.

This project was initiated to:

- Conduct a comprehensive literature review of available polymers, emulsion modification methods, comparison with non-modified asphalt emulsions, applicability for low and high volume roadways as well as to parking lots, hiking trails, and biking trails, and applicability in the differing environmental zones.
- Develop polymer modified emulsion use recommendations in various surface treatments, including chip seals, slurry seals, and cape seals based upon traffic and climate conditions.
- Develop a draft testing plan to verify the developed recommendations.
- Suggest areas where further investigation is needed.
- Develop a guide with best-practice recommendations and specifications.

During this investigation, researchers determined that performance-related specifications should greatly improve the predictability and performance of the polymer modified asphalt emulsion surface treatments. A series of field trials on Federal Lands Highway projects was conducted, with field samples tested in laboratories according to the draft testing plan developed with input from government, academic and material supplier experts. The laboratory results are to be compared to evaluations of the field performance, with expectation that this analysis will continue over the lifespan of the surface treatments. Best practices were used in the specifications for the construction of the experimental field projects, and a draft performance-related asphalt emulsion materials specification was developed. This report includes the knowledge collected over the course of the project, including the test plan for the field trials, the draft performance-related specification and laboratory results. It is intended to aid Federal, State and local agencies using polymer modified asphalt emulsion pavement preservation treatments.

F. David Zanetell, P.E., Director of Project Delivery

James Sorenson, Construction & System Preservation

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Composition, Uses and Specifications for Surface Treatments

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7. Author(s)

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Gayle King, GHK, Inc.

8. Performing Organization Report No.

9. Performing Organization Name and Address

National Center for Pavement Preservation
2857 Jolly Road
Okemos, MI 48864

10. Work Unit No.

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Lakewood, CO 80228

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Final Report, September 2005 - March 2009

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15. Supplementary Notes
COTR: Michael Voth, FHWA CFLHD; Advisory Panel Members: Gary Evans FHWA WFLHD, Jim Sorenson FHWA HIAM, Joe Gregory FHWA HIAM, Larry Galehouse NCPP, Koichi Takamura BASF, Chris Lubbers BASF. This project was funded under the FHWA Federal Lands Highway Technology Deployment Initiatives and Partnership Program (TDIPP), Functional Discipline Leader (FDL) initiative, and the Office of Infrastructure.

16. Abstract

This document gives a comprehensive review of best practices and recommendations for the use of polymer modified asphalt emulsions in surface treatments such as chip seals, slurry seals, microsurfacing and cape seals on low and high volume roadways, trails, paths and parking lots in varying climates. It includes the test plan for field trials using the report-only draft specifications developed during the course of this project, as well as the test results. It is expected that the laboratory test results and field performance of these projects will be used in the development of performance-related specifications for asphalt emulsions.

17. Key Words

ASPHALT EMULSION; POLYMER; SURFACE TREATMENT; PAVEMENT PRESERVATION; CHIP SEAL; SLURRY SEAL; MICROSURFACING; CAPE SEAL; POLYMER MODIFIED ASPHALT EMULSION; SBR; SBR LATEX; SBS; MSCR; PERFORMANCE SPECIFICATIONS; SWEEP TEST

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Unclassified	Unclassified		

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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EXECUTIVE SUMMARY

The Central Federal Lands Highway (FLH) Division of the Federal Highway Administration (FHWA) initiated this study to provide a guide for the use of polymer modified asphalt emulsions in surface treatment applications, specifically chip seals, slurry surfacings and cape seals. Although FLH has much experience with best practices using conventional asphalt emulsions, there was no definitive guide for selecting, specifying and using polymer modified asphalt emulsions. Based on the experience of many users and producers of polymer emulsions over the last 25 years, it was generally accepted that polymer modification resulted in better short- and long-term performance, and ultimately cost savings over the life the pavements treated. This study consisted of a comprehensive literature review and information gathering from government, academic and industry experts. These experts were then called upon to develop recommendations.

During the course of the investigation, it became evident that the industry felt a need for updated test methods, specifications and recommendations that are better predictors of performance, that is, performance-related specifications. The investigators developed draft specifications based on the best available information from experts on both asphalt emulsions and the performance-based test methods for Superpave hot mix asphalt developed by the Strategic Highway Research Program (SHRP).

Several field trials were run in the summer and fall of 2008 on Federal Lands Highway projects, with more planned for 2009. Field samples were and will be tested in several laboratories using the draft protocols. This report gives the test plan. Federal Lands Highway will evaluate the field performance of these projects over time and the results will be compared to the laboratory test results to determine the applicability of the test methods and the appropriate specification limits. The preliminary results of performance rheometry and sweep testing included in this report are very promising, and are being shared with other researchers working on related on-going projects. The recommendations and draft specifications for materials included in this report should be of value to those users and producers wishing to improve performance of asphalt emulsion surface treatments on all types of pavements. Preliminary results are very promising, and the data collected is being shared with other researchers to characterize and specify the performance of the modified residue. Other researchers, suppliers and users will benefit from the results obtained by this testing plan, and it is envisioned that performance specifications for polymer modified asphalt emulsion surface treatments will be the norm in the not too distant future. Current activities are being fully coordinated with the FHWA Pavement Preservation ETG's Emulsion Task Force and with the FHWA Superpave ETGs to advance recommendations to the AASHTO Highway Subcommittee on Materials, with a goal of provisional emulsion performance specifications in 2010.

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1.0 INTRODUCTION

1.1 Background

Polymer modification of asphalt emulsions offers improvements in performance and durability, mitigation of pavement distress, and reduced life cycle costs when compared to unmodified asphalt emulsions or hot mix asphalt surface dressings. Such modifications have exhibited demonstrable reductions in rutting, thermal cracking, and increased resistance to many forms of traffic-induced stress. Conversely, polymer modifiers, when used in chip seal applications, have demonstrated some problems associated with accelerated stripping when placed over a moisture sensitive hot mix. Asphalt emulsions frequently provide a lower cost, efficient, and more environmentally-friendly alternative to hot mixes due to their low energy consumption, reduced hydrocarbon emissions, ease of implementation at remote sites, and preventive maintenance/life-extending benefits when applied to mildly distressed pavements.

Although best-practice manuals and specifications for conventional asphalt emulsions are plentiful, there is no single document available which offers guidance on the proper use, application techniques, and benefits of high-performance polymer modified asphalt emulsions. Similarly, the preponderance of the published research on polymer modifiers has focused primarily upon their use in hot mix asphalt (HMA) applications.

This research includes an exhaustive review of the literature to collect and analyze polymer modified emulsion practices and specifications, coupled with a laboratory testing and verification program designed to validate the findings and recommendations developed from the literature review. Guidance is provided on proper project selection, polymer dosing rates and methods, applicability under varying traffic load and environmental conditions, and contraindications to the use of polymer modifiers.

1.2 Study Objectives

The principal objectives of this study were to:

1.) Compile published research on the types of polymer modifiers, dosage rates, and modification methods.

2.) Compare and contrast the performance, cost, and benefits of polymer modified with non-modified asphalt emulsions.

3.) Determine the applicability of polymer modified asphalt emulsions to low (i.e., generally < 400 ADT), medium (400 to 1000 ADT), and high (> 1000 ADT) volume roads (as defined by Federal Lands Highway Division), and varying environmental conditions.

4.) Evaluate the applicability of polymer modified asphalt emulsions to non-roadway applications such as parking lots, hiking and bike trails.

5.) Analyze information obtained from the literature review and develop recommendations and guidelines relating to the proper application, modification, and contraindications of polymer modified asphalt emulsions (PME).

6.) Perform laboratory testing and verification to evaluate the recommendations and data gaps identified from the analysis of information obtained from the literature review.

7.) Prepare a Federal Lands Highway (FLH) manual of best practices for polymer modified asphalt emulsions.

1.3 Scope

Electronic and physical literature searches were performed using a variety of sources, including the National Center for Pavement Preservation (NCPP) on-line library; the Transportation Research Information Service (TRIS) database; the National Technical Information Service (NTIS); the COMPENDEX engineering research database; the Michigan State University College of Engineering Library; the State Library of Michigan; the websites of the Asphalt Emulsions Manufacturers Association (AEMA), the International Slurry Surfacing Association (ISSA), and the Asphalt Recycling and Reclamation (ARRA); the Asphalt Institute's on-line document collection; the Federal Highway Administration's (FHWA) technical document and reference collection; and the Google™ search engine. Numerous pavement and polymer research publications were also examined, including publications of the Transportation Research Board (TRB), the *Journal of the Asphalt Paving Technologists* (AAPT), the *International Journal of Pavement Engineering*, the *Journal of Materials in Civil Engineering*, *Polymer Engineering and Science Journal*, and the *Journal of Applied Polymer Science*.

Although this review focuses principally on polymer modified asphalt emulsions (PME), literature and research dealing with polymer modified asphalt (PMA) binders (such as those used in hot mix) have also been used in cases where the results could reasonably be extrapolated. For example, some polymer modifiers occur only in solid form, and must be added directly to the asphalt regardless of whether the binder will be hot-applied, or emulsified; whereas liquid modifiers may be added either to the soap mix; co-milled; or in some cases, post-added to the emulsion mix either at the plant or in the field. Thus, research dealing with the impact of polymer modification on asphalt binders may have some implications with respect to both hot mix and emulsion applications.

Information was also collected from a series of meetings with industry experts, who represented many years of experience with specifying, manufacturing, using and researching many types of polymer modified asphalt emulsions. There was general consensus that current test methods and specifications needed to be updated; and while there has been much work in the field of asphalt (and PMA) characterization in the last decade, little of that has been applied to asphalt emulsions. Further knowledge sharing at a series of industry teleconferences, conferences and meetings led to development of draft performance-related specifications for PME. A series of field projects was constructed by the Federal Lands Highway (FLH) division of the Federal Highway Administration (FHWA). Samples from the field projects were sent to several independent and supplier laboratories for testing with the draft testing protocols. The test results are included here and are being shared with researchers working on related on-going projects.

1.4 Report Organization

Section 1 of this report presents an introduction and overview. A discussion of the literature review is provided in Section 2. Section 3 presents the recommendations for the laboratory testing program and specifications, and a summary of the recommended application guidelines derived from the literature review. Section 4 gives the test plan and draft performance-related specifications used for the field and laboratory study. Section 5 gives the test results, and Section 6 gives the conclusions and recommendations for future work. Additional field trials are planned for 2009; once the testing on these is completed an additional report will be written. The compiled meeting notes giving the input of various industry experts consulted are on file in the Central Federal Highway Lands office. Appendix A gives the details on the user and producer survey, and Appendix B gives the materials and construction specifications used for the field trials. The final section gives the references consulted.

2.0 LITERATURE REVIEW OF POLYMER MODIFIED ASPHALT EMULSIONS

This section presents the results of the literature review with respect to the types, modification methods, demonstrated performance, surface treatments, environmental applicability, materials selection, and cost-benefit analysis of polymer modifiers. A brief overview of polymer and emulsion chemistry is provided, as is a discussion of the pavement conditions and applications which contraindicate the use of polymer modifiers. Some excellent general information on asphalt emulsions is available in the "Basic Asphalt Emulsion Manual" and the "Recommended Performance Guidelines" published by the Asphalt Institute (AI) and the Asphalt Emulsion Manufacturers Association.(1) The Caltrans Maintenance Technical Advisory Guide is one of the most comprehensive sources for information on using maintenance treatments.(2)

2.1 Basics of Polymers and Asphalt Emulsions

This section introduces and describes some of the basic terms and concepts related to polymers, polymer chemistry, and asphalt emulsions. While the purpose is not to provide a comprehensive narrative of the complexities of polymer chemistry, a grasp of the essential terminology and processes is beneficial in understanding the formulation, advantages, and applications of polymer modified asphalt emulsions.

2.1.1 Polymer Terminology and Chemistry

A polymer is a natural or synthetic high-molecular weight organic compound which consists of a chain of smaller, simpler repeating units known as monomers. For example, the monomer "ethylene" may be "polymerized" (i.e., individual ethylene molecules chained together) to form "polyethylene". When two or more distinct types of monomers are combined, the resulting compound is termed a "copolymer."(3)

The structure of copolymers may be random, or may repeat in blocks of polymers (block copolymers) as illustrated in figure 1. An example of a block copolymer is "styrene-butadiene" or SB, which consists of blocks of polymerized styrene (a monomer) and polymerized butadiene (another monomer). SB is further categorized as a "diblock" copolymer, because it consists of two different polymerized monomers. Polymer structures include straight, radial, crosslinked, and irregularly branched chains. Factors which can influence the behavior and performance of polymers include chemistry, structure, bonding types, and the manufacturing process.

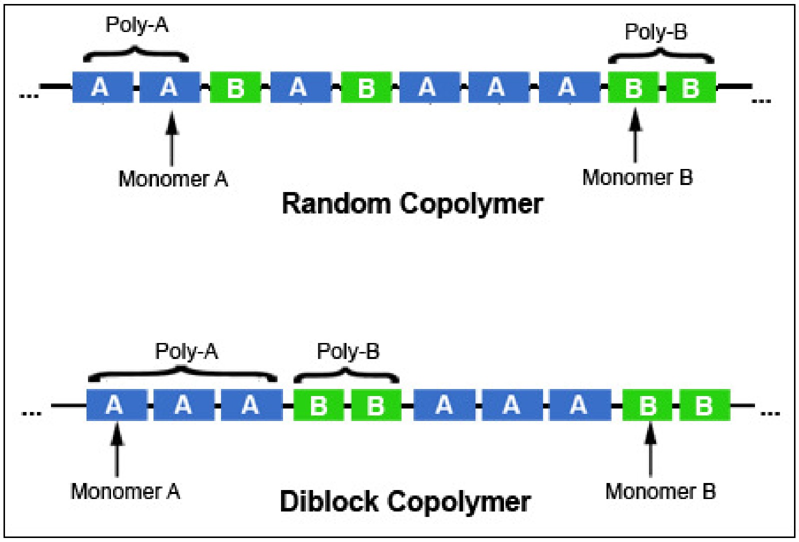


Figure 1: Examples of Copolymers

2.1.2 Asphalt Emulsions

Asphalt emulsions are formed by the milling of raw asphalt into microscopic particles which are dispersed in water with the aid of a chemical emulsifying agent called a "surfactant" (sometimes referred to as "soap"). In such cases, the dispersed asphalt forms discrete droplets which are intrinsically insoluble in water. The emulsion is said to be "stabilized" if the asphalt droplets remain well-dispersed such that phase separation does not occur. Stabilization is achieved through the use of surfactants, which consist of polar molecules comprised of a hydrophilic (water loving) "head" and hydrophobic (water avoiding) "tail." The tail of the surfactant molecule is attracted to the asphalt particles, forming a coating around each particle which consists of the hydrophilic heads of the emulsifying agent. The hydrophilic portions of these surfactants strongly associate with water and aid in keeping the droplets dispersed and in suspension. Formulators can use other additives to enhance properties of the emulsion during storage, shipping, application and the asphalt's end use.

Surfactants are classified as anionic, cationic or nonionic based upon the charge of the hydrophilic portion of the molecule. Anionic and cationic emulsifiers are the most commonly used in pavement surface treatment applications. The electrical potential that exists between the surface of the surfactant-coated asphalt particles and the emulsion solution is measured as the "Zeta potential". The Zeta potential is determined by measuring the velocity of emulsion particles when an electric field is applied. Some researchers believe high zeta potentials indicate potentially greater electrostatic repulsion between asphalt particles, and therefore greater stability of the emulsion (i.e., less of a propensity to phase-separate). Some suppliers use chemistries which have confused the issue of classification. For example, nonionic emulsifiers can be used with certain additives to produce materials which pass specifications for cationic emulsions, and quaternary amines produce cationic emulsions that show behavior more similar to nonionic emulsions.

In cationic asphalt emulsions, the positively charged layer of surfactants coating the asphalt particles are attracted to negatively charged aggregate mixed with the emulsion. “Breaking” of the emulsion is said to occur when the asphalt separates from the water phase and coalesces to coat the grains of the mineral aggregate. This may occur by 1) simple evaporation of the water, 2) a chemical destabilization of the hydrophilic portion of the surfactant by the aggregate, the existing road surface or chemical additives or 3) a combination of evaporation and chemical destabilization. To achieve breaking in anionic asphalt emulsions, the asphalt and aggregate particles must be sufficiently close to overcome the repulsive forces which exist between the negatively charged outer layer surrounding the asphalt particles and the negatively charged surface of the aggregate (when using aggregates with negatively charged surfaces). The timing and rate of breaking of all asphalt emulsions is controlled by several factors, including the chemistry of the surfactant, the type of aggregate used, the emulsion formulation, chemical additives, the temperatures of the emulsion, air, aggregate and pavement surface at time of application and the ambient humidity.

Generally, cationic asphalt emulsions maintain their positive charge at low pH but lose the charge at pH>8-10. The emulsion is typically produced, stored and applied at pH<4. In contrast, anionic asphalt emulsions possess a high negative charge at high pH, but become neutral under acidic conditions. The emulsion pH changes when contacted with aggregate and/or with the addition of other additives, such as Portland cement which is often added for slurry seals and microsurfacing. This change in the emulsion pH is one of key parameters controlling the timing and range of breaking.

After the break occurs, the water phase of the applied emulsion drains and evaporates, allowing the residual asphalt to coalesce and achieve its full strength (curing). Asphalt is a very viscous liquid, and therefore it flows very slowly. The emulsification process improves flow. Once the water has separated from the asphalt, warm air and surface temperatures facilitate the flowing together of the asphalt particles to form its most stable cohesive and adhesive binder state. Factors influencing the quality and performance of asphalt emulsions include, but are not necessarily limited to:

- Chemical properties, particle size, hardness, and concentration of the base asphalt.
- Chemistry, ionic charge, and concentration of the surfactant.
- Manufacturing conditions such as temperature, pressure, milling shear, and the order in which the ingredients are combined.
- The type of manufacturing equipment used.
- The types and amounts of other chemical modifiers (such as polymers) which are added to the emulsion.
- Chemistry and quality of the bulk emulsion water solution.[1]

2.1.3 Asphalt Composition

Asphalts have been characterized as colloids containing high molecular weight, relatively insoluble and nonvolatile compounds and associations of lower molecular weight polar molecules known as "asphaltenes" dispersed within a liquid, continuous, lower viscosity phase comprised of low molecular weight compounds called "maltenes". Asphaltenes are believed to be the component of asphalt that imparts hardness, while maltenes provide ductility and facilitate adhesion. Maltenes consist predominately of oils (aromatics and saturates) and resins (compounds which represent a transition between asphaltenes and oils). Typical asphalts normally contain between 5 percent and 25 percent by weight of asphaltenes. Newer theories are a bit more complex, defining sol and gel types of asphalts. The chemistry of the asphalt depends upon the crude oil source and the refining method. The chemistry also determines the stability of the colloidal structure and its physical characteristics, including temperature susceptibility, cohesion and adhesion.

The asphaltene content of asphalt cements is chiefly responsible for influencing the overall viscosity of the composite system – that is, higher asphaltene contents generally lead to higher composite viscosities. In addition, research has shown that maltene phases possessing a comparatively high aromatic content generally result in better dispersal of the asphaltenes, leading to high ductility, low complex flows, and lower rates of age-dependent hardening.(2)

Conversely, low aromatic maltenes generally lead to the formation of agglomerates of asphaltenes which form a network-like structure and are referred to as "gel-type" asphalt cement. Asphalts containing high percentages of non-reactive saturated paraffinic, waxy molecules tend to be temperature susceptible; they become fluid at high pavement temperatures causing rutting and bleeding and brittle at low temperatures causing cracking and shelling. Gel-type asphalt may also be formed from mixtures where the asphaltene to maltene ratio is inordinately high, because maltenes are needed to disperse the asphaltene fractions. Gel-type asphalts are generally characterized by low ductility, increased elastic component content, thixotropic behavior, and rapid age-dependent hardening.(3) In this sense, the addition of polymer modifiers when used in conjunction with compatible asphalts, can lead to improved high and low temperature performance combined with increased flexibility and resistance to deformation. Compatible polymer/asphalt systems produce a two-phase mixture that is characterized by a well dispersed polymer phase that is stable at high temperatures. The most effective polymer networks maintain integrity at both high and low temperatures.

Asphalt's polarity is due to the presence of alcohol, carboxyl, phenolic, amine, thiol, and other functional groups on the various molecules making up the asphalt. As a result of this polarity, the molecules self-assemble to form effectively large, complex structures with molecular weights ranging up to 100,000. The adhesion of asphalt to mineral aggregate particles is also thought to depend on the polar attraction between asphalt particles and the charged surfaces of most aggregates. Although asphalt is not a polymer in the strict sense of the word, it is regarded as a thermoplastic material because it becomes soft when heated and hardens upon cooling. Asphalts also exhibit viscoelastic properties which can be improved upon with the addition of polymer modifiers.

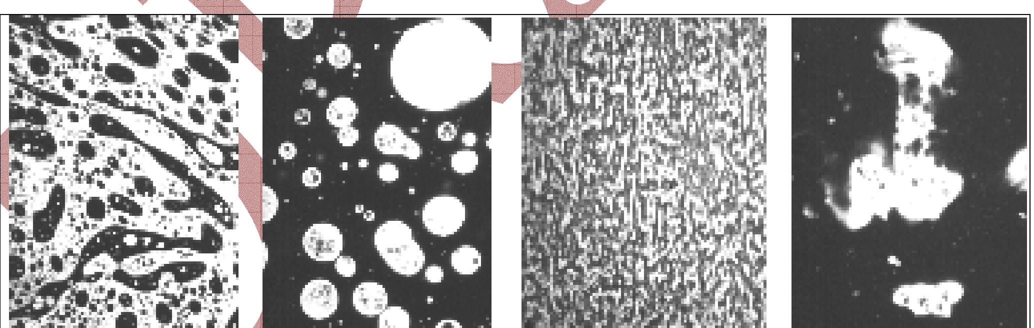
2.1.4 Polymer Modified Asphalt (PMA)

In general terms, the addition of polymers to asphalt binders results in the modification of certain key physical properties including the:

- Elasticity.
- Tensile strength.
- High and low temperature susceptibilities.
- Viscosity.
- Adhesion and cohesion.

Depending upon the form of modification desired, improvements in pavement longevity can be achieved through the reduction of fatigue and thermal cracking, decreased high temperature susceptibility (e.g., rutting, shoving and bleeding), and enhanced aggregate retention in applications such as chip seals. Polymer modifiers are used to extend the lower and/or upper effective temperature operating ranges of pavements and to add elastic components that allow it to recover from loading stress.

The physical and chemical characteristics of the polymer and its compatibility with the chemistry of the asphalt determine the physical property enhancements. Figure 2 shows ultraviolet light reflective photomicrographs of two different asphalts modified with differing SB block copolymers, all at the same percent polymer added.(4) The dark is the asphalt and the light colored material is the polymer. In the compatible cases, the polymer is swollen by the oils in the asphalt and entangles itself within the asphalt to form a continuous network. In the incompatible blends, the polymer balls up into itself and is discontinuous. In most cases, the polymer has a lower density than the asphalt, and these polymer particles will rise to the top of the storage tank without constant agitation.



Asphalt G	Asphalt G	Asphalt A	Asphalt A
SB Polymer	**SBS Polymer**	**SB Polymer**	**SBS' Polymer**
Compatible; Good Physical Properties; Homogeneous	Incompatible; Poor Physical Properties; Polymer Separates	Compatible; Good Physical Properties; Homogeneous	Incompatible; Poor Physical Properties; Polymer Separates

Figure 2: Photomicrographs of 6% of Three Polymers in Two Asphalts

A variety of testing techniques and equipment are available which may be used to evaluate and quantify the performance characteristics of polymer modified binders and emulsion residues. A few of the most common of these include:

- Dynamic Shear Rheometer (DSR) which is used to measure the shear modulus (resistance and phase angle) of asphalt within intermediate to high operational temperature ranges. DSR testing distinguishes between elastic (recoverable) and viscous (non-recoverable) responses of the test material when placed under stress, and is often used as an indicator of rutting resistance and other forms of permanent deformation. While there is much work with DSR testing and specification of asphalts for HMA, there has been relatively little work done with emulsion residues until this study.

- Bending Beam Rheometer (BBR) and Direct Tension Test (DTT) which are used to determine the stiffness/flexibility of asphalt binders at low temperatures, and thus, their susceptibility to thermal cracking. As with the DSR, there is relatively little in the literature about the use of these tests for asphalt emulsions.

- Ring and Ball Softening Point which is used to determine the temperature at which an asphalt allows a metal ball to fall through the asphalt. This test provides another measure of high temperature susceptibility.

- Direct Tensile Test which is a measurement of the force that is required to deform an asphalt sample; tensile strength testing allows the stress applied to the sample to be plotted against its resulting elongation (strain).

- Elasticity after Ductility Testing where the sample is elongated into a thread, cut, and the resulting recovery after a given time is measured. Elasticity measures have important implications related to the resiliency of the pavement under repeated cycles of loading and unloading.

- Rotational Viscometry (RV) is used to measure the viscosity of modified and unmodified asphalts and is directly related to the workability of the HMA mixture during field application. It may also be used to determine emulsion viscosity.

- Modified asphalt emulsion testing can be carried out by either testing the binder prior to emulsification, or by obtaining a sample of the properly cured emulsion residue. A more thorough treatment and evaluation of performance testing methodologies and criteria is provided in Section 2.4.

2.2 Types of Polymer Modifiers

2.2.1 Overview and Classification

Based upon their strain performance characteristics at low temperatures, polymer modifiers are generally separated into two broad categories: elastomers and plastomers. Elastomeric polymers can be stretched up to ten times without breaking, but quickly return to original shape once the load has been removed.[3] Typical elastomeric polymers used to modify asphalt include natural and synthetic rubbers, styrene-butadiene-styrene (SBS) and crumb rubber modifiers (CRM) reclaimed from scrap tires. Worldwide, elastomeric polymers comprise approximately 75 percent of all the asphalt polymer modifiers used (not including recycled crumb rubbers).

Unlike elastomers, plastomeric polymers attain very high strength at a rapid rate, but are brittle and resistant to deformation once set. Examples of plastomeric polymer modifiers include low density polyethylene (LDPE), ethylene-propylene-diene-monomer (EPDM), and ethyl-vinyl-acetate (EVA). Plastomeric polymers currently comprise about 15 percent of the global market for asphalt polymer modifiers.

Elastomeric and plastomeric polymer modifiers are further classified as either “thermoset” or “thermoplastic”, based upon their temperature-dependent structural formation and reformation characteristics. When initially heated, thermoset polymers develop a complex, cross-linked structure which is retained upon cooling, but which cannot be reversed when reheated.(3) In contrast, thermoplastic polymers also develop a well-defined, linked matrix when cooled, but the resultant structures can be reversed or “reset” with reheating.

Thermoplastic Rubbers (TPR) or Thermoplastic Elastomers (TPE) such as SBS combine the hard, resistant characteristics and re-settable structure of plastics with the elastic recovery of thermoset elastomers like natural or synthetic rubber. TPE’s exhibit this unique blend of properties through the structural integration of rigid, generally steric (i.e., styrene-containing) components with rubbery domains such as found in polybutadiene.

Table 1 presents a summary of the most commonly used polymer modifiers, classified according to their deformational and thermal properties. It is important to note that many of these polymers may be blended with other types to achieve the appropriate combination of thermal and deformational properties. The following subsections provide detailed discussions of the published literature covering each of these polymer modifiers.

Table 1: Types and Classifications of Polymer Modifiers

Polymer Type	Examples	Classification	References
Natural Rubber (Homopolymers)	Natural Rubber (NR), Polyisoprene (PI), Isoprene, Natural Rubber Latex (NRL)	Thermoset Elastomers	(5) (6)
Synthetic Latex / Rubber (Random Copolymers)	Styrene-Butadiene (SBR)	Thermoset Elastomers	(5) (6)
	Polychloroprene Latex (Neoprene)	Thermoset Elastomers	(3) (6)
	Polybutadiene (PB, BR)	Thermoset Elastomers	(5)
Block Copolymers	Styrene-Butadiene-Styrene (SBS)	Thermoplastic Elastomers	(6)
	Styrene-Isoprene-Styrene (SIS)	Thermoplastic Elastomers	(6) (8)
	Styrene-Butadiene Diblock (SB)	Thermoplastic Elastomers	(3) (5)
	Acrylonitrile-Butadiene-Styrene (ABS)	Thermoplastic Elastomers	(7)
	Reactive-Ethylene-Terpolymers (RET)	Thermoplastic Elastomers	(9)
Reclaimed Rubber	Crumb Rubber Modifiers	Thermoset Elastomers	(5) (6)
Plastics	Low / High Density Polyethylene (LDPE / HDPE), Other Polyolefins.	Thermoplastic Plastomers	(6)
	Ethylene Acrylate Copolymer	Thermoplastic Plastomers	(3) (6)
	Ethyl-Vinyl-Acetate (EVA)	Thermoplastic Plastomers	(6)
	Ethyl-Methacrylate	Thermoplastic Plastomers	(8)
	Polyvinyl Chloride (PVC)	Thermoplastic Plastomers/ Elastomers	(6)
	Ethylene-Propylene-Diene-Monomer or EPDM	Thermoplastic Elastomers	(6)
	Acrylates, Ethyl methacrylate (EMA), Ethyl butyl acrylate (EBA).	Thermoplastic Plastomers	(4)
Combinations	Blends of Above	Varies	(6)

2.2.2 Natural Rubber and Latex

Natural rubber latex (NRL) is an elastomeric hydrocarbon polymer of the isoprene monomer (polyisoprene) that exists as a natural milky sap produced by several species of plants. The "sap" has a water-based colloidal structure. Natural rubber (NR) is produced from NRL by coagulation to form a solid material.

The first commercial process that was developed to modify asphalt emulsions with NRL was the Ralumac® system. [10] The Ralumac® process involves mixing naturally anionic NRL with cationic surfactants, and emulsifying the resulting liquid with asphalt using a colloid mill.[10] This type of NRL modification is usually a two-stage process using a continuous-feed emulsion plant to achieve the desired results. However, when compatible NRL is used (with respect to asphalt microstructure) the process can be reduced to a single stage, and the latex added pre- or post-emulsification as shown in figure 3.[4][10]

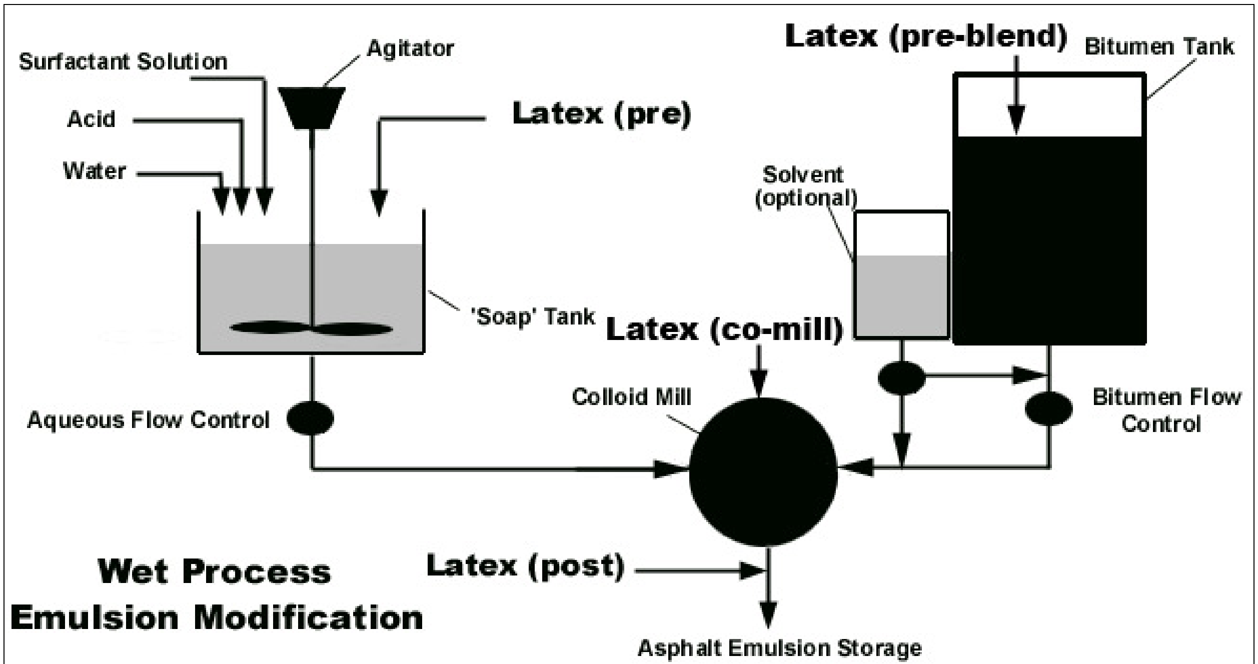


Figure 3: Typical Emulsion Modification Processes [10]

The resulting cationic emulsion is attracted to the anionic surfaces of the aggregate, latex, and filler material; this increases the oil-wettability and ensures better adhesion of the coagulated asphalt to the mineral grains once cured (figure 4).[(11)] This breaking process is essential in ensuring rapid adhesion and strength development. The polymer component of a properly formulated and stabilized emulsion is dispersed throughout the bituminous cement to form an elastic, foam-like lattice upon curing (figure 5).

NRL modification of asphalt yields similar performance benefits to those obtained in hot mix, including increased thermal stability, higher resistance to load deformation and reduced thermal cracking.[(10)] The resulting rubberized asphalt acts like an elastic membrane which holds residual asphalt particles together, thereby retarding crack propagation and increasing stone retention (figure 5). Crack pinning also contributes to retarding the crack growth.

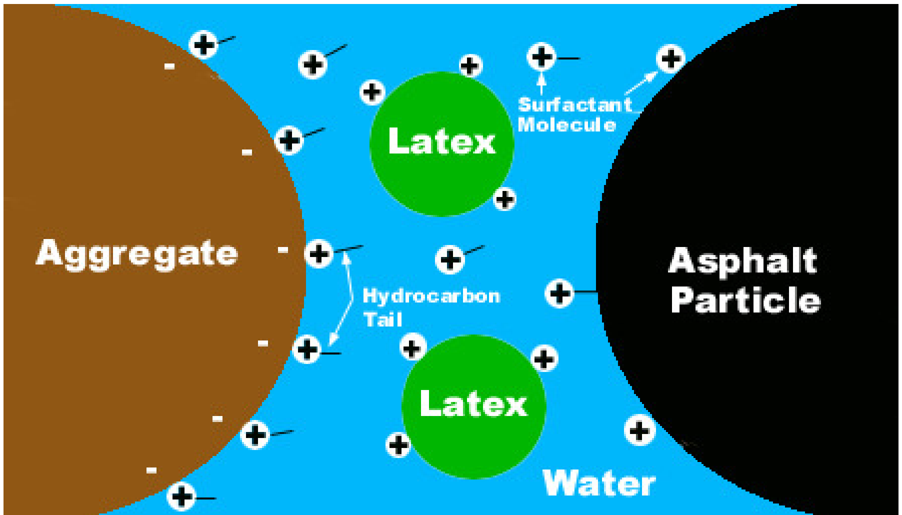


Figure 4: Surfactant Action in NRL Modified Asphalt Emulsion

At higher temperatures, the NRL's lattice resists flow in the asphalt matrix, which increases the pavement's resistance to deformation. Microsurfacing, slurry seals, chip seals, and tack coats all may benefit from the use of NRL modified asphalt emulsions. Figure 5 illustrates the distribution of a latex lattice within the asphalt binder used in a typical chip seal.

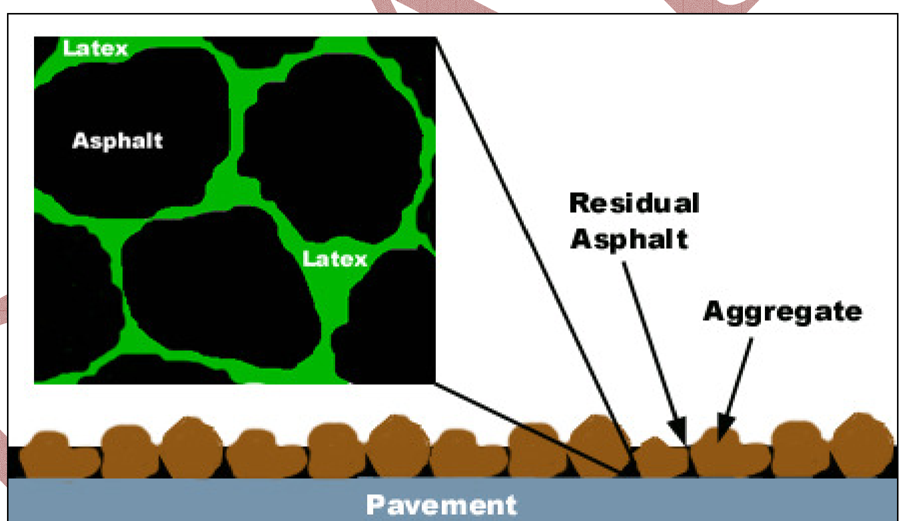


Figure 5: Polymer Modified Chip Seal

2.2.3 Synthetic Rubber and Latex

Synthetic latex is a thermoset elastomer which consists of a mixture of polymer particles dispersed in water. Commonly used varieties of synthetic latex rubber include styrene-butadiene rubber (SBR) which is a random copolymer, polychloroprene (Neoprene), and polybutadiene (PB). Common uses of latex modified asphalt emulsions include microsurfacings, chip seals, and slurry seals. Lubbers and Watson (2005) note that the handling and blending of SBR latex is particularly facile, and is amenable to a variety of pre- and post-modification methodologies.[(5)] When sufficient quantities of synthetic latex are added to compatible asphalts, the cured mixture is commonly characterized by a continuous polymer network which envelops the bitumen particles (see figure 5). Benefits of properly blended latex polymers included improved stone retention, increased skid-resistance and improved low temperature performance (i.e., less brittleness, better elasticity and better adhesion to aggregates).

Like NRL, SBR latex that is uniformly dispersed in the emulsion during blending forms elastic lattices within the bituminous cement when cured. More specifically, as water within an applied emulsion evaporates, droplets containing SBR coalesce along the surfaces of asphalt particles, which results in the formation of a continuous, honeycombed polymer network which extends throughout the binder.(12) In this way, SBR particles form "welds" between asphalt particles, which result in an increase in tensile strength, stone retention and resistance to cracking.(12) (13) SBR modification of asphalt emulsions may be accomplished by co-milling at the colloid mill, post-blending after emulsification, or by mixing at the application site through the distributor (a field variation of the post-blending method).(13) Compatibility of the SBR with the asphalt should be verified to ensure the success of single-stage mixing methods.

Takamura (2001) has demonstrated the benefits of SBR modification of asphalt emulsions and microsurfacing mixes, with significant increases in rutting resistance temperatures observed with increasing polymer content, as illustrated in figure 6. Figure 6 gives test results from laboratory aging at elevated temperature, in an attempt to simulate long term field aging.(14) Similarly, Takamura shows that a latex modified asphalt chip seal emulsion exhibits better early chip retention than the unmodified emulsion (figure 7).

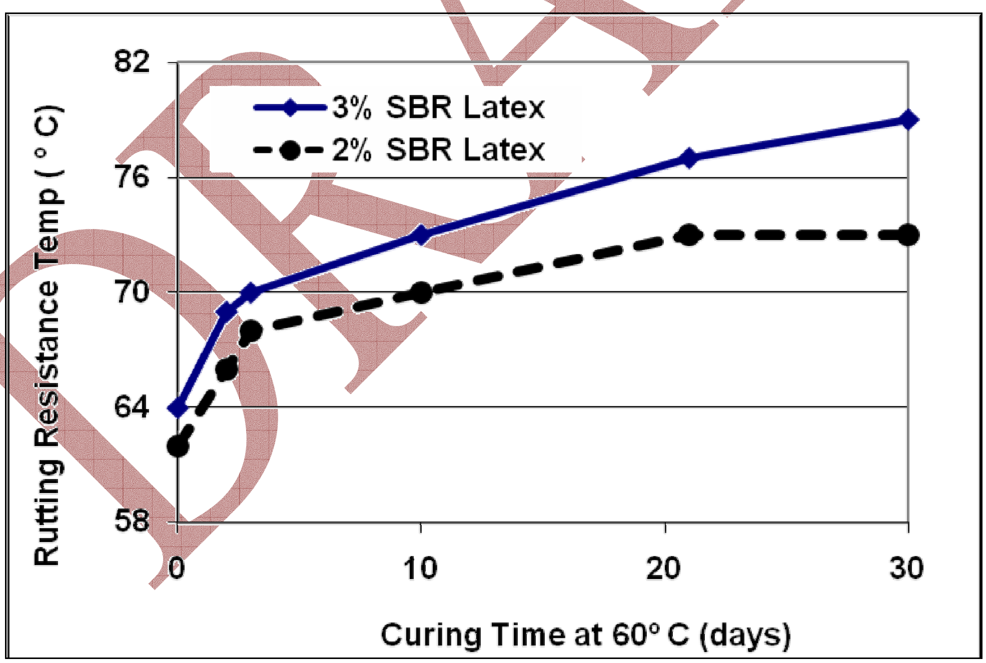


Figure 6: Curing of a CRS-2P Emulsion (14)

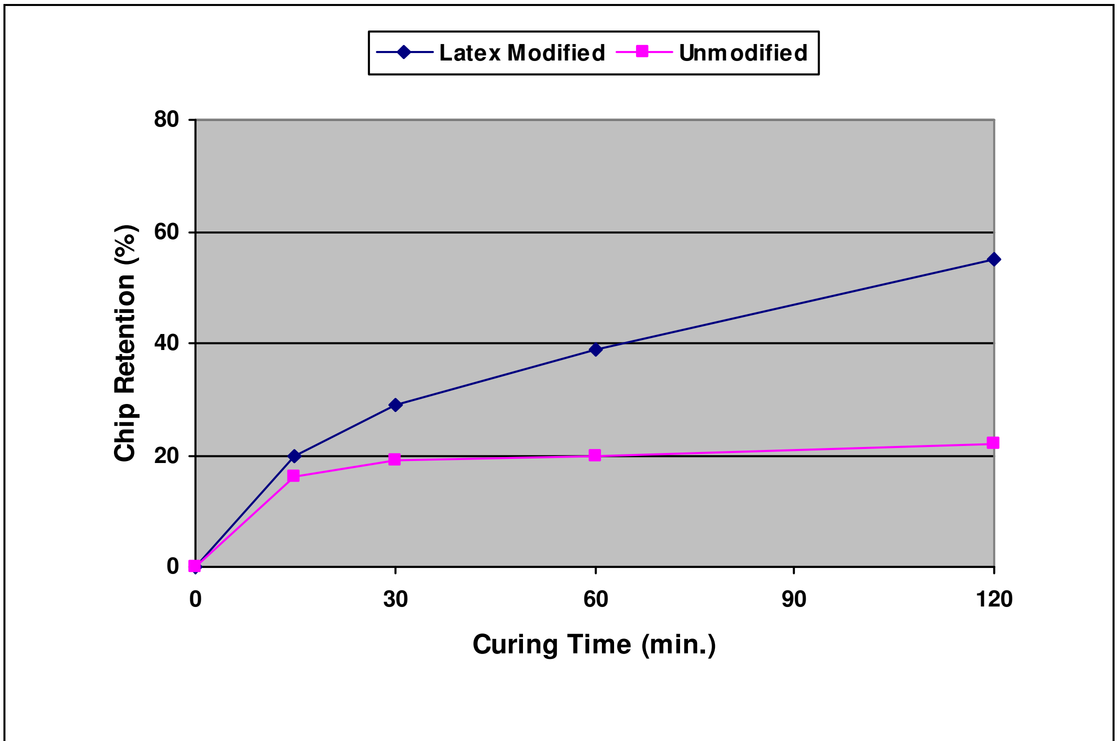


Figure 7: Stone Retention over Curing Time (14)

Further, Takamura illustrate in figure 8 that an SBR latex modified microsurfacing mix gave better results than SBS, EVA or Neoprene (in the same asphalt) in wet track abrasion losses and wheel track deformation, indicating better stone retention and reduced flow characteristics.

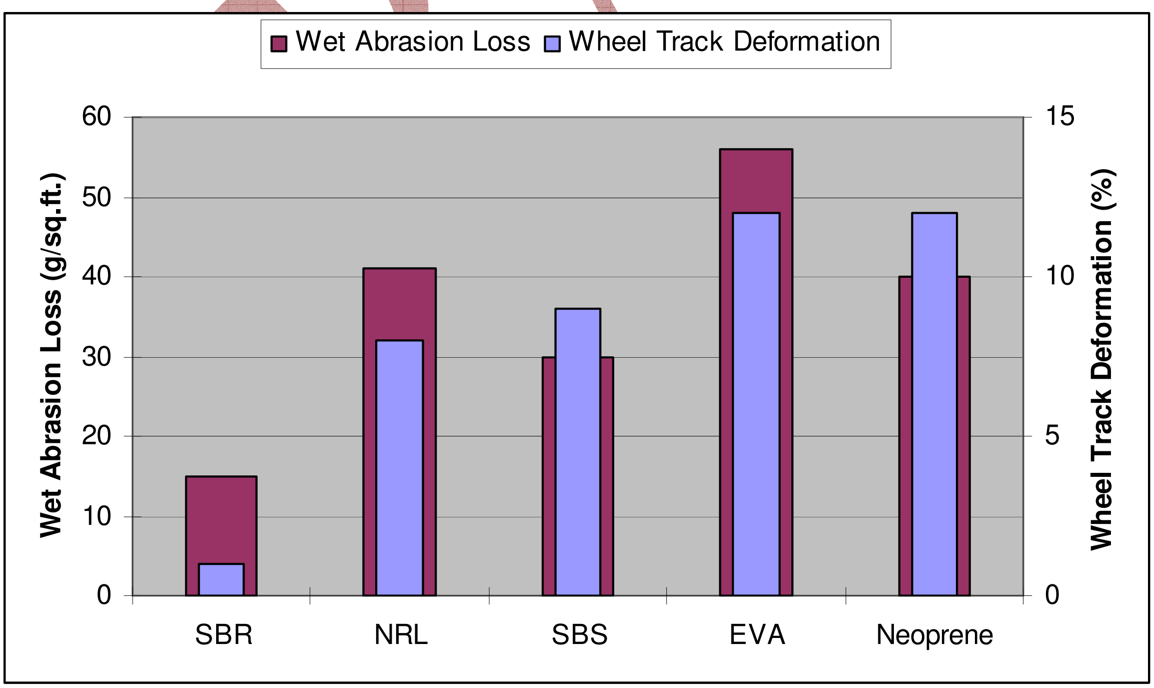


Figure 8: Wet Track Abrasion and Loaded Wheel Test by Polymer Type (14)

2.2.4 Block Copolymers

When hard, styrene containing polymers are co-polymerized with small molecules such as butadiene in structurally discrete connected blocks, the result is a block copolymer.(15) Typical examples of block copolymer modifiers include SBS, SIS, SB, ABS, and RET. SBS (a triblock) is the most commonly used because of its desirable properties and comparatively low cost.(16)(17) The elasticity and strength benefits imparted by SBS modifiers are attributable to the molecule's rubbery polybutadiene (PB) "mid-blocks" capped at either end by polystyrene end-blocks which provide strength and rigidity and increased compatibility with most asphalts.(16) Most block copolymer modifiers behave as thermoplastic elastomers, returning to their original shape upon removal of the loading stress.

Block copolymers are typically lower molecular weight than typical formulations of SBR latex, and generally consist of a comparatively narrow distribution of chains of similar monomer chain lengths. Whereas in SBS or SB, the monomers (building blocks) are randomly positioned and can exhibit a wide variety of regular and well-defined molecular morphologies including linear, star-shaped, and radial structures.(4) Generally, random SBR polymer modified asphalts elongate more (have better ductilities, especially at very low temperatures) than SBS block copolymers because of the double bond structure, but SBS block copolymer modified asphalts show more strength at elongation (elastic recovery, especially at high temperatures) because of the structure of the styrene blocks.(4) The exact performance, however, depends upon the structure of the specific polymers used and their compatibility with the specific asphalt used, as was illustrated in figure 2. Formulators have the ability, therefore, to design polymer asphalt blends for specific performance needs, such as for durable microsurfacing and chip seal applications.

When triblock copolymers such as SBS and SEBS are raised above the glass transition temperature of their polystyrene end-blocks, these rigid domains soften, thereby weakening the crosslinked structure of the polymer. At temperatures above 150ºC, block copolymers are pliable in molten form in contrast to NRL modifiers which begin to undergo crosslinking at this temperature. (3) Work by Wegan (2001) suggests optimal mixing temperatures of approximately 180º C for SBS modifiers.(16) Because block copolymers are workable at higher temperatures, the styrene domains comprising the typical SBS modifier can readily be segregated under shear force during the milling process, promoting the dispersion of individual chains throughout the asphalt binder. Consequently, as the polymer/asphalt blend is cooled, these styrene domains begin to reform, establishing a pervasive polymer network throughout the residual asphalt matrix.(3)

Stroup-Gardiner and Newcomb (1995) report that sufficient quantities of SBS polymer modifiers are required to promote effective crosslinking during the cooling phase to ensure that reactive portions of the styrene domains are close enough together to permit bonding. Termed the "critical concentration" or "c*", Stroup-Gardiner and Newcomb recommend SBS contents of at least 2 percent, and in some cases greater than 4 percent by weight of residual asphalt depending upon the chemistries and compatibility of the specific polymer and asphalt.(3) Additionally, as the ability of the polymer components to be swollen by a given asphalt increases, less polymer additive is needed (by weight) to achieve c*. Polymer swelling is generally believed to be caused via interaction with aromatics contained within maltene fractions, and will eventually lead to the formation of a continuous network.(18)

Factors influencing c* include the quantities of diblock (SB) versus triblock (SBS) copolymer used, mixing temperatures, the chemical compatibility between the asphalt and polymers utilized, and blending time.[3] "Compatibility" refers to the degree of molecular interaction occurring between the asphalt and polymer modifier components of the mixture, with more compatible asphalt being characterized by a higher degree of polymer swelling and increased homogeneity and dispersion of the polymer fractions when mixed. Block copolymer modifiers must be matched to a compatible asphalt which will readily dissolve the end-block styrene domains at typical mixing temperatures to ensure thorough dispersion of the polymer during the emulsification and milling process.[3]

Stroup-Gardiner and Newcomb report that the complex modulus of 6 percent SBS-modified AC-10 decreases significantly with increasing SB diblock content at higher temperatures.[3] Moreover, the researchers note that as the concentration of the diblock SB increases within a particular SBS modifier, the resultant complex modulus decreases substantially, leading to increased pavement rigidity, particularly at higher temperatures.[3]

Studies by Serfass *et al.* (1992) show that SBS-modified asphalt emulsions exhibit excellent adhesion properties with a diverse variety of aggregate, and they can be applied over a much longer working season than similarly modified hot mixes.[19] Moreover, emulsified asphalt applications were also shown to tolerate higher polymer dosing levels than modified hot mixes, resulting in improved stone retention, cohesion and viscoelasticity, especially in crack sealing applications.

Investigation into the effects of SBS and SEBS triblock copolymers on asphalt rheology conducted by Gahavari (1997) shows a substantial increase in dynamic shear rheometer complex moduli at low to intermediate testing frequencies as polymer content is increased and when compared to unmodified asphalts.[20] Using time-temperature superposition, the low frequencies correspond to high temperature properties (i.e. resistance to rutting). Gahavari also reports a significant decrease in loss tangent values (i.e., decreased viscous, flow-type behavior) over low to intermediate frequencies with the addition of polymer–an indicator of increased elasticity. However, at higher testing frequencies, it has been shown that the aging condition of modified asphalts may reduce the preferential elastic response effects obtained via the addition of polymer modifiers which were observed at lower frequencies.[20]

2.2.5 Reclaimed Rubber

With the abundance of used tires and their associated disposal problems, there are undeniable incentives to use reclaimed rubber to improve pavement performance and/or as a means of facilitating disposal. CRM consists of scrap tire rubber that has been mechanically ground and reduced in size to particles generally less than or equal to 6.35 mm (0.25 inches) in diameter. Although most commonly used in HMA applications, reclaimed CRM has been used successfully on a limited basis in asphalt emulsions, particularly in those areas of the world where their lower cost and simplified application in remote locales are viewed favorably as compared to hot mixes.

Reclaimed tire rubbers are not pure polymers, but represent blends of SBR latex, polyisoprene (natural rubber), carbon black, and other additives.[21] CRM is extensively crosslinked and is not very compatible with the asphalt nor is it readily swollen. Devulcanization and use of high shear both reduce molecular weights and open up the polymer structure thereby improving the solubilization/emulsification. While CRM can be successfully emulsified if particle size is sufficiently fine or if predigested, the cross-linked structure of the compounds in tire rubber generally result in the formation of two distinct phases upon blending (i.e., asphalt and rubber). This makes stabilization of the final emulsion difficult to achieve. Phase separation in CRM modified asphalt emulsions is characterized by two distinct mechanisms: coalescence and creaming.(22) Coalescence occurs when polymer particles aggregate together within the emulsion through the process of molecular diffusion. Creaming occurs when polymer particles rise to the surface of the emulsion due to density differences between the modifier and binder components.

Sabbagh and Lesser (1998) note that the phase stability of CRM modified asphalt emulsions is governed in large part by both particle size and morphology. In unstable modified asphalt emulsions, polymer particles tend to coalesce, gradually increasing in size over time until they become sufficiently large for creaming to occur.(22) Sabbagh and Lesser have experimentally determined the critical particle transition radius (between coalescence and creaming) to be approximately 4 µm at 110ºC for polyolefins. Polymer particles in unstable asphalt emulsions have a predominately teardrop-shaped morphology, whereas those in stabilized asphalt emulsions are characterized by either spherical and/or cylindrical shapes. The irregular, nonspherical shaped polymer particles which characterize unstable modified asphalt emulsions are commonly observed under high shear mixing conditions. Additionally, the use of steric stabilizing copolymers has been shown to promote more thermodynamically stable spherical polymer particle shapes.(22) Sabbagh and Lesser have noted that while polymer particle sizes in stabilized asphalt emulsions are generally larger than those in unstable asphalt emulsions, the former are not more susceptible to creaming. The authors attribute this to the increased density of the particles in stabilized asphalt emulsions created by the use of steric stabilizers.(22) Thus, stabilized asphalt emulsions are those which are characteristically stable with respect to both creaming and coalescence. Paradoxically, Sabbagh and Lesser have shown comparable increases in fracture toughness and improved high-temperature viscoelastic behavior with increasing polymer content for both stable and unstable asphalt emulsions. This suggests that actual field performance is relatively insensitive to initial polymer particle morphology.

CRM can be added as a dry ingredient to slurry mixes to avoid problems of phase separation, but in such cases it serves primarily as a filler material. When used as filler, CRM fails to form a pervasive matrix or network, and thus does not impart the cohesive and viscoelastic benefits associated with most other forms of polymer modification.

One solution to the phase separation problems associated with CRM modifiers involves the use of solvents to partially predigest the rubber particles prior to their introduction into the emulsion. High boiling point petroleum-based solvents that are high in aliphatic content are generally preferred because they promote swelling and softening of the rubber which improves particle wetting and increases adhesion, (21) while also meeting U.S. Environmental Protection Agency (EPA) emissions requirements. "RG-1" represents a mixture of 40-50 percent CRM dispersed in a petroleum-based solvent, which is post-added to the emulsion through simple mixing. RG-1 modifiers exhibit good stability when blended with either cationic or anionic asphalt emulsions, with typical treatment applications including chip seals and slurry surfacing.(21)

Laboratory and short-term field testing of RG-1 modified asphalt emulsions indicate improved crack and rut resistance, higher viscosity, lower thermal susceptibility, better stone retention, and improved elasticity when compared to unmodified asphalt emulsions, though results are generally less impressive than conventional forms of polymer modification. (21) As far as processing RG-1, some research shows that it does not adversely impact setting times for slurries or microsurfacings. (21) When used in chip seals, RG-1 costs are approximately 2 to 5 cents per square foot, and for slurry or microsurfacing the cost is about 1.5 to 3 cents per square foot. (21) No special equipment is required to add RG-1, and standard batch plant transfer pumps are adequate for the task.

Another use of reclaimed rubber and emulsions involves the direct addition of 15-22 percent of CRM to the hot asphalt binder used in some chip seals. In such instances, the modified binder is sprayed on top of the pavement surface followed by an overlay of stone, and then rolled. A fog seal of asphalt emulsion (generally, a 1:1 dilution) may then be applied over the top of the chip seal to improve stone retention. (23) Cape seals may be constructed using CRM in a similar fashion, by modifying the chip seal binder coat prior to the application of the overlying microsurfacing or slurry seal.

2.2.6 Plastics

The plastic polymer modifiers are typically thermoplastic plastomers (and sometimes elastomers) which are commonly based upon the polyolefins or copolymers of ethylene. Typically, polyolefin modifiers include polyethylene and its variants such as high density polyethylene (HDPE) and low density polyethylene (LDPE). Although polypropylenes are also considered part of this group, they are generally not recognized as imparting significant improvements in elasticity or crack resistance in asphalt paving applications. (7, 24) Among the ethylene copolymers, ethyl-vinyl-acetate (EVA), ethylene-propylene-diene-monomer (EPDM), ethyl-butyl-acrylate (EBA), and ethyl-methacrylate (EMA) are the most common.(4)

Characteristically, the plastomers impart rigidity to asphalt pavements leading to rapid early tensile strength and decreased high temperature susceptibility, but depending upon the formulation, may also fail to exhibit the desired elastic response when deformed (i.e., decreased resistance to strain). The Strategic Highway Research Program (SHRP) guidelines call for a maximum fatigue resistance value of 5,000 kPa (as tested at standard temperatures) in order to decrease the propensity of the in-place pavement to crack at intermediate ambient temperatures.(25) Some formulations with these plastomers may fail this parameter. Although many believe the SHRP fatigue parameter may not be the best measure for fatigue resistance, the ability of the material to withstand repeated loadings has a major contribution to its effective life cycle. In general, the higher the degree of crystalline structuring possessed by a plastomer, the higher the resulting tensile strength and the lower the elastic response. (3) While plastomer systems may be somewhat brittle, their plastic polymer structure may give them the ability to resist crack propagation. Additional modifiers may be introduced as copolymers which can serve to partially disrupt this crystalline structure, thereby increasing the ability of the pavement to flow. The goal of inducing modest increases in flow potential is to reduce excessive binder stiffness at low (less than 10ºC) temperatures, thereby mitigating the potential for thermal and fatigue cracking. (25) Moreover, the principal function of plastomeric modifiers is usually not to form a pervasive and continuous elastic network as with the block copolymers or latex; rather it is to produce a dispersal of discrete plastic inclusions throughout the bitumen which can impart increased rigidity that provides better resistance to high temperature (greater than 30ºC) permanent deformation (rutting) and modest improvements in fatigue cracking caused by repeated loading and unloading at intermediate (10º–30ºC) temperatures. [25, 26] In addition, these plastic inclusions can also aid in interrupting and therefore arresting the propagation of cracks. [3]

Comparing unmodified binders and those modified with polyethylene plastomers and various elastomers (SBR and CRM) Morrison *et al.* (1994) have shown that the plastomeric modifiers provide for substantial increases in the penetration index and measures of rutting resistance. [25] These results suggest that the polyethylene-modified binder tested (Dow Chemical Company's Tyrin® 2552) would offer enhanced rheological performance in those environments and during seasons where pavement temperatures meet or exceed 30ºC.

Some of the plastic modifiers such as EPDM represent hybrid combinations of elastomeric and thermoplastic characteristics. Indeed, EPDM is often classified as a form of synthetic rubber as well as a plastic, and it can be mixed with plastomeric additives such as HDPE to yield pavements that possess high temperature rutting resistance, and sufficient ductility at low temperatures to inhibit thermal cracking. [27] Greater detail on the use of polymer blends is given below.

Work with polyolefin modifiers indicate asphalt compatibility problems resulting in binder instability (segregation) when stored at temperatures in excess of about 150ºC. [28] Perez-Lepe, *et al.* (2006) have shown that segregation of the polymer phase occurs at comparatively short storage times in the form of creaming, and that this creaming is immediately preceded by widespread polymer coalescence brought about by the immiscibility between the bitumen and polyethylene fractions. Morrison *et al.* (1994) have demonstrated that the use of virgin or recycled tire rubber SB as a steric stabilizer in polyethylene modified asphalt emulsions, can interrupt this coalescence mechanism and yields a more stabilized mix.[29]

Yousefi (2003) suggests that as the melt flow index (MFI) of linear polyethylene polymers such as HDPE decreases, instability increases, making thorough dispersal within the bitumen problematic.[26] Moreover, branched polyethylene modifiers such as LDPE are easier to disperse than linearly structured equivalents. While high MFI polymers are easier to disperse, they have less of an effect on high temperature performance, but were shown to significantly improve low temperature behavior.[26]

Hesp and Woodhams (1991) note that polyolefin modifiers impart a wide range of beneficial characteristics to applied asphalt emulsions, including decreased thermal cracking and high temperature rutting, greater fatigue resistance, improved skid-resistance and increased stone retention.[30] Hesp also observes problems related to gross phase separation at elevated storage temperatures have inhibited the widespread adoption of polyolefin compounds in PME. Indeed, the authors note that without the use of a stabilizer, polyolefin-modified asphalt emulsions commonly have stable life-spans of only one hour or less. The findings of Hesp and Woodhams are in general agreement with those of Perez-Lepe, and they indicate that the primary mechanism of instability in polyolefin-modified asphalt emulsions is the coalescence of the polymer phase which eventually leads to creaming.[28,30] The most promising and cost-effective method for achieving mixture stability in such cases, is regarded to be the addition of steric stabilizers which are thought to secure stability by being preferentially absorbed at the polyolefin-asphalt interface.[28,30]

EVA is a commonly used plastomeric modifier which represents a copolymer of ethylene and vinyl acetate. By co-polymerizing ethylene and vinyl acetate, the latter serves to reduce the crystalinity of the former, resulting in increased elasticity and better compatibility with the base asphalt.[3] In EMA and ethylene acrylate modifiers, the crystalline structure of polyethylene is similarly reduced via the introduction of acrylic acid.[3] Panda and Mazumdar (1999) report decreased penetration and ductility and improved temperature susceptibility in EVA modified versus unmodified binders.[31] Additionally, EVA modified asphalts have been shown to retain their desirable physical properties even after prolonged periods of storage, and they do not appear to be adversely affected by minor variations in mixing methods or temperatures. [31]

Reclaimed waste plastic modifiers such as HDPE and LDPE have been shown to be somewhat effective in improving fatigue resistance, and in reducing penetration.[24,32] However, it is noteworthy that some stability problems with these mixes have also been reported, particularly at higher additive concentrations.[24]

Gerard *et al.* (2001) have compared the performance of plastomer-modified, elastomer-modified, and unmodified asphalt binders with respect to fracture toughness and crack propagation characteristics at low (-20ºC) temperatures.[33] It has been demonstrated that the use of polymer modifiers generally increases the facture toughness of asphalt binders. However, SB- and SBS-based modifiers exhibited substantially better fracture toughness than did comparable EVA and EMA modified mixtures owing to respective differences in crack propagation behavior as shown in figure 9. More specifically, Gerard *et al.* report that EVA and EMA modified mixes propagate cracks at the interface between the polymer and asphalt phases, leading to brittle behavior and stone pull-out (shelling). In contrast, the continuous polymer network formed in binders modified with elastomeric additives tends to stretch as the energy from the crack propagates through the polymer domains, impeding crack development in a phenomenon referred to as "crack-bridging".[33] In summation, the results suggest that SB and SBS modifiers provide for diminished low temperature susceptibilities as compared to similar EVA and EMA mixtures.

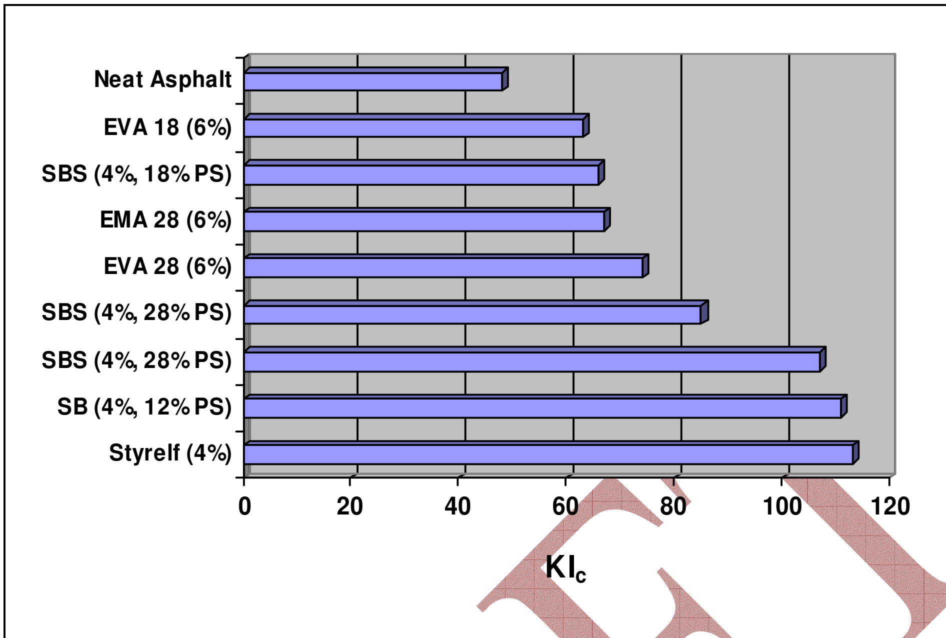


Figure 9: Fracture Toughness at -20° C(33)

2.2.7 *Polymer Blends*

Select polymer additives may be blended together to achieve desired composite properties that cannot be obtained from a single polymer modifier alone. Blending may prove a viable option when a particular polymer modifier has attractive availability and costs but does not give the rheological and performance characteristics that satisfy design requirements. In such cases, the addition of complementary modifiers may provide the means to satisfy the design specifications while permitting the use of the desired primary modifier. Additionally, supplemental modifiers are frequently added to improve the overall compatibility between the polymer and bitumen phases and to improve long-term mixture stability. While practical considerations preclude the exhaustive documentation of the numerous potential polymer combinations, examples of some of the most common blends in the literature are presented here.

Applications which use polyethylene as the primary modifier are frequently augmented via the addition of elastomers such as PB, to improve mixture stability and prevent segregation. (29) Morrison *et al.* (1994) report that polyethylene-modified asphalt emulsions can be effectively stabilized with either virgin PB or lower-cost de-vulcanized CRM. (29) The mechanism for increasing stability is the attachment of steric stabilizer molecules at the polyethylene-asphalt interface.

Ait-Kadi *et al.* (1996) report that blends of HDPE and EPDM produce improved penetration, loss of aromatics (aging) and viscosity, when compared to neat asphalt. (27) Comparisons of HDPE/EPDM blends to straight HDPE-modified asphalt in this study indicate little performance difference, although microscopic evaluation suggests that the blends generally yield a better distribution of the polymer phase. This characteristic has important cost and handling implications, since modifiers which are difficult to disperse translate into significantly higher energy requirements and longer mixing times. (34) More thorough and homogeneous dispersal of the polymer phase within the bitumen generally leads to improved mixture stability and increased potential storage life.

2.3 Polymer Modification Methods and Dosage Rates

The performance of polymer modifiers can be greatly affected by blending techniques, the percentage added, the types of aggregate used and the methods and temperatures of emulsion storage. This section discusses the impacts of mixing methodologies and conditions, dosing rates and storage and handling practices on the demonstrable field and laboratory qualities of polymer modified asphalt emulsions.

2.3.1 Polymer Modification Methodology

Table 2 is a summary of representative polymer modification methods and recommended dosage rates found in the literature. Table 2 shows that the modifiers may be added before emulsification to the emulsifying solution or asphalt, added to the finished emulsion product or "co-milled" at the colloid mill with the various component streams during production (see figure 3). The discussions below on test results of polymer modification methods are generally based on blends of specific polymers and specific asphalts. As mentioned above, the chemical and physical interactions of various polymer/asphalt blends can have significant affects on such results.(4)

Premixing with the soap solution is the generally preferred method of adding liquid latex to asphalt emulsions, followed by co-milling at the colloid mill. Becker *et al.* (2001) observe that the phase separation and stability problems associated with using solid polymer modifiers generally necessitate preblending the solid polymer in the asphalt at elevated temperatures prior to emulsification. (8)

Post-addition of the modifier to the final emulsion product either at the plant or the application site is sometimes discouraged due to the need for vigorous, continual and thorough mixing to ensure proper and homogeneous polymer dispersion. One notable commercial exception is the use of CRM-based RG-1, which is predigested with an organic solvent prior to being post-added to the emulsion.

Table 2: Polymer Modification Methods and Dosages

Type	Method	% Polymer Solids	Application(s)	Ref.(s)
SBR	Soap pre-batching. NO post or field addition.	3 – 4% of residual asphalt content.	Slurry Seals	(36)
SBR	Not Specified	3% of residual asphalt content	Various	(5)
SBR (Ultracoat™)	Dilute with water to 15% latex solids and blend with aggregate at collection hopper.	15% of total emulsion weight	Polymer anti-strip increases chip seal stone retention	(37)
SBR (Butonal LS 198®)	Soap pre-batching. NO post or field addition.	2 - 6% of residual asphalt content, usually 3%.	Various	(9, 38)
SBR	Soap pre-batching.	>=3% of residual asphalt content.	Microsurfacing	(39)
SBR, NRL, Neoprene, SBS, EVA	Preblend latex solids with bitumen using a high-shear blender. If latex in form, then use soap pre-batching.	2% of residual asphalt content.	Microsurfacing	(40)
SBR, NRL	Soap pre-batch, co-mill, or post add.	3 – 5% of residual asphalt content.	Various.	(41)
SBS	Preblend with asphalt.	5 – 12% of residual asphalt content.	Various	(42)
SBS	Preblend with asphalt binder.	> 5% of residual asphalt content (forms continuous polymer matrix).	Various HMA applications.	(17)
SBS, SB	Preblend with asphalt.	6% of residual asphalt content.	Various.	(43)
SBS, SB	Preblend with asphalt.	4% by weight of asphalt content.	Various low temperature applications.	(33)
CRM (RG-1)	Post-blended in-line directly with emulsion at plant and remixed before application.	5 – 8% of total emulsion weight.	Asphalt Rubber Slurry Surfacing	(44)
NRL (1497C)	Ralumac Process – Soap pre-batching.	4% of total emulsion by weight.	Various	10
EGA (Elvaloy®)	Preblend directly with binder.	1.5 – 2.0% of residual asphalt content.	Various HMA applications.	(45)
EVA	Preblend with binder.	5% by weight of asphalt content.	Various	(31)
EVA / EVM	Preblend with binder.	6% by weight of asphalt content.	Various low temperature applications.	(33)
EPDM, LDPE, HDPE	Preblend directly with binder.	5% of residual asphalt content.	Various HMA applications.	(34)
EVA, LDPE	Preblend directly with binder.	4 – 8% of asphalt content by weight.	Various	(35)
Any Appropriate	Soap pre-batch or preblend with bitumen.	3% of residual asphalt content	Microsurfacings	(46)
Polyethylene (Tyrin® 2552)	Preblend directly with binder.	3 – 5% of residual asphalt content.	Various	(25)
Various	Various	2 – 10% of residual asphalt content, 2 -3% most commonly.	Various	(8)

Forbes *et al.* (2001) examined the effect of four distinct and commonly used polymer modification techniques on asphalt binder microstructure at high temperatures:(47)

1.) Preblending – The polymer modifier is added directly to the bitumen prior to emulsification. This method is required for solid forms of polymer.

2.) Co-milling – Separate streams of polymer, bitumen, and emulsifier solution (soap) are co-milled together simultaneously.

3.) Soap Pre-batching – The polymer modifier is added to the soap solution (water and emulsifier) prior to milling with the bitumen.

4.) Post-Modification – The polymer modifier is added to the final asphalt emulsion either at the plant or in the field.

Properly cured residues from asphalt emulsions prepared using each of these methods were examined using laser-scanning microscopy to ascertain the structural network and distribution of polymer within the test samples. Microstructure comparisons were also performed with non-emulsified polymer-modified HMA binders.

Forbes *et al.* found that asphalt emulsions produced using either soap pre-batching or co-milling produced a slightly better distribution of the polymer than did post-modification. (47) Bituminous particles created within the colloid mill were found to have polymer modifier droplets layered around their surfaces (figure 10). When asphalt emulsions are prepared by soap prebatching or co-milling, latex particles are prevented from coalescing in the presence of the soap solution, but result in the formation of a thin film or matrix around the asphalt particles upon drying (figure 11).

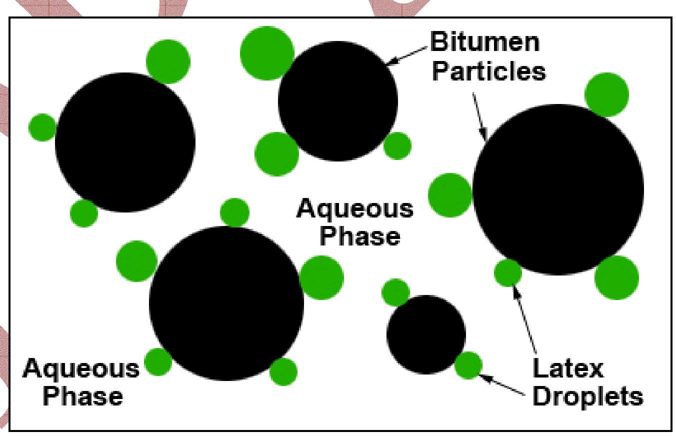


Figure 10: Bi-Phase Modified Emulsion (47)

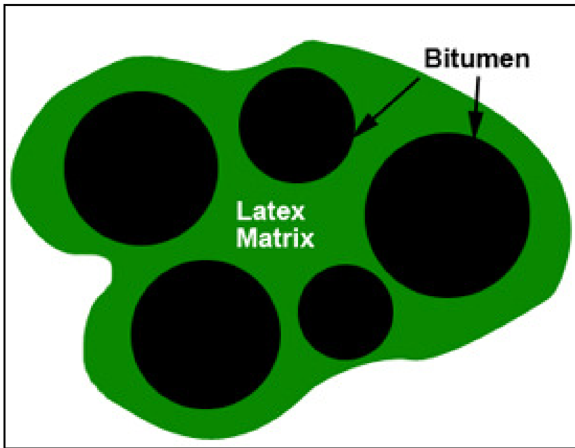


Figure 11: Polymer Network in Cured, Co-Milled Emulsion[47]

Examinations of non-emulsified asphalt binders which have been subjected to direct polymer modification indicate the presence of widely distributed polymer droplets of varying size, and numerous occurrences of discrete "swollen" polymer particles, indicating incompatibility between the polymer and bitumen phases. However, when preblended asphalts are emulsified, the resulting mixture exhibits well-distributed and discrete fine particles of polymer, areas of swollen polymer, and aggregated asphaltenes, representing a marked improvement in bitumen-polymer compatibility.[47] While co-milling and soap prebatch modification yield a biphase of asphalt and polymer, preblending produces a monophase of asphalt and polymer <u>after</u> emulsification as illustrated in figure 12. Preblending was shown to ultimately yield a much more homogeneous and more thorough distribution of polymer than did modified hot binders, suggesting that preblended polymer-modified asphalt emulsions may lead to more consistent cohesive strength performance, better elasticity, and improved stone retention characteristics than modified hot mix asphalt.[47] When lateral shear stress was applied to a dried preblended modified emulsion sample in the Forbes study, the polymer network was found to predictably elongate and resist deformation.[47] However, Forbes *et al.* caution that preblended asphalt emulsions do not produce a continuous polymer network as seen in co-milling or soap pre-batching mixes, and they recommend further investigation to determine if this structural difference might impact performance.

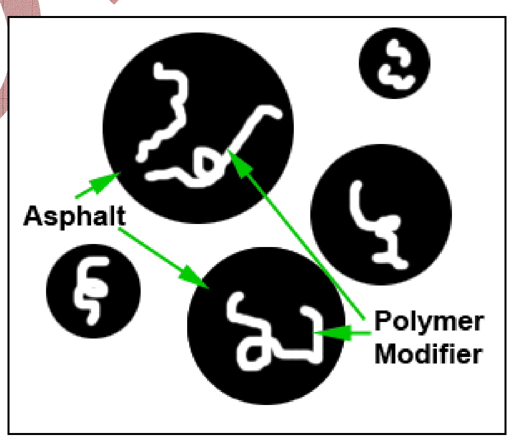


Figure 12: Pre-Blended Asphalt-Polymer Monophase [54]

Lubbers and Watson (2005) presented the results of analyses performed at BASF Corporation using stress-strain testing developed by Dr. Koichi Takamura[48] to gauge the relative fatigue performance of unmodified, preblended, and co-milled asphalt emulsion residues, as well as unmodified hot mixes. [5] The BASF testing consisted of the following steps:

1.) Strain sweep from a low of 0.1 percent to high of 5.0 percent applied for 30 minutes.
2.) Constant strain of 5 percent applied for 30 minutes.
3.) Strain reduced to 0.1 percent for 15 minutes to monitor potential recovery.
4.) Repeated steps 2 and 3 and measured change in residual strength.

A similar test sequence was performed on duplicate samples using a maximal stress of 10 percent. The test results indicate that unmodified asphalt emulsions are substantially weaker than neat hot-mix asphalt, due in large part to the failure of asphalt droplets in the former to fully coalesce, even within a 24-hour period. Conversely, asphalt emulsions modified with 3 percent SBR latex performed significantly better than did unmodified emulsions or neat non-emulsified asphalt cement. Of particular interest was the performance of the preblended SBS-modified emulsion samples, which demonstrated diminished viscoelastic recoveries as compared with conventionally co-milled SBR-modified emulsions. The reduced performance of the preblended asphalt emulsion was especially evident at the higher 10 percent strain level. [5] These results suggest that using preblended modified asphalts without continuous polymer networks in emulsions may yield reduced residual asphalt performance. Figure 13 illustrates fatigue resistance test result comparisons between unmodified, conventionally co-milled, and preblended modified asphalt emulsion residues.[5]

Similarly, an evaluation of preblended and co-milled SBR modified asphalt emulsions in chip seals performed by Takamura (2001) indicates that the formation of a honeycombed polymer network around the asphalt particles, results in a one to two Performance Grade (PG) improvement in rut resistance as compared to polymer-asphalt monophase mixtures.[14] Figure 14 illustrates a comparison of rutting resistance temperatures for neat asphalt, hot mix, emulsion residue, and cured residue after one week at elevated temperature (60ºC).

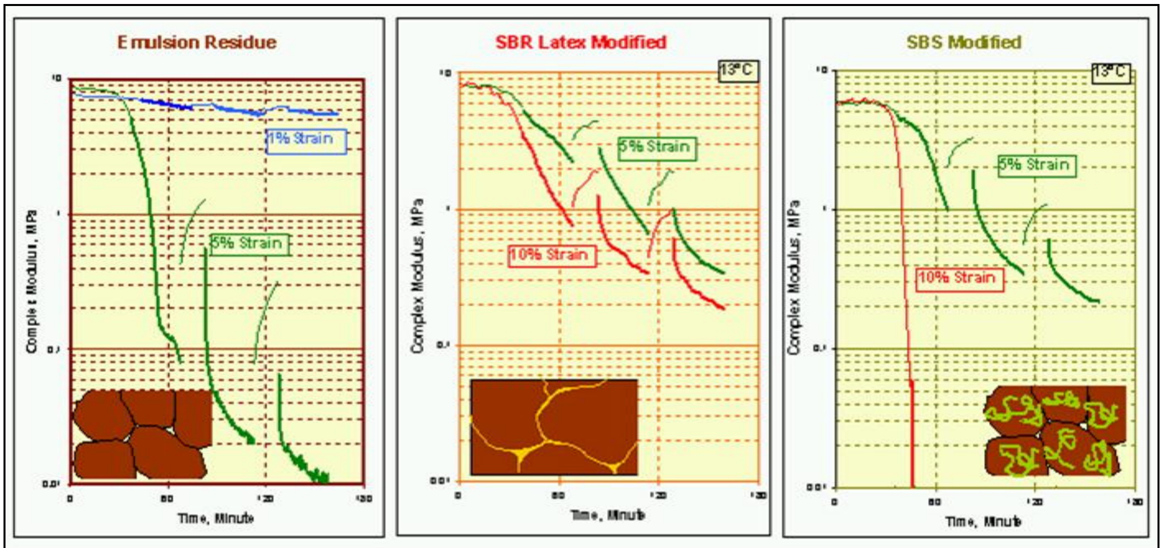


Figure 13: (L to R) Unmodified, Co-Milled, and Preblended Emulsion Test Results [5]

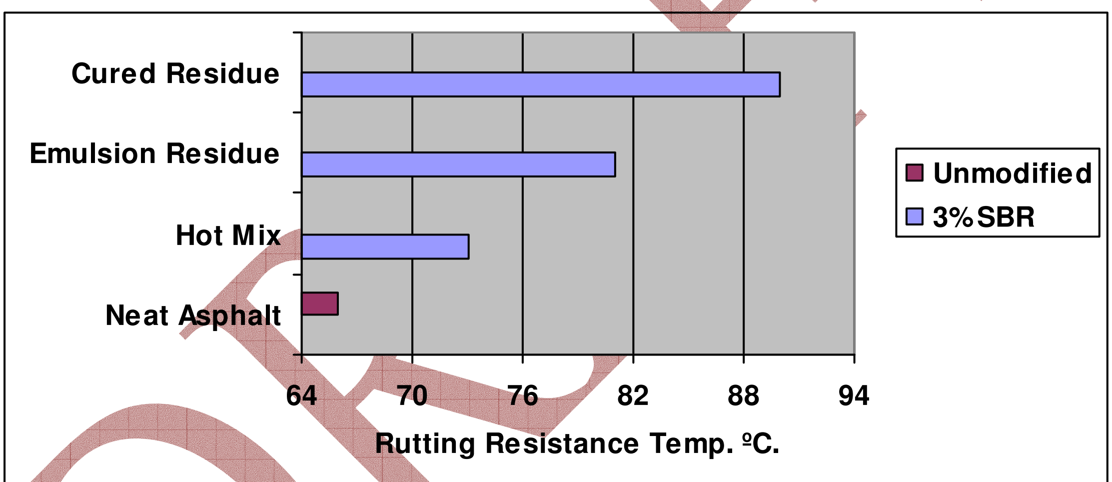


Figure 14: Advantages of SBR Network [12]

Takamura and Heckmann (1999) suggest that SBR latex has the advantage over SBS modifiers because SBR latex is able to be successfully added using co-milling, soap pre-batching or post modification methods, while solid SBS generally must be preblended with hot asphalt before emulsification. The researchers report observing the successful formation of a continuous polymer network in asphalt emulsions prepared with post-added 3 percent Butonal® NS198 (an SBR modifier), as well as significant improvements in laboratory measures of rutting resistance over unmodified binders, particularly at high (greater than 50ºC) temperatures. [49] However, no comparisons were provided between the performance characteristics of the various polymer mixing methodologies. Takamura and Heckmann further demonstrated that once formed, the resultant polymer network will remain intact, even when reheated to "hot mix" temperatures (200ºC).

Wegan (2001)[(50)] examined the impact of different polymer modification techniques, mixing times and temperatures and filler and aggregate types on the distribution of polymer additives in modified asphalt binders. This study involved the formulation of a variety of mix designs in the laboratory which upon curing, were cut and prepared as ultrathin sections which were subsequently subjected to UV light microscopic analysis. Polymer modifiers tested in the Wegan study included EVA, SBS, and a waste product material based on polyethylene (PE). Results indicate that polymer swelling increases substantially in cases where modifiers are preblended with the binder, versus those which are added directly to the final bituminous mixture (post-modification to asphalt and aggregate mix). Preblended polymer modified asphalt binders were also shown to provide increased contact and adhesion between polymer components and the surfaces of mineral grains in those mixtures where coarse-grained aggregate was used. Polymer was similarly found to be more pervasively distributed and to exhibit better aggregate contact characteristics in cases where mixing times and/or the quantity of the modifier used were increased. Wegan's temperature-related studies indicated that a mixing temperature of approximately 180º C provided for more homogeneous polymer distribution than did substantially cooler (160ºC) or hotter (200º C) temperatures.[(50)]

In test mixes where 7 percent EVA was preblended with the asphalt binder, Wegan reports observing the formation of a partial, yet distinct polymer network structure. Test samples containing 18 percent preblended EVA exhibited an even greater degree of polymer network formation. These results appear to suggest that in contrast to the findings of the BASF and Forbes studies, modified asphalt binders produced by preblending may produce a cross-linked network structure, providing that the polymer content is sufficiently high. However, no information is provided by Wegan with respect to comparing the performance of high polymer content preblended binders to conventionally modified lower content mixtures, or whether the increased materials cost of this form of preblend justifies its use. Wegan's studies were on binders for hot mix asphalt, not asphalt emulsions. The presence of the aqueous phase may account for the differences with the BASF and Forbes studies. The water-based latex emulsion facilitates dispersion of low percentages of polymer among the emulsified asphalt particles.

Hussein (2005) has examined the impact of polymer-asphalt blending time on PMA performance for varying molecular weight LDPE and EVA additives.[(35)] Figure 15 summarizes the change in complex shear modulus for various modified and neat asphalts relative to mixing time. Polymer modified mixes exhibit significant and well-defined increases in complex shear modulus (G*) as mixing time is lengthened, until a critical point is reached where upon these improvements begin to stabilize (and can decrease). For example, the steady-state points for 8 percent LDPE1, 8 percent EVA1 and 8 percent EVA2 are approximately 30, 15, and 20 minutes, respectively. In contrast, neat asphalt exhibits a virtually flat-line G* response over the same period. Hussein proposes that the point which represents stabilization in the magnitude of G* is indicative of the optimal blending time for that polymer-modified mixture. Results indicate that the optimal blending time for EVA-modified binders was generally less than for LDPE-modified mixtures, owing in part, to the lower weight-average molecular weights of the former. Hussein also found that binders containing low vinyl acetate content EVA additives exhibited the best high temperature susceptibility and long-term storage stability of the mixtures tested. However, little if any benefit was identified for these polymer additives at low temperatures.

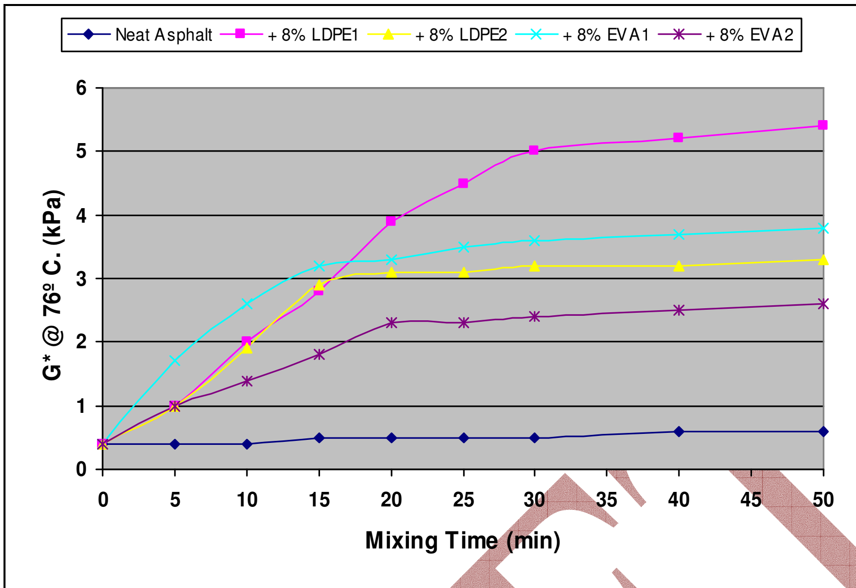


Figure 15: Complex Modulus over Mixing Time [35]

2.3.2 Polymer Dosing

As table 2 illustrates, the range of polymer content dosing recommended for most applications generally varies between about 2 percent and 10 percent by weight of the residual asphalt content with most research, standard, and manufacturer specifications calling for a polymer concentration of approximately 3 percent to 5 percent. The optimal percent depends upon the specific polymer, the specific asphalt and their interaction.

Chen *et al.* (2002) have examined the effect of SBS polymer content on laboratory-determined PMA performance. SBS contents were varied from 0 percent to 9 percent, and the resulting cured mixtures tested for ring-and-ball softening point, penetration and complex modulus by dynamic shear rheometer (DSR).[17] In addition, test samples were also subjected to structural analysis via transmission electron microscopy. Results of the Chen *et al.* study reveal that increasing SBS content resulted in increased polymer swelling, which in turn increased apparent asphaltene percentage (caused by maltene absorption by the polymer phase), leading to a harder matrix. Figure 16 presents the results of the softening point and penetration tests.

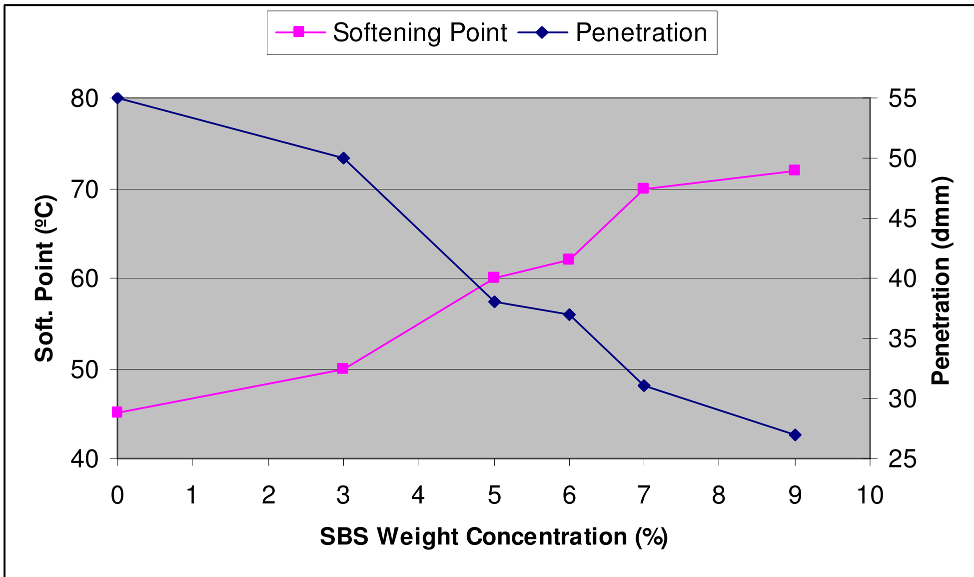


Figure 16: Effect of SBS Concentration on PMA [16]

As figure 16 illustrates, increasing SBS content resulted in substantially increasing softening point and lowering penetration characteristics up to a critical concentration of about 5 percent to 6 percent. Chen *et al.*[17] note that as the concentration of polymer reaches about 5 percent, the asphalt and polymer phases both become continuous; each phase forms an interconnected and interwoven matrix. At polymer concentrations in excess of 5 percent, the SBS becomes the dominant matrix, forming a continuous film around droplets of almost pure asphalt. Moreover, because improvements in softening point and penetration begin to stabilize at concentrations higher than about 6 percent, Chen suggests that this level of SBS is optimal for the particular asphalt tested (an AC-30).[17]

Figure 17 depicts the effect of SBS content on the complex shear modulus of test samples as measured using the DSR. As figure 16 illustrates, adding about 5 percent SBS results in an approximately 6-fold increase in the complex modulus over neat asphalt cement. Furthermore, increasing SBS content from 3 percent to 5 percent yields a proportionally larger increase in complex modulus than do increases in excess of 5 percent.

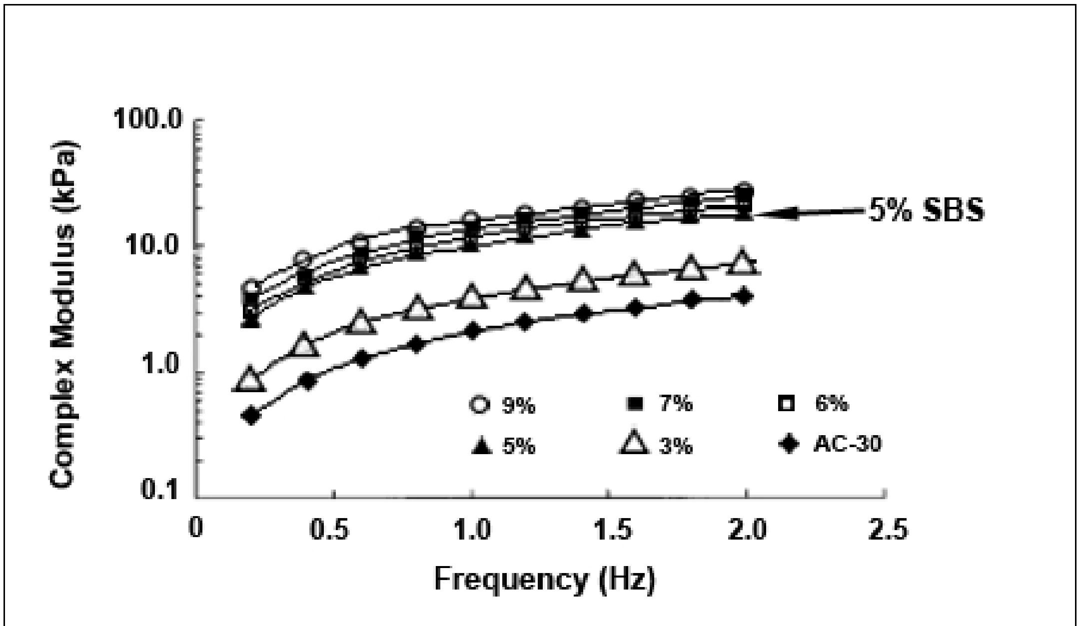


Figure 17: Effect of SBS Concentration on Complex Modulus at 60º C (16)

Thus, it is suggested that a polymer content of around 6 percent is required to generate the continuous polymer network which is believed to impart the desirable rubber-like elasticity characteristics associated with polymer modified binders. It should be noted however, that preblending was used to prepare samples for this study. Similar results were obtained by Airey *et al.* (2002), which indicate that SBS concentrations of 4 percent to 8 percent are required to establish a continuous polymer network with direct bitumen modification.[18] However, as previously discussed, others have shown that preblending certain systems may fail to result in the formation of a continuous polymer network unless the content of polymer added is sufficiently high to promote phase separation and swelling.[5,12,50] This would suggest optimal polymer contents presented in the Chen and Airey studies might prove to be higher than necessary than polymer modifiers such as SBR latex which can be co-milled or soap prebatched in an analogous PME emulsion application. It should be noted that at the point where the polymer becomes the sole continuous phase, the blend exhibits more of the physical characteristics of the polymer than the asphalt. That is, it becomes more cohesive and may have a softening point higher than typical use temperatures, making pumping and emulsification difficult for emulsions, and coating of aggregates difficult for HMA.[4]

Chen *et al.*[17] have also examined the impact of variable SBS concentrations on Brookfield viscosity (ASTM D789, D4878) as shown in figure 17. The researchers note that polymer modified binder pumping generally does not become problematic until mixture viscosities begin to exceed about 3,000 cP.[16] Thus, as figure 18 illustrates, SBS weight concentrations in excess of 6 percent appear to be contraindicated with respect to the materials handling and placement practicalities for modified AC-10 and A-30 asphalt binders.

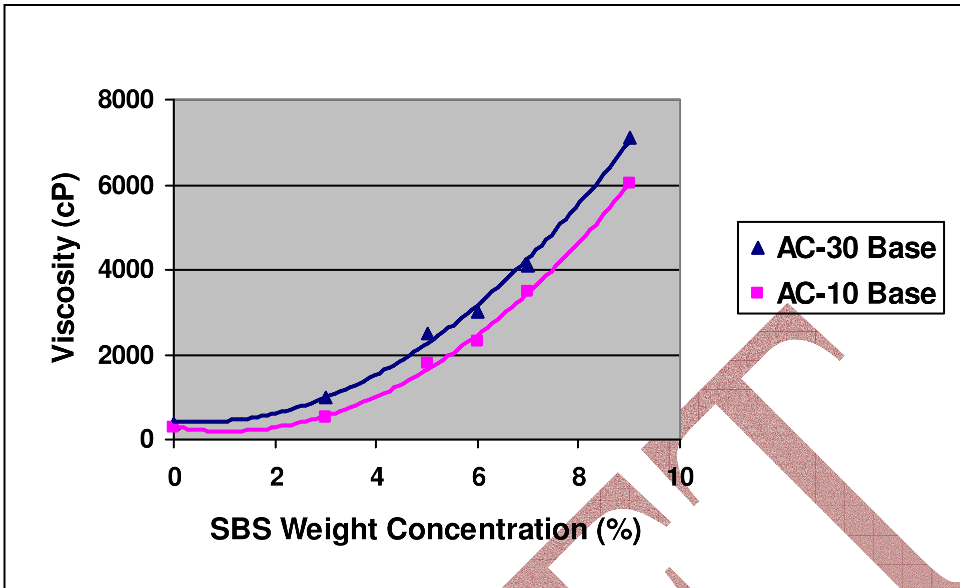


Figure 18: Viscosity as a Function of SBS Concentration [(12)]

Serfass *et al.* (1992) report that adequate SBS concentrations are required to ensure proper formation of a continuous polymer and asphalt network, and that it is this network which imparts the most desirable viscoelastic properties to modified asphalt binders. Within this context, the authors note that "adequate" is highly dependent upon asphalt compatibility, but is generally within the range of 3 percent to 5 percent by weight of residual asphalt.[(19)]

2.3.3 Storage and Handling Considerations

Proper storage conditions represent one of the most common problems associated with the use of PME. The mixing processes used are complex and often proprietary, and as such, modified binders are generally acquired in an already-blended form from the supplier. Once batched, some polymer modified asphalts and some polymer modified emulsions must be placed in special holding tanks that can be continuously agitated to prevent phase separation problems. Temperatures during storage also need to be strictly controlled to prevent setting, premature breaking (emulsions), and/or thermal destruction of the polymer modifier. As has already been demonstrated, the effective length of storage of polymer modified asphalt emulsions, even under ideal conditions, can vary widely depending upon the modifier and bitumen types, the degree of polymer-asphalt compatibility and the surfactant system used. Emulsions formulated for chip seals (rapid-setting) are designed to break quickly for early chip retention, while emulsions for slurry seals and microsurfacing (slow- and quick-setting) are designed to be stable enough to mix with aggregates and additives. Chip seal emulsions are therefore generally less stable than microsurfacing or slurry seal emulsions. When storing and handling prepared asphalt emulsions, the following general guidelines are recommended.[(1,51,52)]

- In general, store the emulsion between 10º and 85ºC, depending upon the intended use and the particular grade of emulsion. Specifically, rapid setting cationic chip seal emulsions should be stored at temperatures above 50°C to prevent premature breaking.

- Do not heat the emulsion above 85ºC during storage as this may cause excess water evaporation. Similarly, excessive and prolonged temperatures above 100ºC can cause breakdown of the emulsion and/or destruction of its polymer components.
- Avoid prolonged periods of storage, and make sure the mixture is gently and continuously agitated.
- Maintain an accurate temperature history and collect frequent measurements.
- Do not allow the asphalt emulsion to freeze, as this breaks the emulsion and causes phase separation and mixture instability.
- Do not use forced air to agitate the emulsion as this, too, may cause premature breaking.

DSR testing conducted after simulated aging with a Rolling Thin Film Oven (RTFO-DSR) of properly cured polymer modified emulsion residue indicates that unmodified asphalt emulsion contamination present within storage tanks or product transfer lines may adversely impact performance.(52) Similarly, some reductions in RTFO-DSR performance were noted with increasing storage times, which, when combined with product contamination, resulted in even more pronounced degradation of RTFO-DSR results.(52,53)

However, when modified non-emulsified asphalt binders were tested using comparable protocols, results indicate that the impact of prolonged storage, elevated temperatures, and contamination were substantially greater than were found during asphalt emulsion residue trials. It is hypothesized that this performance differential between modified asphalt binders may be due to the evaporation of water from the former, which provides a better barrier to oxidation, and hence aging.(52) Therefore, it is suggested that modified asphalt emulsion storage and handling protocols should focus primarily on preventing excessive water loss and phase separation rather than on aging-related problems.(53)

2.4 Performance

2.4.1 Performance Criteria

The performance enhancing characteristics of polymer additives are generally twofold: increased resistance to permanent deformation such as rutting, shoving and bleeding (high temperature susceptibility); and improved durability against load-associated types of pavement distress (fatigue cracking, aging and shelling). Polymers can also afford additional benefits by reducing the formation of non-load associated cracks caused by roadway brittleness which often occur in pavements that become excessively stiff and hard at low temperatures. In this regard, properly modified asphalts demonstrate improved temperature susceptibility characteristics by remaining flexible at low temperatures, while retaining sufficient stiffness at high temperatures to resist flow and permanent deformation.

Some initiatives have been undertaken to develop a "Superpave™-like" specification for surface applied asphalt emulsions. At present, ASTM D977-05 Standard Specification for Emulsified Asphalt uses few aspects of Superpave™ in its testing and characterization protocols. Hazlett (1996) asserts that many of the Superpave™ performance criteria, such as rutting resistance, thermal cracking, and RTFO aging, are not applicable to surface applied treatments.[55] Moreover, while some forms of Superpave™ testing could be extrapolated to polymer-modified emulsified asphalts, certain specification limits may not be appropriate for pavement surface conditions. However, Clyne *et al.* (2003) used Superpave™ specifications to test polymer modified asphalt emulsion residue for cold in-place recycling applications, in a manner similar to that of asphalt binder.[56] Comparisons of resulting data trends from emulsified and non-emulsified asphalt binder tests were similar enough to suggest that PG test protocols could be adapted to emulsion characterization, although further investigation is required to establish whether experimental results can be successfully correlated to field performance.[56]

Takamura noted that polymer modified asphalt emulsions can be successfully used in microsurfacing applications for filling ruts up to 5 cm deep.[54] The Portland cement used in microsurfacing significantly improves the rutting resistance of the asphalt binder, as shown in figure 20. This contradicts the contention by some that rutting resistance is an inconsequential measurement parameter when assessing polymer modified asphalt emulsion performance. Indeed, rutting resistance should prove a valuable indication of a rut-filling mixture's ability to resist future high temperature deformation.

Epps *et al.* (2001) have developed a Surface Performance Grading (SPG) system for asphalt emulsions based upon the modification of existing test protocols used under the standard PG system for HMA.[57] The SPG is designed to take into account the unique forms of distress common to surface course mixes, such as extreme high and low temperature performance, susceptibility to aging, stone loss (from chip seals), storability, and handling characteristics. Modifications to the standard PG system generally include adjustments to constant limiting values, as well as some changes to the actual testing protocols. For example, the PG procedure specifies that the designed high temperature limit should be determined at a depth of 20 mm below the pavement surface – a depth limitation which is not applicable to surface treatments. Thus, high and low design temperatures under the SPG are taken to be directly at the pavement surface.

Determinations of in-place asphalt emulsion performance are dependent upon the identification of key performance variables and the measurable physical and chemical properties of the asphalt binder or emulsion residue which relate to those variables. An extensive literature review conducted by the Strategic Highway Research Program (SHRP) has identified five (5) key variables for assessing pavement performance. These are:[58]

1.) Low Temperature Cracking (low temperature susceptibility).
2.) Fatigue cracking (repetitive loading/unloading).
3.) Raveling (stone loss).
4.) Rutting (permanent deformation, high temperature susceptibility).
5.) Aging.

Table 3 presents a matrix adapted from the SHRP review, depicting the reported relationships between various asphalt physical and chemical properties and each of the performance variables enumerated above. The arrows in table 3 indicate whether the performance criteria increases or decreases in magnitude as the corresponding physical or chemical property increases or decreases. For example, when viscosity increases, so do measured fatigue and low temperature cracking.

Table 3: Asphalt Properties and Pavement Performance [58]

Performance Criteria	Viscosity	Penetration	Ductility	Temperature Susceptibility	Binder or Mix Stiffness	Softening Point	Asphaltene Content	Naphthalene Aromatics
Low Temp. Cracking	↑	↓	↓	↑	↑			↑
Fatigue Cracking	↑	↓	↓					
Raveling	↓		↓				↑	↓
Rutting				↓			↓	
Aging	↑	↓	↓	↓		↑	↑	↓

However, in developing the SPG, Epps generally discounts the importance of rutting and thermal cracking in surface treatments, focusing instead on the more typical emulsion requirements of:

1.) High and low temperature behavior which can lead to aggregate loss.
2.) Aging performance.
3.) Application and handling characteristics of the prepared emulsion.[57]

Conversely, rutting resistance can prove a valuable test parameter when assessing the performance of rut-filling mixes such as microsurfacing.[54] Takamura has observed that the action of radial truck tires actually produces higher than average critical shear stresses on thin surface treatments such as chip seals and microsurfacing, as compared to full or partial thickness HMA (see figure 19). This underscores the importance and value of estimating the high temperature susceptibility and stone retention capacity of modified surface treatments.

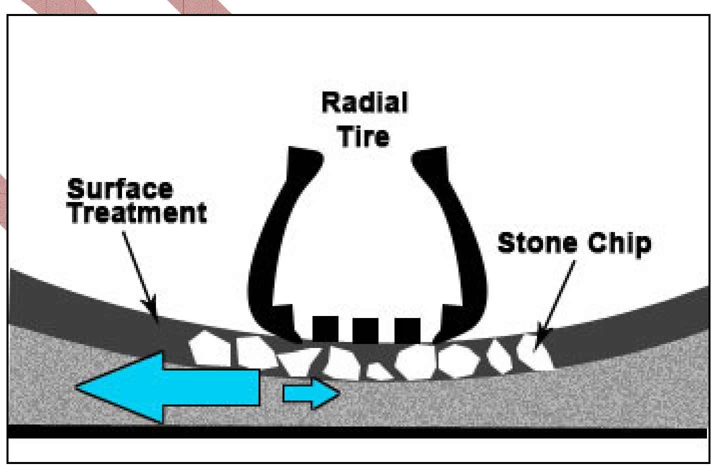


Figure 19: Influence of Radial Tire on Surface Treatment [54]

It is noteworthy that the relationships between laboratory-determined binder physical properties and actual field performance are not always clear, and substantial evidence exists which is often contradictory. For example, it has been shown through stress-controlled fatigue tests that stiffer mixes are more resistant to fatigue cracking, whereas strain-controlled tests indicate that softer mixes are more fatigue resistant. [16] Moreover, because polymer modified asphalt binders are used as thinly-applied surface treatments, the physical parameters used to characterize the performance of HMA mixes (such as the PG specification) may not always be applicable.

The search for physical parameters and related laboratory tests which can be used to accurately characterize the performance of PME is on-going. The following section discusses some of the information obtained from the literature review which pertains to the use and adaptation of various innovative and routine testing protocols that have been, or which may be utilized for the analysis of PME residue performance. Section 4 gives some protocols developed to use Superpave type methods that more accurately characterize the desired properties of polymer modified asphalt emulsion applications.

2.4.2 Testing Protocols and Considerations

For successful PME applications, the emulsion must exhibit acceptable performance during storage, shipping and construction. It must remain stable, it must lend itself to effective construction and it must break (phase separate) at the appropriate time. The cured emulsion residue must also exhibit the expected performance for its end use on the pavement. It is necessary, therefore to test both the emulsion and the residue as it would be on the finished pavement. Analysis of the cured residue properties may be accomplished by directly collecting a sample of the non-emulsified binder or by extracting the properly cured residue from a prepared emulsion sample. Typical residue extraction techniques include:

- Stirred Can Method – This method involves constantly stirring a sample of the emulsion for 170 minutes at a temperature of 163ºC to evaporate and drive off the water. A blanket of nitrogen gas is used to dampen the effects of oxidation. Although this method yields abundant quantities of testable residue in fairly short-order, it has been criticized as not accurately representing actual field conditions due to the high continuous temperatures which are used. [59]

- RTFO Method – This methodology described by Takamura (2000) is a variation on the RFTO test used to simulate aging in the hot mix plant. [60] Samples of the emulsion are rolled in bottles in a temperature-controlled environment at 85º C for 75 minutes with a stream of heated nitrogen gas jetted over the emulsion film to facilitate water evaporation. This method has also received criticism because it can lead to incomplete water evaporation in certain asphalt emulsions such as CRS-2P, producing inconsistent follow-up test results. [59] However, some suggest that this method may be useful for quality control purposes at emulsion production sites since it permits for the rapid extraction of testable quantities of residue. [60]

- Forced Air-Drying Method – This extraction technique uses forced air flow at ambient (22ºC) temperatures to facilitate water evaporation. Although this method is generally regarded as the most representative of actual field conditions, it is a lengthy process to complete (300 to 360 minutes) and approximately one day is required to prepare the sample for extraction.[59]

- Vacuum Distillation Method – The sample is placed into a vacuum distillation unit at a temperature of 115ºC. Takamura (2000) has noted that microscopic examinations of samples extracted through distillation exhibit undesirable changes in polymer network morphology including cross-linking and polymer decomposition owing to the application of excessive heat.[60] These changes can lead to viscosity inconsistencies and the degradation of other performance measures. Thus, it is suggested that vacuum distillation may only be appropriate for determining the presence of polymer, not for ascertaining the placed network structure.

- Forced Draft Oven – This technique is described in more detail in chapter 3. It has the advantage of obtaining cured emulsion residue at a temperature closer to field temperature, but it also takes longer than traditional residue recovery methods.

- Moisture Analyzer - A technique used to determine asphalt content, but currently yielding very little residue.

Key factors which should be considered when selecting a residue extraction methodology include:

- Reproducibility – Residue samples repeatedly extracted from the same emulsion mix should yield statistically similar results when subjected to testing techniques such as DSR, softening point, penetration, etc. Extraction techniques that tend to yield widely divergent physical property test results are not suitable for insuring accurate characterization of modified emulsion performance.

- Time – Various extraction methods have different processing time requirements which must be considered from a logistical standpoint. For example, lengthy extraction techniques may not be appropriate for use at the emulsion production site if accurate test results cannot be obtained in a timely manner prior to field placement.

- Cost – Differences in sample preparation time, extraction time, and equipment requirements can translate into varying costs between methods.

- Accuracy and Representativeness – A balance must be achieved between time/cost considerations, and the testing accuracy that can be realized with an associated extraction methodology. Similar consideration should also be given to how representative an extraction technique is relative to actual field evaporation and curing conditions and whether a particular method might fundamentally alter the character of the residue in a way that distorts physical property test results.

As discussed previously, extracted residues or samples of non-emulsified binder material may be subjected to a wide variety of testing modalities to estimate field performance. Typical forms of performance testing include (but are not necessarily limited to):

- DSR – to predict rutting resistance and high temperature susceptibility. Useful for polymer modified asphalt emulsions employed in rut-filling applications.
- RTFO – to simulate the effects of aging/oxidation.
- PAV – to simulate the effects of long term field aging.
- Ductility – to estimate the potential for fatigue and thermal cracking and/or raveling.
- RV – used to gauge cracking susceptibility, and raveling potential through viscosity measurements.
- BBR – low temperature susceptibility and thermal cracking potential.
- Vialit – measures stone retention characteristics.
- Penetration – to estimate cracking potential and mixture consistency.
- Wheel-Track Test – used to simulate wheel traffic loading and unloading to ascertain rutting-resistance.
- Loaded Wheel Test – used for slurry seals and microsurfacings to compact the sample as a means of assessing the mixture's susceptibility to flushing.
- Wet Track Abrasion Loss – used to measure the wearing characteristics of slurry seals and microsurfacings under wet track abrasion conditions.
- Ring and Ball – to determine stiffness failure at high temperature. Usually used as a consistency check on polymer modified asphalts.
- Schulze-Breuer-Ruck – used to evaluate the compatibility between bitumen, aggregate, filler and polymer modifier in microsurfacing.
- Zero Shear Viscosity – proposed as an alternative to $G^*/\sin\delta$ as a measure of rut-resistance. Also used in highly modified mixtures to estimate the degree of polymer network formation.
- Infrared Spectroscopy (IR) and Nuclear Magnetic Resonance (NMR) – used to verify the presence and relative abundance of polymer modifiers.(4)
- High Performance Gel Permeation Chromatography (HPGPC) – used to characterize the molecular weight and physical size of polymer modifiers.(4)

Emulsion recovery tests are run to determine asphalt content and the properties of the cured material on the pavement. The former can be evaluated using one of the extraction procedures described previously to determine residual asphalt content. The Long-Term Asphalt Storage

Stability Test (LAST) was proposed to estimate thermal degradation and phase separation potential.[4]

Typical physical property testing techniques for asphalt binders and emulsion residue have traditionally focused on determinations of viscosity, penetration, ductility, and softening point temperature. However, these tests often fail to accurately and comprehensively characterize the performance characteristics associated with PME.[16, 48] Most researchers now advocate oscillatory DSR testing as the method of choice for characterizing the viscoelastic properties of modified residue and binders. [16] In this procedure, binder or emulsion residue sample is placed between two plates in a DSR device and subjected to oscillating shear stress and strain for the purpose of determining the complex modulus (G*, a relative measure of stiffness) and the phase angle (δ, the elastic response) of the material. Takamura (2005) has further proposed a variation on the DSR procedure specifically for modified emulsion residues, which consists of the following sequence of three testing intervals:

1.) Strain Sweep - Strain is gradually increased from 0.1 to 5.0 percent in 35 minutes and is used to evaluate rheological properties of the binder at wide strain levels.

2.) High-Constant Strain – constant strain (1 percent, 5 percent, or 10 percent) is applied immediately after the first period of strain sweep for a period of 30 minutes.

3.) Relaxation – After the end of the second period of high-constant strain, the sample is permitted to relax for a period of 15 minutes with only a minimal strain of 0.1 percent which is used to observe the recovery of G*.[48]

The sequence above is typically repeated at least two more times on the same sample to illustrate the progressive loss of G* as shown in the example provided in figure 13. The results of this test provide an indication of the relative fatigue resistance of various mixtures under the high-strain deformation forces which might be created by radial truck tires and/or snowplow blades.[48]

In contrast, Airey (2004) reports that the phase angle (delta, δ) is usually considered to be much more sensitive to the structure of the binder than is G*, and as such, provides a better indication of the type and extent of polymer modification.[16] Within this context, smaller δ values are indicative of a greater elastic (less viscous) response, and thus, suggest a higher degree of polymer network formation, particularly at higher temperatures.

King *et al.* (1998) noted that at comparatively high polymer levels, viscosity can increase substantially, leading to an over-prediction of rutting resistance, while DSR high temperature parameters and Wheel-Tracking test results are generally found to be more representative and in good agreement with one another.[4] Moreover, ductility testing on binders modified with elastomeric polymers can exhibit significant variability at low to intermediate temperatures (4º–25ºC). In this regard, Neoprene and SBR modifiers generally produce comparatively high ductility, while SB and SBS additives yield much lower ductility values.[4] King characterized the low ductility of the latter as a function of "too much" rather than "too little" strength, as the elongated strands of SB/SBS modified asphalts in the ductility test are comparatively thick and snap back much in the way a thick rubber band does when pulled too far. [4] This suggests that with some SB and SBS modified mixes, ductility testing could under-predict performance measures of strength.

Desmazes *et al*. (2000) have developed a testing protocol for measuring the zero shear viscosity (ZSV) which the authors assert provides for a more accurate estimate of rut-resistance in binders modified with certain elastomeric polymers (e.g., SBS). [61] Conceptually, ZSV represents the viscosity of a fluid which is at rest. In elastic mixes at very low shear rates, the structures of the fluid deform slowly enough to reach equilibrium. Measurements are collected at lower and lower shear rates, and the results are extrapolated to yield the zero shear viscosity. Demazes observes that rutting is a demonstrably slow process, and, as such, the "resting" viscosity of a modified binder more closely approximates its capacity to resist permanent deformation. [61] In contrast, studies have shown that conventional DSR testing tends to underestimate high temperature performance in modified binders characterized by high delayed elasticity.

The SPG developed by Epps (2001) uses the following modified testing program: [57]

- <u>Residue Recovery</u> – the researchers use the Stirred-Can method.

- <u>Aging</u> – pavements located at the surface are most susceptible to aging. RTFO developed for simulating aging in a hot mix plant was discarded due to the comparatively low application temperatures associated with emulsion surface treatment applications. A Pressure Aging Vessel test (PAV) was used instead for long-term aging only.

- <u>RV</u> – viscosity was determined for unaged binders, as this parameter generally reflects how easily the resulting asphalt emulsion can be pumped and sprayed. Multiple temperatures were used to simulate the wide range of typical surface treatment application temperatures, as opposed to the single temperature (135ºC) used to determine workability for HMA binders under the standard Superpave PG protocol.

- <u>DSR</u> – DSR testing was performed in accordance with AASHTO TP5 on the unaged binders to determine G* and δ values to assess early, high temperature performance. The researchers believe aggregate loss is of greater significance for surface treatments than are rutting or shoving at high temperatures.

- <u>PAV-DSR</u> – residues were long-term aged using PAV and then tested using the DSR to assess intermediate temperature range performance. More specifically, this test was intended to evaluate the potential for aggregate loss rather than fatigue cracking.

- <u>BBR</u> – BBR testing was performed on long-term aged residues to evaluate low temperature behavior. For this test, the fastest BBR loading time (8 sec.) was used to simulate critical traffic loading conditions, rather than to gauge thermal cracking.

The final recommended limiting values proposed for the SPG are presented in table 4.

Table 4: Recommended SPG Limiting Values [57]

Viscosity	DSR	BBR
ASTM D 4402 Max: 0.15; Min: 0.1 Pas	G*/Sin δ , Min: 0.750 kPa Test Temp. @ 10 rad/s, °C	Creep Stiffness, TP1 S, Max: 500 MPa m-value, Min: 0.240 Test Temp., @ 8 s, °C

2.4.3 Evaluation of Existing Federal Lands Standards

The Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects (FP-03) calls for cationic and anionic emulsified asphalts to conform to AASHTO M 208 and AASHTO M 140, respectively. [62]

Polymer modified asphalt emulsions used for microsurfacing are further specified to meet the requirements of AASHTO M 208 as well as the following:

- Residue by distillation: 62 percent minimum.
- Softening point: 57° C minimum.
- Penetration at 25ºC: 40-90.

Current Federal Lands Highway (FLH) specifications direct that polymer additives are to be blended either into the asphalt directly or the emulsifier prior to emulsification.

Table 5 presents the key physical property parameter requirements specified under AASHTO M 208 and M 140 (i.e., ASTM D2397-05 and ASTM D977-05, respectively) for comparison and discussion purposes.

Table 5: Summary of M 208/140 Specifications

Emulsion Type	Viscosity, Saybolt at 22º C †	Viscosity, Saybolt at 50º C †	Demuls †	Min. Residue by Distillation †	Penetration at 25º C ‡	Ductility at 25º C ‡ (cm)
Anionic Emulsions and Residues (M 140-86)						
RS-1	20 - 100	--	60	55 percent	100 - 200	40
RS-2	--	75 - 400	60	63 percent	100 - 200	40
MS-1	20 - 100	--	--	55 percent	100 - 200	40
MS-2	100	--	--	65 percent	100 - 200	40
MS-2h	100	--	--	65 percent	40 - 90	40
HFMS-1	20 - 100	--	--	55 percent	100 - 200	40
HFMS-2	100	--	--	65 percent	100 - 200	40
HFMS-2h	100	--	--	65 percent	40 - 90	40
HFMS-2s	50	--	--	65 percent	200	40
SS-1	20 - 100	--	--	57 percent	100 - 200	40
SS-1h	20 - 100	--	--	57 percent	40 - 90	40
Cationic Emulsions and Residues (M 208-86)						
CRS-1	--	20 -100	40	60 percent	100 -250	40
CRS-2	--	100 - 400	40	65 percent	100 - 250	40
CMS-2	--	50 - 450	--	65 percent	100 - 250	40
CMS-2h	--	50 - 450	--	65 percent	40 - 90	40
CSS-1	20 - 100	--	--	57 percent	100 - 250	40
CSS-1h	20 - 100	--	--	57 percent	40 - 90	40

† Applies to liquid asphalt emulsion
‡ Applies to asphalt emulsion residue

As has already been covered in some detail, the literature review unequivocally illustrates that polymer modified asphalt binders (i.e., PME and PMA) exhibit significant performance benefits over unmodified equivalents.[4,5,12,14,20,21,24,25,31,33,48,49] Demonstrable benefits include increased rutting resistance, improved chip/stone retention, improved elasticity and ductility, increased fracture toughness, improvements in the penetration index, decreased low and high temperature susceptibility and improved fatigue resistance. Although polymer blending techniques appear to impact mixture performance, all of the methods examined performed better when compared to unmodified binders.

2.4.4 Modified versus Unmodified Asphalts

Khosla and Zahran (1988) compared the performance of unmodified and Styrelf® polymer modified mixtures of three commonly used asphalt cements: AC-5, AC-10, and AC-20.[63] Styrelf® is a proprietary blended modified asphalt product produced by Total™, which uses a cross-linked SB elastomeric polymer additive. Khosla and Zahran evaluated each asphalt preparation under varying load conditions and operating temperatures using the resilient modulus test, and reported that they were able to predict the fatigue, deformation and brittleness of each of the binders. These test results were then used to simulate the predicted service life using the VESYS III computer model in each of the four major climatic regions as shown below in table 6.

Table 6: Predicted Service Life (years) After Khosla [(63)]

Region	Temp. Range	AC-5	AC-5 Styrelf®	AC-10	AC-10 Styrelf®	AC-20	AC-20 Styrelf®
1	18-30ºC (0-90ºF)	9.83	15.90	11.96	17.13	15.10	19.01
2	4-30ºC (40-90ºF)	6.24	14.39	8.04	16.55	11.94	18.53
3	4-49ºC (40-120ºF)	5.02	12.81	6.04	14.92	10.40	16.39
4	4-60ºC (40-140ºF)	NA	10.32	NA	12.76	6.63	14.21

As table 6 suggests, in each case the Styrelf® asphalt mixtures appeared to yield significant improvements in overall predicted service life as compared to their unmodified parent asphalts. The performance impacts of polymer modified binders were further evaluated specifically with respect to predicted rut depth, fatigue cracking, and low temperature cracking for various service year benchmarks. Quantitatively, Khosla and Zahran estimated the approximate resulting magnitude of rut depth and the degree of fatigue cracking (using cracking indices) over time. Additionally, low temperature cracking susceptibility was determined by a stiffness value that was formulated based upon creep tests conducted at temperature benchmarks of -29ºC (-20ºF), -18ºC (0º F), -7ºC (20º F), and 4ºC(40ºF), respectively. Khosla and Zahran conclude that:[(63)]

- Styrelf® mixtures have better low temperature susceptibility than their unmodified counterparts and are therefore less brittle.
- Styrelf® asphalts are more resistant to low temperature cracking.
- The Styrelf® samples exhibited a reduced propensity for rutting deformation at higher temperatures than the unmodified asphalts.
- Polymer modification of Styrelf® asphalts results in improved fatigue life.

In figure 20, Takamura (2002) compares the high temperature performance of modified and unmodified asphalt emulsions in microsurfacing applications, as shown by the temperatures where Superpave rut failure criteria are met. The modified asphalt emulsion residues show significantly better rutting resistance than unmodified mixtures.[(54)]

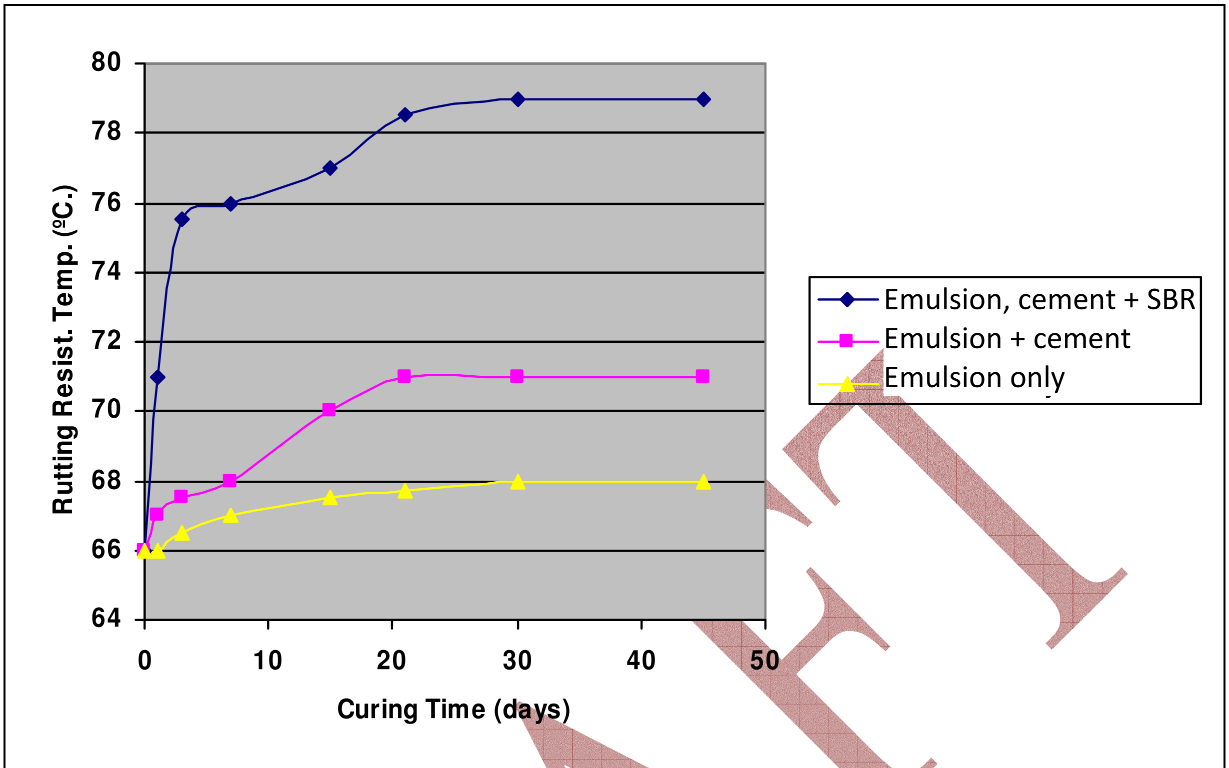


Figure 20: Microsurfacing Emulsion Residue Curing Time [54]

2.4.5 Modified Emulsion versus Modified Hot Mix Binders

Serfass *et al.* (1992) have compared the performance of SBS modified hot mix and emulsified asphalt in thin surface treatments using laboratory tested rheological properties, cohesion, stone retention, tensile strength, and durability.[19] Results from this study indicate that the studied SBS modified hot mixes exhibit poor adhesion to the study aggregate and require the use of an antistripping agent. Moreover, the use of anti-stripping agents in SBS modified hot mixes yields only modest improvements which decline under more adverse climatic conditions.[19] In this regard, Serfass *et al.* report that the use of SBS modified hot mixes is contraindicated in cooler environs, and that SBS modified asphalt emulsions offer a longer application season, performing well under cool and even damp conditions. The authors also note however, that SBS-modified asphalt emulsions require a much longer set time than do their hot mix counterparts. In addition, Serfass *et al.* report that higher SBS contents may be used in asphalt emulsions, since modified hot mixes exhibit decreased adhesion and problematically high viscosities when higher SBS concentrations are used.[19]

Gransberg and Zaman (2005) examined the relative performance and cost effectiveness of 342 chip seal projects in the State of Texas to compare the efficacy of hot mix binders to asphalt emulsions.[64] The results of this study indicate that PME performs at least as well as modified hot mix binders, and that the former does so at a lower cost while offering modest improvements in skid resistance and ride quality.[64] The Texas Department of Transportation (TxDOT) generally uses asphalt emulsions in their chip seals on lower volume (< 2,000 ADT) roadways. Moreover, these asphalt emulsions are typically applied to pavements that are generally in poorer condition as compared to hot applied chip seal projects. In such cases, TxDOT differentiates between asphalt emulsion and hot applied chip seal applications based primarily on traffic volumes, because the latter requires a shorter curing time and as such, reduces lane closure times and traffic delays.

2.5 Surface Application Types

2.5.1 General

This section presents those findings of the literature review specific to common surface treatment applications where polymer modified asphalt emulsions may be employed. Among the treatment applications examined are chip seals, slurry seals / microsurfacing, and cape seals. The benefits and limitations of PME are examined with respect to each specific treatment type, and where applicable, compared to the performance of non-modified asphalt emulsions.

2.5.2 Chip Seals

Chip seals (sometimes called seal coats or bituminous surface treatments) consist of an asphalt emulsion which is spray applied to the pavement surface and then immediately covered with a layer of aggregate (chips) and rolled to seat the aggregate. Chips seals are commonly employed as an inexpensive treatment for minor forms of pavement surface distress such as cracking or raveling and as a cost-effective preventive maintenance (pavement preservation) treatment.

The advantages of using polymer modified asphalt emulsions in chip seal applications over non-modified mixtures include:(14)

- Better early and long-term stone retention.
- Quicker traffic return.
- Reduced rates of flushing and bleeding.
- Increased durability on higher volume roadways (due to improved stone retention).
- Greater design tolerance for chip and asphalt emulsion quantities and aggregate embedment factor.

Takamura (2003) demonstrates the impact of polymer modifiers on improving stone retention in chip seals.(65) Figure 21 presents a comparison of retained aggregate percentages between modified and unmodified variants of eight mixtures—each containing different aggregates—from an early strength sweep test. As figure 21 illustrates, improvements in aggregate retention range from modest to dramatic in the polymer modified (BASF's Butonal™ NX1118) chip seal mixes in all eight test cases, with percentages near or above 90 percent.

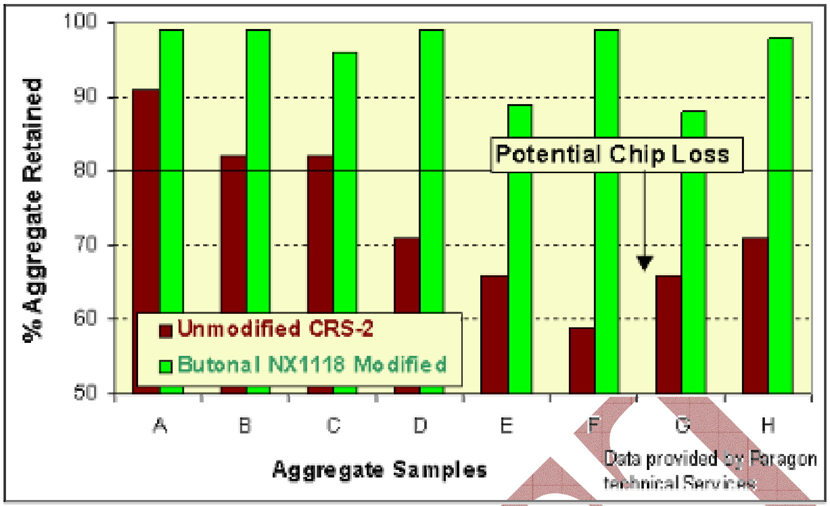


Figure 21: Chip Seal Aggregate Retention with SBR Latex Polymer [65]

Windshield damage caused by the displacement of stone is perhaps the most widely reported early difficulty with of chip seals. For this reason, many agencies restrict the use of chip seals to relatively low volume (< 2,000 ADT) roadway pavements. Therefore, because polymers offer demonstrably improved rates of aggregate retention, it is suggested that modified chip seals could provide acceptable performance on higher volume roads. Several field studies have shown excellent performance of chip seals on very high volume roads.[4,66]

Moreover, Lubbers and Watson have also shown that Vialit chip retention test results are markedly better in modified chip seals at low temperatures than are comparable unmodified mixtures, indicating polymers may similarly prove valuable in cold weather climates (figure 22).[5]

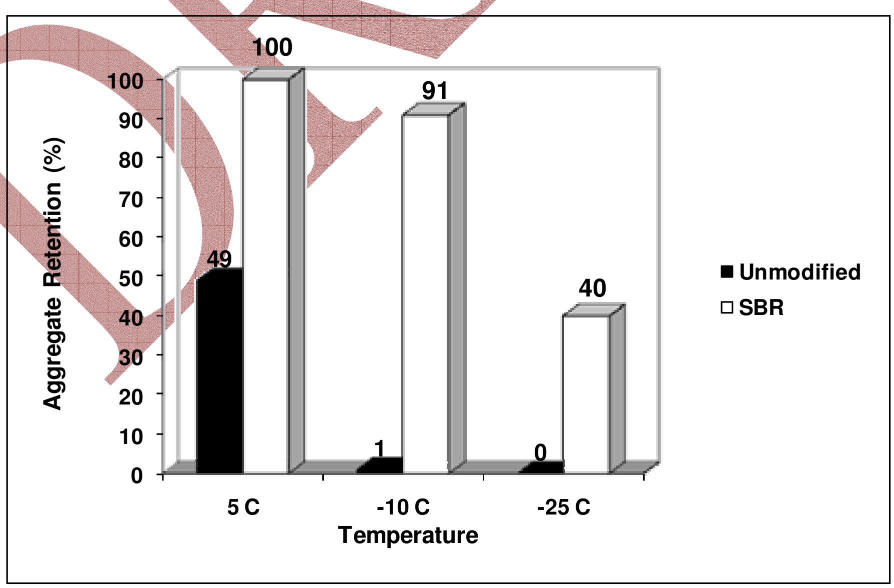


Figure 22: Vialit Chip Retention at Low Temperatures Chip Seals [5]

Wegman (1991) notes that the improved early chip retention offered by polymer additives when used in chip seals allows for greater variation in aggregate and emulsion application rates, and permits earlier sweeping of the applied surface which serves to mitigate windshield damage. [67]

A survey of chip seal best practices by Gransberg and James (2005) indicates that early brooming of chip seals immediately after rolling to remove loose stone may be ill-advised since curing at this stage is generally insufficient to permit proper binder to aggregate bonding.[68] More specifically, although polymer modifiers can significantly enhance stone retention, research has shown that adequate cure times are needed to realize this benefit (see figures 7 and 21).[14, 65] Gransberg observes that chip seals can be successfully applied to high volume roads, providing allowances are made for adequate curing time, and that the underlying pavement condition of the roadways selected for treatment are fundamentally sound.[68] Moreover, detailed assessment of chip seal performance nationwide indicates that the best performing chip seals are those where design specifications are meticulously prescribed, implemented and verified by the highway agency.[68]

2.5.3 Slurry Seals and Microsurfacing

Slurry seals consist of a homogeneous mix of crushed aggregate and an asphalt emulsion which is applied to the pavement surface as a single-pass monolayer. Some slurry seals contain polymer, others do not. Curing of the slurry seal coat occurs as the water evaporates, leaving only the residual asphalt to coat the aggregate surfaces. In general, slurry seals contain a high proportion of fines which generally improves skid-resistance and water-resistance. Slurry seals are generally applied to only lower-volume (< 1,000 ADT) roads.

Microsurfacing is a commonly used form of slurry sealing consisting of a combination of mineral aggregate and fillers, a polymer modified asphalt emulsion and other additives. The primary difference between microsurfacing and other forms of slurry sealing is the chemical formulation which generally yields an instantaneous, chemical break. Generally, the specifications and design procedures for microsurfacing are more stringent than those for slurry seals. By definition, microsurfacing contains polymers, while slurry seals may or may not contain polymers. Slurry seals are generally laid at thicknesses of 1 to 1.5 cm, whereas microsurfacing can be thickly applied in multiple layers. Slower breaking slurry seals cure on the surface "skinning over" and preventing thorough breaking and curing when they are applied at greater thicknesses. The PME used in microsurfacing break chemically instead of through evaporation which occurs in slurry seals and some other asphalt emulsion applications. This permits the microsurfacing to gain cohesive strength rapidly, thereby minimizing lane closures and traffic delays. [69] Microsurfacing is commonly used to correct wheel-path rutting and improve skid-resistance, and can be applied to either high or low volume roadway pavements, and may be used over both asphalt and Portland cement concrete pavements. [40, 70] Takamura (2002) reports that polymer enhanced microsurfacings can be used to fill ruts up to 5 cm in depth using a rut-box. [54] When applied in rut-filling applications, it is desirable to assess the rut-resistance potential of the PME (at a minimum) through the performance of DSR testing on the extracted asphalt residue.[48,49, 54]

Takamura (2000) also provides comparisons of varying latex polymer concentrations. [54] As stated earlier, achieving a fine, networked structure of polymer within the asphalt provides a stronger and more elastic binder and is dependent upon the type and concentration of polymer, the asphalt source, and the compatibility between polymer and asphalt. Figure 23 illustrates the change in rutting resistance temperature versus percent polymer over a prolonged laboratory curing period at elevated temperature. The rutting resistance temperature for the 5 percent microsurfacing mixture is improved over the 3 percent mix with prolonged curing, but exhibits little initial difference. As with all asphalt surfaces, the strength (rutting resistance) of microsurfacing continues to increase with time. The 5 percent polymer asphalt binder provides the strength equivalent of PG-76 rutting resistance within a few days of curing.

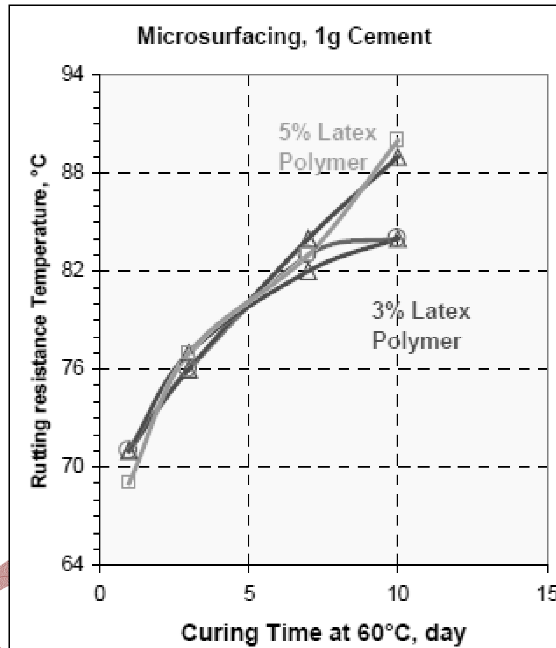


Figure 23: Curing Time and Rut-Resistance (54)

Microsurfacing curing times are highly dependent upon a number of factors, including the pH of the asphalt emulsion, the type and amount of surfactant, the type of bitumen and aggregate, and the application temperature. (71) Most manufacturers advise that microsurfacing has developed sufficient strength and is ready for full traffic return within an hour of construction.

Takamura used the same method to test latex polymer chip seal binders, as shown in figure 24. Although rutting is not usually associated with CRS-2P chip seal emulsions, this is a measure of the strength of the binder, and its ability to resist flushing. As would be expected, the 3 percent polymer binder is consistently stronger than the 2 percent.

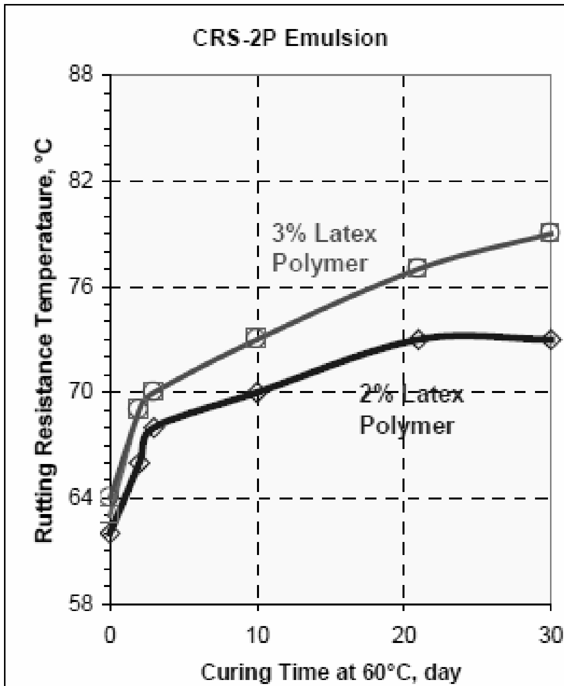


Figure 24: Prolonged Laboratory Curing of Latex CRS-2P at Elevated Temperature [54]

Setting agents such as Portland cement or lime can be added to microsurfacing mixes to control curing time by reducing the rate at which water evaporates and the asphalt emulsion breaks. When used with polymer modifiers, these setting agents aid in promoting the formation of the continuous polymer networks associated with quantifiable improvements in the viscoelastic characteristics of thin surface treatments discussed previously. [71] Work by Takamura (2001) proposes substituting aqueous-phase alkali metal hydroxides or salts in place of Portland cement to facilitate independent control of curing and mixing times based upon aggregate and bitumen type. [71] In addition, mixing accuracy is improved and handling made much easier owing to the difficulty in metering powdered Portland cement on the paving machine.

Holleran (1996) recommends using SBR or EVA in microsurfacings at a concentration of 1-5 percent depending upon the application; noting that 3-5 percent polymer concentrations will offer the most significant improvements. [70] Figure 25 presents wet track abrasion losses for 3 percent SBR, SBS, Neoprene and NRL modified surfacing treatments in comparison to an unmodified asphalt emulsion. A mixture modified with 3 percent SBR can reduce abrasion losses by up to 67 percent over unmodified asphalt after a 6 day soaking period. Similarly, Neoprene and SBS modifiers improve abrasion losses by 40-50 percent. These results indicate that PME offers significantly increased adhesion (translating into better stone retention) and water resistance than unmodified asphalt emulsions in slurry seal applications.

With respect to flushing, Holleran has shown that loaded wheel test results produce significant improvements in vertical displacement for 3 percent PME over neat asphalt, particularly for SBR and EVA modified mixtures (figure 26). [70]

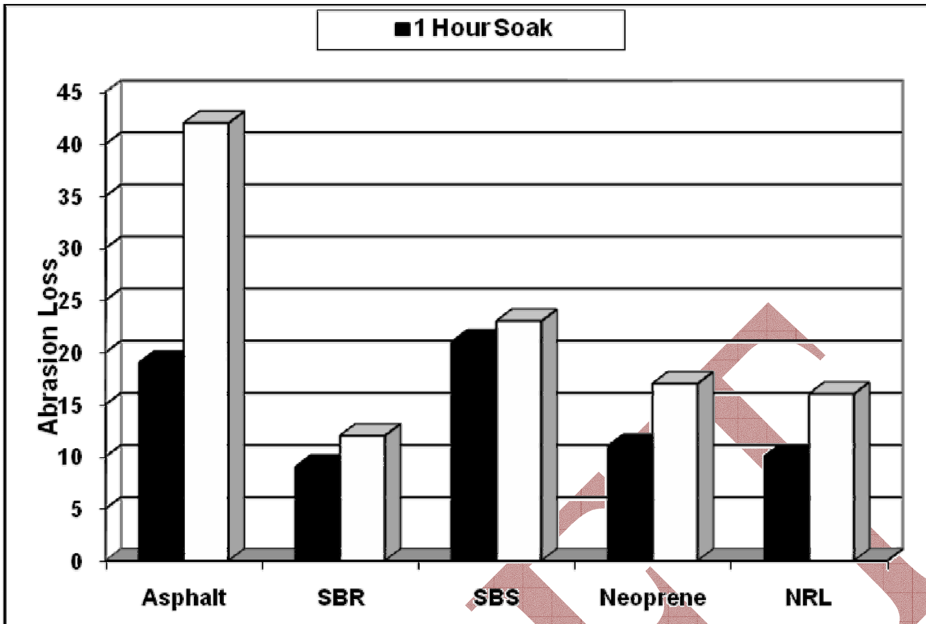


Figure 25: Wet Track Abrasion Losses [70]

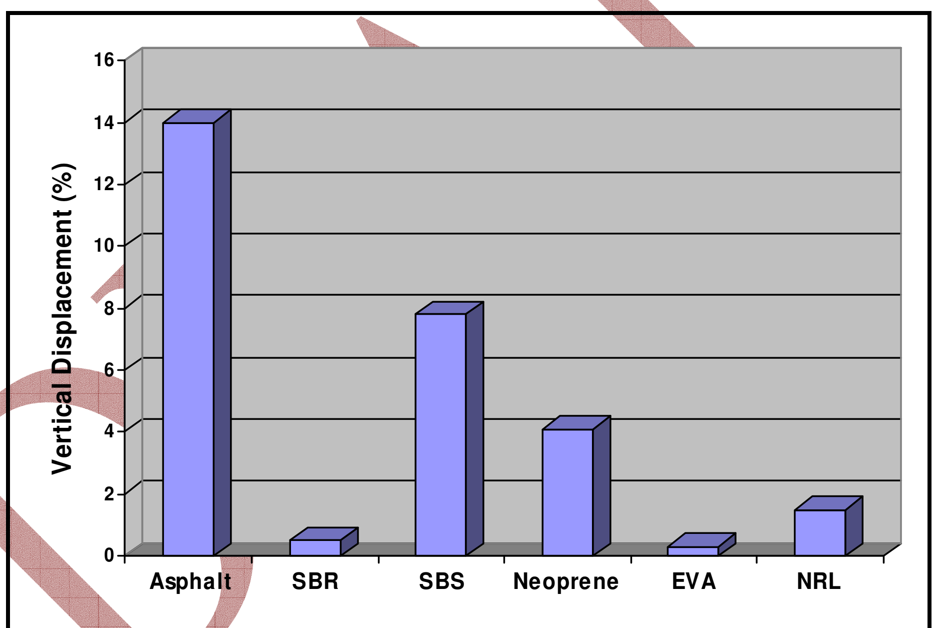


Figure 26: Loaded Wheel Test Results [70]

Jones and Ng (1989) have demonstrated similar results, with SBR, NRL and SBS modifiers offering the greatest improvement in vertical displacement for microsurfacing emulsions as shown in figure 27.[40] Jones further subjected these same mixtures to the Schulze-Breuer-Ruck abrasion test, which provides estimates of water absorption (soaking), loss (rotary tumbling), adhesion (water boiling), and integrity (largest remaining fragment after tumbling). Measurement parameters from Schulze-Breuer-Ruck are used to derive an overall numerical grade or rating for each test sample, with higher values representing greater compatibility (and thus better adhesion) between the aggregate, binder, filler, and polymer components. Schulze-

Breuer-Ruck results from the Jones study are provided in table 7. As table 7 illustrates, SBR and SBS modifiers provide for the most significant improvements in abrasion loss. Moreover, SBR demonstrates the highest degree of integrity and the highest overall grade for the microsurfacing mixtures tested.

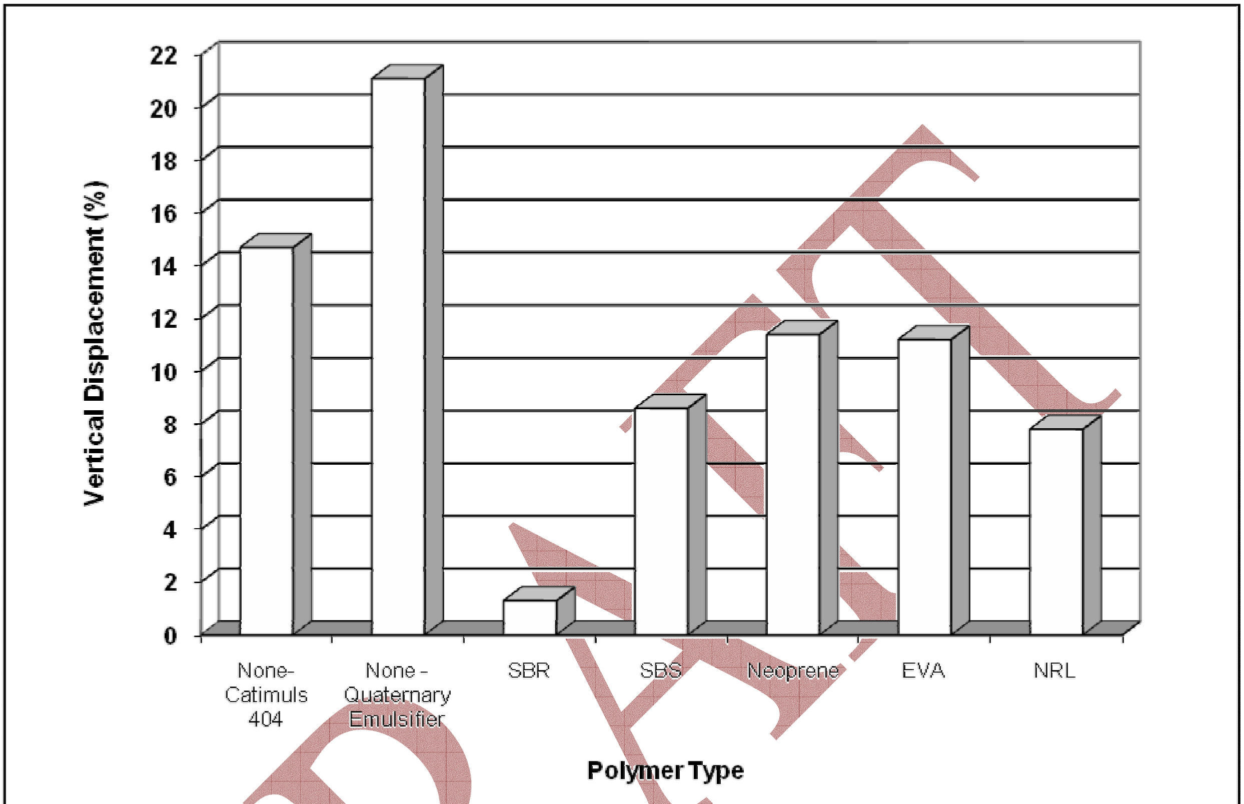


Figure 27: Loaded Wheel Test Results[40]

Table 7: Schulze-Breuer-Ruck Test Results [40]

Polymer	Absorption (g)	Loss (g)	Adhesion (g)	Integrity (percent)	Rating
SBR	1.25	0.96	99	98	11
NRL	2.30	1.49	99	95	9
SBS (Fina 416)	2.18	0.82	99	40	8
EVA (150W)	1.64	1.13	99	67	8
Neoprene (671 A)	2.06	1.51	99	96	9
None Catimuls 404	1.35	1.97	99	62	7
None EM26	1.59	2.01	99	33	5

Jones concludes that among the modifiers tested, SBR offers the best laboratory and long-term field performance in microsurfacing applications.[40] While the Schulze-Breuer-Ruck test appears to be a promising method of assessing the performance of polymer modifiers, it is noted that resulting measures of adhesion and absorption provide little or no correlation or distinction between modified and unmodified mixes (table 7). Jones also notes that latex modifiers were generally found to outperform solid polymers in microsurfacings. This likely relates both to the necessitated differences in mixing methodology (preblending for solids) and the manner and relative efficiency with which latex may be dispersed relative to bituminous fractions.[5,7,12,47,54] In addition, it has been shown that preblending of solid polymers may necessitate the addition of higher polymer concentrations than in soap batching or co-milling in order to achieve the formation of a continuous polymer network.

2.5.4 Cape Seals

Cape seals represent a combination of a large aggregate chip seal topped by a slurry seal coat (or microsurfacing) which is applied approximately 4 to 10 days later. Cape seals provide a dense, water-resistant surface which exhibits superior ride quality and skid resistance.

Solaimanian and Kennedy (1998) evaluated the field performance and design characteristics of 20 cape seal projects in the State of Texas over a period of one year.[72] During this study, bleeding, shoving, and flushing were identified as the most significant forms of distress in cape seals. Insufficient binder stiffness and failure at the interface between the chip seal and underlying pavement surface were generally found to the primary causes of permanent deformation. Moreover, the infiltration and entrapment of water were indicated to be substantially involved in early cape seal failure.

It has been demonstrated that resistance to deformation can be increased significantly through the addition of polymer modifiers to surface applied asphalt emulsion treatments.[12,14,47,48,54] This indicates that the use of polymers in the surface seal or microsurfacing overlays of cape seals can increase pavement life and high temperature performance. PME slurry seal overlays are also useful to increase chip seal stone retention and to provide a more water-resistant, smoother riding surface. Polymer modifiers in general have been shown to improve water resistance.[69, 70] However Solaimanian notes that microsurfacing cannot be used to correct an underlying water problem present in an incorrectly constructed chip seal or deficient base pavement. Indeed, in such cases the use of polymers in surface treatments can actually exacerbate underlying deficiencies, entrapping water which can lead to stripping and freeze-thaw related damage.[72]

2.6 Polymers and Traffic Volumes

The Context Sensitive Roadway Surfacing Selection Guide (2005) specifies roadway volume classifications based upon ADT used in practice by the FLH Division.[73] Table 8 presents this classification system for reference.

Table 8: Federal Lands Traffic Volume Classification[73]

Design Volume (vehicles/day)	Suggested Descriptive Term	Design Speed (mph) Preferred	Design Speed (mph) Minimum
< 200	Very Low	40	30
200 -400	Low	50	40
400 – 1,000	Medium	50	40
1,000 – 4,000	High	55	45
4,000 – 8,000	High	60	50
> 8,000	High	60	50

A survey of chip seal best practices performed by Gransberg and James (2005) reveals that many U.S. highway agencies restrict their use of chip seals to roadways with maximum traffic volumes of < 2,000 ADT.(68) The primary reason cited for confining chip seal applications to lower volume roads is the loss of stone which can result in inordinately high levels of windshield damage. It has been well-established, however, that when properly formulated, applied and cured, polymer modifiers can substantially increase stone retention and allow for earlier brooming without excessive losses.(4,5,12,13,14,23,30,51,65,68) This suggests that polymer modifiers are an essential (though not the only) component in the successful application of chip seals to high volume roads. Table 9 presents a summary of the maximum ADT volumes used for chip seal construction projects which were reported by U.S. and select international highway agencies surveyed during the Gransberg study.

Table 9: Chip Seal Maximum Traffic Volumes (68)

Maximum ADT	U.S.	Canada	Australia	New Zealand	South Africa	U.K.
< 500	2	1	0	0	0	0
< 1,000	1	1	0	0	0	0
< 2,000	12	2	0	0	0	0
< 5,000	11	2	0	0	0	0
< 20,000	12	3	3	1	0	0
> 20,000	7	0	1	1	1	1
Agencies Reporting:	45	9	4	2	1	1

Of the U.S. state agencies surveyed, approximately 64 percent specify the use of polymer modified emulsions in all chip seal applications. Moreover, Gransberg indicates that of the states self-reporting "excellent" levels of chip seal performance (32 percent for in-house, 17 percent for contractors), all were found to use polymer modifiers (including CRM), and all generally prescribe chip seals for only those roads attaining a pavement condition rating (PCR) of "fair" or better.(68) Chip seals are indicated to work best when they are applied as part of the regular pavement maintenance cycle, and they are not a suitable replacement for roads requiring rehabilitation even when polymer modifiers are used.

Microsurfacing applications by definition always include the use of polymer modifiers and are widely regarded as appropriate for use on medium to high volume traffic (> 1,000 ADT) roadway pavements.(40, 70) Because microsurfacing treatments are augmented with setting additives such as Portland cement, breaking can be controlled even at significant layer depths of up to 5 cm. This chemically-controlled curing mechanism allows microsurfacing to be used for comparatively "deep" treatment applications such as rut-filling, and permits expedited opening of the roadway to vehicular traffic.(54,71)

The South African National Roads Agency (SANRA) states that traffic volumes are important to ensuring proper stone embedment and to keeping the binder "alive and flexible", particularly in chip seal applications.(74) It is noted that since polymers impart increased rigidity to the binder, the demands for an appropriate level of traffic loading are even higher in PME based surface treatments and base pavements. However, SANRA also observes that polymer modified binders offer superior stone retention in the early stages of seal placement, thereby having the additional benefit of reducing asphalt bleeding. This latter benefit of PME is especially relevant on steep grades and at intersections where bleeding problems are most frequently encountered.[74]

2.7 Non-Roadway Applications

One of FLH's objectives is to determine the applicability of PME for non-roadway applications such as parking areas, hiking trails and bike paths. Although the authors could find no directly pertinent literature, the evidence of effectiveness of PME in addressing the same distresses encountered on both roadway and non-roadway pavements leads to the conclusion that judicious selection of PME applications can provide the same enhanced performance.

For example, prevalent forms of pavement distress, deformation, and weathering observed in FLH parking areas include:

- Block cracking.
- Rutting (caused by high pavement temperatures in combination with tight, relatively stationary wheel turns).
- Oxidation.

Cracking and oxidation are also found on hiking trails and bike paths, with the former representing the most common and problematic form of distress.

FLH reports that slurry seals in particular, are the favored preventive maintenance treatment applied to parking lot pavements, owing to their ability to waterproof the underlying base pavement while reducing closed-to-traffic times, reducing energy consumption, and minimizing environmental impacts.

As the research presented elsewhere in this report clearly illustrates, the use of PME in thin surface treatments does appear to enhance stone retention, improve low temperature susceptibility, and reduce the effects of high temperature deformation (rutting). Moreover, PME-based slurry has been anecdotally found to cure at a somewhat faster rate than its non-modified counterparts (thereby reducing closed-to-traffic times). Thus, it is reasonable to conclude that the use of PME could be expected to provide similar benefits in non-roadway applications, although it is not possible at this time to assess the resulting cost-benefit implications.

2.8 Climate, Environmental and Timing Considerations

Serfass *et al.* (1992) have examined the impact of climate on stone retention in surface treatments using SBS modified hot applied and emulsified asphalt.[19] In modified hot applied chip seals, the researchers note that an adequate period of warm weather is required to facilitate the evaporation of volatiles to allow aggregate to "firm" into its final position. The researchers recommend an application period extending from late May to late August in northern or mountainous climates, and mid-May to mid-September in southern regions for modified hot applied asphalt binders.[19] Conversely, SBS-modified emulsions were found to exhibit good stone retention characteristics even at relatively cool temperatures and high humidity as determined through Vialit cohesive testing. Thus, the cohesive properties of SBS modified emulsions appear to offer a longer application season when used for surface treatments, although Serfass does not provide a specific application calendar.

For chip seals, minimum ambient air and pavement application temperatures of at least 10ºC and 21ºC, respectively, are generally accepted standards to prevent excessive and prolonged stone loss.[75,68] Indeed, early stone loss as a result of late season application under cool temperatures is perhaps the most common reason for chip seal failure. Not only does the emulsion need to break, but the asphalt also needs to cure. For complete curing, the temperature needs to be high enough for a long enough period to allow the asphalt particles to fully flow together and coat the aggregate in a continuous, cohesive and adhesive binder. In general, low application ambient and/or pavement temperatures can result in high binder viscosity which hampers bitumen-to-aggregate adhesion.[68] At very high ambient air and pavement temperatures, problems have been reported with emulsions curing on the surface ("skinning over"), leaving emulsion trapped beneath the skin. The trapped water based emulsion does not bind to the surface or aggregate and causes problems when it bleeds through and releases chips under early traffic. Also, cured, low viscosity or solvent extended asphalt residues can bleed on very hot days. There is little consensus concerning maximum pavement temperatures for chip seal application projects, but most recommendations vary between approximately 54ºC and 60ºC.[68] Typically, a maximum ambient air temperature of approximately 43ºC is recommended for most chip seals.[68]

In hot climates, the primary issues that impact bituminous pavements and surface treatments are 1) deformation caused by high temperature susceptibility and 2) binder oxidative aging. [76] Vonk and Hartemink (2004) have shown that when comparing the accuracy of ring-and-ball softening point and zero shear viscosity (ZSV) test results, the latter produces a much more reliable measure of high temperature deformation potential in modified binders than does the former, as illustrated in table 10.[76]

Table 10: Physical Properties and Deformation Results [76]

Binder	Ring & Ball Temp. ºC	ZSV Pa.s 40º C	ZSV Pa.s 50º C	Deformation Rate in Test Road, 40º C	Deformation Rate in Test Road, 50º C
100 pen	45.5	2.5×10^3	6.3×10^2	24.0	56.2
100 pen + 3% SBS	49.5	3.2×10^5	1.0×10^4	4.0	12.6
60 pen	51.0	7.9×10^3	2.0×10^3	10.1	23.6

The results in table 10 show:

- The Ring & Ball test results do not correlate with the test road deformation for the polymer modified binder.
- The ZSV results do correlate with the test road deformation.
- The reduced high temperature susceptibility imparted by the polymer modifier translates to reduced rutting.

In high temperature applications, Vonk recommends SBS concentrations of at least 5 percent to insure that the polymer phase forms a resilient and continuous network throughout the mixture.[76] As has been suggested previously, it is this network that ultimately imparts the elastic response desired to resist permanent deformation. [4,12,14] Vonk's work focuses primarily on the modification of asphalt binders for HMA, and as such, the implications for desirable polymer concentrations in soap pre-batched or co-milled emulsions are uncertain. However, this research undoubtedly has valid implications in emulsion applications where the bitumen is subjected to direct forms of modification (i.e., preblending) prior to emulsification. Moreover, the interplay between polymer concentration, ZSV and the measurement of high temperature deformation potential have significance in emulsion treatments such as microsurfacing which are commonly used to fill wheel rut paths.

Vonk (2004) and Demazes *et al.* (2000) note that the measurement of ZSV in binders with a substantial polymer network is inaccurate because one requirement of this test is the development of steady-state viscosity under constant stress–a state which the elastic components of such a mix cannot attain (viscosity appears to grow infinitely).[61, 76] Although Desmazes offers an extended ZSV testing protocol that may yield improved accuracy and reliability, Vonk suggests that this phenomenon could be used to evaluate proper polymer dosing. More specifically, as ZSV begins to trend toward infinity, this provides a solid indication that a pervasive, 3-dimensional polymer network is present within the mixture, thereby insuring that the optimal modifier content has been achieved.

Vonk notes that accelerated binder aging in hot climates is dominated by the following characteristics:

- The binder becomes harder and less compatible.
- There is polymer-polymer cross-linking, polymer chain-scission, and reactions between bituminous components. [76]

Vonk observes that even in cases where polymer chains are shortened through age-related scission, the smaller polymer segments still contribute to maintaining elastic flexibility, albeit to a lesser degree than in unaged modified binders. Indeed, work by Davies and Laitinen (1995) demonstrates that aged SBS modified binders harden less than unmodified / differently-modified mixtures as measured via the wheel tracking test.[77]

Vonk asserts that SBS modified binders used for chip seal applications also offer demonstrable benefits in hot climates: increased stone retention, and high ZSV which indicates the presence of a continuous polymer network to retard permanent deformation and aggregate displacement.[76]

In arid climates however, the potential for hydrogenesis can pose a significant challenge to the use of PME. Hydrogenesis is defined as "the upward migration of water vapor in the road pavement which, under certain climatic conditions, condenses under the road surfacing".[78] In such cases, ambient air which penetrates through the roadway shoulders into the pavement aggregate layer, may transfer water to the stone surfaces via condensation to form a thin film. Although the full implications of hydrogenesis are not yet fully understood, anecdotal evidence provided by State highway agency (SHA) practitioners suggests that PME used in thin surface treatments may inhibit this trapped water from evaporating, thereby hastening the development of stripping, surface distress and/or structural failure.

2.9 Impact of Materials Selection

2.9.1 Polymer Type

A review of the available research indicates no clear empirical evidence that one type of polymer modifier is inherently superior to another with respect to performance, at least between the most commonly used types (SBR and SBS). A recent study of stone retention in chip seals performed by Kucharek *et al.* (2006) indicates that while latex-based PME may require more curing time than preblended PME to fully achieve the aggregate retention benefits associated with polymer modification, performance between the two binder types is comparable after only 24 hours. [80] Moreover, Kucharek concludes that "no special benefit has been observed so far from having the SBR polymer both inside and around the asphalt binder;" citing the need for additional research.[80] With a correct design with compatible materials, quality aggregates and best-practice construction methods, research has shown that a number of different polymers will give successful projects.

2.9.2 Surfactants & Emulsion Type

Surfactant chemistry is a complex and multifaceted area of study and as such, is well beyond the scope of the current review. Although published literature on the variation in PME thin surface treatment performance with respect to surfactant types is relatively scant (much of these data are proprietary in nature), a few researchers have attempted to identify high level differences between modified anionic and cationic emulsions.

Kucharek *et al.* (2006) assessed the chip retention characteristics of a variety of anionic and cationic emulsions modified with different polymers. [80] In this study, emulsion and whole system (i.e., chip seal) performance evaluations were accomplished using DSR, the Frosted Marble Cohesion Test, and the Sweep Test for Thin Surface Treatments. Overall, cationic PME mixes demonstrated considerably higher moduli during the first few hours of curing than did similarly modified anionic preparations. Moreover, although the moduli of the anionic group did gain some ground on the cationic test samples as curing progressed, the modulus values of the anionic mixes were not found to reach the same levels as the cationic group, even after a 24 hour cure period.[80]

Kucharek reports that cationic emulsions consistently demonstrated better chip retention characteristics (as measured in the sweep tests) than anionic emulsions for all the aggregate types studied. Cationic mixes also showed less sensitivity towards the varying chemical composition of the aggregates tested than did those prepared using anionic emulsions.[80]

2.9.3 Aggregates

One of the few issues identified during the literature review with respect to aggregate-polymer interactions pertains to the use of moisture-sensitive aggregate in thin surface treatments. In this regard, aggregates such as moisture-sensitive gravels may exacerbate the effects of hydrogenesis in arid climates, leading to water film buildup beneath a relatively impermeable polymer modified surface treatment.[78] Moreover, in cooler climates pre-existing excess water retention problems can lead to freeze-thaw damage.[72] Arguably, these potentially negative interactions are representative of an indirect relationship between aggregates and polymers. That is, the use of PME may be contraindicated in certain climates when placed atop a base course containing moisture-sensitive aggregate or one that already has a pre-existing water retention problem.

Overall, the impact of polymers on moisture sensitivity is not well understood at this time. In fact, some polymers are used as adhesion promoters. Moreover, chemical sensitivity issues between aggregate and various types of polymers could also present some challenges in certain cases. But the literature review presented herein turned-up little to no information regarding chemically sensitive aggregates and the use of PME. Indeed, the available research points overwhelmingly toward the ability of polymers to impede moisture penetration, enhance stone retention and increase overall pavement durability. However, caution should be used to determine whether the base course has a fundamental water retention problem prior to the application of any PME based thin surface treatment.

2.9.4 Fillers

Airey *et al.* (2002) present the findings of a laboratory investigation into the effects of mixing SBS modifier with CRM to produce impact absorbing asphalt (IAA) surfaces.(17) The results of this study show that the polymeric viscoelastic characteristics of the SBS are lost due to precipitation and phase-separation caused by the absorption of light aromatics contained within the maltene fractions by the CRM particles. In properly mixed SBS PMA which does not contain CRM, the SBS particles absorb these light maltene fractions, which results in the swelling of the polymer phase, thereby producing a continuous elastic network.

Other types of fillers have proven very effective in polymer modified stone matrix asphalt (SMA) HMA serving to increase the film thickness of the binder mastic on aggregates, improving adhesion, cohesion, strength and resistance to oxidative aging.(4) The fillers used in microsurfacing serve similar purposes.

2.10 Surface Treatments, Distress, and Cost-Effectiveness

The selection of appropriate surface treatments and the decision on whether or not to use polymer modifiers are dependent upon a number of factors, including:

- The effectiveness of a given treatment in rectifying a particular form of pavement distress.
- The cost-effectiveness of a particular treatment relative to the benefits and cost of other alternatives (including material, construction, life cycle and user delay costs).
- The environmental conditions under which the treatment is to be applied.
- The functional classification and/or traffic loading conditions of the roadway to be treated.
- The current condition of the underlying roadway, the type of pavement involved and its construction and maintenance history.
- The availability of appropriate materials, equipment and well-trained maintenance forces to insure proper placement.

Numerous decision tools and best practices have been developed by state highway agencies and industry trade organizations for matching the type and degree of pavement distress with the appropriate form of surface treatment. Hicks *et al.* (2000) provide a review of some of the best known of these practices, and present a framework which can be used to determine the most cost-effective treatment alternative.[79] This section of the report focuses on those treatments which are regularly employed using PME including chip, slurry and cape seals and microsurfacing.

One of the simplest and best known approaches to determining cost-effectiveness is the Equivalent Annual Cost method or EAC. EAC is determined as follows:

EAC = (unit cost of treatment) / (expected life of treatment in years).

Table 11 is from a 2000 paper and presents examples of the equivalent annualized cost (EAC) cost-effectiveness of various treatment types. Because of changing economics and supply as well as the improved materials and construction of recent times, the numbers given here may not be representative of those today. However, they provide information for comparisons. While the cost of the polymer emulsion may be thirty percent higher than an unmodified emulsion, the relative cost increase is much less when considering the total costs—including materials, construction, traffic control and user delay—and increased service life. More recent data from the Minnesota Department of Transportation (Mn/DOT) has found the total project cost of chip seals is seven percent higher with polymers, and Mn/DOT now uses only PME chip seals statewide, citing better early chip retention, faster traffic return (sweep and open in one hour), significantly reduced claims to the state for windshield damage, and significantly reduced damage from snow plows. They believe that "properly constructed chip seals are the most cost effective application we use to preserve our highways."

Table 11: Examples of EAC Cost-Effectiveness

Treatment	**Approx. Average Cost per s.y.** (*)	**Avg. Longevity (years) < 100 ADT**	**Avg. Longevity (years) 100 – 500 ADT**	**EAC (100-500 ADT)**
Chip Seal	$1.30	8	5	$0.26
Chip Seal Modified	$1.69	--	6.5 (**)	$0.26
Slurry Seal	$1.08	7	5	$0.22
Slurry Seal Modified	$1.40	--	6.5 (**)	$0.22
Cape Seal	$2.08	11	7	$0.30
Cape Seal Modified	$2.70	--	9 (**)	$0.30
Microsurfacing	$1.40	11	6	$0.23

*Costs may vary widely depending on materials used, location, etc.
**Number of years of longevity needed to achieve EAC break-even point assuming average cost increase of 30 percent for PMA emulsions.

While it was hoped the field projects in this study would provide additional cost effectiveness information, they were bid and placed during an unprecedented asphalt and polymer shortage and spike in asphalt and fuel prices, further emphasizing the difficulty in estimating cost differential. Most agencies estimate that a typical PME project costs less than ten percent more than an emulsion project without polymer, when all project costs are considered (including materials, construction, traffic control, striping, etc.) Table 12, which gives the costs of PME FLH projects in 2007 and 2008, is further verification of this figure, and shows the spike that occurred in 2008. The information gathered led to the conclusion when best practices are used for specification and construction, the additional cost of the polymers is more than offset by the improvements in performance both during and after construction.

Table 12: Federal Lands Highway Projects' Cost of PME for 2007-2008

Project	PME Cost	Chips & Placement	Total Project Cost	PME% of Total Costs	Cost increase by polymer*
Chickasaw National Recreation Area Oklahoma (2007)	$147,525	$579,530	$1,152,750	13%	4%
Yosemite National Park California (2007)	$395,568	$457,957	$1,986,451	20%	6%
Joshua Tree National Park California (2007)	$252,000	$374,100	$1,221,159	21%	6%
Death Valley National Park California (2008)	$243,338	$204,978	$741,130	33%	10%
Arches National Park Utah (2008)	$605,475	$345,540	$1,619,535	37%	11%
Dinosaur National Monument Colorado/Utah (2008)	$274,565	$222,750	$797,858	34%	10%
*Assuming PME emulsion cost is 30% higher than unmodified.					

Other forms of determining cost-effectiveness include life-cycle costing, longevity cost index and cost-effectiveness analysis using pavement performance curves.

As table 11 suggests, the increased longevity realized through the appropriate use of PME in thin surface treatments can offset somewhat higher initial material costs associated with the addition of polymer modifiers. This table assumes a 30 percent higher cost for polymer versus unmodified emulsions, which is fairly typical. However, that translates to approximately a 10 percent higher overall project cost when considering total costs (including aggregate, construction, traffic control, striping, etc.).

In 2007 and 2008, the FHWA developed the Transportation System Preservation (TSP) Research Roadmap by garnering the input of numerous State highway agencies, private industry and academia at three workshops held across the U.S. Several of the resulting problem statements generated by the Roadmap working groups were specifically targeted at identifying research needs that would better quantify the cost-effectiveness of preventive maintenance treatments in general and of specific material components more specifically. The literature review contained herein serves to further emphasize the need for additional research in the area of assessing the cost-benefit relationships between polymer modifiers and thin surface treatments. However, it is worthwhile to note that the comparatively small cost of polymer modifiers relative to overall material and construction costs, coupled with the demonstrable benefits of polymer modification illustrated throughout this report, indicate that the benefits of PME likely far outweigh its additional cost.

3.0 LABORATORY TESTING AND SPECIFICATION RECOMMENDATIONS

Task 2 in the Statement of Work articulated four areas for recommendations. Following the literature search, there were several industry outreach initiatives to collect information from current practitioners. Presently, there are several other in-progress research projects addressing some of the same issues as this work, and the principal investigators of those projects were contacted for idea sharing and possible coordination of on-going and future efforts.

There is a general consensus that current test methods and specifications can be greatly improved, and there are several performance-related protocols and methods currently being evaluated that look very promising. Because the proposed performance tests are not yet ASTM or AASHTO approved, and because there are still major data gaps, these protocols are not yet ready for full implementation by FLH.

Based on the findings of this investigation, it is recommended that FLH continue to use the best practices of existing specifications for acceptance and pay supplemented with the "report only" performance tests listed in table 13. It is further recommended that the data thus reported be combined with field performance evaluations, and that those results be used to gain statistical validation and acceptance as AASHTO/ASTM standards. More detailed information on the background for these recommendations is given in the following sections.

3.1 Industry Outreach Initiatives

3.1.1. Initial Discussions with Industry Representatives

Asphalt emulsion material suppliers, study participants from the NCPP and FLH representatives participated in an initial information gathering session on September 25, 2006 in St. Louis, Missouri. Koichi Takamura and Chris Lubbers of BASF Corporation (BASF), Joe Thrasher and Barry Baughman of Ultrapave, Dennis Muncy and Jon Wingo of SemMaterials, Paul Morris of Ergon and Roger Hayner of Terry Industries represented the industry viewpoint. Gary Evans, Scott Saunders and Mike Voth represented FLH, and the NCPP participants were Larry Galehouse and John Johnston. Following this meeting, Gayle and Helen King were brought into the project as consultants to contribute asphalt emulsion materials expertise and a better understanding of supplier needs and concerns. Several teleconference calls and meetings have followed since the initial meeting in St. Louis to garner relevant input from other industry representatives, academics and FHWA personnel. A summary list of these meetings includes:

- September 2006 Meeting in St. Louis, Missouri.
- March 2008 Meeting in Okemos, Michigan.
- Teleconference calls with industry and FLH in October, November, and December 2007; and July 2008.

Discussions of the input received during these meetings are presented in the following subsections, and detailed meeting minutes are on file at the Central Federal Lands Highway office.

3.1.2 Survey and Follow-up Communication

Based on comments gathered from the previously referenced meetings, the study participants developed a survey for the industry at large. Invitations were sent to members of the Binder Expert Task Group (ETG); the Transportation Research Board (TRB) committee AFK20 (Asphalt Binders); the TRB Pavement Preservation Task Force; and the International Technical Committees of the American Emulsion Manufacturers Association (AEMA), the Asphalt Recycling and Reclaiming Association (ARRA), and the International Slurry Surfacing Association (ISSA) to respond to a web-based questionnaire. Appendix A contains the full survey results. In support of the survey, numerous research resources and proposed test procedures were posted on the NCPP website. While a majority of the 33 survey respondents were technical people, there was a good cross-section of industry leaders and experts representing state highway administrations (SHA), suppliers, contractors, academics and consultants involved in regulatory, technical, construction, marketing, management and business roles. Industry had previously opposed innovative ideas for polymer modified emulsion testing and performance-related specifications, often because of concerns about shipping and payment delays or extensive testing requirements. The survey indicates that the private sector of the asphalt emulsion industry would be willing to accept more performance-based methods and specifications, so long as emulsion suppliers and contractors are included in the change process and their existing operations can continue to produce and place products efficiently. To make this happen, emulsion suppliers generally support a standardized certified pre-compliance testing and acceptance program. Overall, there was a good mandate for contractor, supplier and laboratory certification, but not individual certification. Representative samples of the specific comments on test protocols are given below. All of the experts consulted agreed that the ASTM D-244 specification covering test procedures for asphalt emulsions needs to be updated. Changes discussed in the survey include the following.

3.1.2.1 Emulsion Viscosity–Lab Test

Experts agree the Saybolt-Furol method for measuring asphalt emulsion viscosity is antiquated and unable to measure shear rate. Brookfield rheometers are used to determine asphalt viscosities at high temperatures for prediction of HMA mix and compaction temperatures, and are therefore standard equipment in asphalt laboratories. Although asphalt emulsion viscosity can be measured with this same rheometer, survey comments revealed that recent work by Salomon indicates some problems with Brookfield testing that might be overcome with a Paddle Rheometer as used by the paint industry.[81] Survey comments on the Paddle Method were generally favorable, but a follow-up phone call indicated that one lab (Flint Hills Resources) conducting work in support of the ASTM committee on asphalt emulsion test methods had problems with temperature control and suggested that additional work is required to validate the method. Improving the method for measuring asphalt emulsion viscosity in the lab remains a data gap. Although not critical for the improvement of FLH PME specifications as outlined in this study, it would be appropriate to include any new viscosity test methods under review by ASTM in the report-only field study.

3.1.2.2 Asphalt Emulsion Viscosity–Field Test

Asphalt emulsion viscosity as measured in an agency laboratory at some time well after project application is not necessarily informative, because particle size and resulting emulsion viscosity changes with storage and agitation, particularly when asphalt emulsions are kept at ambient temperatures. Another data gap recognized by many experts is the need for a field viscosity test to be run on an asphalt emulsion at the time of delivery to the project. Wyoming DOT has already implemented such a field test, which is available on the project website.[82] The Wyoming procedure should be considered for the report-only field study.

3.1.2.3 Optimizing Emulsion Viscosity

Respondents from cooler climates don't want chip seal emulsion viscosities raised from the standard 100-400 seconds Saybolt-Furol (SSF), but a number of agency and industry representatives from hot climates expressed concern that the 100-400 SSF minimum is too low. Other comments referenced problems with lower viscosity asphalt emulsions on pavements with steep slopes. It is important for the viscosity to be such that the asphalt emulsion sprays uniformly through the distributor and stays in a thick enough film on the pavement for optimal chip embedment. Another data gap revealed by the literature review is that optimum seal coat emulsion viscosity may need to vary with climate and pavement slope.

3.1.2.4 Residue Recovery Method

Support for low temperature asphalt emulsion residue recovery was strong, but significantly increasing the testing time for product certification may only be practical in combination with a delayed acceptance or a precertification program to overcome shipping delays. The need to eliminate distillation methods with recovery temperatures of 177°C (350°F) and higher was emphasized in a recent presentation by Kadrmas to the 2008 AEMA Annual Meeting. [83] He showed that binder moduli for PME microsurfacing residues as recovered using a Forced Draft Oven Procedure at 60°C (140°F) were consistently twice as high as the moduli for the same residues as recovered using 177°C (350°F) distillations. This data shows conclusively that asphalt emulsion residue performance specifications must not be based on current residue recovery practices. This conclusion is consistent with findings obtained from several European studies. During follow-up discussions, Dr. Didier Lesueur, an asphalt emulsion research manager for Eurovia and participant on European Normalization Committees for asphalt emulsion specification, shared new European Community for Standardization (CEN) standards for residue recovery[84] and a framework for cationic emulsion specifications based on performance parameters.[85] CEN also has a third relevant specification for recovery of emulsion residues which contain solvent.[86] The CEN standard for emulsion recovery is very similar to the Forced Draft Oven Procedure that Takamura and Kadrmas plan to submit at the next ASTM meeting.

Both methods first evaporate the asphalt emulsion at ambient temperature for 24 hours, and then place the residue in a forced draft oven for another 24 hours. The only major difference is that the CEN standard uses an oven temperature of 50°C (122°F), whereas the ASTM proposal will use 60°C (140°F). Although many lower temperature recovery methods have been proposed, the Forced Draft Oven procedure has the advantage of curing materials at conditions that most closely simulate conditions on the pavement. Furthermore, residue can be removed from the silicone mold without reheating. Although other potential recovery methods such as Stirred Can, Vacuum Recovery, Microwave Moisture Analyzer and others may be faster or may yield more emulsion residue, Forced Draft Oven will remain the method of choice until other methods are proven to match all resulting residue performance properties.

3.1.2.5 Residue Testing Using Superpave Binder Technology

There is strong support for using Superpave binder tools to specify performance properties of asphalt emulsion residues, with an accompanying climate driven grading system. However, legitimate concerns were expressed regarding additional equipment costs, extended testing time, lack of aging protocols and the need for a residue recovery method that yields a binder consistency equal to that of a pavement-cured material. More importantly, there is little consensus regarding the definition of performance parameters and specific testing conditions for PME chip seal and PME microsurfacing/slurry applications. Unfortunately, current practice is only loosely tied to variability in climate and traffic. For example, the penetration range for a binder in current microsurfacing specifications is 40 to 90 dmm, a range that would typically represent three full grades in the PG grading system for HMA binders (i.e., PG 58, PG 64, & PG 70). Implementation of performance specifications is a huge data gap that remains to be filled.

Although many issues remain to be resolved before asphalt emulsion residues can be characterized with reliable performance tests, a number of guidelines for future research can be established based on input received during the survey and related discussions. Residue performance properties to be characterized:

- High temperature grade based upon climate, traffic, and appropriate failure parameters (rutting, bleeding).
- Low temperature grade based upon climate and appropriate failure parameters (cracking, aggregate loss).
- Polymer identifier which is able to rank performance at different levels of polymer modification.
- High float gel identifier.

3.1.2.6 Aging protocol and handling during sample preparation

Because asphalt emulsions are applied at ambient temperatures, and high temperatures are known to change the physical properties of many polymers, PME residues should not be exposed to elevated temperatures during recovery or sample preparation. Any procedure requiring curing or reheating temperatures above 60ºC (140°F) must be validated by showing performance properties comparable to those from Forced Draft Oven Residues.

The rolling thin film oven (RTFO) procedure was definitively rejected by all respondents, since hot mix plants are not used for cold emulsion applications. The Pressure Aging Vessel (PAV) is clearly the aging tool of choice, but it has a number of limitations.

One concern is polymer/asphalt compatibility and stability during aging. It is known that certain polymer/asphalt blends are incompatible, such that the polymer will tend to separate or lose its elastic network over time. For modified HMA binders, such unstable systems are typically eliminated by specifying heat stability tests such as the Long-Term Asphalt Stability (LAST) test or the Separation Test. Because there is no heated storage of emulsion residue, experts reject these methods as performance indicators. Another good indication of compatibility comes from various microscopic methods such as fluorescence or scanning electron microscopy. Again, experts suggest such methods are useful to the formulator, but should not be adopted for specifications. As another data gap, a method is needed to insure polymer network stability under the conditions experienced by aging emulsion residues on the pavement surface.

3.1.2.7 Optimization of Testing Time, Cost and Reliability

Several respondents emphasized the need to minimize the quantity of residue needed for performance testing, ideally completing all residue tests with the recovered binder from a single silicone mold as cured in the Forced Draft Oven Recovery Method.

It was also thought important to minimize equipment costs and testing time, using common tools wherever possible. Survey comments and AEMA discussions emphasized the concern that there are many small companies supplying emulsion from one or two plants, and those facilities only manufacture approximately 10-20 percent of volumes shipped by refineries or liquid asphalt terminals supplying PG-graded binders. Amortizing expensive laboratory equipment and testing costs over small volumes can significantly increase product cost and disadvantage smaller producers.

Several comments emphasized the need to maximize use of the dynamic shear rheometer. DSR appears to be a critical tool for defining performance standards based upon rheology. One goal of the planned FLH report-only field study will be to maximize the capabilities of this instrument. Conversations with other research teams lead project leaders to believe it may be possible to use DSR to meet each of the four critical residue performance properties, as well as determine polymer-asphalt compatibility after aging. DSR also offers other important advantages including small sample size and no reheating for sample preparation. As discussed later in this report, the DSR methods developed by the Binder ETG and adapted by Kadrmas [87] will be used for high temperature residue properties and for polymer identification. Although most experts consider it logical to use the BBR for low temperature performance testing, several disadvantages make its use problematic for asphalt emulsion residues. For example, BBR would necessitate reheating recovered residue to pour relatively large test specimens. The equipment itself is costly, requires significant lab space for testing and temperature control units, and needs volatile solvents that often require access to fume hoods or vents.

An ongoing field-aging study led by Harnsberger and Huang at the Western Research Institute (WRI) encountered similar problems with the need to reduce sample size and so developed DSR protocols that include rheological measurements around 0-20°C (32-68°F). [87] The WRI researchers then fit and extrapolate rheological CAM models[88] to predict binder modulus and phase angle at the lowest pavement temperatures. Full details of Harnsberger's and Huang's work have not yet been published. WRI has significant funding within their current FHWA "Fundamental Properties" contract for development of rheological methods. Discussions are ongoing with project managers to determine whether WRI work plans and resources can be modified to develop specific DSR methods for testing emulsion residues at low temperatures. Of particular interest is a new DSR test method using 4-mm parallel plate geometry to directly measure G* and phase angle at the low pavement temperatures usually tested using BBR. Results to date are encouraging, and these methods will be evaluated as part of ongoing report-only field studies. With the recent addition of a second rheologist to their staff, WRI may also be willing to take on the task of developing a DSR test method to characterize the non-linear gel-like characteristics of anionic high float residues. John Casola of Malvern Instruments has also expressed interest in pursuing rheological studies of gelled asphalts. He cites classic criteria such as yield stress or non-linear response to strain rate can be used, but newer DSR techniques enable more sophisticated analyses such as using harmonics to define gels. SemMaterials has agreed to supply a series of gelled emulsion residues for testing. WRI also has broad experience with asphalt aging, and could be asked to adapt PAV protocols for emulsion residues. If WRI work plans can be altered accordingly, the report-only format of the FLH projects will be used to validate their findings.

3.1.2.8 Defining Polymer Content

Industry experts overwhelmingly favor physical performance tests over analytical chemistry methods to define the amount of polymer in various PME residues. Performance-related testing should give better information on predicted performance than recipe specifications. Elastic Recovery in a Ductilometer (ER), the most common method used by FLH and most AASHTO agencies, received lukewarm support as the preferred method. However, there was no strong support for other currently available alternatives such as force ductility, toughness and tenacity, torsional recovery or DSR phase angle. Most industry experts would prefer DSR testing if equipment costs could be controlled and the right parameters selected. Most of the survey comments favored use of a strain recovery parameter from the newly developed DSR Multi-Step Creep Recovery (MSCR) Procedure as recommended by the Binder Expert Task Group and recently adopted as AASHTO test method 7405-08.[89] Kadrmas' research presented to AEMA in February 2008 outlines a path forward that should satisfy the many comments received in this area. His results also showed the importance of physical testing rather than polymer quantification to assure equal performance.[83] This study was discussed in some detail at the Okemos, Michigan meeting, and further testing plans to identify polymer for the FLH report-only study will be based on Kadrmas' recommendations.[83]

3.1.2.9 Polymer/Asphalt Compatibility

Although widely used by suppliers as a formulation tool, there was very little support for the use of microscopy in product specifications to verify polymer network formation or asphalt/polymer compatibility. Among the primary objections to microscopy were the increased equipment acquisition and training costs, as well as potential delays in testing. If such a tool were to be included, it should be used as part of product qualification in a certified supplier program rather than as a PME specification tool.

3.1.2.10 PAV Tests to Simulate Field Aging of Emulsion Residues

It is easy to reject RTFO since this laboratory aging procedure is meant to simulate oxidation occurring at elevated temperatures in the hot mix plant. The Pressure Aging Vessel (PAV) is clearly the tool of choice for asphalt emulsion residue aging, but the direct translation of PAV procedures from asphalt concrete (AC) binders to PME residues is not as straightforward as most experts might expect. Issues to be considered include:

- Residue recovery for PAV testing: In order to avoid reheating the recovered residue to pour the sample into the PAV pan, it would be preferable to pour asphalt emulsion directly into the PAV pan and then cure the pan using methods established for the Forced Draft Oven. The cured residue would then be placed into the PAV oven for a defined time and temperature. Although seemingly straightforward, such a method has not yet been developed.

- PAV aging time & temperature: It would be ideal to hold PAV temperatures to 60°C (140°F) so that polymer modified residues would never be damaged by temperatures higher than those encountered in the field. The problem is that oxidation reaction rates double for each 10°C increase in temperature. Therefore the rate of oxidation in the PAV should be approximately 16 times slower at 60°C than at the 100°C (212°F) condition used for most Superpave binders. To reach an equivalent level of oxidation, the PAV testing time would have to be increased from 20 hours to 320 hours if temperature were reduced to 60°C. Extensive time-temperature PAV aging studies were conducted at WRI during SHRP. Such data would be valuable in evaluating alternatives for asphalt emulsion residues. Further research will be needed to determine the maximum temperature to which residues can be heated without damaging latex-induced polymer networks.

Performance tests to be run on PAV aged residues should include:

- Low Temperature Performance Specification: As asphalt ages, it becomes more brittle and prone to cracking at low pavement temperatures. Hence, low temperature physical properties should ideally be measured on appropriately aged residues. For surface applications such as slurry/microsurfacing or chip seals, the level of asphalt oxidation should be comparable to that observed near the surface of HMA. Physical tests on the aged residue should report both a hardness parameter and a relaxation parameter. For example, low temperature specifications could be based upon Stiffness (S) and "m-value" as measured by the Bending Beam Rheometer (BBR), or dynamic modulus (G*) and phase angle as measured by a DSR.

- PAV Aging to Control Polymer Compatibility/Degradation: Because standard test methods which control polymer/asphalt compatibility have been removed, there is some risk that unstable polymer/asphalt blends might prematurely degrade or separate. One possible means to control this could be to evaluate the polymers contribution to physical properties both before and after aging. For example, if the strain recovery in the MSCR test falls off rapidly with PAV aging, there would be some concern that the polymer system is unstable. Such a test method has not been considered in the literature, and this issue remains a data gap yet to be defined.

3.1.2.11 Aggregate Specifications

It is clear from the survey responses that aggregate requirements must fit the asphalt emulsion application. For example, chip seal experts typically prefer to specify fines by assigning a maximum P200 percent, while microsurfacing designers want a methylene blue test to control the surface activity of those fines. Although survey respondents generally favor LA Abrasion over MicroDeval, the few who have actually used the latter think it is a much better test, particularly for surface applications where more moisture is present. It is also generally believed that more aggregate and aggregate/emulsion compatibility testing will yield better performance. A recent study by Kim has shown how to optimize aggregate gradation for surface treatments.[90]

Although the primary objective of this study relates to the use and specification of polymer modified emulsions, some effort was also directed towards reviewing FLH aggregate specifications for chip seal and slurry/microsurfacing applications. Tables 13 through 20 show how current FLH standards compare to other agency specifications (specifically, TxDOT and CALTRANS), as well as to recommendations coming from active research projects and unique industry sources. A quick inspection indicates that FLH aggregate specifications use ASTM/AASHTO standard versions of common test procedures. Overall aggregate quality requirements are consistent with or exceed those of most state agencies. Specification of the Adherent Coating test to control the quantity of P-200 washed from the aggregate is particularly notable as a less common procedure that plays a very important role for insuring early aggregate adhesion to the emulsion residue.

Because the industry survey and other discussions led to a consensus belief that aggregate quality should be tied to traffic, some effort was made to identify aggregate quality standards that might be used to differentiate such use of materials.

3.1.2.12 Aggregate Specifications for PME Chip Seals

Table 13 compares current and proposed chip seal aggregate specifications for five sources.

Table 13: Comparison of Chip Seal Aggregate Quality Specs

Agency / Organization	**FLH (703.10)**	**Caltrans**	**TxDOT (Not AASHTO Standards)**	**Colorado State Study**	**RoadArmor® recommendations** (High Performance)
General Specifications	Furnish hard, durable particles or fragments of crushed stone, crushed slag, or crushed gravel. Use only one type of aggregate on a project.	Screenings shall consist of broken stone, crushed gravel or both. >90% by weight of the screenings shall be crushed particles as per Cal Test 205. Screenings shall be clean & free from dirt & deleterious substances.	Uncontaminated materials of uniform quality meeting plans &specifications. Special requirements for lightweight ag: pressure slaking, freeze-thaw loss, water absorption		
Gradation	Table 703-7	See below	See below		1/2 inch 100 - 100 3/8 inch 100 - 100 #4 0 - 12 #200 0 - 1
Los Angeles abrasion, AASHTO T 96	40% max.		35 max* 40 max*LRA	<25 for high volume	
Los Angeles Rattler, CA 211 Loss at 100 Rev. Loss at 500 Rev.		10% max 40% max			
Sodium sulfate soundness loss, AASHTO T 104	12% max.				
Mg sulfate soundness, 5 cycle, %, Tex-411-A			25 max		
Fractured faces, one or more, ASTM D 5821	90% min.		2 faces, >85%		1 face >98% 2+ >95%
Flat and elongated particles, 1:3 ratio. +⅜ inch sieve, by mass, average, ASTM D4791	10% max.				
Clay lumps and friable particles, AASHTO T112	1.0% max.				
Deleterious Materials Tex-217-F , P-200			2.0 max*		0.5 % max. 1.0 % max.
Cleanness Value, CA 227		80 min			
Decantation,%,Tex-406A			1.5 max		
Adherent coating, ASTM D 5711	0.5% max.				
Film Stripping CA 302		25% max			
	No lightweight aggregate; AASHTO M 195.	Samples for grading & Cleanness Value from spreader conveyor belt prior to application.			
Micro-Deval			For screening, not for acceptance		17% max
Flakiness index Tex-224F			17 max		17 max.
Absorption					2% max.

Tables 14, 15 and 16 give several agencies requirements for size, grade and combining the aggregate fractions in the given mix proportions.

Table 14: FL Table 703-7 Ranges for Surface Treatment Aggregate Gradation

Percent by Mass Passing Designated Sieve, (AASHTO T 27 & T 11) Grading Designation

Sieve Size	A	B	C	D	E	F
1½ inch	100 (1)					
1 inch	90 – 100 (3)	100 (1)				
¾ inch	0 – 35 (5)	90 – 100 (3)	100 (1)			
½ inch	0 – 8 (3)	0 – 35 (5)	90 – 100 (3)	100 (1)		
⅜ inch	---	0 – 12 (3)	0 – 35 (5)	85 – 100 (3)	100 (1)	100 (1)
No. 4	---		0 – 12 (3)	0 – 35 (5)	85 – 100 (3)	85 – 100 (1)
No. 8	---			0 – 8 (3)	0 – 23 (4)	---
No. 200	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 - 10 (1)

(1) Statistical procedures do not apply.
( ) The value in parentheses is the allowable deviation (±) from the target value.

Table 15: CALTRANS Chip Seal Screenings Sizing

Seal Coat Types	Size of Screenings
Fine	1/4" x No. 10
Medium fine	5/16" x No. 8
Medium	3/8" x No. 6
Coarse	1/2" x No. 4
Double	
1st application	1/2" x No. 4
2nd application	1/4" x No. 10

Sieve Size	Percentage Passing			
	Coarse	Medium	Medium Fine	Fine
	1/2" x No. 4	3/8" x No. 6	5/16" x No. 8	1/4" x No. 10
3/4"	100	—	—	—
1/2"	95-100	100	—	—
3/8"	50-80	90-100	100	100
No. 4	0-15	5-30	30-60	60-85
No. 8	0-5	0-10	0-15	0-25
No. 16	—	0-5	0-5	0-5
No. 30	—	—	0-3	0-3
No. 200	0-2	0-2	0-2	0-2

Table 16: TxDOT Aggregate Gradation Requirements (Cumulative Percent Retained)

Sieve Size	Grade 1	2	3S2	3 Non-lightweight	3 Lightweight	4S2	4	5S2	5
1"	–	–	–	–	–	–	–	–	–
7/8"	0–2	0	–	–	–	–	–	–	–
3/4"	20–35	0–2	0	0	0	–	–	–	–
5/8"	85–100	20–40	0–5	0–2	0–2	0	0	–	–
1/2"	–	80–100	55–85	20–40	10–25	0–5	0–5	0	0
3/8"	95–100	95–100	95–100	80–100	60–80	60–85	20–40	0–5	0–5
1/4"	–	–	–	95–100	95–100	–	–	65–85	–
#4	–	–	–	–	–	95–100	95–100	95–100	50–80
#8	99–100	99–100	99–100	99–100	98–100	98–100	98–100	98–100	98–100

Notes: Round test results to the nearest whole number; Single-size gradation.

Aggregate specifications for chip seals vary widely, and not all agencies differentiate aggregate quality for traffic. Even the definition of high-volume traffic for chip seals varies markedly, with experts somewhat arbitrarily choosing anywhere from 1,000 ADT to 10,000 ADT as a minimum level which might require higher quality materials. A high volume chip seal study by Shuler elected to construct field test sections with ADTs exceeding 7,500 ADT.[66] Recommendations from that study, and the new NCHRP project also led by Shuler (Manual for Emulsion-Based Chip Seals for Pavement Preservation: NCHRP 14-17) should be considered. Since FLH has graciously agreed to support this latter NCHRP project with field trials, Shuler's results and recommendations should be available and pertinent to FLH needs. Although the study is ongoing, Shuler has already made some recommendations to the FLH research team based upon earlier work. One example of note is to reduce the LA Abrasion maximum from 40 percent to 25 percent for high volume traffic.

As another example, SemMaterials (formerly Koch Materials) developed a high performance chip seal system under the trademark RoadArmor® for higher volume traffic. This system includes a new piece of construction equipment which applies both emulsion and then aggregate in a single pass. It also includes upgraded emulsion and aggregate specification recommendations consistent with faster curing and longer wear. RoadArmor® was actually developed for chip seal applications that need a quick return to traffic, a concept which may be more appropriate than ADT to FLH needs on pavements such as narrow mountain roads or isolated areas where detours are unavailable and traffic control is difficult. Hence, RoadArmor® guidelines do not define high volume traffic with a specific ADT. However, the aggregate guidelines supplied with this system offer some insight as to recent trends applicable to differentiating material quality. As can be seen on the comparative table for chip seal aggregates (table 13), RoadArmor® guidelines reduce P-200 fines and deleterious materials and require more crushed faces than most agency specifications. Interestingly, this guideline specification also appears to be among the first in the U.S. to replace LA Abrasion with Micro-Deval.

Although the industry survey received more favorable votes for LA Abrasion, the respondents who actually had experience with using Micro-Deval to screen aggregate durability strongly favored it. Since FLH has the Micro-Deval apparatus available in the Denver lab, it is recommended that Micro-Deval be required in the report-only portion of the experimental materials testing plan. Results should be compared against the RoadArmor guideline of 17 percent maximum loss to determine whether similar limits might fit FLH needs on higher volume chip sealed pavements.

3.1.2.13 Aggregate specifications for PME microsurfacing/slurry

ISSA offers separate aggregate quality guidelines for slurry seal and microsurfacing applications. The industry survey indicated that ISSA guidelines represent best current practice, and should be adopted where possible as minimum requirements. More recent research for CALTRANS led by Fugro Consultants proposes that all slurry systems be redefined in essentially three categories based upon traffic, climate, and application. These three classifications should provide better definition for use of microsurfacing, PME slurry seals and conventional unmodified slurry seal emulsions. Aggregate and mix design guidelines should be adjusted accordingly for these three distinct uses. Preliminary information on aggregate quality guidelines was provided by Fugro on the CALTRANS study for slurry seals and microsurfacing. These data are compared to both the ISSA guidelines and existing FLH specifications in tables 17 and 18.

Table 17: Comparison of Slurry Seal Aggregate Quality Specifications

Test Method	FLH	ISSA	FUGRO / CALTRANS Study
General	Furnish natural or manufactured sand, slag, crushed fines, or other mineral aggregate conforming to AASHTO M 29 and the following:	The mineral aggregate used shall be the type and grade specified for the particular use of the slurry seal. The aggregate shall be manufactured crushed stone such as granite, slag, limestone, chat, or other high-quality aggregate, or combination thereof. To assure the material is totally crushed, 100 % of the parent aggregate will be larger than the largest stone in the gradation to be used.	
Los Angeles abrasion, AASHTO T96	35 % max.	35 % max. Abrasion test run on aggregate before it is crushed	30 % max. high traffic 35 % max. low traffic
Sand equivalent value, AASHTO T176, alternate method no. 2, reference method	45 min.	45 min.	45 min. low traffic 65 min. high traffic
Smooth textured sand with < 1.25 % water absorption content by weight of total combined aggregate	50 % max.		
Soundness, AASHTO T104		15 % max using Na_2SO_4 25 % max using $MgSO_4$	20 % max using $MgSO_4$
Polishing		Meet approved polishing values	
Gradation	See below	See below	See below

Table 18: Comparison of Microsurfacing Aggregate Quality Specifications

Test Method	FLH	ISSA	FUGRO / CALTRANS Study	TxDOT
General	Furnish natural or manufactured sand, slag, crushed fines, or other mineral aggregate conforming to AASHTO M 29 and the following:	The mineral aggregate used shall be of the type and grade specified for the particular use of the Micro-Surfacing. The aggregate shall be a manufactured crushed stone such as granite, slag, limestone, chat, or other high-quality aggregate, or combination thereof. To assure the material is totally crushed, 100 % of the parent aggregate will be larger than the largest stone in the gradation to be used.		
Los Angeles abrasion, AASHTO T 96	30 % max.	30 % max. To be run on parent aggregate	30 % max. high traffic 35 % max. low traffic	
Sand equivalent value, AASHTO T 176, alternate method no. 2, reference method	65 min.	65 min.	45 min. low traffic 65 min. high traffic	70 min.
Sodium sulfate soundness, AASHTO T 104	15 max. Using NA_2SO_4 25 max. Using $MgSO_4$	15 max. Using NA_2SO_4 25 max. Using $MgSO_4$	20 max Using $MgSO_4$	30 max.
Polishing		Meet state-approved polishing values		
		Proven performance may justify the use of aggregates that may not pass all of the above tests.		
Gradation, type II or III	Table 703-8 (See below)	See below	See below	

Recommended aggregate gradation comparisons for slurry seals and microsurfacing applications are provided below in tables 19 and 20.

Table 19: Slurry Seal & Microsurfacing Aggregate Gradation & Application Rates

Percent by Mass Passing Designated Sieve, (AASHTO T 27 & T 11), Slurry Seal Type

Sieve Size	**I (Slurry Only)**		**II**		**III**	
	FL	ISSA	FL	ISSA	FL	ISSA
3/8 inch	—	100	100	100	100	100
No. 4	100	100	90-100	90-100	70-90	70-90
No. 8	90-100	90 – 100	65-90	65-90	45-70	45-70
No. 16	65-90	65 – 90	45-70	45-70	28-50	28-50
No. 30	40-65	40 – 65	30-50	30-50	19-34	19-34
No. 50	25-42	25 – 42	18-30	18-30	12-25	12-25
No. 100	15-30	15 – 30	10-21	10-21	7-18	7-18
No. 200	10-20	10 – 20	5-15	5-15	5-15	5-15
Application rate, pounds per square yard	6 - 10	8-12	10 - 15	10–18 slurry 10-20 micro	15 or more	15-22 slurry 15-30 micro

Note: Statistical procedures do not apply to gradations. Application rates are based on the dry mass of the aggregate.

Table 20: TxDOT Microsurfacing Aggregate Gradation Requirements (Washed)

Sieve Size	**Cumulative percent Retained**
1/2 in.	0
3/8 in.	0–1
#4	6–14
#8	35–55
#16	54–75
#30	65–85
#50	75–90
#100	82–93
#200	85–95

TxDOT Microsurfacing JMF Requirements have been provided for comparison purposes below in table 21.

Table 21: TxDOT Microsurfacing JMF Requirements

Property	**Test Method**	**Requirements**
Wet track abrasion, g/sq. ft., max. wear value	Tex-240-F, Part IV	75
Gradation (aggregate and mineral filler)	Tex-200-F, Part II (Washed)	Table 1
Mix time, controlled to 120 sec.	Tex-240-F, Part I	Pass

3.1.2.14 Emulsion/Aggregate Performance Tests

It is widely recognized that asphalt emulsion and residue properties alone cannot define performance. Similarly, mixture performance parameters as typically measured using Superpave mix design and performance tools are not sufficient to describe most Pavement Preservation applications. As pointed out by Leach and Blankenship [91], asphalt emulsions require time to cure. Therefore, one critical performance issue is establishing the amount of time an asphalt emulsion system must cure before a road can be reopened to traffic.

3.1.2.15 Sweep Test - Chip Seal Curing Time for Traffic – ASTM 7000 [92]

The survey indicated some concerns with the Sweep Test, particularly with respect to repeatability of the standard ASTM method. Takamura has investigated this test in some detail, and reports that three minor revisions to the procedure can reduce variability from 20 percent to 5 percent. [93] Such improvement would almost certainly overcome expressed concerns if these results can be duplicated in multi-lab round-robin studies. The survey also indicates that confusion exists as to the performance characteristics being measured. As originally developed by Barnat, the sweep test was intended to rank emulsion/aggregate systems for curing time before a chip seal can be opened to traffic. [94, 95] Since temperature and humidity play an important role in curing, the predictive value of this test is only accurate when the conditioning protocol is able to simulate field conditions at the time of placement. However, when conditioning occurs under the constant environmental conditions designated by the ASTM procedure, the test does seem to provide a reasonably correct rank-ordering of curing times as needed for purchase specifications. It is important to further clarify that the sweep test might predict aggregate loss or potential for windshield damage as the emulsion cures, but it is not intended to be a predictive tool for long term chip loss.

3.1.2.16 Chip Seal – Long Term Aggregate Loss

There was no expert agreement on a good test for evaluating long-term chip loss. Suggestions from Davidson at McAsphalt included the Vialit Plate Shock Test [96] and the Frosted Marble Test, [80, 97] whereas French experts recommended the Vialit Pendulum Test. [98] The best tool to date appears to be the MMLS3 procedures as developed by Dr. Richard Kim's group at N.C. State for the North Carolina DOT. [99, 100] Although too expensive to advance for specification purposes, it remains an excellent research tool against which the predictive capabilities of less expensive performance tests can be compared. This subject remains a significant data gap, with no specific project recommendations at this time.

3.1.2.17 Microsurfacing vs. Polymer Modified Slurry

Microsurfacing is formulated to provide significantly higher performance than Slurry Seals either with or without polymer. From a use perspective, microsurfacing should be used for rut-fill applications and for high-traffic pavements with ADT exceeding 1000. Microsurfacing also contains emulsifier packages that break quickly so that traffic can usually be returned in one hour or less. Where traffic control is a problem due to urban traffic, narrow roads or long detours, the faster curing microsurfacing might be specified for lower volume roads.

3.1.2.18 Microsurfacing Performance Tests

The ISSA document A143 "Recommended Performance Guidelines for Micro-Surfacing" was cited by survey respondents as the best available current practice for performance-related test procedures. [39] Performance tests include wet cohesion, Excess Asphalt by LWT Sand Adhesion, Wet Stripping, and Wet Track Abrasion Loss after one day soak and after six day soak. These tests should be used as pay items.

3.1.2.19 Newly Proposed Tests for Mix Design and Performance

At a recent ETG meeting, Jim Moulthrop of Fugro, Inc. provided an update of a soon-to-be-completed research study updating mix design methods for microsurfacing. [101] Significant contributions from this study include an automated test for cohesion, a German method to predict mixing time by measuring mixer torque and a French adaptation of the wet track abrasion test using wheels in place of the rubber tube. It is recommended the FLH report-only format be used to evaluate new tools recommended by the Fugro study.

3.1.2.20 Polymer Modified Slurry Seal

Since polymer modified slurry seal asphalt emulsions will only be used on roads carrying lower traffic levels (<1000 ADT), the wet-track abrasion test is probably sufficient as a pay item for mixture performance testing. However, it will be important to insure an adequate amount of polymer has been added for PME slurry applications. This can best be done with a residue polymer identification test. Elastic Recovery should remain in formal specifications for now, but Kadrmas' DSR MSCR protocol reporting recoverable strain [83] appears to be the best choice for report-only criteria. The ultimate strain recovery for a PME slurry seal residue would be significantly less than that expected for microsurfacing. From limited data, Kadrmas recommends the following test conditions and specification limits to differentiate microsurfacing from PME slurry as shown in table 22.

Table 22: Microsurfacing and PME Slurry Report Only Performance Tests [83]

Testing Protocol	Specification Latex/Polymer Modified	Specification Microsurfacing
Original DSR, G*/sin δ	3 (minimum)	5 (minimum)
Original DSR, Phase Angle	80 (maximum)	75 (maximum)
MSCR, % recovery at 3200 Pa	15 (minimum)	25 (minimum)

3.1.2.21 Manufacturing and Construction: Construction Controls on Climate

Because of problems with curing when asphalt emulsions are applied at lower temperatures, the application window should be carefully restricted. Pavement temperatures continue to be important until the emulsion residue is fully cured.

Chip seals frequently fail if freezing occurs while there is still moisture within the binder. Controlling pavement temperature at time of application may not be sufficient to insure full curing. Given improvements in weather forecasting, it might be more appropriate to stop projects based upon predicted freezing temperatures for a few succeeding nights rather than raising pavement temperature requirements or narrowing seasonal limits for construction.

Because excess water dilutes and displaces emulsions, break time should be tied to requirements to stop construction for pending inclement weather.

It is also known that sealing high concentrations of moisture into a pavement can result in catastrophic stripping failures. Therefore, entrapped water resulting from recent rainfall before construction or other sources of subsurface moisture can lead to unexpectedly poor performance of sealed pavements.

Use of fog seals over new chip seals can improve short- and long-term aggregate retention, perhaps even to the point of extending the construction season modestly.

Each of these observations, although obvious to the experienced practitioner, represent data gaps needing further research so that effective construction controls can be objectively managed.

3.1.2.22 Manufacturing and Construction: Rolling/Compaction

Recent research by Kim evaluated the effect of compactor type and roller pattern on the performance of chip seals.[(100)] Recommendations from this work should be included in FLH guidelines.

3.1.2.23 Manufacturing and Construction: Controls on Polymer Addition

Good support was noted in the survey for preblending/co-milling polymers at the emulsion plant. Based on field practice, almost no one indicated support for adding polymer latex to the emulsion distributor or field tanks, with comments noting viscosity drop, polymer latex separation, and lack of uniformity leading the negatives. If post-blending latex is to be allowed at all, specification language should insure controlled metering and complete blending of latex and asphalt emulsion at the supplier's plant to attain a uniform consistency that continues to meet minimum viscosity requirements.

3.2 Follow-up Discussions with Larger Industry Audience

The goals of the FLH project and the need for industry response to the survey were introduced to several Transportation Research Board (TRB) committees at the January 2008 annual meeting in Washington, D.C., including the following:

- AFK10 – General Issues in Asphalt Technology.
- AFK20 – Asphalt Binders.
- Task Force on Roadway Pavement Preservation.
- AHD20 – Pavement Maintenance.

Survey results and suggested specification test methods were presented to several groups who were then solicited for their comments. These groups included:

- Joint Annual Meeting of the Asphalt Emulsion Manufacturers Association (AEMA), The Asphalt Recycling and Reclaiming Association (ARRA) and the International Slurry Seal Association (ISSA) in February, 2008, including two presentations and a one-hour breakfast meeting with the International Technical Committee. By the end of the AEMA meeting, industry response was sufficiently positive for Jim Sorenson of the FHWA Office of Asset Management to form the ETG Emulsions Task Force.

- Asphalt Binder Expert Task Group in February, 2008.

- Emulsion Task Force of the FHWA Pavement Preservation ETG in April, 2008 (see discussion in Section 3.3.3.4).

- TRB Committee AFK10 (General issues in Asphalt) in April 2008.

- Discussions with Dr. Scott Shuler, principal investigator of NCHRP Project 14-17, "Manual for Emulsion-Based Chip Seals for Pavement Preservation".

- Discussions with Drs. Hussein Bahia and Peter Sebaaly of the Asphalt Research Consortium.

- Discussions with Dr. Richard Kim, Principal Investigator of an on-going chip seal performance study for the North Carolina DOT (Project HWY 2004-04). Dr. Kim summarized his research at the project review meeting in Okemos, Michigan. He reported that many NC DOT districts are already converting all chip seals to polymer modified asphalt emulsions based upon their own experience and Dr. Kim's findings to date, even though research is not complete and no state mandate requiring polymers has been published.

- Discussions with European emulsion experts and Standards Committee members, including Didier Lesueur of Eurovia and Francois Chaignon of Colas.

- Discussions with Darren Hazlett (TXDOT) and Dr. Amy Epps (Texas Transportation Institute, TTI) on their efforts to develop Superpave PG-type performance-related emulsion specifications.

- Discussions with Jim Moulthrop regarding progress with Fugro's pooled-fund microsurfacing mix design study.

- Discussions with McGraw (MnDOT), Maurer (PennDOT), Hosseinzadeh (CALTRANS) and other SHA personnel on the status of delayed acceptance for certified asphalt emulsion suppliers and modified asphalt emulsion performance specification development.

- Discussion with Roger Olson (MnDOT) regarding an upcoming pooled-fund pavement preservation study for MnROAD that may provide a second opportunity to evaluate performance testing protocols as recommended for this FLH study.

- Discussions with Dr. Jack Youtcheff, Leader of FHWA's asphalt research team at Turner-Fairbanks. [Note: Dr. Youtcheff oversaw the asphalt chemistry research and the development of Superpave binder specs as a member of the SHRP staff, and now has responsibility for approving research projects and work plans developed by the WRI Asphalt Research Consortium, as well as defining asphalt research to be conducted at Turner-Fairbanks. He is also a member of the Binder ETG and the Emulsions Task Forces.] Dr. Youtcheff states that he is interested in funding studies that would advance performance-based asphalt emulsion specifications. He has some ideas as to how the WRI and ARC work plans can be reworked to fit identified research needs, and is prepared to pursue money to support some related activities within FHWA's labs at Turner-Fairbanks. However, Youtcheff feels it is important that any defined research needs for asphalt emulsion applications come from the newly-formed FHWA ETG Emulsions Task Force, rather than from individuals or single projects. Dr. King chairs the emulsion residue testing subcommittee of the Emulsions Task Force (ETF), and will initiate efforts accordingly. Further discussions with Dr. Youtcheff, WRI/ARC investigators, and ETF subcommittee members took place at the Association of Asphalt Paving Technologists (AAPT) meeting in April 2008.

- Recommendations from the FHWA/FP^2 "Spray Applied Polymer Surface Seals Study." The recently completed FHWA/FP^2 study "Spray Applied Polymer Surface Seals" recommends that new chip seals be fog-sealed immediately after brooming if problems from windshield damage or long term chip loss are anticipated. [(102)] Roger Olsen of MnDOT reports that they now fog seal almost all new chip seals, and as a result, windshield and snowplow damage have been reduced, and customer acceptance is unusually high because the black color leads to a perception among the driving public that a new HMA overlay has just been placed. To maintain optimal embedment, the initial application of CRS-2P chip seal emulsion should be reduced by the amount of asphalt to be applied during the ensuing fog seal.

- At ISAET, the International Symposium on Asphalt Emulsion Technology in Washington D.C. in 2008, there were two presentations to technical sessions on the ETF scope and framework and this FLH study.

3.3 Specific Recommendations

To specifically address the four items enumerated in the Statement of Work, recommendations are made in the following subsections.

3.3.1 Task 2A. Use of Modified vs. Unmodified Asphalt Emulsions

Polymer modified asphalt emulsions should be used for chip seal and slurry seal / microsurfacing applications for all traffic and climate conditions. While non-modified materials are less expensive than modified products, the construction, mobilization, traffic control costs and the improved initial and long-term performance of PME usually justify the higher costs of using elastomeric PME.

Moreover, specifications for traffic conditions should be differentiated as follows:

- Microsurfacing vs. PME Slurry. Microsurfacing is polymer modified slurry seal with additives which result in a much faster chemical cure rather than atmospheric evaporation emulsion break. This study recommends microsurfacing for rut-filling, high traffic areas (>1000 ADT), roads that require quick return to traffic and for high durability needs. PME slurry specifications typically require less polymer, but still significantly upgrade the performance above that expected from conventional slurry. PME slurry emulsions are recommended for low-volume roads (<1000) for which microsurfacing is not otherwise justified.

- PME Chip Seals. As mentioned above, cationic or anionic polymer modified chip seal asphalt emulsions are justified regardless of traffic level, as demonstrated by recent studies performed by the Ontario Good Roads Association (OGRA) and Gransberg *et al.* (2005) on the cost-effectiveness of CRS-2P on low volume roads, (64) as well as Dr. Kim's research results discussed previously. Traffic levels and speed should be considered when selecting aggregates and performance criteria. A quick cure and return to traffic, as potentially differentiated by the sweep test, are particularly desirable for high traffic areas, as are durable, polish-resistant aggregates. It is common to have individual asphalt emulsion specifications for cationic (CRS-2P), anionic (RS-2P) and high float anionic (HFRS-2P) PMEs. Local agency names for these emulsions will vary throughout the country.

For climate considerations, it is recommended that strict windows for application temperatures be specified, but this area also needs further investigation as there is clear evidence that curing, shelling and bleeding of chip seals are associated with climatic conditions occurring well after the time of application. Superpave PG-type specifications for HMA are based on climatic temperature ranges, which may also be useful for asphalt emulsion surface treatments, especially microsurfacing. Although the concept of 6°C grade increments based upon LTPPBind climate maps is attractive to practitioners, failure properties have not yet been defined and failure limits have not been established. For this reason, the FLH report-only lab testing format will only be useful if measured physical properties can be tied to actual performance on the pavement. It will be important to have longer-term pavement management data and frequent video tapes of pavement condition so that field performance can ultimately be used to set specification limits on promising laboratory performance measures.

As discussed in the literature review, polymers are believed to be advantageous for use on hiking or biking trails and parking lots because of resistance to permanent deformation, raveling surface aggregate, oxidative aging and damage caused in parking lots when front wheels are turned with no concurrent forward motion. Polymer modified materials have also been shown to retard cracking, particularly the block cracking typically seen in older parking areas. Bikers prefer microsurfacing/slurry seals over rougher chip seals for trails. Small-sized aggregates should be used, and loose chips avoided. Although microsurfacing and slurry seals are not typically compacted for paving applications, they are compacted on airport runways and taxiways to eliminate FOD damage caused by raveling surface aggregate.

If loose aggregate is perceived to be a problem on trails, use of small rollers on slurry/microsurfacing applications should be evaluated. Also, polymerized seals generally cure faster, meaning faster reopening for its intended use. However, there is not much data in the literature on the use of polymerized asphalt emulsions on trails and parking lots, as noted previously.

3.3.2 Task 2B. Identifying and Specifying Polymer Percentages

Experience has shown that specifying polymer percentage does not necessarily result in the expected performance because of differences in compatibilities between asphalt and polymers from different sources. Moreover, feedback received from industry participants at the St. Louis meeting in 2006 clearly indicates that suppliers view polymer quantity specifications as a practice which serves to inhibit innovation, a problem which can be remedied with the adoption of appropriate performance specifications.

Thus, performance testing rather than recipe specifications should result in the longest lasting, most cost-effective treatments, affording suppliers the opportunity to prescribe the polymer types, formulation methods, and mix design flexibility to meet agency and end-user requirements. Specific methods which are currently under consideration are discussed elsewhere in this report. Because of the importance of uniformity and compatibility to performance, it is recommended that the polymer not be post-blended with the asphalt emulsion in the field, particularly since both SHA and industry stakeholders have openly discouraged this practice.

Low temperature recovery of asphalt emulsion residues will simulate emulsion curing much more effectively than current recovery methods which are performed at temperatures that are far higher than these products will ever experience in the field. The high temperatures associated with currently used recovery methods have been shown to change the residue rheological properties, as the modulus is usually cut in half by heating the sample to 350º C, as opposed to using a low temperature Forced Draft Oven Method. Also, phase angles from high temperature distillation suggest that heating can cause cross-linking and damage to polymer additives. Therefore, it is recommended that a low-temperature method be adopted which is more representative of field curing conditions. Several such methods are under investigation by various researchers, with the leading candidate being a Forced Draft Oven Procedure that is similar to a recent European standard and which is expected to be presented to ASTM later this year.

Rheological performance tests on the residue should identify the polymeric properties as well as high-float gel structures. While there is some concern that performance testing will be more time-consuming and result in shipping, construction and acceptance delays, a supplier pre-certification or delayed-acceptance program should facilitate the process.

3.3.3 Task 2C. Projected Performance and Cost

Costs vary significantly from region to region, depending upon the local costs and local availability of emulsified asphalt and aggregate materials, contractors and expertise. Section 2.10 and tables 11 and 12 above give more information on the projected cost-effectiveness and extended performance of PME. For the field projects completed in this study, the emulsion costs were atypical because of an unprecedented shortage and spike in costs of asphalt, fuel and polymers in 2008. Costs vary according to geographical location, project size, mobilization, time of year and availability of materials and contractors.

3.3.4 Task 2D. Further Investigation

There are several data gaps in the available information. Nearly everyone in the industry believes that specifications for PME chip and slurry seals need to be changed so that they better predict field performance. While Superpave greatly improved the specifications for HMA, the tests and specifications developed are not necessarily the same criteria needed to specify performance for PME applications, but the tools may prove useful, albeit in some modified form. In fact, there are several studies independently investigating these. A “PG-type” system consistent with the base asphalts used by the binder industry and dependent upon binder rheology and climatic and traffic conditions would be generally acceptable, if it does not disrupt the supply and truly relates to PME surface treatment performance.

The "Strawman" specification given in table 23 suggests a promising series of protocols, but data gaps are significant. When collected for "report only", this data will be used to validate or adjust these methods as related pavement performance dictates. Most suppliers indicate they would be willing to pay for or perform these tests on upcoming FLH projects. Two laboratories (PRI and Paragon) have committed to run the tests (as specified by FLH but paid for by the suppliers) for those suppliers that currently do not have the in-house testing capability. FLH routinely evaluates pavements as part of its Pavement Management System. The laboratory data and field performance information collected will be evaluated at a later time to prescribe tests that are effective, repeatable, and have definable physical properties that can be tied to pavement performance. Hence, there will be an ongoing need for project oversight beyond the conclusion of the current study.

3.4 Delayed Acceptance - Approved Supplier Certification

The length of testing time has been one of the main obstacles to implementation of low temperature residue recovery and rheological testing (low temperature recovery procedures can take two or more days). Suppliers, contractors and agencies are all concerned that a lengthy test procedure would greatly disrupt the current way asphalt emulsions are manufactured and shipped. Suppliers also do not want different specifications and pre-certification requirements for different geographic regions or markets. Similar concerns during the development of Superpave resulted in an Approved Supplier Certification Program to allow shipping from authorized suppliers before testing is completed. AEMA and FHWA have been contacted, and FHWA's ETG Emulsion Task Force has assigned a sub-committee to develop such a program for emulsions.

Due to unique purchasing requirements for FLH, this program would be written under guidelines for "Delayed Acceptance" rather than in the format of an Approved Supplier Program as preferred by AASHTO.

3.5 Strawman "Report Only" Draft Specification

To simulate field performance, all protocols will ideally avoid heating to temperatures above possible field conditions. That means a low-temperature recovery method should be used, and the residue recovered should not be reheated for further testing. A Forced Draft Oven procedure using a silicone mold is preferred, because the residue can be easily removed from the mold without reheating.

Table 23 illustrates a draft Strawman "report only" testing protocol for recovery and eventual specification of PME residues. It includes rheological testing using a DSR for a minimum $G^*/\sin\delta$ and a maximum phase angle to determine polymer properties. The DSR is further used in the MSCR mode to determine recoverable strain and j_{nr}. High temperature testing will be done at the high temperature (T_h) grade for the base asphalt if known, and two additional temperatures in 6° C increments above that. It is suggested that new DSR test methods be developed to predict low temperature physical properties so that the BBR would not be needed for specification of asphalt emulsion residues. One logical approach to this problem is to use cone and plate geometry in the DSR to evaluate G* and phase angle at temperatures ranging from 0-20º C, and then use the CAM model to predict low temperature properties.

If DSR extrapolation methods cannot achieve sufficient accuracy, then new sample preparation procedures would be needed to make BBR a viable tool for classifying asphalt emulsion residues. High-float gel characteristics will be captured through some yet-to-be-determined method of defining non-linear pseudo-plastic behavior. DSR plots of ln(G*) versus shear rate or determination of a yield stress should be able to replace the antiquated float test with more quantitative measures of gel strength. For long-term residue aging, the PAV is believed to be the best alternative.

Although questions remain as to a specific aging protocol, rheological tests on PAV residue should characterize low-temperature behavior after aging (brittleness, raveling potential) and answer the question of what happens to the modified binder as it ages. Other research teams at WRI and ARC have been approached regarding the possibility of altering their second year work plans to develop methods for the DSR low-temperature specifications, the gel characterization and PAV aging of asphalt emulsion residues. Discussions with project principal investigators and FHWA project managers are ongoing. The expectation would be that the ongoing FLH report-only field study would be used to evaluate proposed methods and specifications that might come from that research.

It is expected that samples will be collected and tested from three FLH field projects during the summer of 2008 and at least one in 2009 as a test run of the report-only concept.

Table 23: Strawman "Report Only" Draft Specification - PME Residue

Purpose	**Test**	**Conditions**	**Report**
Residue Recovery	Forced Draft Oven	24 hrs @ambient + 24 hrs @60ºC	% Residue
Tests on Residue from Forced Draft Oven			
High Temperature (Rutting/Bleeding)	DSR-MSCR DSR freq sweep	T_h T_h	Jnr G* & phase angle
Polymer Identifier (Elasticity/Durability)	DSR-MSCR	T_h @3200 Pa	% Recoverable Strain
High Float Identifier (Bleeding)*	DSR – non-linearity	T_h	*Test to be developed*
Tests on PAV (run on emulsions evaporated in the PAV pan using the Forced Draft Oven procedure)			
Low Temperature (Aged Brittleness)*	DSR freq sweep	10ºC & 20ºC Model Low Temperature	G* Phase Angle
Polymer Degradation (Before/After PAV)*	DSR-MSCR	T_h @3200 Pa	Recoverable Strain Ratio

3.6 Design and Performance Testing

This section presents guidance on design and performance testing. Covered areas include aggregate-asphalt interactions and laboratory design procedures.

3.6.1 Aggregate-Asphalt Interactions

Both the short and long term performance (curing time, adhesion, skid resistance, long term chip retention and durability) are dependent upon the aggregate physical properties and the asphalt-aggregate compatibility as well as the physical properties of the emulsion. Performance testing is needed on both aggregates and the combination of PME and aggregate.

There are several well-accepted performance tests for aggregates. It is clear that cleanliness, shape and durability (as tested by MicroDeval or LA abrasion) are directly related to performance. Aggregate surface chemistry becomes increasingly more important when cure-time-to-traffic is critical to performance.

3.6.2 Laboratory Design Procedures

Chip Seals: The literature review mentions a few of the many design procedures for chip seals, most of which have evolved from McCloud's original work. Dr. Kim's recent studies for NCDOT specifically address aggregate quality, evaluate various design procedures for chip seals and offer excellent recommendations that should be considered for FLH guidelines.[(103)] Although the current ASTM method needs modest revision, the Sweep Test is viable for ranking curing time, and should be included in the FLH field study. While there are several laboratory test methods for long-term chip seal performance, none has universal acceptance. This is an area where further study is needed, and that is currently being investigated by other research projects such as NCHRP 14-17. If possible, the FLH report-only study should remain flexible to include recommendations from such projects as they become available. The MMLS3, as developed in S. Africa and as investigated by Dr. Kim and Dr. Epps, remains a valuable performance testing tool.[(100)] It can be run wet or dry and its rubber tires simulate unidirectional traffic loading on samples. At approximately $100,000, the machine cost is prohibitive as a specification tool, but it can serve as an accelerated simulator for field performance to accelerate validation of other methods.

Microsurfacing / PME Slurry: Current ISSA mix design and performance testing guidelines offer acceptable performance standards for microsurfacing.[(39)] However, better residue specifications and improved mix design protocols are still needed. As discussed elsewhere, the Fugro pooled-fund study should serve as a source for new tests and methods applicable to microsurfacing mix design.

4.0 PME TEST PLAN AND STRAWMAN SPECIFICATION

4.1 Strawman Specification for Emulsion Residues

With input from a number of researchers and users and approbation from Federal Lands Highway, the suggested Strawman Specification was developed (see table 23). Note that the BBR will be replaced by low temperature parameters modeled from intermediate temperature DSR results.

4.2 Testing Plan

To verify the format of the Strawman specification, a testing plan was developed as part of this study for use as report-only for several Federal Lands Highway field projects scheduled for completion in 2008 and 2009. Additional tests will be run during this time to better define the test conditions and limits. Laurand Lewandowski of PRI Asphalt has worked closely with the project research team to develop the proposed testing plan presented herein.

PRI will be equipped to run all proposed tests for those suppliers or agencies that do not currently have the capability. Several suppliers have indicated that they do have the test equipment and expertise needed. Several laboratories have and will perform the testing (PRI, Inc., BASF Corp., Paragon Technical Services, Inc. and SemMaterials, LLC). While the testing during this evaluation has an estimated cost of \$2,000 to \$3,000 per asphalt emulsion, it is expected that the final specification tests will cost approximately \$1,000.

The full list of PME Testing Plan protocols for the 2008 and 2009 evaluations is provided below in table 24. The labs used 1.) the proposed ASTM low temperature forced draft oven method modified by Lubbers, Takamura, and Kadrmas to recover original residue, and 2.) a newly developed method using pressure aging vessel (PAV) pans to recover residue prior to PAV-aging. To determine resistance to rutting and bleeding, G* and sin delta were obtained from dynamic shear rheometer (DSR) frequency sweeps on the residues using standard Superpave protocols. Creep compliance and percent residue recovery were determined via multiple stress creep recovery (MSCR) testing. Three rheological tests were run to measure resistance to low temperature cracking, including:

1) Frequency sweeps a 0, 10, and 20°C.
2) DSR using 4-mm plates at the low pavement temperature (Western Research Institute).
3) Low temperature Bending Beam Rheometry (BBR).

For resistance to aggregate loss (shelling) on original and PAV-aged residue, participants ran strain sweep tests at 25°C and measured loss in G*. Further, sweep testing (ASTM D 7000) using project aggregates and emulsions was used to determine chip seal curing time. FLH will use their road rating trailer to track initial and long-term field performance over a minimum 3-year interval. These field results will be correlated with lab data to 1) validate the test procedures and 2) to determine appropriate failure limits to allow for the development of performance-related specifications for polymer-modified emulsion pavement preservation applications for FLH projects.

Table 25 is a summary of the field project information and assigned responsibilities.

Table 24: Testing Plan Protocols for 2008 and 2009 Evaluations

PROPERTY	TEST METHOD		SPEC	RESULT
Asphalt Emulsion as Received				
Standard AASHTO or ASTM tests:	AASHTO M-140 Emulsified Asphalt or AASHTO M-208 Cationic Emulsified Asphalt			
Field Viscosity Test	WYDOT 538.0		Report	
Evaporative Method Residue (24 hours @ 25°C, 24 hours @ 60°C, Forced Draft Oven)				
Frequency Sweep (25 mm, 0.1 – 100 rad/sec, 10% Strain)	HTG*	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Multiple Stress Creep Recovery (100, 1000, 3200 & 10,000Pa)	HTG*	TP 70-08	Report	% Recovery & Jnr at each stress level
Frequency Sweep (25 mm, 0.1 – 100 rad/sec, 10% Strain)	HTG - 6°C	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Multiple Stress Creep Recovery (100, 1000, 3200 & 10,000Pa)	HTG - 6°C	TP 70-08	Report	% Recovery & Jnr at each stress level
Frequency Sweep (25 mm, 0.1 – 100 rad/sec, 10% Strain)	HTG -12°C	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Multiple Stress Creep Recovery (100, 1000, 3200 & 10,000Pa)	HTG -12°C	TP 70-08	Report	% Recovery & Jnr at each stress level
Test Strain Sweep, 1 – 50% strain, 10 rad/s	25ºC		Report	Resist to Deformation: G*/sind @ 12% Strain Strain Tolerance: Strain Level at G* <90%G*ini Failure Properties: Strain Level at G* <50%G*ini
Pressure Aging Residue (100°C, 300 psi, 20 hours) R 28 (PAV run on residue obtained by Forced Draft Oven Method run in PAV pan)				
Frequency Sweep (25 mm, 0.1 – 100 rad/sec,1% Strain)	HTG*	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Multiple Stress Creep Recovery (100, 1000, 3200 & 10,000Pa)	HTG*	TP 70-08	Report	% Recovery & Jnr at each stress level
Frequency Sweep (25 mm, 0.1 – 100 rad/sec,1% Strain)	HTG - 6°C	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Multiple Stress Creep Recovery (100, 1000, 3200 & 10,000Pa)	HTG - 6°C	TP 70-08	Report	percent Recovery & Jnr at each stress level
Frequency Sweep (25 mm, 0.1 – 100 rad/sec,1% Strain)	HTG -12°C	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Multiple Stress Creep Recovery (100, 1000, 3200 & 10,000Pa)	HTG -12°C	TP 70-08	Report	percent Recovery & Jnr at each stress level
Frequency Sweep (8 mm, 0.1-100 rad/sec, % Strain (TBD))	0°C	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Frequency Sweep (8 mm, 0.1-100 rad/sec, % Strain (TBD))	10°C	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Frequency Sweep (8 mm, 0.1-100 rad/sec, % Strain (TBD))	20°C	AASHTO T 315	Report	Frequency Sweep (G*, delta, etc…)
Test Strain Sweep, 1 – 50% strain, 10 rad/s	25ºC			Resist to Deformation: G*/sind @ 12% Strain Strain Tolerance: Strain Level at G* <90%G* ini Failure Properties: Strain Level at G* <50%G*ini
Bending Beam Rheometer	-12°C + -18°C	AASHTO T 313		Stiffness + m-value
Performance tests for Chip Seals				
Sweep Test	Modified ASTM D-7000		Report	
Performance tests for Polymer Modified Slurry Seals and Micro-Surfacing				
Recommended Performance Guidelines for Emulsified Asphalt Slurry Seal Surfaces	ISSA A105		ISSA	
Recommended Performance Guidelines for Polymer Modified Micro-Surfacing	ISSA A143		ISSA	
Tests recommended by Caltrans Slurry/Micro-Surface Mix Design Procedure Project /Contract 65A0151	TBD			

The projects include numerous project sites, at least three emulsion suppliers and multiple contractors. Climates ranged from very hot and dry (Death Valley National Park) to cold and wet, as well as extreme temperature ranges. Detailed information on the projects is given in table 25. Appendix B gives the specifications used to construct the projects.
The field projects constructed for this study include numerous project sites, at least three emulsion suppliers and multiple contractors. Climates ranged from very hot and dry (Death Valley National Park) to cold and wet, as well as extreme temperature ranges. Construction information on the projects is given in table 25, and the test plan is in table 24. The test results are in Chapter 5. The specifications used to construct the projects are in the appendix.

In late September, 2008, an 11-mile neoprene modified asphalt emulsion chip seal was placed at Dinosaur National Monument which spans the borders of Utah and Colorado.

The "Utah Parks" project included 90 miles of application of SBR latex modified CRS-2L and natural rubber modified microsurfacing to locations in Arches National Park, Canyonlands National Park, Natural Bridge National Monument, and Hovenweep National Monument in September and October 2008.

Death Valley National Park was the site of a 21-mile SBR latex modified asphalt chip seal project in November, 2008.

An additional project is planned in spring 2009 for Crater Lake National Park in Oregon. It is important to include the most commonly used and available polymer modified technologies. Because of the unusual industry supply situation during the oil crisis of 2008, it was not possible to include an SBS modified emulsion chip seal in the 2008 projects. It is hoped that an SBS modified chip seal will be included in this project. The data from this additional project should help complete the data set.

Photos of the projects are shown in figures 28-45.

Table 25: Project Construction Information and Testing Responsibilities, December 2008

Project and Status	Contractor	Supplier / Technology	Project Quantities & Costs	Lab Testing	Testing Completion Date
Dinosaur Project #: UT NPS DINO-PRES-1(08) Contract signed (8a small business negotiated). Production 9/23/08- 9/30/08 Project Engineer: Nick Maximoff	Hardrives Construction, Inc. 4800A Helfrick Rd, Billings, MT 59101	Chip seal emulsion: PASS® (solvent-less emulsion modified with neoprene) Asphalt Systems, Inc. – Salt Lake City, UT plant	~ 11.4 mile project ~ 165 tons of emulsion @ $1664/ton ~135,000 yd^2 chip sealing @ $1.65 per yd^2	PRI: emulsion & aggregates CFLHD Lab: acceptance testing only	Chip seal emulsion and aggregates has been received by PRI.
Utah Parks Project #: CO IMR-PRES-1(08) ARCH, CANY, NABR, & HOVE Production 9/6/08 - 10/17/08 Project Engineer: Joe Kosine 303-358-1915 (mobile)	Intermountain Slurry Seal, Inc 585 W. Beach St. Watsonville, CA 95075 Paul Foster, contact	Chip Seal Emulsion: CRS-Latex modified (SBR) Ergon – Snowflake, AZ plant Micro-surfacing: Ralumac® (natural rubber) SemMaterials – Salt Lake City, UT plant	~90 mile project ~1290 tons of CRS-LM @ $1495/ton ~1,140,000 yd^2 chip sealing @ $0.95 to $1.85 per yd^2 ~60,000 yd^2 micro-surfacing @ $4 to $5.75 per yd^2	PRI: Testing both chip & micro emulsion & aggregates. Paragon: chip emulsion & aggregates BASF: chip emulsion & aggregates SemMaterials: Micro emulsion NCHRP study (Shuler): chip emulsion & aggregates CFLHD Lab: acceptance testing only	Chip seal emulsion/ aggregates received by PRI, Paragon, and BASF Micro emulsion received by PRI (PRI will forward portion to SemMaterials) Shuler has completed field testing
Death Valley Project #: CA NPS DEVA 15(3). Contract signed (8a small business negotiated). Production started 11/11/08, completed 11/14/08 Project Engineer: Nick Maximoff	Hardrives Construction, Inc. 4800A Helfrick Rd, Billings, MT 59101	Chip Seal Emulsion: CRS-Latex modified (SBR) Western Emulsions – Irwindale, CA plant	~ 21 mile project ~ 290 tons of emulsion @ $1350/ton ~271,000 yd^2 chip sealing @ $1.27 per yd^2	PRI: emulsion & aggregates Paragon: emulsion & aggregates BASF: emulsion & aggregates CFLHD Lab: acceptance testing only	Chip seal emulsion and aggregates have been sent to PRI and BASF. (PRI will need to forward a set of samples to Paragon)
Crater Lake Project #: CA PWR –PRES-1(08) Contract signed (8a small business negotiated). Chip sealing in late spring 2009 Project Engineer: TBD	De Los Santos 444 SE Maple Dr. North Bend, WA 98045-9421	TBD (may be opportunity to include SBS technology)	~23 mile project ~420 tons of emulsion @ $1497/ton ~367,000 yd^2 chip sealing @ $1.98 per yd^2	TBD (Kraton potentially may fund if SBS technology used) CFLHD Lab: acceptance testing only	

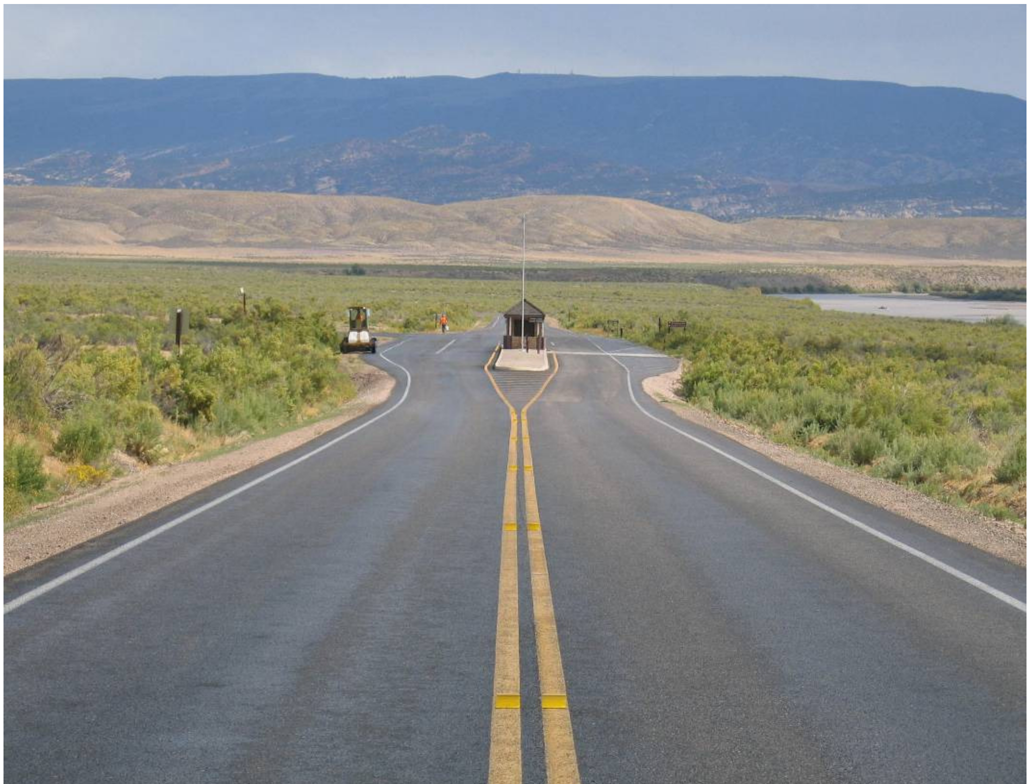


Figure 28: Dinosaur Project - Route 10 Park Pay Station

Figure 29: Dinosaur Project - Green River Campground, Loop 'B'

Figure 30: Dinosaur Project - Pay Station Chipsealing

Figure 31: Dinosaur Project - Loop 'B" After Completion

Figure 32: Dinosaur Project - Green River Access Road after Completion

Figure 33: Dinosaur Project - Route 10 after Completion

Figure 34: Utah Parks Project – Microsurfacing at Arches National Park

Figure 35: Utah Parks Project – Arches NP Partially Fogged

Figure 36: Utah Parks Project – Canyonlands NP Chip Seal Emulsion Application

Figure 37: Utah Parks Project – Canyonlands NP Chip Seal Chip Application

Figure 38: Utah Parks Project – Canyonlands NP Chip Seal Construction

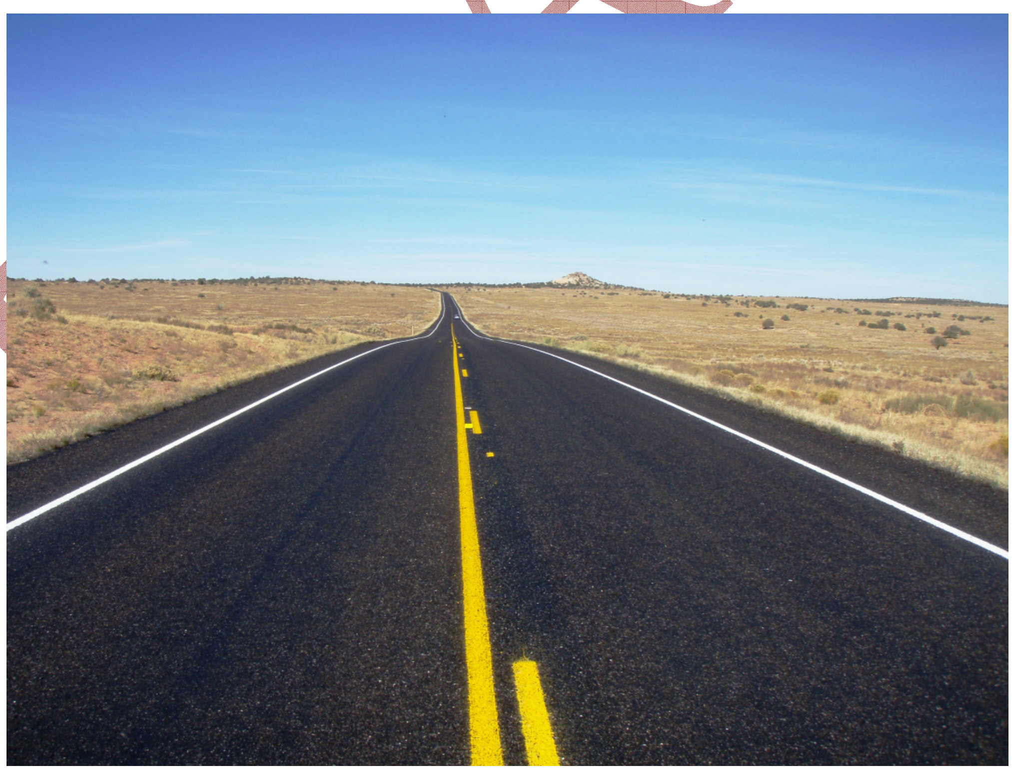


Figure 39: Utah Parks Project – Canyonlands NP Finished Chip Seal After Fog and Striping

Figure 40: Utah Parks Project – Hovenweep National Monument Microsurfacing

Figure 41: Utah Parks Project – Natural Bridges National Monument Chip Seal

Figure 42: Death Valley Project – Chip Seal Emulsion Application

Figure 43: Death Valley Project – Chip Seal Aggregate Application

Figure 44: Death Valley Project – Chip Seal Construction

Figure 45: Death Valley Project – Rolling the Chip Seal

5.0 FIELD TRIAL TEST RESULTS AND DISCUSSION

The tests were run according to the protocol in table 24. The emulsions tested were Ralumac LMCQH-1h (natural rubber latex modified), CRS-2P (SBR latex modified), CRS-2LM (SBR latex modified) and Pass Emulsion (neoprene modified).

5.1 Conventional Test Results

The Central Federal Lands Highway's Laboratory conducted conventional emulsion testing on field samples from all four projects, and Paragon Technical Services, Inc evaluated the CRS-2LM from the Utah Arches National Park project and the CRS-2P from the Death Valley National Park project. Paragon then tested these same emulsion samples using the full report-only protocol. The results of the conventional emulsion tests run by the Central Federal Highway Lands laboratory and Paragon are given in table 26. The microsurfacing tests run on the Ralumac project are in table 27. Other project quality control data, including aggregate testing, is available on the National Center for Pavement Preservation website at www.pavementpreservation.org.

All emulsions exceeded the minimum residue requirement of 65%, although many lab samples failed the minimum viscosity requirement of 100 SFS. This was not unexpected, as late season emulsions are often manufactured at the low end of the viscosity range, and emulsions viscosities tend to fall rapidly in unheated sample bottles. These failing results emphasize the previously recommended need for a field test for emulsion viscosity. No problems typical of low emulsion viscosity such as run-off or pooling were reported from the field trials, so there is no reason to believe these emulsions were not delivered to the project in specification. Similarly, no problems were reported with sieve or particle charge.

The three key residue tests in current PME specifications are penetration (25°C or 4°C), ductility (25°C and 4°C), and elastic recovery in a ductilometer. As noted in table 26, 25°C penetrations were 54 dm for Ralumac, 49 dm for CRS-2LM, and ranged from 57 to 77dm for CRS-2P. The penetrations for the Ralumac and CRS-2LM are typical of a PG 64-22 or AC-20, and the pen of the CRS-2P is in the range that would be expected for a softer PG 58-28 or AC-10. Pass Emulsion typically contains a blend of asphalt and rejuvenator oils, so the residue is typically much softer than conventional PME specifications would allow. It is therefore sold under its own somewhat proprietary specification and uses penetration at the lower 4°C to control consistency. Ductility at 25°C means very little for PME residues, because the strength of the polymer network can actually decrease ductility at higher temperatures. Ductility at 4°C is much more relevant for PME. Results for the CRS-2P emulsion supplied to Death Valley were quite erratic. Three field samples pulled from 10-18 cm, but the other two failed almost immediately. No other emulsions were tested for low temperature ductility. The CRS-2P was also the only emulsion tested for elastic recovery. Results for four field samples ranged from 48% to 68%, with two of those samples passing and two failing the specification minimum of 58%.

Like the FLH lab results, the Paragon emulsion viscosities for the two products tested were marginal to failing, but the long interval between application and testing renders these results relatively useless. Sieve, storage, settlement, demulsibility and particle charge results were all well within specification. Penetrations were somewhat softer than those reported by FLH (60 and 90 dm respectively), but the difference in consistency between these two residues remains about one full grade as defined by pen grading systems. Paragon used the California Torsional Recovery test (CA 332) as required by Utah specifications to define elastomeric properties of the polymer. The CRS-2LM residue recovery of 23.5% exceeded the 18% minimum required by the Utah specification; the Death Valley product would have failed this specification with a recovery of 15.7%, but torsional recovery was not part of the specification for this location. The torsional recovery test is regarded by the research team to be a very poor indicator for polymer content because it arbitrarily eliminates the early part of the recovery period from total relaxation.

Table 26: Conventional Emulsion Test Results on Field Trial Samples

Tests on Emulsion, T 59	Speci-fications	Death Valley CRS-2P						Dinosaur Pass Emulsion			Utah CRS-2LM	Utah Ralumac
		Field #1	Field #2	Field #3	Field #5	Field #12	Field #16	9/23 sample	9/24 sample	Supplier QC	Field #1	
Saybolt Furol Viscosity at 25C, s										120		
Saybolt Furol Viscosity at 50C, s	100-400	68.2	54.8	58.5	178	268	222	50.8	41.8		258	
Sieve Test, %	<0.1									<0.1%		
Particle Charge Test	Positive	Pass	Pass	Pass	Pass	Pass	Pass	Pass	Pass	pH 2.81	Pass	
Residue by Evaporation, %	>65	69.5	69.8	69.7	69.6	67.5	69.2			66	70.9	64.9
Tests on Residue												
Penetration at 25C (100g, 5s)T49	<86	67	67	77	72	57					49	54
Penetration at 4 C (100g, 5s) T49								20	19			
Ductility at 25 C, cm T51		132	122	113	150+	150+	150+	62	59		150+	
Ductility at 4 C, cm T51		8	1	10	18	17	1					
Elastic Recovery at 25 C, %, ASTM D6085	<58	58	55	68	68	48						
Rotational Viscosity, 275 F, cPa T316												2517

Paragon Test Results (T-59)	**Utah Specs**	**CRS-2P**		CRS-2HLM	
Sieve, %	<0.3	0.02		0.01	
50°C SFS Viscosity, Seconds	140-400	125		90.7	
24 Hour Storage, %	<1	0.03		0.06	
5 Day Settlement, %	<5	0.1		0.37	
Demulsibility, %	>40-	91.25		100	
Particle Charge	Positive	Positive		Pos.	
Distillation:					
Residue, %	>65	69.15		70.68	
Oil Distillate, % by volume	<0	0.25		0.125	
Test on Distillation Residue:					
25°C Pen, dmm	40-200	93		60	
25°C Ductility, cm	>125	150		150	
Torsional Recovery (CA 332)	>18	15.7		23.5	

Table 27: Microsurfacing Test Results

Test	Results	Min	Max
ISSA TB 113 Mix time	180 sec +	180 sec	
ISSA TB 139 Wet Cohesion	12 kg-cm @ 30 min 20 kg-cm @ 60 min	12@30 20(NS)@60	
ISSA TB 114 Wet Stripping	>95%	90%	
ISSA TB 100 Wet Track Abrasion, 1 hour	80.5 @ 9% emuls 26.9 @ 11 % emuls 25.7 @ 13% emuls 22.6 @ 15% emuls		75 g/ft^2
ISSA TB 106 Slurry Seal Consistency	2.9 cm	2 cm	3 cm
ISSA TB 102 Set Time	45 min		60 min
AASHTO T 176 Sand Equivalent	66	45	
AASHTO T *27* / T 11 Gradation			
3/8"	100	100	100
No.4	85	70	90
No.8	55	45	70
No. 16	39	28	50
No.30	29	19	34
No. 50	21	12	25
No. 100	15	7	18
No. 200	10.4	5	15

5.2 Report-only Test Results and Discussion

The proposed test plan protocols given in table 24 were run on samples from the field projects. This plan included more testing than would be expected for a performance specification (such as the strawman specification given in table 23), in order to gather information useful to determine the effectiveness, reliability, optimal test conditions, and potential specification limits of the proposed tests. PRI Asphalt Technologies led the lab testing phase of the performance-based report-only testing program. Laboratories at Paragon Technical Services and BASF also supported the study by providing results for the Forced Draft Oven recovery method, Sweep Test, and other procedures that needed multi-lab results to evaluate test reproducibility. The goals are to tie the test results to the performance of specific emulsion application, minimize the exposure of emulsion residue to excess heat and agitation (which are not present in the field), and maximize the use of the DSR to replace all other emulsion residue test equipment. The results are given below.

5.21 Recovery of Emulsion Residue by Forced Draft Oven

There is general agreement conventional emulsion residue recovery tests do not simulate field curing. The high temperatures are not seen in the field, they break down some polymers and cause additional cross-linking with others. The agitation of the hot, cured residue does not occur in the field. Such industry groups as AEMA, ASTM and European agencies have all been evaluating alternative methods, including the Forced Draft Oven (FDO) procedure, the Stirred Can Test and the Moisture Analyzer. The FDO was selected for the strawman (table 23) because it is run at conditions most closely simulating field conditions, and has given acceptable results with interlab reliability testing and comparison of residue properties with the properties of the base asphalt. It is also currently being considered by ASTM for adoption. Table 28 compares the results of the percent residue from the proposed and conventional tests.

Table 28: Comparison of Residue Recovery Test Methods

Test	Test Temp., C	Procedure	Spec	Ralumac LMCQS-1H	CRS-2P, Death Valley Project	CRS-2LM, Utah Arches	PASS Emulsion
Evaporative Method Residue (24hours @ 25C, 24 hours @ 60 C, Forced Draft Oven)							
Residue by Evaporation, %	25, 60	FDO Draft Method	Report	64.8	68.9	70.2	66.4
Conventional AASHTO Method							
Residue by Evaporation, %		T59		64.9	67.5-69.7	70.9	66.0

The FDO was run by Paragon Testing Laboratories, with slight modifications to the procedure currently under consideration by ASTM. There is still work to be done to determine how much aging the FDO procedure produces, i.e. if the FDO alters the initial base asphalt and polymer properties.

5.22 Residue Aging by Pressure Aging Vessel (PAV)

The aging protocol for performance-graded testing on asphalts for hot mix includes a Rolling Thin Film Oven Test (RTFO) to simulate aging in the hot mix plant and the Pressure Aging Vessel test (on residue obtained by RTFO) to simulate long term on-the-road aging. The RTFO is obviously not applicable to emulsions, but the PAV is now standard for long term field aging. This study prepared the samples for PAV by running the 48-hour FDO in the same PAV pans to be placed in the PAV. The residue from the completed PAV was then scraped and tested in the DSR, with no reheating or agitation required.

There are still some issues. Sufficient emulsion must be placed in the PAV pan to allow adequate film thickness of the FDO cured emulsion for the standard PAV test. There is some question if all the water is evaporated during the FDO run in the PAV pans. Thinner films age faster, which may be needed. The 100°C standard PAV temperature exceeds high pavement temperatures, which may alter cured latex structure. The procedure as outlined here appears to be viable, but more data needs to be collected to determine the optimal conditions for aging time and temperature for a given application.

5.23 Residue Testing – Residue Before and After PAV Aging

As mentioned above, the goal is a performance-based specification using a testing protocol that is efficient, reliable and accurately characterizes the field behavior. The report-only testing (table 24) performed in this study is meant to collect data over a broad range of temperature and loading conditions at a cost of approximately $4000 per sample. The ultimate specification will only use the test conditions needed for a specific application with a target testing cost of $1000 per individual certification. The results of the testing are given in table 29.

Table 29: Test Results from Test Plan Protocol

Test	Test Temp., C	Procedure	Spec	Ralumac LMCQS-1H	CRS-2P, Death Valley Project	CRS-2LM, Utah Arches	PASS Emulsion
Evaporative Method Residue (24hours @ 25C, 24 hours @ 60 C, Forced Draft Oven)							
Residue by Evaporation, %		Draft Method	Report	64.8	68.9	70.2	66.4
Water Content, %		ASTM D 95	Report	0	0	0	0
Frequency Sweep (25 mm, 0.1 - 100 rad/sec, 12% Strain		AASHTO T 315	Report	*	*	*	*
MSCR - % Rec (100 Pa)	70	TP 70-08	Report	31.46	16	21.9	10.72
MSCR - % Rec (1,000 Pa)	70	TP 70-08	Report	16.17	7.5	12.51	0.85
MSCR - % Rec (3,200 Pa)	70	TP 70-08	Report	11.12	5.85	7.26	0.11
MSCR - % Rec (10,000 Pa)	70	TP 70-08	Report	7.07	0.9	7.08	0.03
MSCR - Jnr (1,000 Pa) kPa-1	70	TP 70-08	Report	3.53	12.26	2.11	53.46
MSCR - J_{nr} (10,000 Pa) kPa^{-1}	70	TP 70-08	Report	4.71	16.2	2.89	74.06
MSCR - Jnr (100 Pa) kPa-1	70	TP 70-08	Report	2.7	10.32	1.81	40.53
MSCR - Jnr (3,200 Pa) kPa-1	70	TP 70-08	Report	4.09	13.12	2.52	60.09
Frequency Sweep (25 mm, 0.1 - 100 rad/sec, 12% Strain		AASHTO T 315	Report	*	*	*	*
MSCR - % Rec (100 Pa)	64	TP 70-08	Report	34.75	17.24	21.94	28.66
MSCR - J_{nr} (100 Pa) kPa^{-1}	64	TP 70-08	Report	1.34	4.67	0.94	16.48
MSCR - % Rec (1,000 Pa)	64	TP 70-08	Report	24.59	7.39	17.59	3.79
MSCR - J_{nr} (1,000 Pa) kPa^{-1}	64	TP 70-08	Report	1.59	5.5	1.01	27.06
MSCR - % Rec (3,200 Pa)	64	TP 70-08	Report	17.25	8.65	10.14	0.71
MSCR - J_{nr} (3,200 Pa) kPa^{-1}	64	TP 70-08	Report	1.92	5.74	1.19	32.09
MSCR - % Rec (10,000 Pa)	64	TP 70-08	Report	13.86	4.45	9.39	0.05
MSCR - J_{nr} (10,000 Pa) kPa^{-1}	64	TP 70-08	Report	2.2	6.59	1.38	39.25
Frequency Sweep (25 mm, 0.1 - 100 rad/sec, 12% Strain		AASHTO T 315	Report	*	*	*	*
MSCR - % Rec (100 Pa)	58	TP 70-08	Report	38.05	16.93	25.81	37.27
MSCR - J_{nr} (100 Pa) kPa^{-1}	58	TP 70-08	Report	0.63	2.068	0.45	7.29
MSCR - % Rec (1,000 Pa)	58	TP 70-08	Report	33.3	10	22.69	12.39
MSCR - J_{nr} (1,000 Pa) kPa^{-1}	58	TP 70-08	Report	0.68	2.3	0.46	11.78
MSCR - % Rec (3,200 Pa)	58	TP 70-08	Report	25.88	7.36	16.56	3.73
MSCR - J_{nr} (3,200 Pa) kPa^{-1}	58	TP 70-08	Report	0.81	2.53	0.52	14.68
MSCR - % Rec (10,000 Pa)	58	TP 70-08	Report	18.86	8.06	10.92	0.57
MSCR - J_{nr} (10,000 Pa) kPa^{-1}	58	TP 70-08	Report	0.99	2.71	0.63	18.63

Test	Test Temp., C	Procedure	Spec	Ralumac LMCQS-1H	CRS-2P, Death Valley Project	CRS-2LM, Utah Arches	PASS Emulsion
PRESSURE AGING RESIDUE (100 c, 300 psi, 20 hr)							
Frequency Sweep (8 mm, 0.1 – 100 rad/sec, 1% Strain)		AASHTO T 315	Report	*	*	*	*
MSCR - % Rec (100 Pa)	70	TP 70-08	Report	N/A	N/A	-	N/A
MSCR - Jnr (100 Pa) kPa-1	70	TP 70-08	Report			-	
MSCR - % Rec (1,000 Pa)	70	TP 70-08	Report	36.16	8.58	23.67	
MSCR - Jnr (1,000 Pa) kPa-1	70	TP 70-08	Report	0.54	1.55	0.75	
MSCR - % Rec (3,200 Pa)	70	TP 70-08	Report	23.36	0.56	10.25	
MSCR - Jnr (3,200 Pa) kPa-1	70	TP 70-08	Report	0.72	1.818	1.05	
MSCR - % Rec (10,000 Pa)	70	TP 70-08	Report	12.98	0	5.5	
MSCR - J_{nr} (10,000 Pa) kPa^{-1}	70	TP 70-08	Report	1.01	2.804	1.52	
Frequency Sweep (8 mm, 0.1 – 100 rad/sec, 1% Strain)	64	AASHTO T 315	Report	*	*	*	*
MSCR - % Rec (100 Pa)	64	TP 70-08	Report	N/A	N/A	-	N/A
MSCR - J_{nr} (100 Pa) kPa^{-1}	64	TP 70-08	Report			-	
MSCR - % Rec (1,000 Pa)	64	TP 70-08	Report	33.9	19.43	42.55	
MSCR - J_{nr} (1,000 Pa) kPa^{-1}	64	TP 70-08	Report	0.24	0.574	0.15	
MSCR - % Rec (3,200 Pa)	64	TP 70-08	Report	32.97	11.68	31.42	
MSCR - J_{nr} (3,200 Pa) kPa^{-1}	64	TP 70-08	Report	0.25	0.657	0.19	
MSCR - % Rec (10,000 Pa)	64	TP 70-08	Report	19.89	3.43	19.8	
MSCR - J_{nr} (10,000 Pa) kPa^{-1}	64	TP 70-08	Report	0.36	1.106	0.25	
Frequency Sweep (8 mm, 0.1 – 100 rad/sec, 1% Strain)	58	AASHTO T 315	Report	*	*	*	*
MSCR - % Rec (100 Pa)	58	TP 70-08	Report	N/A	N/A		N/A
MSCR - J_{nr} (100 Pa) kPa^{-1}	58	TP 70-08	Report				
MSCR - % Rec (1,000 Pa)	58	TP 70-08	Report	43.33	30.52	44	
MSCR - J_{nr} (1,000 Pa) kPa^{-1}	58	TP 70-08	Report	0.099	0.211	0.06	
MSCR - % Rec (3,200 Pa)	58	TP 70-08	Report	42.54	24.04	43.24	
MSCR - J_{nr} (3,200 Pa) kPa^{-1}	58	TP 70-08	Report	0.1	0.236	0.06	
MSCR - % Rec (10,000 Pa)	58	TP 70-08	Report	33.26	14.29	36.61	
MSCR - Jnr (10,000 Pa) kPa-1	58	TP 70-08		0.12	0.299	0.07	
Frequency Sweep (8 mm, 0.1 – 100 rad/sec, 1% Strain)	10	AASHTO T 315	Report	*	*	*	*
Frequency Sweep (8 mm, 0.1 – 100 rad/sec, 1% Strain)	20	AASHTO T 315	Report	*	*	*	*
Strain Sweep(8 mm, 1 – 50 % Strain, 10 rad/sec	25	New Method	Report	*	*	*	*
Stiffness, MPA (60 sec.) -18 C	-18	AASHTO T 313	300 max.	272	243	315	68
m- Value -18 C	-18	AASHTO T 313	0.300 min.	0.308	0.228	0.282	0.338
Stiffness, MPA (60 sec.) -12 C	-12	AASHTO T 313	300 max.	120	100	142	18
m- Value -12 C	-12	AASHTO T 313	0.300 min.	0.371	0.384	0.348	0.376
* These results are data sets currently under analysis by researchers working on related on-going projects. It is expected this data will be useful in combination with the data from those projects in developing future specifications and limits.							

5.24 Report Only Testing - MSCR

The AASHTO "Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)" TP 70-1 was selected to define the high temperature flow and the elasticity of PME residues. This procedure has been under development by FHWA, and has been published by AASHTO. The FHWA sponsored Binder Expert Task Group is currently evaluating target test criteria for hot mix asphalt binders. The current AASHTO test is run original, residue from RTFO and PAV aged binders. For these emulsion tests, it was run on FDO residues with as little manipulation of the sample as possible. The results are in table 29 above. Figures 46 through 48 are plots of the Jnr (compliance) versus the four tested stress levels at the three test temperatures.

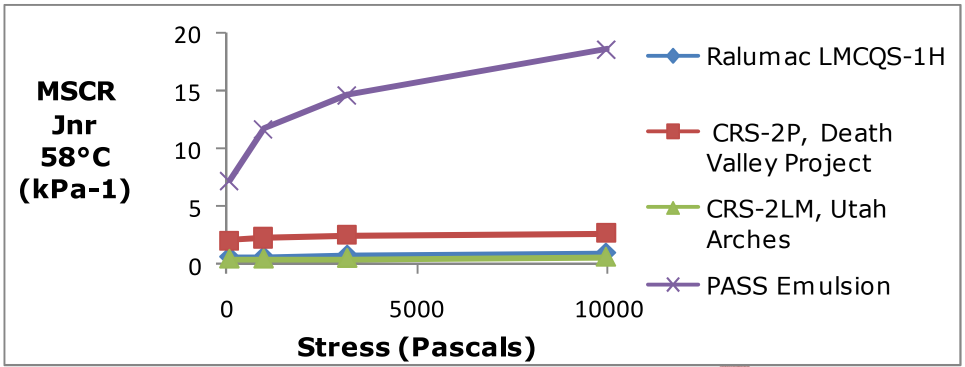


Figure 46: MSCR - Jnr vs. Stress for FDO Residues at 58°C

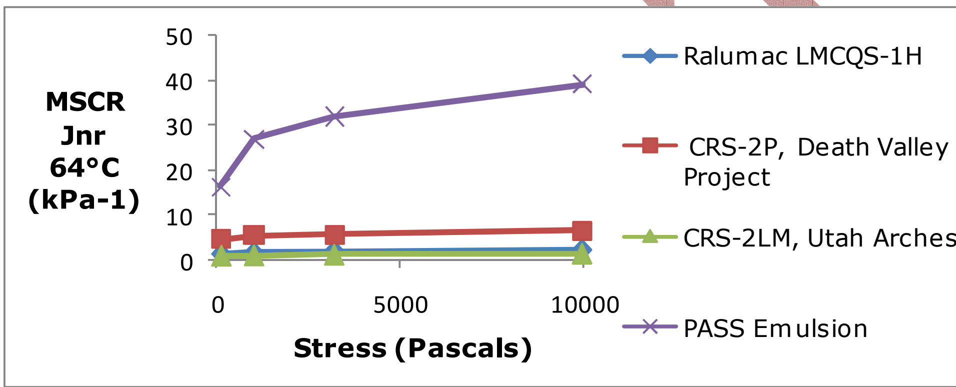


Figure 47: MSCR – Jnr vs. Stress for FDO Residues at 64°C

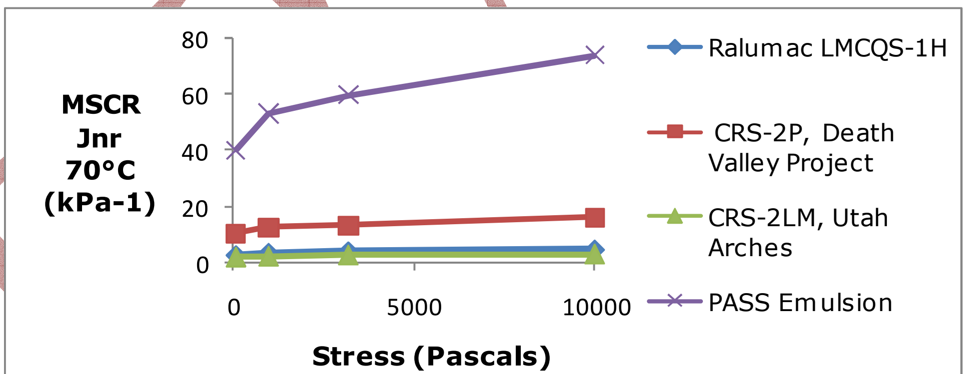


Figure 48: MSCR – Jnr vs. Stress for FDO Residues at 70°C

The differences in Jnr for the three chip seal emulsion residues were extremely high. For a stress of 3200 kPa applied at 64°C, Jnr values were 1.2 for Utah Arches (CRS-2LM), 5.7 for Death Valley (CRS-2P), and 32.1 for Dinosaur National Monument (Pass Emulsion). When grading HMA binders, a doubling of the Jnr represents a softening by approximately one full binder grade. This rule of thumb would suggest that the Death Valley CRS-2P is more than two grades softer than the Utah Arches CRS-2LM residue, and the Dinosaur National Monument Pass emulsion residue another two or three grades softer yet. This range seems excessive, and the grades as used have no relation to the high temperatures for the respective climates. These surprising results accentuate the need for urgency in developing performance based emulsion specifications. The Jnr for the microsurfacing emulsion (Ralumac with natural rubber latex) was 1.9, which was surprisingly higher than the Utah Arches chip seal emulsion. Normally, microsurfacing would contain a harder base asphalt and more polymer than a chip seal emulsion, both of which should push Jnr lower.

Figures 49 through 51 give the test results for the MSCR percent recovery versus the four tested stress levels at the three test temperatures.

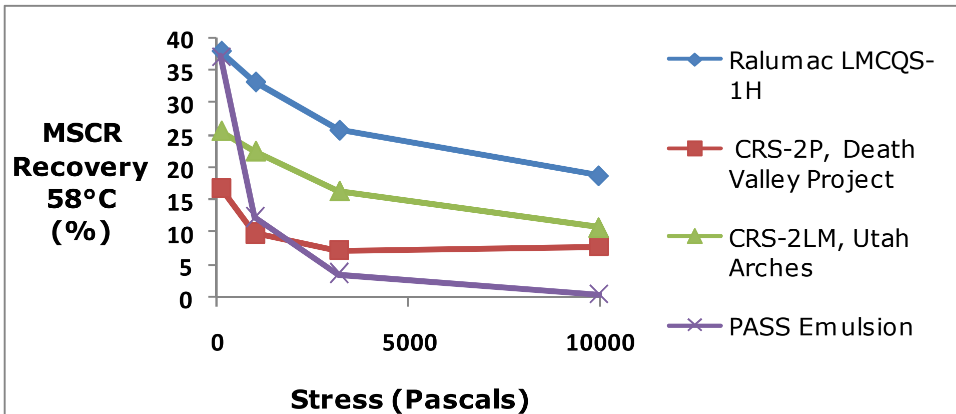


Figure 49: MSCR – % Recovery vs. Stress for FDO Residues at 58°C

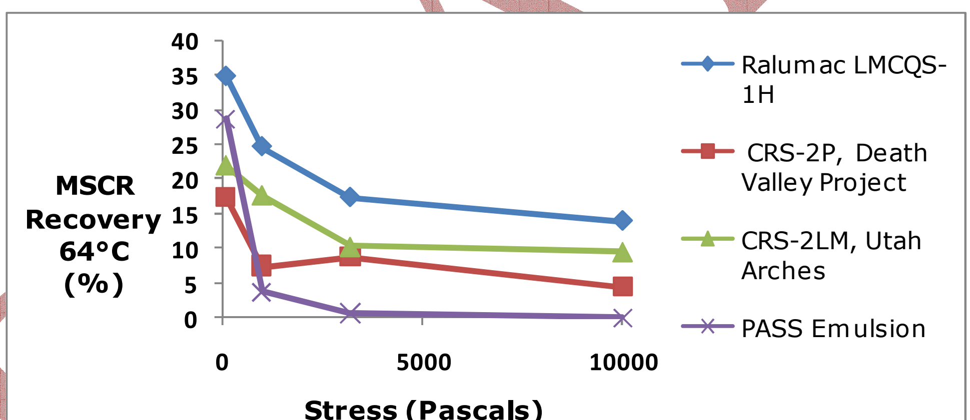


Figure 50: MSCR – % Recovery vs. Stress for FDO Residues at 64°C

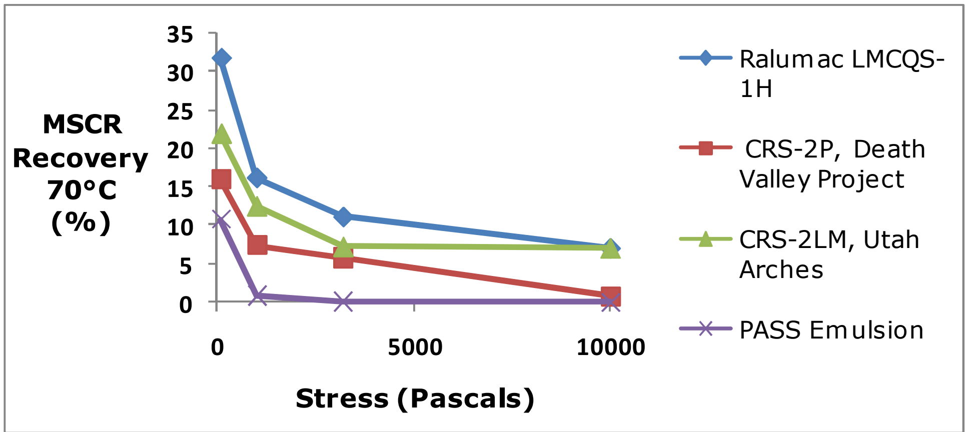


Figure 51: MSCR – % Recovery vs. Stress for FDO Residues at 70°C

Each of these emulsion residues contained polymer, and should therefore exhibit substantial recovery in the MSCR test. Again, however, there were huge differences in performance, particularly at the higher stress levels as recommended by FHWA for hot mix asphalt binders and by researchers for microsurfacing (table 22). Using a stress level of 3200 Pa at 64°C, the recoveries ranged from 0.7% for Dinosaur, 8.6% for Death Valley, 10.1% for Utah Arches, and 17.2% for microsurfacing. (The recommendation from table 22 is 25% for microsurfacing.) The higher value for microsurfacing is in line with expectations that it would contain more polymer. The low recovery of the Death Valley project can possibly be explained by the fact that it is a much softer residue at 64°C, and softer residues tend to recover less in the MSCR test. The plastomeric polychloroprene product (Neoprene) used for the Dinosaur project is not only very soft, but it also exhibits an almost gel-like tendency to completely loose elasticity as the stress increases. At 100 Pa, it has the best recovery of the three chip seal emulsions; however, at 3200 Pa, the Neoprene exhibits virtually no elasticity. It seems probable at this time that no single performance specification for emulsion chip seal residues could possibly cover the breadth of consistency and elasticity as evidenced by the elastomeric styrene-butadiene emulsions and the plastomeric Neoprene. If both families of products are found to be useful in the marketplace, independent performance specifications will be needed to define their respective residues. Of course bids could always allow either/or in competitive bidding situations where the user had no preference for the type of polymer to be used.

Figures 53 through 55 are plots of the Jnr (compliance) versus the four tested stress levels at the three test temperatures for the PAV aged residues of the products. Results for Pass emulsion PAV residue are not available; lab work is ongoing to understand testing issues that resulted in problematic data. Data from the aged residues for the other three emulsions were consistent and ranked in the same order as their unaged counterparts.

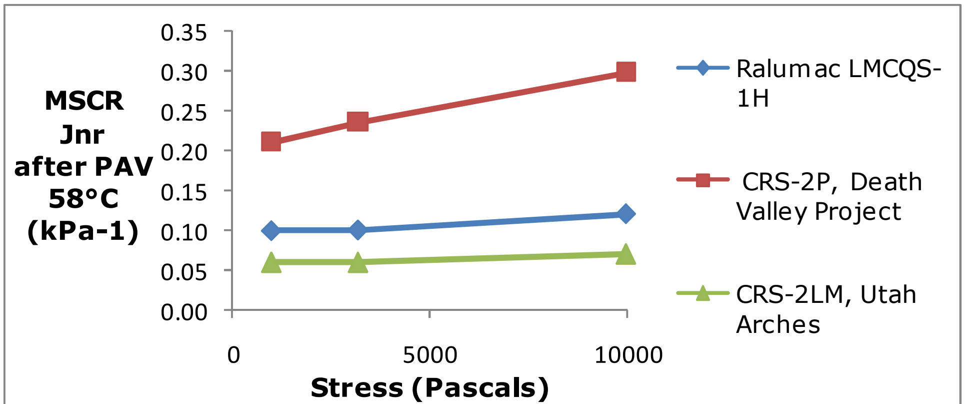


Figure 52: MSCR – Jnr vs. Stress for PAV Residues at 58°C

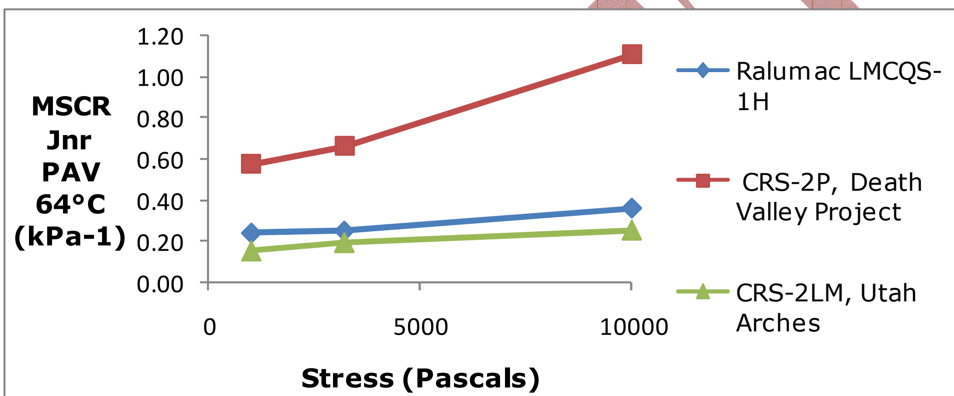


Figure 53: MSCR – Jnr vs. Stress for PAV Residues at 64°C

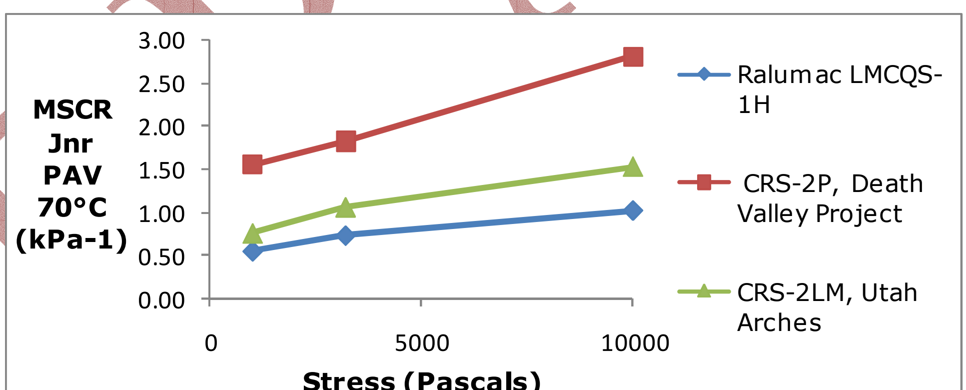


Figure 54: MSCR – Jnr vs. Stress for PAV Residues at 70°C

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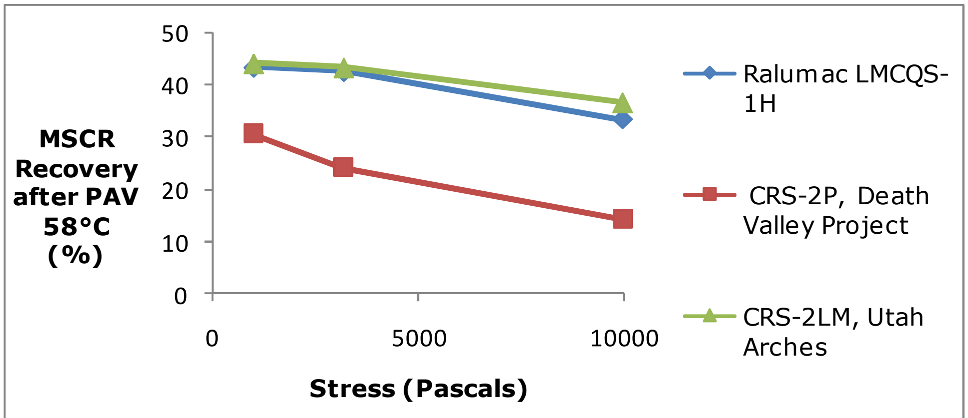

Figure 55: MSCR – % Recovery vs. Stress for PAV Residues at 58°C

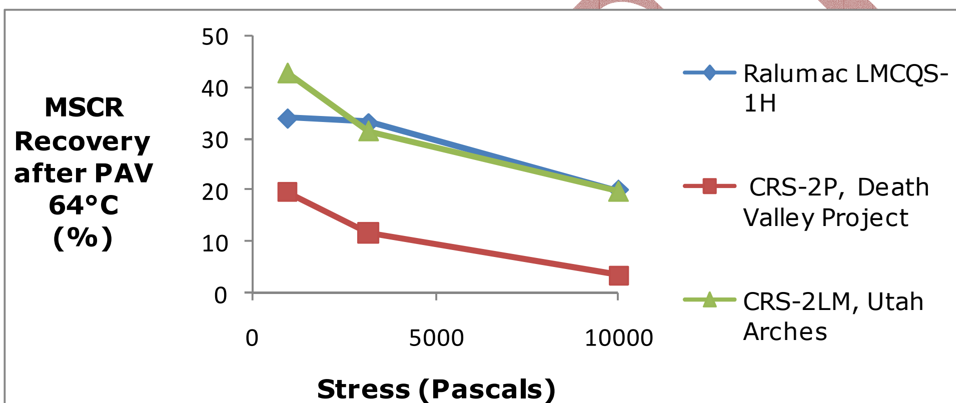

Figure 56: MSCR – % Recovery vs. Stress for PAV Residues at 64°C

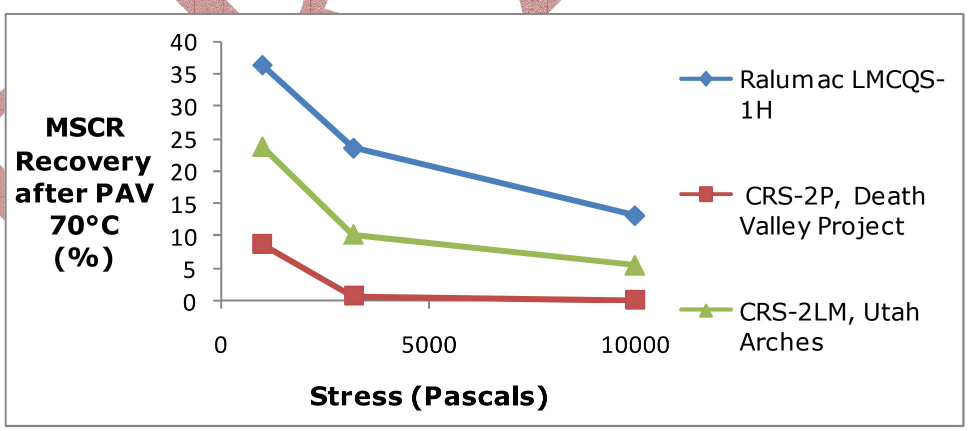

Figure 57: MSCR – % Recovery vs. Stress for PAV Residues at 70°C

Percent recoveries usually improve following PAV aging. First, harder residues produced through the aging process naturally exhibit better recovery at a given temperature. Secondly, some elastomeric polymers may cross-link to some degree during aging. This cross-linking should strengthen the polymer network and improve elasticity. The CRS-2LM product improved in elasticity relatively more than the other two latex products.

Figure 58 shows the change in Jnr with temperature at 3200 Pa.

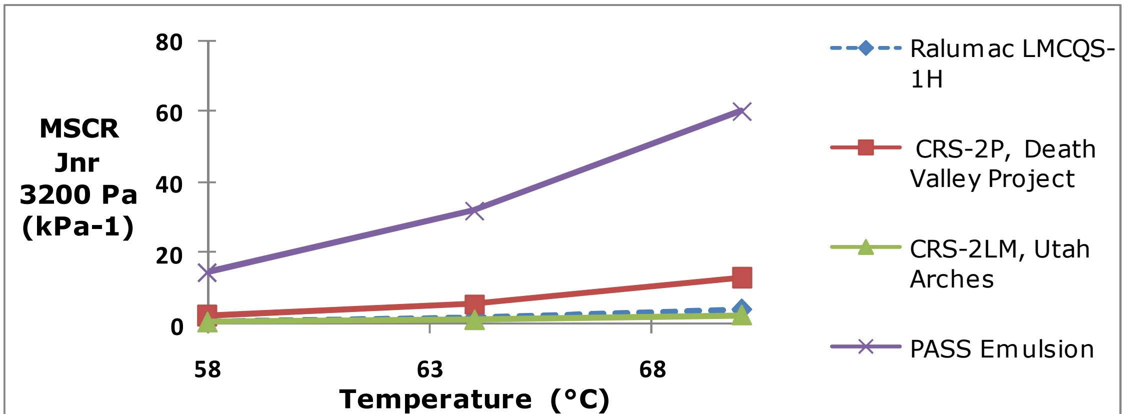

Figure 58: MSCR – Effect of Temperature on Jnr at 3200 Pa

As expected from newly developed HMA binder grading protocols, figure 58 confirms that the residue Jnr approximately doubles with each 6°C incremental increase in test temperature. There is every reason to believe it will be possible to use the climate maps created in LTPPBind to define and select appropriate emulsions grades for a given locale. However, the test conditions and specification limits must be adjusted to best fit the application.

Figure 59 shows the effect of temperature on MSCR recovery for the emulsion residues.

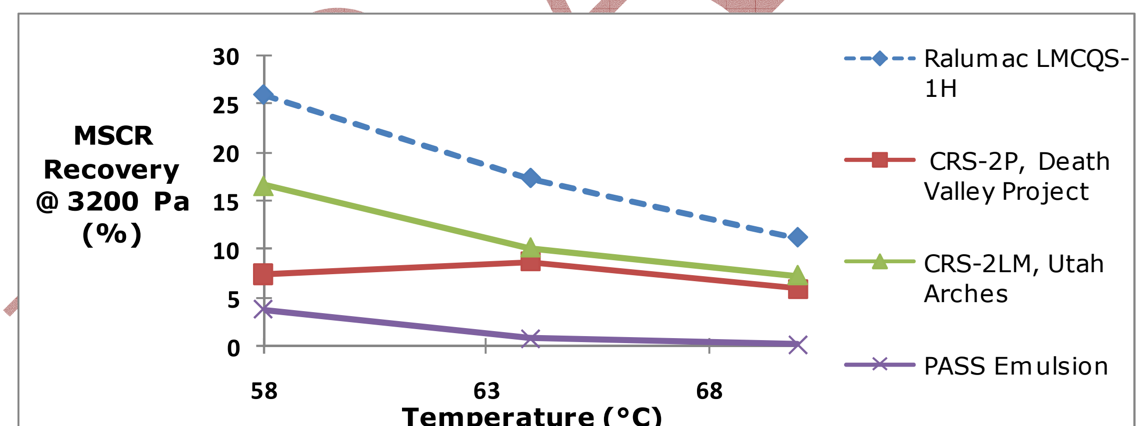

Figure 59: MSCR – Effect of Temperature on Recovery % at 3200 Pa

As mentioned earlier, percent recovery has a very strong dependence on the compliance (inverse modulus), or Jnr of the residue. For example, the highly modified microsurfacing emulsion has a recovery of 25.9 percent at 58°C, 17.2 percent at 64°C, and 11.1 percent at 70°C. The high susceptibility of the MSCR percent recovery to temperature is a disadvantage for specifications, because it will always be possible to improve acceptance results somewhat by making the residue harder rather than by adding polymer to improve recovery. Of course temperature dependence is also a problem for the current methods to define polymer elasticity, such as Elastic Recovery (ER) in a Ductilometer. At least new performance specifications will change the test temperature with each 6°C increment in climate temperature, whereas the classic ER test is always run at a single temperature regardless of grade. Further research might consider running MSCR at lower test temperatures where recoveries would be higher and possibly less sensitive to temperature.

Two sets of figures (figures 46-48 and 60-63) represent different ways to view the impact of stress on Jnr. One important reason for replacing the traditional high temperature PG grading parameter G*/sin delta with the MSCR parameter Jnr is that the latter enables the product specifier to select test conditions under the higher stress conditions associated with high traffic volumes on HMA or turning rubber tires on chip seals. Earlier PG specifications based only upon conventional asphalt could assume that asphalt is a linear viscoelastic material, and therefore the asphalt modulus G* should be constant for all applied strain rates, and therefore for all applied stresses. The MSCR test as developed during NCHRP 9-10 showed clearly that these fundamental assumptions do not apply to polymer modified asphalts. Nonlinearity is particularly evident for the softest, most highly polymer modified materials including PME residues.

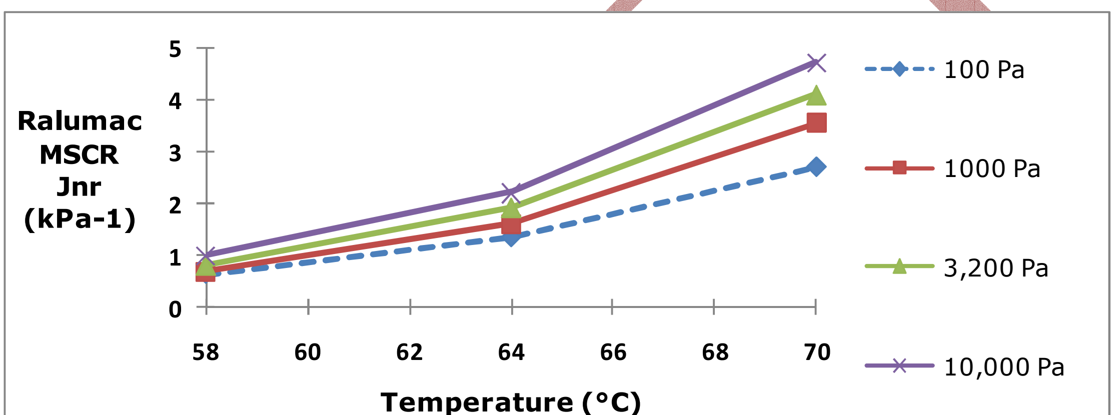

Figure 60: MSCR – Effect of Stress on Jnr for Ralumac

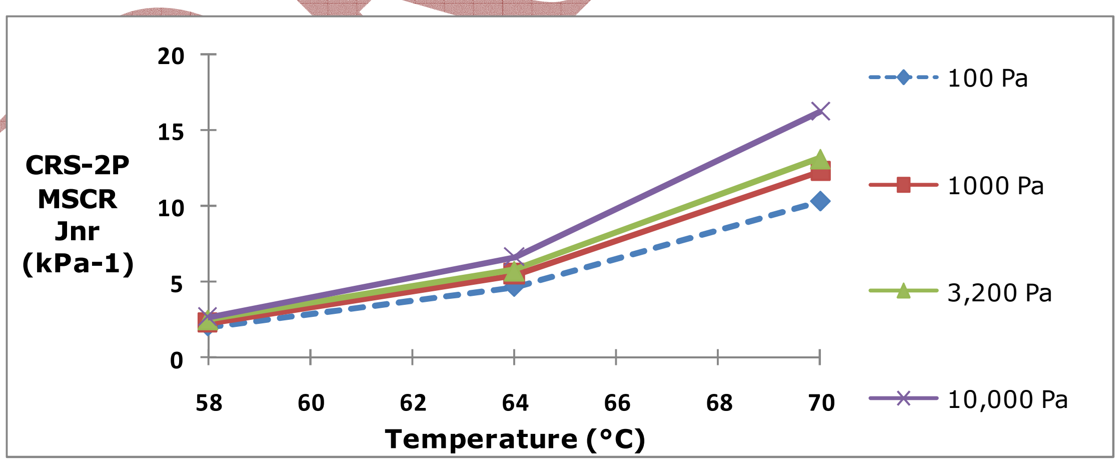

Figure 61: MSCR – Effect of Stress on Jnr for CRS-2P

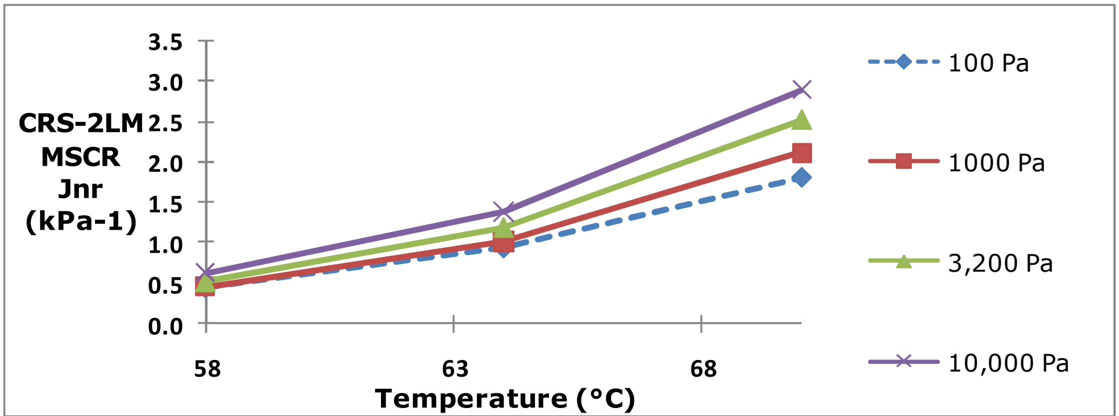

Figure 62: MSCR – Effect of Stress on Jnr for CRS-2LM

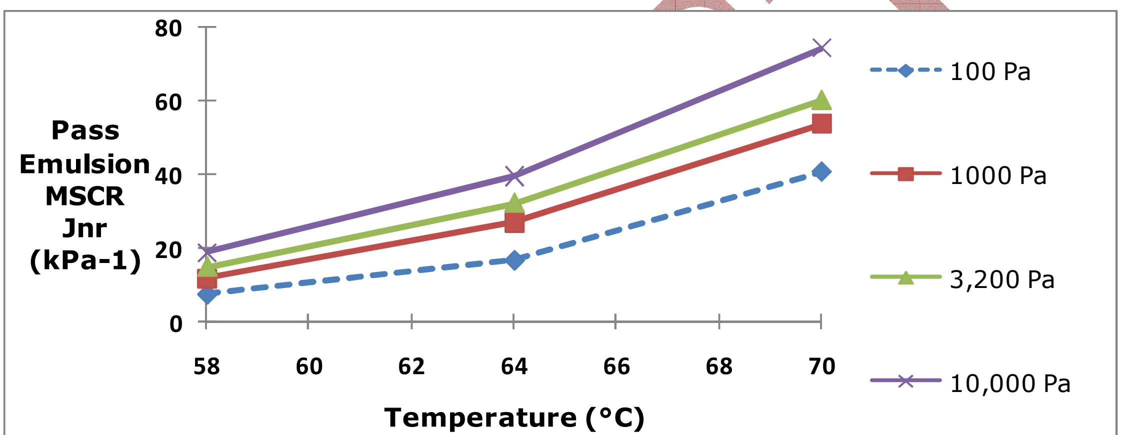

Figure 63: MSCR – Effect of Stress on Jnr for Pass Emulsion

The previously described plots in figures 46, 47 and 48 demonstrate how significant this non-linear behavior might drive material performance for each of the four products studied. The Jnr for all products increases with applied stress at all temperatures, but the relative non-linearity as expressed by the slopes varies dramatically from one PME residue to another. In layman's terms, all PME residues get softer as increasing load is applied, but the amount of load-induced softening is highly dependent upon the amount and type of polymer, as well as the grade of the base asphalt. Because the Pass emulsion residue is very soft and contains a plastomeric modifier, it is particularly sensitive to this stress softening effect at high temperatures. Two examples from the field study emphasize the importance of this effect. First, CRS-2LM at 58°C represents the hardest base residue at the lowest test temperature, and Pass emulsion at 64°C represents the softest base residue at the next higher test temperature. Under these respective conditions, the CRS-2LM Jnr changed from 0.45 to 0.52 when applied stress was increased from 100 Pa to 3200 Pa. With the same stress change, Jnr for the much softer and hotter Pass emulsion increased from 16 to 32. On a temperature grading scale typical of the PG grading system, the CRS-2LM in a PG 58 climate would lose two degrees and the Pass emulsion in a PG 64 climate would lose 6°C, or one full high temperature grade, due to this non-linearity at the higher 3200 Pa stress level recommended by the FHWA's Binder Expert Task Group for HMA applications. This same phenomenon can be viewed in another way be reviewing the four plots in figures 60, 61, 62 and 63. The widening gap in JNR as temperature increases is consistent with the fact that softer materials exhibit more non-linearity. Higher applied stress results in higher strains, while softer materials or hotter liquids yield more at any given stress. Hence, increasing stress, increasing temperature, or softening the base binder all push the results further into the non-linear region. This effect, when viewed from a chemist's point of view, is really a strain dependent issue related to the polymer structure. Very long polymer molecules entangle much like long hair blowing in the wind. These entanglements enable the polymer network to resist flow to a degree much higher than its molecular weight alone would imply. However, as these tangled chains are stretched and unwound, the additional elasticity provided as entropy effects re-entangle chains is lost. Hence, the polymer network becomes weaker and less elastic as it is stretched to the point that chains begin to disentangle. These effects are tied to the higher applied strains, regardless of cause (higher stress, higher temperatures or softer base asphalts). Since polymers vary widely in chain length and molecular structure, the strain at which these effects become important can vary dramatically. This is not surprising; the behavior is the same as woven cloth being much stronger than the individual threads.

Figures 64 through 67 illustrate the effect of stress level on MSCR recovery percent.

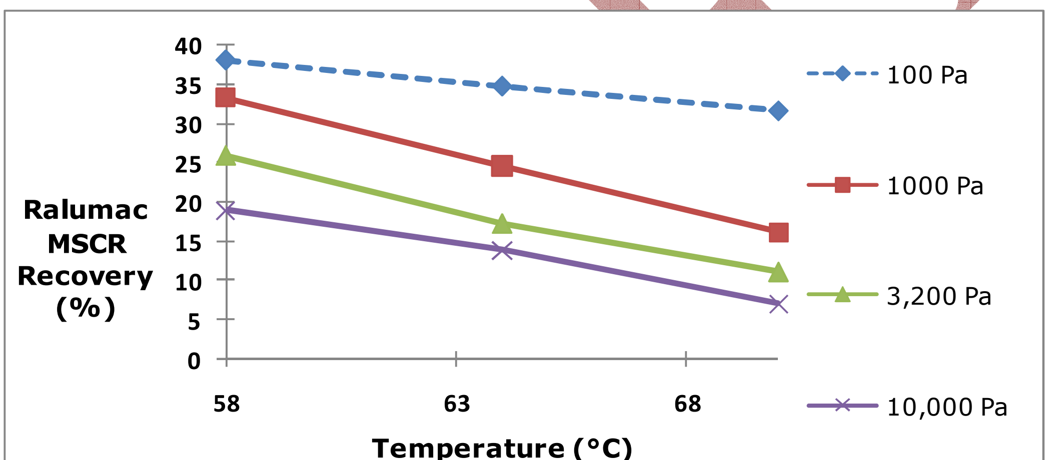

Figure 64: MSCR – Effect of Stress on Recovery for Ralumac

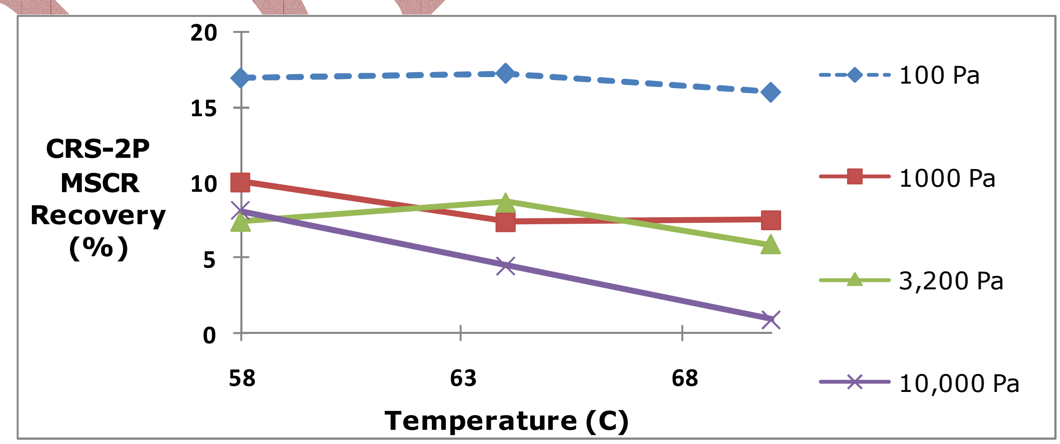

Figure 65: MSCR – Effect of Stress on Recovery for CRS-2P

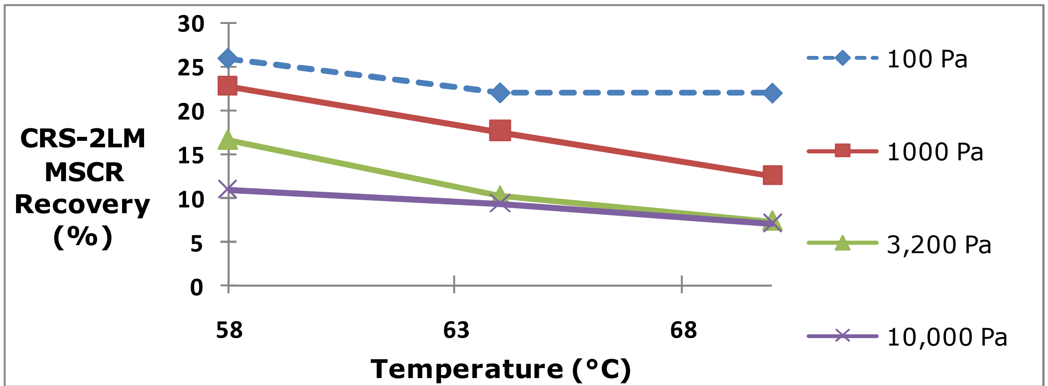

Figure 66: MSCR – Effect of Stress on Recovery for CRS-2LM

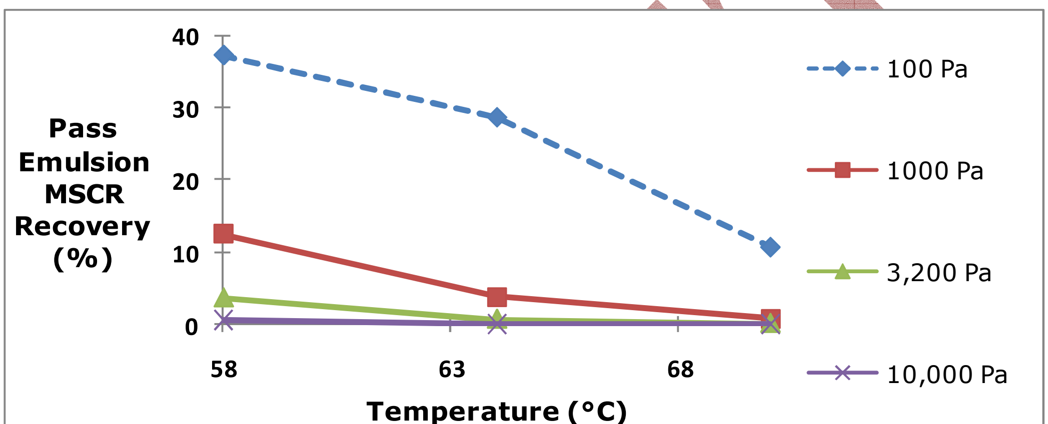

Figure 67: MSCR – Effect of Stress on Recovery for Pass Emulsion

The effects of increasing applied stress on percent recovery are considerably more dramatic than those impacting Jnr. As mentioned above in the discussion of figures 49, 50 and 51, recovery is always reduced when higher stresses result in higher strains which dislodge polymer chain entanglements. However, the percent recovery for the Pass emulsion at 64°C fell from a relatively high 28.8 percent to less than one when the applied stress was increased from 100 to 3200 Pa. The elastomeric materials were also highly sensitive to stress, but maintained reasonable elasticity even at the highest stress levels. It is also interesting to note from figures 64, 65, 66 and 67) that the percent recovery for CRS-2P at different temperatures is surprisingly insensitive to applied stress up to 3200 Pa. CRS-2LM and Ralumac show moderate declines in percent recovery as temperature increases, while percent recovery for the Pass emulsion is extremely sensitive to both temperature and applied stress. It seems most logival to compare recoveries of different products using an equi-stiffness approach. Unfortunately, lab procedures would be too time consuming and costly for product specifications. A simpler climate-based grading system for percent recovery does appear to be a workable solution.

Figures 68 through 70 show the change in Jnr at 64°C after PAV aging.

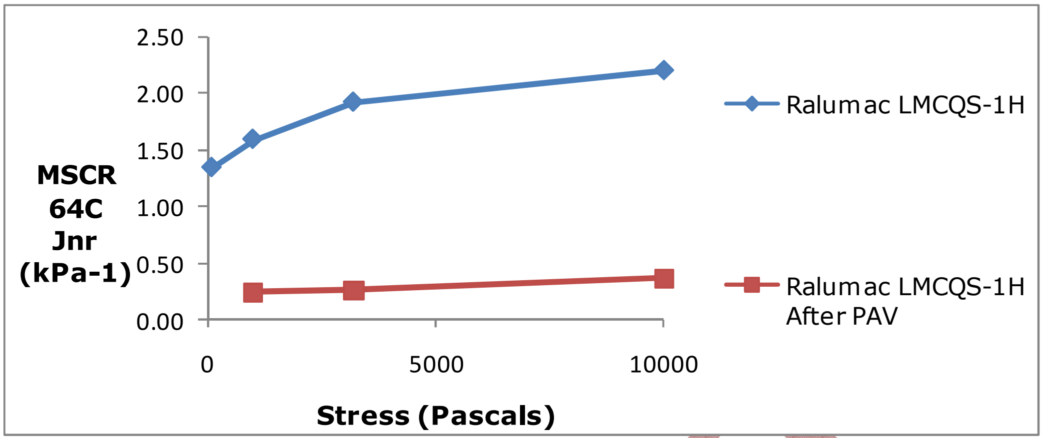

Figure 68: MSCR – Effect of Aging on 64°C Jnr for Ralumac

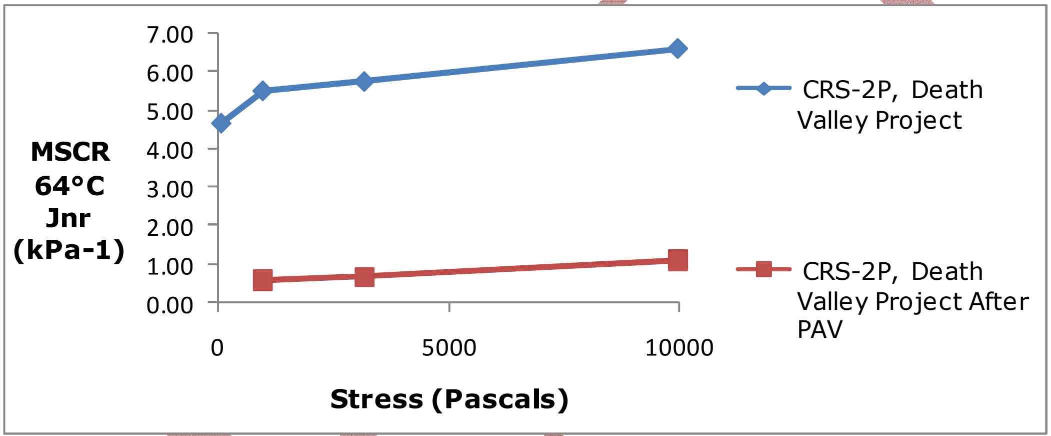

Figure 69: MSCR – Effect of Aging on 64°C Jnr for CRS-2P

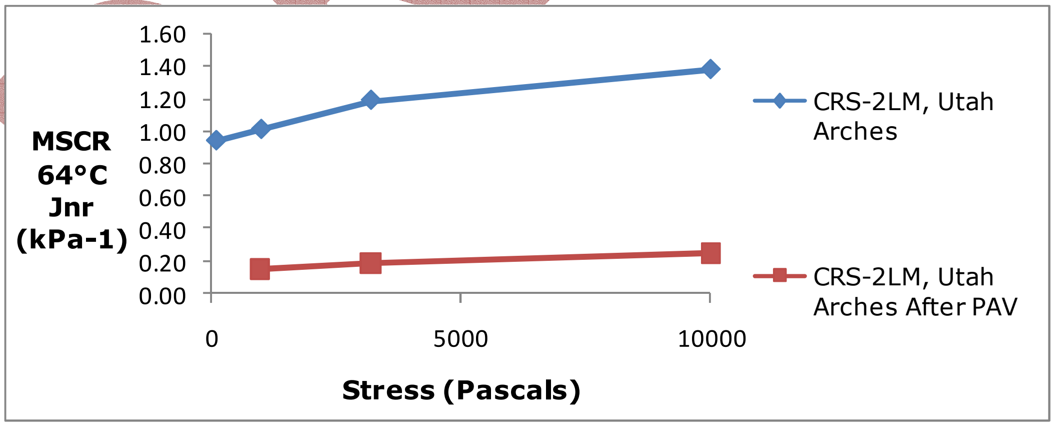

Figure 70: MSCR – Effect of Aging on 64°C Jnr for CRS-2LM

The evolution in Jnr with PAV aging (20 hr, 100°C) was evaluated for three of the four products (The aging data for the Pass emulsion was not reproducible, so the results are not reported here.) For the intermediate test conditions of 64°C and 3200 Pa, the Jnr fell with aging as follows:

- Ralumac: from 1.92 to 0.25.
- CRS-2P : from 5.5 to 0.66.
- CRS-2LM: from 1.19 to 0.19.

Based upon previously cited rules of thumb, the Ralumac and CRS-2P hardened by three high-temperature grades in the PAV(approximately 18°C change in equi-service temperature), and the CRS-2LM hardened by approximately 2½ grades (15°C).

The PAV aging induced change in percent recovery was evaluated for the same three products at all three test temperatures and all four stress levels, as shown in figures 71 through 73.

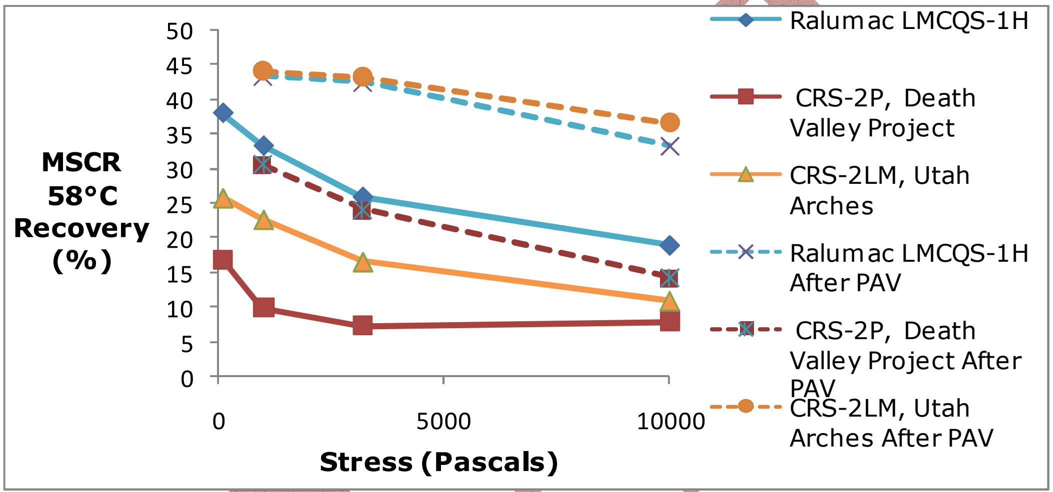

Figure 71: MSCR – Effect of Aging on Recovery at 58°C

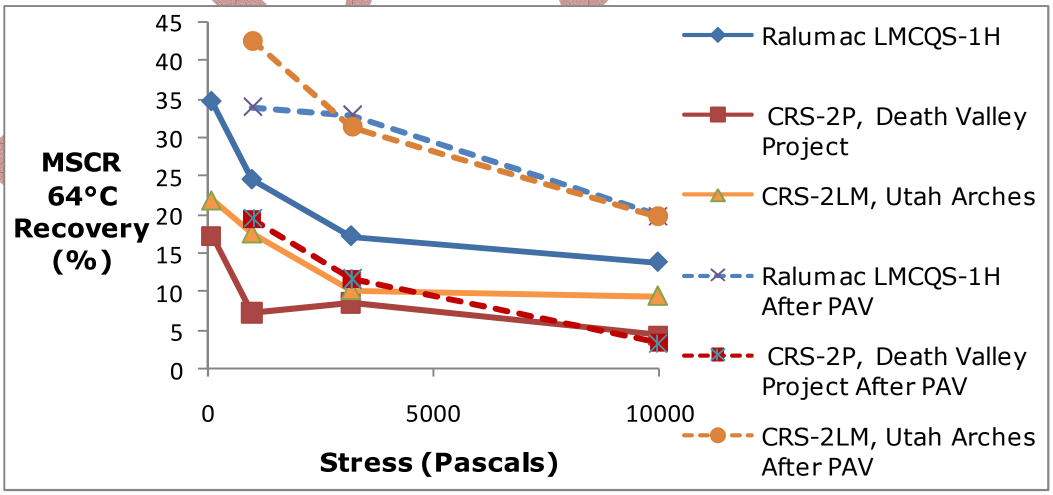

Figure 72: MSCR – Effect of Aging on Recovery at 64°C

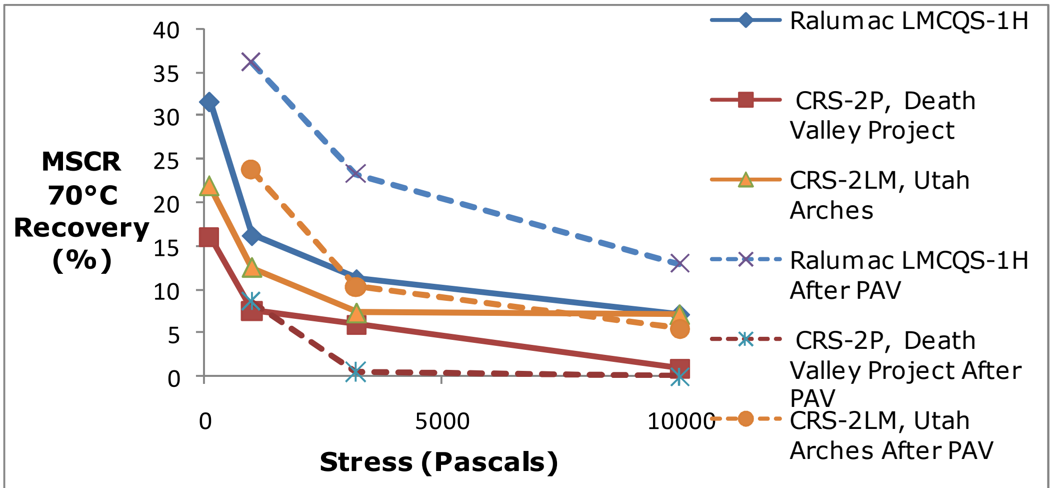

Figure 73: MSCR – Effect of Aging on Recovery at 70°C

Each of the products show significant increases in the percent recovery with aging, but the relative change is quite different. For example, unaged recoveries for the Ralumac microsurfacing residue with its higher polymer content are considerably higher than those from the CRS-2LM. However, after aging, the percent recoveries of the two products are almost equal under most test conditions. As discussed earlier, recoveries should increase as the binder stiffens. However, both residues experience very similar relative increases in Jnr as measured on a log scale, but CRS-2LM exhibited a much high increase in recovery than did the Ralumac. This would suggest that at least one of these two products may have experienced some changes in the polymer network structure with PAV aging. More work is needed to understand how the variables of time and temperature impact aged properties in the PAV oven as compared to field aging.

5.25 Low Temperature Bending Beam Rheometer Testing and Continuous Grading

AASHTO T 313 BBR tests were run at two temperatures on the FDO residue. The tests were then used to predict the temperature at which the passing criteria of 300 MPa S and 0.300 m-value. The results, given in table 30, show that the low temperature grading of the three latex modified emulsions were similar, meeting the specification requirements at -28.8, -30.6 and -26.3°C. The neoprene modified Pass emulsion is much softer, as was indicated in the MSCR testing, with a low temperature of -34.7°C.

Table 30: Bending Beam Tests and Rheology Limiting Temperatures

Test	Test Temp., C	Procedure	Spec	Ralumac LMCQS-1H	CRS-2P, Death Valley Project	CRS-2LM, Utah Arches	PASS Emulsion
Stiffness, MPA (60 sec.) -18 C	-18	AASHTO T 313	300 max.	272	243	315	68
m- Value -18 C	-18	AASHTO T 313	0.300 min.	0.308	0.228	0.282	0.338
Stiffness, MPA (60 sec.) -12 C	-12	AASHTO T 313	300 max.	120	100	142	18
m- Value -12 C	-12	AASHTO T 313	0.300 min.	0.371	0.384	0.348	0.376
Temperature at Which FDO Residue Meets SHRP PG Grading Specification Limits							
Temperature where residue meets G*/sin delta of 1.0, kPa (°C)			AASHTO T 315	76.9	67.6	81.8	54.6
Temperature where residue meets G* X sin delta of 3000 Pa			AASHTO T 315	20.7	19.3	21.7	9.3
Temperature where residue meets BBR Stiffness of 300 MPA(°C)			AASHTO T 313	-28.7	-29.3	-27.6	-34.7
Temperature where residue meets BBR m-value of 0.300 (°C)			AASHTO T 313	-28.8	-30.6	-26.3	-34.7
SHRP PG Temperature Grade (continuous grading)			AASHTO MP 1	76-28	67-29	81-26	54-34

Because PG binders are graded in 6°C temperature increments, it is easiest to understand differences in asphalt consistency by comparing the temperatures at which materials have the same consistency as measured by the current PG standard, G*/sin delta. Because those using PG specifications are familiar with the temperature as defined for HMA applications using a frequency of 10 radians per second and a specification limiting modulus of 1.0 kPa for unaged binders, these test conditions were used to define comparable limiting temperatures for the emulsion residues. Although not in this report, it should be emphasized that full frequency sweep data is available on the FLH project website for all unaged and aged samples at high and intermediate temperatures, so rheological master curves can be constructed and/or limiting temperatures can be determined at other test conditions ultimately deemed appropriate for chip seal applications. As can be seen from the data in table 30, limiting temperatures for the unaged residue from the three chip seal emulsions ranged from 54.6°C (Pass) to 81.8°C (CRS2-LM), a difference of 27.2°C or 4½ PG binder grades. It is quite surprising that the two extreme binders were both applied to Utah National Parks during the late fall of 2008. The emulsion (CRS-2P) applied during the same period in Death Valley, CA, one of the hottest locations in the US, had a consistency near the mid-point of the range between the two Utah projects. This range of consistencies seems illogical and accentuates the need for improved emulsion residue specifications. From a research point of view, though, the broad range of properties might accelerate differences in performance to better select specification limits in the future.

5.26 Dynamic Shear Rheometer Frequency Sweep at Intermediate Temperatures:

The intermediate temperature at which the specification parameter (G* x sin delta) reaches 5000 kPa for the PAV aged residues from all four emulsions is reported in table 30. As expected, there were large differences in the critical intermediate temperatures, with the Pass residue appearing to be much softer that the others. This fatigue parameter is probably inappropriate, because chip seal emulsion residues are not subject to classic fatigue cracking. It is also worth noting that research project NCHRP 9-10 and numerous other researchers have collectively concluded that G* x sin delta is a very poor performance parameter for any kind of cracking, including fatigue.

Christensen, Anderson, and Marasteanu showed that rheological master curves of Modulus (G*) versus Temperature and Phase Angle versus Temperature can be mathematically modeled using the now well-accepted CAM model. If measured data is precise, and if the CAM model appropriately fits both master curves for a given binder over a broad range of temperatures, it is possible to make a rheological measurement in one temperature range and then extrapolate using the model to predict rheological properties at a very different temperature. For reasons discussed earlier, it is the goal of the project to investigate the use of these intermediate temperature frequency sweeps as a means of replacing the Bending Beam Rheometer as the preferred method for specifying the low temperature performance properties of emulsion residues.

Because low temperature properties are best defined in performance specifications after the binder is subjected to laboratory aging protocols, frequency sweeps were run on all PAV aged residues at 10°C and 20°C using procedures as designated for intermediate temperature PG Binder grading (8 mm plates, 2 mm gap, 5 percent strain, 0.1 to 100 radians/second). All frequency sweep data tables can be found on the Federal Lands and National Center for Pavement Preservations (www.pavementpreservation.org) websites.

5.27 Dynamic Shear Rheometer Strain Sweep

Recent collaborative research between the University of Wisconsin and the University of Stellenboch in South Africa suggests that one important aspect of chip retention is a binder that maintains strength as tire contact moves an embedded chip. Polymers are a very effective means of creating additional tensile strength with elongation, such that the chip is pulled back to its original position when the tire has passed. This is particularly important for problem areas such as intersections or driveway exits where turning tires are most prone to dislodge chips. Rather than adding an expensive tensile test to the specification, it is theorized that this property can be captured by determining how much strain can be applied to a binder in a dynamic shear rheometer before it loses a significant percentage of its modulus.

DSR strain sweeps were run on all PAV residues using the test conditions recommended by Bahia (25°C, 8 mm plates, 2 mm gap, 10 radians/second, 0.01 to 50 percent strain). As can be seen on figure 74, logarithmic plots of modulus (G*) versus % strain indicate that the modulus remains relatively constant as strain increases, and then weakens dramatically as the strain exceeds some critical limit. Full strain sweep data is available on the FLH website mentioned earlier, and detailed data tables have been forwarded to the University of Wisconsin for further analysis as part of ongoing Emulsion Task Force efforts to develop performance specifications.

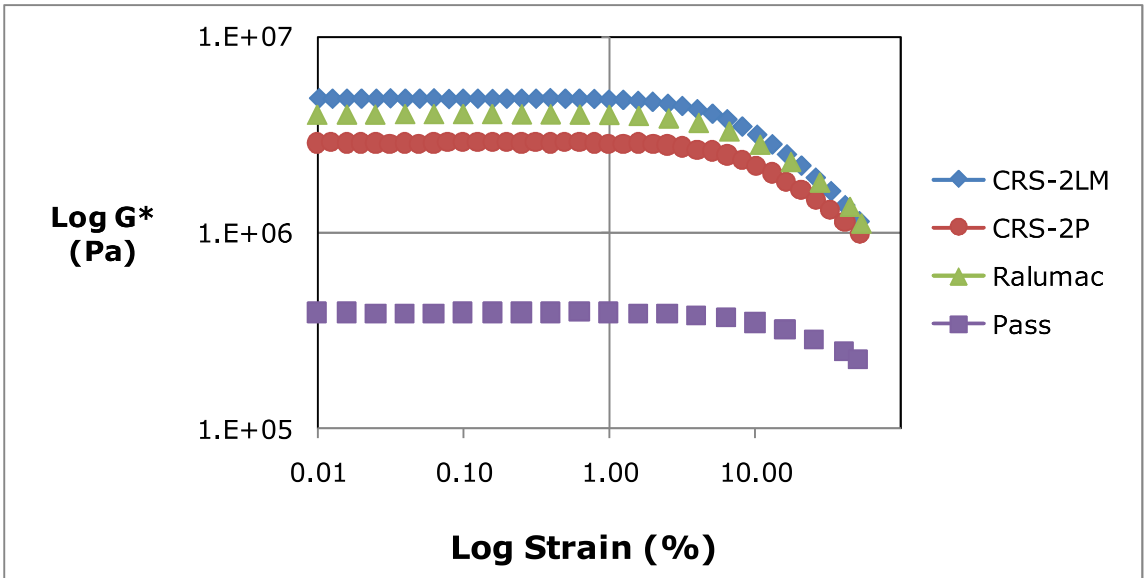

Figure 74: Strain Sweeps on PME Residues

5.28 Sweep Test

ASTM D7000 Standard Test Method for Sweep Test of Bituminous Emulsion Surface Treatment Samples uses project aggregate and emulsion to determine compatibility of the chip seal emulsion and aggregate, and give an indication of how quickly the adhesion and chip retention develops. Following recommendations from Takamura, the ASTM procedure was modified slightly to improve reproducibility. Changes include:

- Preheat felt pad to 35°C in oven prior to use.
- Dampen the aggregate surface with about 4 grams of water prior to spreading into the emulsion on the sweep test pad.
- BASF also uses rectangular felt pads (12'x14") rather than the circular pads.

Samples of emulsion and aggregate from the Utah Arches, Death Valley, and Dinosaur Monument chip seal projects were sent to all three participating laboratories for sweep testing. Five single-lab replicates were run using a two-hour curing period for every trial. Split samples of the emulsion and aggregates from some of the projects were sent to three labs, Paragon Technical Services, Inc., PRI, Inc., and BASF. The results are given in Table 31.

Table 31: Sweep Test Results

Project / Emulsion	Test Lab	Mass Loss (%) Average	STD	Range
Arches / CRS-2HLM	BASF	11.1 %	2.0	5.3
Arches / CRS-2HLM	Paragon	16.5 %	0.4	0.9
Arches / CRS-2HLM	PRI	13.1 %	1.0	2.4
Arches / CRS-2HLM	**All labs**	**13.5 %**	**2.7**	**5.4**
Death Valley / CRS-2P	BASF	9.7 %	1.5	3.2
Death Valley / CRS-2P	PRI	11.9 %	1.1	3.0
Death Valley / CRS-2P	**All labs**	**10.8 %**	**0.2**	**1.1**
Dinosaur NM / Pass emulsion	PRI	Insufficient curing @ 2hrs, all chips lost		

Results from inter-laboratory sweep tests were encouraging, but some questions remain. As can be seen from Table 31, intra-laboratory results for the Arches CRS-2LM and the Death Valley CRS-2P were very consistent, with 5-replicate standard deviations ranging from 0.4 to 2.0 percent mass loss. This precision should satisfy needs for an acceptable specification test. The inter-lab precision is less encouraging. Average results for the Arches CRS-2LM ranged from 11.1 to 16.5 percent mass loss, suggesting there is still room to improve the definition of procedural details in the ASTM draft. For the Death Valley CRS-2P, two labs reported results that were within intra-lab variability, but the third lab reported results that were unacceptably high.

Finally, the Pass emulsion did not cure sufficiently in two hours to hold chips, so mass loss was essentially 100% and testing was abandoned. It should be understood that the residue from Pass emulsion contains rejuvenator oils, and is therefore very soft. Furthermore, the emulsifier is designed to break more slowly than typical CRS-2P emulsions. This kind of product has found an important niche in the marketplace, particularly when applied to low ADT, highly aged bituminous surfaces that need rejuvenation to prevent further surface-initiated cracking. On the other hand, Pass may not an appropriate emulsion for chip sealing roads with high volume traffic or for projects that need early cures to minimize traffic control issues. Hence, such a product would need independent performance specifications written for the applications where it is found to be successful.

6.0 CONCLUSIONS AND NEXT STEPS

Polymer modified emulsions can be very effective in a number of paving applications for all types of pavement. When properly formulated, they resist deformation and bleeding at high temperatures, resist cracking, raveling and shelling at low temperatures, are more durable, and they exhibit improved behavior during construction allowing quicker traffic return and reducing early failure. The best results are obtained when the polymer and asphalt are compatible and the polymer is well dispersed and networked throughout the asphalt. Literature searches, information gathering from industry, academic and government experts and a survey confirmed there is a need for performance specifications of the PME during construction and residue on the pavement rather than recipe specifications for type and content of polymer. A strawman specification was developed using newly developed techniques for setting time (Sweep test), emulsion recovery (Forced Draft Oven test) and rheological characterization (Dynamic Shear Rheometry compliance and recovery in the Multi-Stress Creep Recovery Mode as well as Bending Beam Rheometry). Samples from field trials placed on Federal Highway Lands projects in 2008 were tested using the new protocols. They were also tested for a proposed method of using intermediate temperature rheology testing with mastercurve analysis to characterize low temperature, eliminating the need for expensive and time consuming Bending Beam testing. Preliminary results are very promising, and the data collected is being shared with other researchers to characterize and specify the performance of the modified residue. There is still work to be done. It is hoped that other researchers, suppliers and users should benefit from the results obtained by this testing plan, and it is envisioned that performance-related specifications for polymer modified asphalt emulsion surface treatments will be the norm in the not too distant future.

6.2 Leveraging Resources and Information Sharing

This project has begun leveraging available knowledge and pooling information (test methods, data, and pavement performance) with suppliers and other researchers and agencies (Federal, State, City and County). The recently released "TSP Preservation Research Roadmap" also recognizes the need for improved, performance-related specifications for asphalt emulsions. Because of the high interest by several entities in developing improved emulsion test methods and specifications, an expert task force (ETF) of the Pavement Preservation ETG has been formed by FHWA, with the first meeting held in April 2008. By cooperating on testing procedures and round robin testing, researchers from several projects will be more effective in developing standard procedures. Because funding for the current FLH study ends in September 2008, it is hoped that the ETG ETF in combination with these other ongoing research efforts will continue to monitor and update the report-only testing program and eventually recommend pertinent performance specifications to FLH and to the broader paving industry. It is further expected that the guidelines delivered by this FLH project will be applicable not only to FLH personnel, but to the industry as a whole. It is further recommended that governmental agencies support the creation of a pooled fund study to continue the report-only performance testing using AASHTO agency field projects.

6.3 Other Data Gaps and Future Work

Specific areas identified as currently needing more investigation include:

- Develop performance and specification recommendations for hiking and biking trails and parking lots.
- Provide clearer differentiation of material performance given variability in climate (temperature, humidity) and traffic.
- Update asphalt emulsion test methods in ASTM D-244, including measures for laboratory and field viscosity and low-temperature residue recovery.
- Develop standard asphalt emulsion residue test methods and specifications that correlate with performance.
- Continue the development of rheological methods to insure the presence of optimum levels of polymer modification or gel (high float) formation in the residue.
- Develop aging procedures and polymer/asphalt compatibility or stability tests for asphalt emulsion residues.
- Improve materials selection, including aggregate specifications and mix-design procedures.
- Develop/improve performance methods for PME applications to include interactions between modified asphalt emulsion and aggregate. Efforts to include curing tests establishing time-to-traffic, moisture damage, and longer term performance under specified traffic and environmental conditions.
- Improve controls on environmental and pavement conditions at time of construction.
- Create Delayed-Acceptance or Certified Supplier Programs for asphalt emulsions.
- Conduct formal cost-benefit analyses with and without modifiers for specific asphalt emulsion applications.

It is hoped that an ongoing FLH field study continuing under the report-only format can be used to support some of these research needs. There were several FLH chip seal and microsurfacing projects constructed in the summer and fall of 2008 and another planned for early 2009. The materials used were tested using the suggested Strawman protocols. The data presented here is very promising in support of the strawman, and is a beginning of a database of performance test results on polymer modified asphalt emulsions. Hopefully these results will be used by other researchers to optimize test conditions and specification limits.

Although problems with curing might be visible shortly after construction, ultimate performance cannot be analyzed until many years later. FLH collects video pavement management data every three years. More frequent field inspection may be needed as the Strawman tests are run. Tying the field performance information over time to the test results should be an on-going process. A Materials Library of the tested materials should also be maintained, so that materials may be retested as the test methods are perfected and pavement performance is known.

In conclusion, current activities are being fully coordinated with the FHWA Pavement Preservation ETG's Emulsion Task Force and with the FHWA Superpave ETGs to advance recommendations to the AASHTO Highway Subcommittee on Materials, with a goal of provisional emulsion performance specifications in 2010.

APPENDIX A – SURVEY RESULTS

A.1 Survey Questions

Industry Survey [Note: the questionnaire was web-based and had links to several on-line test procedures that were under evaluation.]
Technology Deployment Study – Modifiers for Asphalt Emulsions, Synthesis of Best Practices.

GOAL: Compile best practices information and develop a Guide and Powerpoint titled, "Using Polymer Modified Asphalt Emulsions in Surface Treatments". The guide should include specification recommendations for Polymer Modified Emulsion Techniques for Pavement Preservation that will result in the expected performance during construction, curing and especially on the pavement. These will include criteria for emulsion and aggregate acceptance, construction quality and field performance.

SPONSORING AGENCY: FHWA Federal Lands Highway Division. The intended audience is project development engineers and maintenance engineers from federal land management agencies (FLMA). However, it is expected that the document will also be a good resource for APWA, NACE, LTAP, and AASHTO. The National Center for Pavement Preservation (NCPP) is the principal investigator for this contract.

SPECIFIC REQUESTS: polymer-modified emulsion chip seals, polymer modified slurry (and micro-surfacing) systems, and polymer modified cape seals be defined for two levels of performance based upon traffic considerations. There should be some verification that the emulsion is indeed polymer modified.

I. APPROVED SUPPLIER CERTIFICATION PROGRAM (ASC)
Problem: Timely manufacture and application of polymer modified emulsions result in the best performance. Acceptance criteria testing using performance-related specifications can take longer than practical. This problem was addressed in the development of Superpave specifications by an Approved Supplier Certification Program (ASC) for binder suppliers.

Pros: the supplier can ship materials before the lab certification is completed, allowing timely shipping and application of emulsions. It has generally been successful with Superpave.

Cons: requires a number of elements, including certification of labs and technicians, detailed reporting requirements, agency inspections and data verification.

Question 1 – How likely would it be that you would support working with the AEMA technical committee to create an Asphalt Emulsion Approved Supplier Certification (AEASC) Program using the AASHTO ASC program for HMA binders as a model? [The AEASC Program would include QC Plan, Self-Certification Plan, Access Plan, and timeframe for testing and technical support.]

Unlikely   Somewhat Unlikely   Neutral   Somewhat Likely   Likely
Other considerations, comments:

II. EMULSION ACCEPTANCE

Part A: Low Temperature Residue Recovery Methods

The Problem: Conventional 500°F distillation procedures can damage polymers in ways that would not occur during normal emulsion curing at ambient pavement temperatures. The industry has evaluated several methods for lower temperature procedures.

Pros:
Reducing temperatures preserve the polymer structures and more accurately reflect field curing.

Cons:
Low temperature recoveries typically require two to three days to completely remove the water so that residues can be tested for physical properties.

Question 2 – How likely would it be that you would support the adoption of one of the proposed low temperature recovery procedures (see reference list below), using AEMA/ARRA/ISSA members and technical panels to test and review alternatives?

Unlikely Somewhat Unlikely Neutral Somewhat Likely Likely

Do you have a preference? If so, why? Please give other considerations, comments:

____

____

Recovery Procedure References

Part B: Liquid Emulsion Specification Tests

The Problem: Changes to Specification Tests have been proposed because for the following reasons: many current specification tests do not relate to performance; there is little or no compatibility check with the job aggregate; emulsion viscosities tested in the lab are often not the same as when they are applied in the field; most asphalt cement is now supplied using Superpave grading; and current specifications were generally developed for non-polymer emulsions (and those that were developed for polymers reflect just one type of polymer). Questions raised by previous industry input during this study are given below (links to specific test procedures are also given below):

Pros: current specification tests are generally quick, easy-to-run, with well-determined repeatability limits.

Cons: there has been limited round-robin and reliability testing of new methods

Question 3 – Is the demulsibility test needed for chip seal emulsions if the sweep test is used to establish cure rates?

Yes No

Considerations, comments:

Question 4 – Should emulsion viscosity be measured with a Brookfield viscometer, rather than Saybolt-Furol?

Yes No
Considerations, comments:

Question 5 – Should chip seal emulsion viscosity be raised from the typical 100-400 sec SF to 200-500 sec SF per specifications in some states, such as ARK?

Yes No
Considerations, comments:

Question 6 – Should field testing of emulsions for compliance testing be addressed in some way? (Note that WY DOT has developed a simple field viscometer to allow for real-time viscosity verification at the job site)

Yes No
Considerations, comments:

Question 7 – Can polymer latex be post-added to the emulsion, or should it be added to the soap or asphalt before processing through the emulsion mill?

Yes No
Considerations, comments:

Question 8 – What other changes would you suggest to insure polymer emulsion specifications really relate to performance during storage, construction, curing and on-the-pavement durability?

Liquid Emulsion Specification Test References

Part C: Emulsion Residue Specifications

The Problems: Emulsion residue specifications remain largely penetration-graded, with material grade selection only loosely tied to climate and traffic. There is a very clear need to narrow penetration range for grades more closely related to climatic and traffic needs. There is a desire to apply the more sophisticated Superpave methods to emulsions. Furthermore, it is difficult to capture the specific advantages of each of the various types of polymer modifiers in PME with a single physical test and specification parameter. Parameters for tensile strength (such as force ductility ratio), tend to favor SBS, whereas low temperature ductility tend to favor SBR. As a compromise, common specifications meant to allow both polymer types frequently use some measure of elasticity, such as the Elastic Recovery Test in a Ductilometer.

Pros: Superpave tests are more closely related to performance and adapted for climate and traffic than conventional tests such as penetration. With newer rheometers, elasticity can be measured more accurately under controlled stress or strain conditions by using a creep recovery test in a dynamic shear rheometer (DSR). One possibility is to apply test conditions as recently developed for hot mix binders under a new AASHTO testing protocol entitled the Multi-Stress Creep Recovery Test.

Cons: Some forms of binder testing such as RTFO are inappropriate or may require significant revision for emulsion residue assessment. Rheological requirements are much different for emulsion and binder applications; the Superpave tests were developed to address rutting and cracking, not chip retention and bleeding.

Question 8 – In your opinion, how accurate or representative is the Elastic Recovery Test (performed in a ductilometer) in assessing polymer presence and relative concentration for polymer modifiers?

Inaccurate   Somewhat Inaccurate   Neutral   Somewhat Accurate   Accurate
Other considerations, comments:

Question 10 – Would you support using the Dynamic Shear Rheometer (DSR) to verify polymer properties?

Yes   No
If yes, how might this be done most effectively? If no, why not? Please give other considerations, comments:

Question 11 – Compatibility can be monitored through microscopy, but such methods are difficult to translate into specifications. Emulsion heat stability, if a concern, could be addressed by the Laboratory Asphalt Stability Test (LAST) (or variation) as developed by NCHRP 9-10. If a certified supplier program is developed for asphalt emulsions, some of these stability/compatibility criteria might be required as part of the certification process, rather than including them in formal product specifications.

Would you support the use of microscopy to assess compatibility if implemented under a program of certification rather than as a formal product specification?

Yes   No
Why or why not? Other considerations, comments:

Question 12 - Is a heat stability test necessary for emulsion residues?

Yes No

Why or why not? Other considerations, comments:

____

____

If yes, would you prefer current standards using the aluminum tube separation test (ASTM D 5976) or incorporating the LAST (Laboratory Asphalt Stability Test) as developed by NCHRP 9-10? Would your thoughts change if storage stability were implemented as part of a precertification program rather than as a formal product specification?

Explain; please give other considerations, comments:

Question 13 – Do you support the use of Superpave binder grading tools such as the DSR, BBR, and PAV for emulsion residue specifications?

Yes No

If yes, please cite references or discuss approaches that might make this possible, and give other considerations, comments:

For suppliers: Would you be willing to provide Superpave binder test data for your own materials following protocols eventually established by this project?

Yes No

Emulsion Residue Test / Specification References

III. AGGREGATE TESTS AND SPECIFICATIONS

The Problem: Even with a perfect emulsion, if it is not chemically or physically compatible with the job aggregate or the aggregate cannot withstand the demands of the construction or application, the project may be a failure. Typically aggregate quality varies widely and testing is minimal for these types of applications, especially testing of the job emulsion with the job aggregate. Therefore, performance tests must address the unique problems and expectations for a given application. The FHWA Central Federal Lands Highway Division has requested that both polymer-modified chip seal emulsions and polymer modified slurry systems be defined for two levels of performance based upon traffic considerations.

Question 14 – Which of the following aggregate tests / specifications do you feel might be appropriate for polymer-modified emulsion based surface treatments (select all that apply)?

LA Abrasion Micro Deval Soundness Polish Size/Gradation

Cleanliness Methylene Blue (clay content)

List other aggregate tests that might apply: ___________________________________

Discuss preferences in detail:

Other considerations, comments:

Aggregate Test / Specification References

Question 15 – Would you support using the Sweep Test to quantify curing time to traffic for chip seals (2-levels of product performance would likely be established based upon separate limits for curing time)?

Yes No
Why or why not? Please give other considerations and comments:

Question 16 – Should performance specifications include chip seal testing procedures which differentiate long-term chip loss (e.g., Frosted Marble Cohesion and Vialit Plate Shock tests)?

Yes No
If yes, what test methods would you prefer (provide references)? If no, why? Please give other considerations and comments:

Question 17 – Should the current ISSA micro-surfacing performance tests for areas needing rapid cure, rut-filling, or with heavy traffic be adopted?

Yes No
Why or why not? Please describe any deficiencies or needed changes, and give other considerations or comments

Question 18 – Should a micro-surfacing PME grade be used specifically for rut-filling with performance tests selected accordingly?

Yes No
Why or why not? Please give other considerations or comments

Question 19 – Should the existing polymer-modified slurry specifications be upgraded with input from ISSA technical representatives for use on lower traffic areas?

Yes No
Why or why not? Please give specific recommendations or other considerations or comments

Question 20 – Fugro Consultants is currently working with a FHWA pooled-fund study to develop new guidelines for micro-surfacing mix design procedures. Recommended changes include modifications to the wet-track abrasion test, automated tests for mixing and cohesion, etc. Are you aware of that research, and do you generally support these changes?

Yes No Don't Know
Why or why not? Please give specifics or other considerations or comments

Application Performance Testing References

IV. CONSTRUCTION
The Problem: Many failures of emulsion surface treatments are caused by poor construction practices and/or construction during inclement weather. There is a Transportation Curriculum Coordination Council (TCCC) panel that is working to implement contractor certification for contractors. Certification requires both training and testing to demonstrate competency. Several states have volunteered to pilot this concept and others have expressed interest.

Question 21 – Which of the following forms of certification would you support (check all that apply)?

Contractor Individual Materials Supplier Laboratory
Please explain or give other considerations or comments

Question 22 – Describe your level of interest in participating in the development of certification criteria, training, testing, etc.?

Not Interested Neutral Somewhat Interested Interested
Please explain or give other considerations or recommendations

Construction Certification References

V: GENERAL INFORMATION

Please fill-in the following items as completely as possible. Items marked with a * are required.

*Name:_________________________

*Company / Organization / Agency:______________________

*Current Job Title:________________________________

*Email:________________________________

Phone:________________________________

*Which of the following best describes your current job function?

Technical Sales/Marketing Managerial Regulatory Industry Representation

*Which of the following best describes your current occupational affiliation?

Contractor Supplier Consulting Government Academia Trade Association

A.2 Raw Data from Survey

See B.1 for the full questions. Comments follow the data tables.

job Function	affiliation	lowTempRecovery	certification	demul-sibility	viscosity	other Viscosity	raise Viscosity	post Add	viscometer	elastic Recovery	DSR
Technical	Supplier	Somewhat Likely	Neutral	No	Yes	Brookfield	No	Yes	Yes	Somewhat Accurate	Yes
Technical	Supplier	Likely	Likely	No	Yes	Brookfield	No	Yes	Yes	Inaccurate	Yes
Regulatory	Government	Likely	Likely	No	No	NA	No	No	No	Somewhat Accurate	Yes
Technical	Consulting	Likely	Neutral	No	No	NA	No	No	No	Somewhat Accurate	Yes
Technical	Supplier	Somewhat Likely	Likely	No	Yes	Paddle	No	No	Yes	Somewhat Accurate	Yes
Technical	Contractor	Likely	Neutral	No	Yes	Brookfield	No	No	No	Neutral	Yes
Regulatory	TradeAssociation	Likely	Somewhat Likely	No	Yes	Paddle	Yes	No	Yes	Neutral	No
Technical	Supplier	Somewhat Likely	Likely	Yes	Yes	NA	No	No	No	Somewhat Inaccurate	Yes
Technical	Supplier	Likely	Likely	Yes	Yes	Paddle	No	Yes	No	Somewhat Inaccurate	Yes
Regulatory	Government	Likely	Somewhat Likely	Yes	Yes	Brookfield	Yes	Yes	Yes	Somewhat Accurate	Yes
Technical	Academia	Likely	Likely	No	No	NA	No	Yes	Yes	Neutral	No
Regulatory	Government	Likely	Likely	No	No	NA	No	No	No	Accurate	Yes
SalesMarketing	Supplier	Likely	Likely	Yes	Yes	Brookfield	Yes	No	Yes	Somewhat Accurate	Yes
Technical	Consulting	Likely	Likely	No	Yes	Paddle	No	No	Yes	Somewhat Accurate	Yes
Technical	Supplier	Likely	Likely	Yes	Yes	Paddle	No	Yes	Yes	Somewhat Accurate	Yes
Technical	Supplier	Somewhat Likely	Somewhat Likely	Yes	No	NA	No	No	Yes	Neutral	Yes
Technical	Academia	Likely	Likely	No	Yes	Brookfield	Yes	Yes	Yes	Neutral	Yes
Managerial	Consulting	Somewhat Unlikely	Somewhat Likely	No	No	NA	No	No	Yes	Neutral	Yes
Managerial	Government	Unlikely	Neutral	Yes	Yes	Brookfield	Yes	No	Yes	Neutral	Yes
Technical	Academia	Somewhat Likely	Yes	Yes	Paddle	Yes	Yes	Yes	Somewhat Accurate	Yes	
Managerial	Supplier	Likely	Somewhat Likely	No	No	NA	Yes	No	No	Neutral	No
Technical	Supplier	Unlikely	Somewhat Unlikely	No	No	NA	No	Yes	Yes	Accurate	No
Technical	Supplier	Somewhat Unlikely	Somewhat Unlikely	No	No	NA	No	Yes	Yes	Inaccurate	Yes
Technical	Supplier	Likely	Somewhat Likely	No	Yes	NA	No	Yes	No	Somewhat Accurate	Yes
Technical	Supplier	Likely	Likely	No	No	NA	No	No	No	Somewhat Inaccurate	Yes
Technical	Supplier	Somewhat Unlikely	No	Yes	Brookfield	No	No	Yes	Somewhat Accurate	Yes	
Managerial	Supplier	Somewhat Likely	Neutral	No	Yes	Brookfield	No	Yes	No	Somewhat Accurate	No
Technical	Supplier	Unlikely	Somewhat Likely	Yes	No	NA	No	Yes	Yes	Somewhat Inaccurate	Yes
Technical	Supplier	Likely	Likely	Yes	No	NA	Yes	No	Yes	Somewhat Accurate	Yes
Managerial	Government	Likely	Likely	No	Yes	Brookfield	No	No	Yes	Somewhat Accurate	Yes
Technical	Supplier	Somewhat Unlikely	No	No	NA	No	No	Yes	Somewhat Accurate	Yes	
SalesMarketing	Supplier	Somewhat Likely	Likely	No	No	NA	No	Yes	No	Neutral	Yes

job Function	microscopy	heat Stability	superpave	super Suppliers	LA Abrasion	Micro Deval	Sound-ness	Polish	Size	Cleanli-ness	MB
Technical	No	No	Yes	No						Yes	Yes
Technical	No	No	No	Yes							
Regulatory	No	No	Yes		Yes	Yes				Yes	Yes
Technical	No		No								
Technical	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	
Technical	No	No	Yes	Yes	Yes			Yes	Yes	Yes	
Regulatory	Yes	Yes	Yes	NA	Yes		Yes	Yes	Yes	Yes	
Technical	Yes	No	Yes	Yes							
Technical	No	No	Yes	Yes				Yes	Yes	Yes	
Regulatory	Yes	No	Yes	NA	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Technical	No	No	No	NA	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Regulatory	No		No							Yes	
SalesMarketing	Yes	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Technical	No		Yes		Yes		Yes	Yes		Yes	
Technical	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Technical	Yes	No	Yes		Yes	Yes	Yes	Yes	Yes	Yes	Yes
Technical	Yes	Yes	Yes	NA	Yes		Yes	Yes	Yes	Yes	Yes
Managerial	Yes	No	Yes	NA		Yes		Yes		Yes	
Managerial	No	No	No		Yes				Yes	Yes	
Technical	No		No							Yes	
Managerial	No	No	No	Yes	Yes	Yes	Yes	Yes		Yes	Yes
Technical	No	No	No	NA	Yes		Yes	Yes	Yes	Yes	Yes
Technical	No	No	Yes	No	Yes		Yes		Yes	Yes	Yes
Technical	No	No	Yes	Yes			Yes	Yes	Yes	Yes	
Technical	Yes	No	No	No	Yes		Yes	Yes	Yes		Yes
Technical	Yes		No							Yes	
Managerial	No	No	No								
Technical	No	No	Yes	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Technical	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Managerial	Yes	Yes	Yes	NA	Yes		Yes		Yes	Yes	
Technical	No		Yes					Yes		Yes	
SalesMarketing	Yes	No	Yes	Yes	Yes				Yes	Yes	Yes

job Function	Agg Others	sweep Test	chip Loss	micro	rut	low Traffic	Fugro	Contractor	Individ	Supplier	Lab	interest
Technical	adhesion type tests, mixing type tests	Yes	No	Yes	No	No	Dont Know	Yes		Yes	Yes	Not Interested
Technical		Yes	Yes	Yes	Yes	Yes	Dont Know	Yes	Yes	Yes	Yes	Somewhat Interested
Regulatory	Flakiness	No	No	Yes		No	Yes	Yes	Yes		Yes	Interested
Technical	Particle shape is one of the most important characteristics in chip seal design.	No		No		Yes		Yes			Yes	
Technical		Yes	No	Yes	Yes	Yes	Dont Know	Yes		Yes	Yes	Interested
Technical	shape, 10% fines or crushing resistance.	No	Yes	Yes	No	No	Dont Know	Yes	Yes	Yes	Yes	Interested
Regulatory		Yes	Yes	Yes	Yes	Yes	Dont Know	Yes	Yes	Yes	Yes	Not Interested
Technical		No	No	No	No	No	Dont Know	Yes		Yes	Yes	Interested
Technical	Sand Equivalent	Yes	No	Yes	Yes	No	Dont Know	Yes		Yes	Yes	
Regulatory	Flat and elongated particles or some other shape factor test.	Yes	Yes	Yes	Yes	No	Dont Know	Yes		Yes	Yes	Neutral
Technical		Yes	No	Yes	No	No	No	Yes	Yes	Yes	Yes	Interested
Regulatory		No		No		No		Yes	Yes		Yes	
SalesMarketing	sand equivelency	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Interested
Technical		Yes		Yes		No			Yes		Yes	
Technical		Yes	No	Yes	No	Yes	Yes	Yes	Yes	Yes	Yes	Interested
Technical		No	Yes	Yes		Yes	Dont Know	Yes		Yes	Yes	Interested
Technical	Adhesion (resistance to stripping)	Yes	Yes	Yes	Yes	Yes	Yes	Yes		Yes	Yes	Neutral
Managerial		No	No	No	No	Yes	Dont Know	Yes			Yes	Interested
Managerial		No	No	No		No	Dont Know	Yes	Yes	Yes	Yes	Neutral
Technical		Yes		Yes		Yes		Yes	Yes			
Managerial		No	No	No	No	No	Dont Know	Yes			Yes	Neutral
Technical	particle charge	No	No	No	No	Yes	Yes					Interested
Technical		Yes	No	No	No	No		Yes			Yes	Interested
Technical		No	No	Yes	Yes	No	Dont Know					Interested
Technical	Sand Equivalent in place of Methylene Blue.	No	No	Yes	Yes	Yes	Dont Know			Yes		Interested
Technical		Yes		Yes		No		Yes			Yes	
Managerial		No	Yes	No		No		Yes	Yes	Yes	Yes	
Technical	a modified Sand Equivalent test (aggregate oven dried then conditioned 24hr@ 2% moisture). much better indication of clay content than reference	No	Yes	Yes	No	Yes	No	Yes	Yes	Yes	Yes	Somewhat Interested
Technical	I think all apply. If you are trying to develop and place high performance seals then good high quality aggregates are needed.	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Interested
Managerial		Yes	Yes	Yes	Yes	Yes	Dont Know			Yes	Yes	Neutral
Technical		No		Yes		No		Yes			Yes	
SalesMarketing		Yes	No	No	No	Yes	Dont Know	Yes	Yes	Yes	Yes	Interested

Responses from the "comments" questions. Note that to protect the integrity of the data the responses here have not been edited. Apparently there was a limit on the number of characters accepted on the website for each response.

Low Temperature Residue Recovery Comments

- Any method that can recover binder suitable for testing accurately and repeatably, and which is representative of in-service properties will have our support
- Keep options open to alternative recovery methods that will duplicate field conditions.
- It is my opinion that a new method that more represents what happen on the roadway needs to be develop. I also believe developing methods to test the base asphalt before emulsification would be helpful.
- Would ultimately depend upon ease of use, accuracy, cost to implement and time to perform.
- We have just completed a research project developing a low temperature procedure. We found that there were variations in polymer modified binder properties based on the methods of test used due to the requirement of remixing and reheating.
- Pre-testing has some risks related to testing the actual product delivered to the filed, however the process of making specified products is usually verified.
- I appreciate the value of low temperature methods. It makes sense to evaluate emulsion based on performance. My hesitation with changing the current system, however, has more to do with quality monitoring, rather than specification validation. We use t
- If the evaporation technique does not exceed 60C, and remove all of the moisture from the sample, this would be a great method to utilize. The BASF procedure 25C for 24 hours and 60C for 24 hours seems to do a very good job.
- The step to an effective performance specification must be a recovery method that doesn\'t modify the base binder. Correlation with other quicker recovery methods may be possible for some systems for use in quality control and acceptance testing. Severa
If the ASC is implemented and successful, the cons discussed here are not applicable.
- the closer to field temperatures that we can obtain residue the better we can relate to performance
- We would look at all potential ways at low temperature.
- I am in favor of performance criteria that measure properties to be expected in service, so using methods that match in service conditions makes sense.

- Any procedure that produces a more representative residue for testing and characterization is desirable. It seems to me that a certification program would eliminate the concern of time delays in recovering residue?
- If there was a certified supplier procedure that would certify plants would possibly eliminate the low temp recovery methods.
- Tests need to be able to be performed in a timely manner. Two to three days is too long.
- As long as the method used is thoroughly evaluated.
- We need to have a test were we are not increasing the time in which to achieve an answer. We would support a vacuum distillation at lower temperatures.
- The methods are too time consuming. We will surport one that can cut the time down to 2 hours.
- This is an important part of moving the science of asphalt emulsion forward. We can support this effort through cooperative work with the researchers.
- If adopted in conjunction with the certification system mentioned in the previous question, I believe this would be beneficial for the industry. Low temperature binder recovery systems are without doubt more realistic as a field performance indicator.
- A procedure with reduced sample preparation time is required. We would support SAFT even though the equipment costs are high if it can reduce the the sample preparation time to hours instead of a day or more. The tests that take longer may be of value
- We would support adoption of low temperature recovery methods. We would NOT support the adoption of only ONE low temperature recovery method. As a manufacturer of both premodified and latex modified PMEs, we do not think that one recovery method will likely be involved in testing
- The EN13074 method seems very cost effective and time efficient. However, I would like to see any methods established as either AASHTO or ASTM procedures (if not already going in that direction).

I would support this as long as the frequecny of the testing (i.e. Elastic Recovery) outlined int the Certification program was set resonalbly (i.e. Twice per month for certification). The percent residue can be obtained through a quicker procedure for recovery.

- Approved Supplier Certification Comments
- This is more likely to be adopted in the USA and may not meet broad acceptance internationally. Hence suppliers outside of the USA, such as ourselves will not benefit from an ASC program
- Will support an Emulsion Approved Supplier Certification Program.
- Consistency in specifications and certification procedures across the states would be a great benefit from a suppliers perspective.
- We are contractor in New Zealand
- I have been retired for two and a half years and am somewhat disconnected with the technicalities of emulsions.
- We manufacture emulsions into small storage tanks, turning over our inventory, in most cases, the same day. We would not only be in favor of such a program, we would like to play an active roll in its development.
- If a good program was in place for the appropriate training of technicians this would be a good practice. I would think agencies would need to be on board if recovery procedures are used that take additional time to get material to test.
- Having all the elements of the program inplace would be a benefit. However, it is unlikely that we would base acceptance on "Approved Supplier Certification" only. We would still do some testing on samples of the delivered product. We have had many cas
- I see no reason to start from scratch. Many of the issues are the same between the two processes so I think it would be highly beneficial to use a similar, and now successful, model. Most emulsion plants are capable of producing a steady, consistent emu
- That the plan should be based on on the existing AASHTO ASC plan aside, supplier certification is clearly needed is some form
- we work in the asphalt industry producing both emulsion and peformancd binder asphalt. We are aware of the plan for PG.
- A supplier certification program has worked quite well in California for asphalt cements. I am not sure if it is also used for emulsions but I believe so.
- This would give it more direction as to what needs to be done and to ensure everyone is doing the same.
- If the AASHTO ASC program is similar to the AASHTO AMRL certification for HMA and Aggregate testing I would probably be somewhat unlikely. AMRL is expensive and

in my opinion overkill. A program similar to NETTCP used in the NorthEast US would be more p

- The problem would be the shelf life of the sample and getting uniform results back.

- Suppppliers have enough self regulation and the current requirements from different agencies are more than sufficient.

- As a supplier the idea of an AEASC program has it merits. The complication occurs when different agencies adopt parts or modify a "national" program to a point where suppliers essentially end up adhering to programs that are effectively different for e
This would be good for the industry as a whole. I believe an approved suppliers program would lift the whole manufacturing segment with regards to delivering a quality product.

- We would support an AEASC program as long as it is usable by small companies.

- Have been involved with AEMA ITC for a number of years

- We are currently evaluating a program for ASC for emulsions based on our ASC program for binders. Our original binder program was based on AASHTO PP26.

- This is a great way to outline what testing needs are required along with the frequency of testing required allowing for timely shipping.

Demulsibility Comments

- The sweep test is intended to simulate field performance and hence is preferred. The demulsibility test artificially breaks the emulsion and may be more representative of the chemical reactivity of the emulsion than the curing behaviour.

- Performance based. Demulsibility can be a tool for the emulsion producer to optimize production.

- I don't believe that demu. test is very accurate.

- Would not be required in the specification portion. Probably would still be used as a QC test if a reasonable corelation was found.

- We are more focussed on performance based testing which looks at the binder which is all that is important for the client how it is delivered to the road is the problem of the contractor and most specs regarding the emulsion should be developed between th
Demulsibility test never gave the field much infomation.

- If we change to a "system based" test, like the sweep test, we also introduce more variables. Are we able to identify a leaking heating coil, for example, as quickly with the sweep test, as we would if we saw a sudden rise in the demulsibility value? Wh

- This is a difficult question to answer. My answer would be no if the appropriate temperatures and curing conditions for the area and time of year were to be used for the sweep test. The demulsibility test (or the breaking index test) is/are good descrip

- Demulsibility may not be the best test, but a test is needed that can be performed on a tank or load basis. Testing the emusion aggregate system on one sample does not insure constistency in production. Measuring break time is especially important with

- But I think that both should be run simultaneously, at least in a few plants, to make sure that the sweep test is practical, and gives the information needed before phasing out the demulsibility test.

- Unless the sweep test is run per batch or lot there is nothing saying ta particular load of emulsion will pass the sweep test. I realize that demul is not run on every batch so the same problem applies. It is my understanding that the sweep test is quite
  The sweep test can take to long to use in the a quality control lab. A program that would use a demulsibility and coorelate the results would be beneficial.

- Demulsibility measures an emulsion characteristic, while Sweep measures system performance. We check both, but from the standpiont of emulsion plant operations, Demulsibility is the better emulsion test.

- However, the sweep test and demulsibility test results should be compared for a wide range of emulsions to be sure we are not missing something from the demulsibility test with this replacement.

- It might depend on what part of the country the material will be sold in and the temperatures and weather conditions it would be susseptable to.

- I still like to have the demulsibility test in place as an option. I haven't seen the sweep test and have concerns about equipment prices, test times. etc.

- I am not familiar with the sweep test.

- If the sweep test was done in similar time frame. Demulsibility is a test used for quality control but would not need to be part of specifications if the sweep test could be very reproducable.

- Demulsibility has only an indirect coorelation to field performance. As a manufacturer it has some value as method to determine if manefacturing is being done in a consistent manner. As a specifying agency the field behavior regarding the property defin
  New emulsifier chemistries can now produce high demulsibility emulsions without affecting stability. I do not agree that the sweep test is a reliable test for field performance in all cases as the field variables are difficult to imitate. Would this mean

- The sweep test is a SYSTEM test. The demulsibility test is an emulsion test. If demulsibility was eliminated as an emulsion requirement, the term CRS, RS, and HFRS would not have meaning. While demulsibility does not correlate to on-road performance

- The parameters for the sweep test have not been established as of yet. Till there is a definite spec some other test will be needed.

- A field test seems the most useful, although refineries should maintain demulsibility information.

- For the majority of chip seal applications I feel the demulsibility test is a good indicator and a very quick test method. Would you need this test if it was replaced probably not from a certification standpoint. The main thing is the material has to work

Viscosity Comments

- The Brookfield can measure viscosity at a number of shear rates and can provide estimates of shear susceptibility and more complex flow behaviour. It is an established method and is also readily available in most asphalt labs.

- A level of comfort is needed for the agency.

- The paddle viscometer looks the most promising. Would likley need to revise the specification ranges.

- We have been using brookfield measurements for at least 20 years and it was in the Transit NZ emulsion spec until recently when all emulsion specs were removed.

- Rotational Paddle Viscometer is a practical test to measure viscosity.

- Either method--I would welcome a new method of measuring emulsion viscosity. Saybolt Furol clean-up between tests is difficult--especially in the production lab, where viscosity is being monitored during production. It is an attractive idea to have a sa

- I think it is time to change, but I would like to see an emulsion viscosity to be done at a range of temperatures from 20-70C and possibly a slope being specified along with a range at a specified temperature. The reason for this is that the emulsion vis

- I am not familar with the Paddle Viscometer so either test may be appropriate. I would also suggest investigating the possibility of measuring emulsion viscosity at a temperature that corresponds to the application temperature range. It would then be po

- The Saybolt-Furol viscosity is so easy to run, with simple, inexpensive equipment. The alternate viscometers listed above are not as easy, and much more expensive.

- only because I have no experience with the paddle

- I do not know much about the paddle viscometer, but I know it is used in the paint industry. Again upon review of other options would be a program that should be considered.
- Although Brookfield measures at constant shear rate, it only measures one shear rate, and therefore does not characterize the rheological profile any better (or worse) than SFS (unless you are thinking about measuring viscosity as a function of shear rate
- Brookfield Viscometer is widely used and easy to run. It is part of specifications for PG asphalts so most labs have basic equipment.
- I'm actually neutral on this. I have no problem with the Saybolt, other than cleaning. If a change was made, I would prefer the Brookfield and/or other rotational viscometers. I don't feel that it is correct to specifiy a brand of equipment in a test p
- We have found in the Pacific Coast Conference round robin that the emulsion results for the Brookfield varied up to 100%. The emulsion started to break in the test. A better suggestion might be to consider raising the temperature at which the saybolt-furo
- What are the extra information that the different tests can get?
- Would like to understand what we intend to test (temperature, shear rate, spindle geometry, etc.)
- Every emulsion chip seal producer is currently equipped with Saybolt Viscometers and I do not believe the Brookfield viscometer takes thixiotropic properties of emulsions into account which the Saybolt Viscometer does.
- If the Brookfield (or equivalent - because Brookfield is a supplier of test equipment) can be used with success then this should be used as most labs have this equipment. Is there any drop in viscosity over time observed when testing emulsions in the Br
- The SSF viscosity method works well and is proven. While the rotational methods (brookfield and paddle) can give more information at different shear rates, neither is adequate to accurately predict run-off. A simpler method would be to add some type of
- Do not have any feel for the two proposed alternatives
- Many labs have the Brookfield equipment already. If it can be used effectively (i.e. more consistent results), it would be cost-effective as a replacement for the Saybolt-Furol.
- At this point I don\'t feel the other test methods have been proven. The Rotational Paddle Viscometer when tested in our lab varied significantly due to temperature control of the device.

Raise Viscosity Comments

- Viscosity is a matter of fitness for purpose and in some situations a low viscosity emulsion may perform equally as well if not better than a higher viscosity emulsion (e.g voidfills). This needs to be considerd before a change is made. Having said that,
- If it can be proved to be performance driven.
- It is my opinion that any raising of mimimum would cause more drilling of the chip seals.
- 100 second vis material works just fiine. No need to raise it.
- Need to have agreed viscosities for the type of treatment as low viscosity is good for flow into voids and around the stone but also across and down the road with steep geometry. You sometimes need thicker emulsion that won't run.
- This warranted in some states and ought to be decision made by each state.
- I see value in changing the specification to 100-500, with the specifying agency having the option of changing the minimum to 200. In some cases it is difficult to maintain even a 100 viscosity when using polymer modified basestock. On the other hand, s
  I don\'t have the data one way or the other, but it should be investigated to determine if the present specifications is allowing for proper application.
- If states think it should be changed based on their experience, they should change it. However, states with success on the lower viscosities should not have to change.
- in my opinion viscosity in the low 100's is not sufficient to obtain 70% embedment of some gradations...especially if the agg spreader is not immediately following the dist truck. Additionally for the larger grades of aggregate the low visc are is not suf
- The Furol Vis should be 150-400 SSF. A vis of 500 SSF would be to thick for some levels in the field. Anything less than 150 SSF would flow on the road.
- Neutral on this one.
- Most likely especially for hot climate regions.
- Depends on if they have had problems in the past.
- Maintenance managers have commented that viscosities of 100 SFS are too low for a good chip seal. However, I do not know if 500 is too high for spraying. That would need to be confirmed.
- If the test is performed at the suppliers plant. No if the testing is done after sampling on the job.

- I believe that the test temperature should be raised to reflex application temperature instead of raising the viscosity. Some agencies in the state have raised the viscosity to avoid run off in this state already.

- Should be 50 to 500.

- Lower range then 100 - 400 SSF being used in many locations with success.

- No. These specifications were primarily identified by past field performance, some regions would benefit from very high viscosity chip seal emulsions, other regions would not benefit due to climate.

- The standard specifications are fine. While certain conditions (steep hills) might require a higher viscosity, we have seen most distributor trucks start to have spray pattern issues (drilling) at 500SSF.

- Most production CRS-2 is at greater than 200 sec. Based on all the pumping which can occur it is better to have a higher minimum to prevent pumping viscosity loss to the point where the product becomes too fluid to spray and hold properly

- Since shipping and pumping can affect viscosity, the truest measure should be viscosity at the point of application. This may be based on visual methods. As the recommendations stated, unless field personnel are complaining about too thick / thin materi

- I don\'t believe this is needed as long as you manufacture stable product. If you raise this viscosity by 100 seconds that still doesn\'t prevent the viscosity drop off issue that some face.

Post Add Polymer Comments

- Once again it is a matter of fitness for purpose. If the mode of addition does not affect the desired emulsion propoerties then there is no reason to prohibit post addition. This should be left up to the manufacturer to decide.

- Beyond the scope of this research. BUT if it does not affect performance, then it should be allowed. With true performance test, we will be able to determine if it can be post added. The current concern is the loss of viscosity in a Chip Seal Emulsion if

- I don't currently believe the latex modified emulsion performs a good as true polymer modified emulsion. We have don't field test in Minnesota and true CRS-2p out preformes CRS-2L

- Should be 2 questions. The specification, if correctly identifying parameters, should allow either manufacturing process. Performance-based specifications should be blind to methods and materials.

- Prefer to see it added before the mil.
- it can be post added but the emulsions produced are different as are the properties of the resultant binder.
- Better processing is achieved by adding before being milled.
- As I understand it, the most valuable place for the latex to be added, is in the soap phase. I would think that there may be some issues with achieving proper mixing if it were post-added.
- It would not be wise to determine any companies manufacturing policies if a performance test is included such as the sweep test for seal coats or the slurry seal/micro-surfacing tests.
- It would be desirable to deveop test methods and performance critera that would be blind to how the latex is added.
- This is not a yes/no question. I think it depends on the plant/state/situation. If people have had success with post-adding polymer latex, I do not think they should be forbidden to continue. However, if the general consensus is that this is not a good
- the latex should be co-milled in some systems though it may not be necessary in other systems or for other applications. for example a mixing grade emulsion may perform equally well in a post add situation. If it has to be yes or no i vote to co-mill.
- I believe that it can be post added with success but should be checked before before it is allowed.
- It should be added to the soap or the asphalt depending on the type of polymer.
- We see better performance when milling the latex.
- I believe in-line blending of polymer latex and asphalt emulsion has been done successfully. Hopwever, a more uniform product is likely if the polymer latex is added at the emulsion mill.
- Not applicable. We are not an asphalt manufacturer.
- Never post added, only co-milled.
- The specifications should be end result driven. The supplier should be able to use what ever method there is out in industry to make a product that meets specifications. We have used all different kinds of polymer and added them many different ways. We ha
- Are there any data that indicate one method is better than the other? I see no difference.

- This is a manufacturing process directive, depending on the end use and chemistry different introductory methods may be preferable.

- A better dispersion is obtained when the latex is passed through the colloid mill.

- "Post added polymer latex is not adequately dispersed in the emulsion. Addition of latex to the soap or asphalt prior to emulsion allows the high shear mill to adequately diperse the polymer, thus forming a truely homogeneous emulsion."

- Suppliers of emulsions should meet specifications. The supplier should be free to elect how to best provide a product that meets specifications and performs in the application.

- The question should be "can polymer latex be post added to the emulsion in the field?" Latex addition works well as a co-milled ingredient and with proper dispersion, latex can be added to emulsion at the emulsion plant. We have manufactured CSS, CQS,

- The post addition of latex could create viscosity and separation issues. Better latex?AC relationship if comilled or is in AC prior to emulsification

- We saw inconsistent performance with in-line blending with latex and binder. Smaller operations might not have the control necessary to give consistent performance with post-mill blending.

- I feel this should be co-milled to add to the stability of the emulsion.

Viscometer Comments

- Any compliance testing, whether in the field or lab needs to be undertaken by competent, suitably qualified staff. Such personnel should be certified or accredited to a suitable quality or competency standard.

- Performance related and can be tested. This is where we can determine if the emulsion will perform.

- It has been my experience that allong as the emulsion does not run off the road before you can put the chips on the viscosity is not an issues. So I don't believe field testing neccessary.

- The field properties are what matters. Not the properties after the emulsion has been transported to the lab.

- Why bother testing if it sprays okay and doesn't flow off the road it is good but if there are issues then there is a problem that needs to be fixed and a field test may not be able to show the difference.

- Some type of field testing is always useful to back up or verify the laboratory testing and for quality assurance purposes. However, field testing probably should not take presidence over the laboratory testing.

- I think that job-site verification would be a great thing. We can certify an emulsion before it leaves our facility. But if a sample is taken and left in some guy's trunk for a few weeks before being delivered to the DOT lab--we are not going to be conf

- This is a slippery slope due to sampling of the material. I would suggest that if there are application problems there should be a means of sampling that would be able to validate the material. If there are no application issues regarding emulsion distr

- There needs to be a method to address consistency from load to load of emulsion and be able to reject material that will cause application issues. The application viscosity is much more important than a viscosity measured hours or days earlier or days after

- Yes. Field testing of emulsions can be easily accomplished with a Zahn cup or equivalent. This would influence question 3, as the viscosity usually drops during shipment.

- from a supplier point of view this is tough. yes it would be helpful in preventing the application of low visc or off spec emulsion but if everything is good at the plant and in the tank how can I convince that the storage, handling and shipping container

- But field testing and acceptance should be verified for reporducibility and accuracy

- AS long as it is repeatable and applicable. There is also contractor acceptance, and also handling the product.

- Many things can go awry once emulsion has left the producing plant. I think this is a good idea.

- This gives the Agency some confidence that the emulsion he is getting at the job site is similar to that in supplier's storage tanks. Helps eliminate contamination problems in shipping, etc.

- There should be some kind of control for in the field. If so then there should also be some solutions to typical problems that have been seen.

- We currently field test for viscosity using the Saybolt Viscometer.

- I am not familiar with the WYDOT method, but most crews know if there is a viscosity problem in the field.

- Any time that an agency is allowed to test the product where price adjustment are applied, then the testing must be repeatable and reproducable. Many of these field viscometer methods are not uniform nor reproducable.

- Viscosity does change. Suppliers should not be penalized if the field test passed and the central lab test failed.

- Emulsion viscosity can be important during the transport and application of emulsion, however other factors may be much more important to the success of chip seal. with the limited amount of field inspectors that most agencies now have in the field they

- I would say it should be at the discretion of the client.

- Field testing of PME allows for a timely determination if the PME used will meet specifications.

- I would need to see more data on the repeatability/reproducibility of the field viscometer results and see how these results related to testing the sample in a QC lab in order to comment.

- We would support this only if tested from a sealed transport. Once emulsion is transferred into the customers equipment, the emulsion manufacture loses control with regard to contamination, temperature control, over pumping, dilution, and added materials

- Allows for protection against low viscosity products being sprayed which could create issues with field performance. Although some type of warranty might resolve that concern.

- A quick field test for contaminants or early breaking would be useful (i.e. Sieve test or some sort of visual viscosity test).

- Compliance testing should be performed from the suppliers tank. However field testing is not a bad practice for useability of the material.

Other Changes Comments

- Identify what is critical for performance of the emulsion then go from there. Start with a blank slate to remove preconceive paradigns.

- I am not sure.

- Define performance characteristics and necessary limits. Be prepared accept new materials and to obsolete current materials.

- Need to focus on the end result not on irrelevant spec that pass or fail an emulsion even when it is delivering the binder as required to the road surface.

- Percent of chip loss after the roadway is opened again to traffic would tell the real story of chip seal performance.

- I really like the idea of evaluating the residue using the DSR, specifically the MSCR values. I think that this would be a great replacement for elastic recovery testing.

- A test for storage stability may be needed in certain circumstances. In some cases such as chip seal or micro-surfacing it is not as important. A sieve test is also important for appropriate application.

- "THERE WAS NOT A GENERAL COMMENT BOX, SO I WILL USE THIS ONE.

- Many of the same issues and proposed products are being addressed under NCHRP 14-17. This project was tasked with developing a chip sealing manual, evaluating some test methods, and developing"

- I think multiple labs/plants around the country should explore the new performance tests (sweep, low temp recovery procedures, etc.) and track the performance in the field. New, more expensive tests in the lab are not necessary if they do not show a cons

- Ensure that field personnel follow SOPs in storing and delivering field samples.

- There are many preformanced based tests in the industry. As the industry grows, these should be used to help distinguish between good and poor products

- The use of some type of performance grading, and also the FHWA use of the MSCR program.

- There are several. Emulsion specifications should focus on performance related issues such as adhesion to existing pavement surface, adhesion to rock (chip seals), abrasion resistance (slurry seals), tackiness (tack coats), controlled break times (chip s

- Some type of seperation test or settleing test for the polymer.

- If an adhesion test were developed and easy to use that would go a long way to performance on the road.

- Much of what constitutes "good" field performance is determined by the applicator and field conditions. We need to understand what materials are successful and what drives their success.

- 1-Some type of temperature/shear stability test for the emulsion that would indicate a minimum handling/pumping temperature for the emulsion. Many CRS and RS grades of chip seal emulsion are manufactured to not be stable at low temperatures. This allows

- some type of performance test done in the lab. Don\'t know what but thinking about it
- As noted in the references, DSR and BBR testing will be useful to determine effects of storage and curing on durability.

Elastic Recovery Test Comments

- Elastic recovery can indicate the presence of polymer but I suggest that there are too many other factors affecting the values obtained to be able to accurately assess relative concentration.
- As stated in the ASTM 6084 "Standard Test Method for Elastic Recovery of Bituminous Materials by Ductilometer" in Section 3. Significance and use: "This test method is useful inconfirming that a material has been added to the asphalt to provide a significa
- It is my belief that testing the base asphalt before emufication would be a better way to make sure that one receives true CRS-2p. It is my experence that the effect of milling the asphalt gives you better Elastic Recovery Test results thant testing the
- ER can display the presence of modifiers, but not concentration when comparing different polymers and systems.
- Various systems do better than others. The test is somewhat messy and the ductilometer takes up a lot of space. Other tests are able to identify polymer as well or better.
- No experience with the ductilometer
- No comment as I'm unfamiliar with the technicalities of these tests.
- There are so many variations of the Elastic Recovery method that sometimes it is like comparing apples to oranges. One concern that I have about the elastic recovery method is that it is sometimes difficult to get consistent results. The "lumpiness" of
- The particular residue testing will need to include application specific tests to identify the appropriate polymer is used. A micro-surfacing needs a different polymer than a chip seal or a latex modified slurry seal. The stretchy-pully tests that are a
- Elastic recovery can be used to accurately indicate the presence of polymer but as mentinoned is sensitive to polymer type and may need some method adjustments based on polymer type. It probably has little relation to performance and needs to be replaced
- This is not in my area.
- in that it indicates some amount of elastomeric polymer present...not how much and not what type to a great degree of accuracy

- good for presence but bad for concentration
- There is some correlation to the percent and the results of ER. This is not necessarily a correlation between elastic property and seal coat performance.
- I'm not sure how accurate it is for relative concentration.
- Gives Customer or Agency some assurance that polymer is present but does not define the amount of polymer present.
- Don't know the test.
- Very limited experience with the Elastic Recovery Test. Currently use torsional recovery.
- I have not had enough experience with this test.
- It is the only good test available that will determine the presents of polymer. All the other tests are only good enough for one type of polymer.
- Especially on latex modifed asphalt.
- The temperature / cut / hold timing all play into what number is generated by the Elastic Recovery Test. If industry is going to use this would like to think through what desireable properties we are trying to rate.
- While the elastic recovery does indicate the presence of polymer, I do not believe it is a good indicator of the relative concentration of polymer. Take three different asphalts with the same PG grading and add polymer, the results will not correlate.
- Elastic recovery measures the presence of elastomers, but is not a quantitative test. It reflects that enough polymer has been added to achieve elasticity, but does not actually measure the amount of polymer.
- ER does correlate only roughly to elastic type polymer concentration. It does not correlate with thermoplastic type polymers. ER is a physical test; there is no expectation that it could be used to assess polymer concentration.
- Somewhat accurate for certain polymer types but not all.
- We use the Elastic Recovery for both binders and emulsion residue (by evaporation) and feel it gives a good general idea of polymer presence. We also use force ductility which will show polymers that have performed well in Louisiana.

DSRComments

- Use similar protocols to Superpave requirements
- TEST by all concerned.
- The obstacle is the residue recovery method. It needs to closely simulate field conditions.
- MSCR test. Would need to define the spec limits but this is a good way to determine polymer.
- can use samples of recovered binder with out the need for reheating and remixing.
- The DSR is a much more time-efficient instrument. We can learn more in a shorter period of time. The amount of residue required is much less, which also makes it attractive.
- If the use is for identifying ploymer content I would say yes a maximum phase angle can be used to determine a certain polymer content or verification there is polymer in the emulsion residue.
- The test method and criteria developed for paving binders should not be used directly without being confirmed that they accurately meet the needs of emulsion applications. The test method and properties need to be modified to address the needs of sealing
- I think the added cost does not give enough added value in the field.
- some but maybe not all properties and maybe not all polymers...need to see whats out there
- phase angle or actual dsr number
- THe use of FHWA MSCR test.
- No sure of the best way.
- Phase angle determinations or stress recovery measurements should be helpful.
- Those that design the machine might have a better chance to answer this than myself.
- I would guess looking at a creep recovery test such as AASHTO TP-70. My only concern would be equipment costs for labs possibly needing additional DSR's.
- Frequency sweep tests on dsr

- I should put maybe. As long as the testing could be done after shipments, I would possibly support this.

- The date that has been used to determine if the DSR is good in from a small pool of suppliers. Most of the date comes from Texas. There is no reproducability on the residue and price adjustments will be made if it becomes specification. The specification

- Change the residue recovery method.

- Multiple Stress Creep Recovery - Run protocol at a temperature that is characteristic of the surface temperature where the emulsion is applied.

- This is currently done for PG graded asphalts and indicates the presence of polymers. I don't know how this translates to field performance of the specific polymer.

- The repetitive creep test being work on currently, once validated, should be a much better tool.

- I would support if it can be validated that DSR relates to performance in the application that the emulsion was being used for.

- I am assuming that you are asking about using the DSR to look at the properties of the PME residue, not to quantify the properties of the polymer separate from the residue. DSR procedure would need to be developed that represent what is happening in the

- MSCRT could be used to determine polymer properties. A proper residue recovery test which simulates curing in the field will be needed to recover the binder. Obviously the RTFOT does not reflex what occurs in the field. A recovery test is needed which

- MSCR should give usable results using equipment currently in most labs.

- As long as the frequency of testing is aligned with the type of instrumentation needed to perform these tests. (i.e. It would be expensive to put a DSR in every plant)

Microscopy Comments

- Physical property testing is more likely to indicate performance than a compatibility test such as microscopy. I see microscopy as being of limited value.

- Microscopy may be a good indicator when looking at PMA, but not necessarily the PME. Compatibility of polymer is not an issue with emulsion modified with different modifiers (Latex). The polymer is in the water phase not asphalt. What will you be looking

- I do not know anything about this test so I am netrual.

- If you cannot specify compatibility parameters you cannot certify them.

- It may or may not relate to performance.

- Compatibility is important in the manufacture process but is it important once the binder is emulsified? I suppose this is more a problem in latex modified emulsions which I don't have much experience with.

- If it could be shown to be relevant to compatibility performance then it might be worth while. Generally, cationic emulsion are compatibile with nearly all aggregates.

- I am not familiar with these tests.

- I think that this is another slippery slope in trying to formulate for suppliers. If the material performs adequately in a performance test such as the sweep test at the appropriate conditions or a loaded wheel test and wet track test for slurry and micr

- It should be adequate to this type of testing on a base asphalt-modifier system for certification. These tests should not be considered for tank or load acceptance testing.

- Again, I think the added cost does not give enough added value in the field.

- good and quick indicator of compatability between polymer and system

- Too variable and it is dependant upon an individuals opinion

- I not sure if I understand the question. Does mean using a microscope for polymer content?

- However, some suppliers may not have the necessary equipment or personnel to carry out this evaluation. They would have to contract it out and would need timely responses.

- Only if the testing does not take to long to get results back for certification.

- Would rather depend on specification tests to ensure product compliance.

- to cumbersome

- Too complicated.

- The microscopy is very subjective. Certification is going to determined by someone who has want no knowledge of how to work a microscopy. Microscopies are not used by engineers.

- It is not readily avaliable and it depends too much on the individual who run the test. The repeatability is too low.

- In emulsion residues - the need to have a "single phase" polymer / asphalt blend seems less evident. It may actually be better to have some degree of separation when the emulsion is acting as a binder

- I believe this "test" is too subjective to be incorporated in a product specification.

- Some form of the measurement of compatibility is necessary to ensure a good performing product. Since microscopy and othere tests are cumbersome, the test should be confined to certification, not as a product specification.

- It is difficult to turn a subjective evaluation procedure into a specification.

- This question is biased to the premodified emulsions. A latex product works by forming a polymer matrix during emulsion curing that is external to the asphalt. This type of compatibility test is meaningless for a latex modified emulsion. In the case of

- A certification or Designated sources list would cover it. It would not have to be run all the time as a part of standard everyday testing. Only when polymers change or crude slates change would it be needed and that would be up to the supplier.

- Until a specification can be developed, proof of compatibility would be sufficient.

- Use MSCR recovery loss between 100 & 3200 Pa instead.

- Again if this is an annual item and not a certification test i think it would be ok.

Heat Stability Comments

- A heat stability test is not needed on the residue. We make both premodified and latex modified emulsions. A heat stability test will change the polymer morphology of a latex PME residue and would be meaningless. In the case of premodified PMEs, since

- Emulsion residues are not exposed to extreme temperatures like a tank of modified binder would be. Therefore, I do not see the need for testing for heat stability. If I am understanding the question correctly, and there are some that are using the aluminum tubes

- Emulsion residues are not subjected to storage at high temperatures for long time periods.

- Emulsions are not subjected to the same conditions as hot binders, so heat stability testing is redundant.

- Heat stability of the residue could be a relevant test for emulsions placed in very hot desert conditions where the surface temperature can get up to 180 degrees F.

- I don\'t understand what you will be certifying.

- lower heat during storage and use of latex rather than SBS seem to reduce the tendency to migrate polymer. Some systems may differ.
- Not generally needed, but I would be in favor of a heat stability criteria for a precertification program.
- Not really that important as if it is not heat stable and falls apart then it is a supplier/contractor problem not a client problem.
- Performance base using feild conditions. How hot in the field?
- precertification would be the way to handle.
- Probably prefer using current standards.
- See the question above.
- Storage stability is the responsiblity of the supplier. It is not required in the specification or the certification program. If you are in the business then it is part of your program and not something that outside people should be judging or reviewing.
- THe product is an emulsion. The heat stability test is used to make sure that asphalt and polymer are compatible in a tank setting at elevated tempatures. Emulsion tanks are low temp and the end use is low temp.
- The storage stability test is sufficient. Easy to perform.
- This is definitely not necessary since the emulsions do not see those temperatures. If you are making an emulsion from a modified asphalt I believe the production limits may dictate the stability in many cases.
- This is not in my area.
- This would depend on storage time, but if the material is expected to be stored for extended periods (weeks) I'd like to see heat / storage stability information. Whether this is done as a specification or pre-certification depends on the program and tes
- We have never experienced any problems with polymer modified emulsions currently in use.

Superpave Comments

- A grading system using these tools was developed by Texas A & M for the Texas DOT. They also conducted a second study to correlate the specification to field performance. The specification was developed for climate conditions in Texas and needs to be ex

- ASTM has discussed these issues in task groups. Superpave grading has proven useful for binders, so it is logical that Superpave emulsified asphalt residue would be useful.

- But do we know what those numbers will actually be telling us based on performance. Might need to compare to similar base asphalts the emulsions are made from.

- Chip Seal Emulsions have been successfully applied for many years in the different States. This is a low cost, effective road preservation treatment. Many of the smaller emulsion manufacturers do not have the means or the staff to purchase and operate Su

- Chip seal specifications will have to figure out how to utilize SuperPave asphalt cements, rather than a whole new set of specifications just for the base stock of emulsion products. If paving-grade ashphalt cement is used neat (without emulsifying) then

- Evaluating residues using Superpave binder tests seems appropriate to better match the appropriate binder with climate. PAV would be useful in evaluating the aging characteristics of certain emulsion residues (surface seals).

- How does the DSR, BBR and PAV relate to early chip lose. or lose of chips after a season. People are trying to take something made for hot mix and apply it to a product that does not see those conditions. The whole idea of PG is to relate to performance.

- i do however support looking at these test methods. it is difficult for most to extend the concept that was developed for 1.5 inch thickness of an encapsulated mix and relate that behavior to applications less thn 3/8 inch where the aggregate may be expe

- I support the use of the DSR for residue, but PAV and BBR tests require too large a sample size to be practical.

- I would say yes if the appropriate specifications are developed. I would say that the rolling thin film oven is definitely not appropriate, but the aging in the PAV and ultimately testing the BBR may be appropriate for some long term performance.

- I would support using the equipment as long as test methods and protocols are developed that relate to PME performance. I would not support an adoption of the PG of SPG grades for PMEs.

- If we use a different method to recover the asphalt. With heating to 500 degrees, I am not sure who accurate the results are.

- Only if we can validate that they relate to performance in the application that the emulsion is being used for.

- Possibly for polymer modified emulsions if they can be tested after shipment.

- Specifying the original binder grade to be used for the emulsion.

- Superpave is the closest, most accepted procedure for assessing performance properties that we currently have. So, I am more than willing to support their use for residue testing.

- The added cost does not give enough added value in the field.

- The removal of water at lower temperature is necessary. Dealing with polymer and chemical modified emulsions would a challenge.

- There would have to be changes in the protocols to reflect how emulsion residues in chip seals age in the field. I have know idea what those changes might be in relation to aging temperatures and residue recovery temperatures.

- To the extent this is frequecy of testing is resonable.

- Too time consuming.

- Tools yes, protocols to be developed that are applicable to polymer modified emulsions.

- Use the current PG spec. Eliminate the plus spec.

- Using the Original DSR to establish the appropriatness of a binder for a given region is a good tool. The RTFOT shuld be excluded due to the lack of a hot plant and therefore the omission of the binder ageing step. The PAV should then be evaluated on th

- Yes but using the SPG and maybe developing some better tests that mean more for thicker surface films that are exposed to direct high frequency impacts in chip seals

Super Supplier Comments

- How to recover to duplicate field conditions? How to condition? Test?

- "If there was clear evidence that the tests related to performance final product on the road then we would supply data. It is not clear that all the SHRP tests developed for hot-mix necessarily apply to emulsion applications.

- Certainly a better way to recover residue

- It depends on how it is shared and used.

- it will begin the process of comparing different equipment and different systems. need to be able to correlate the sample wiht specific jobs to back up performance or lack thereof

- maybe

- No real explanations should be necessary. I think this is in the best interest of the industry. If there are issues with variability this would be discovered quickly and the group could discuss how things could be adjusted.

- Once everything is established I would imagine we would run SHRP testing
- This is a USA initiative and may not be adopted internationally
- We would follow the ASC program or a state program. It would be difficult to test every batch of product.
- We would provide data for our binders both before and after emulsification.
- We would provide if requested.
- We would welcome the opportunity to be involved.

Aggregate Test Comments

- Aggregate quality is essential for successful surface treatments, whether polymer modified or not.
- Aggregate specifications need to be related to traffic conditions. A lower quality aggregate may perform satisfactorily when subjected to low traffic volumes. We successfully use limestone for most chip seals, but it polishes if used in higher traffic a
- Aggregates are obviously very important. I think all of these tests are critical. Usually, aggregate supplies have already run most of these tests on their aggregates for HMA, so it shouldn't be too much more work to incorporate into their chips/slurry
- All are important. LA Abrasion is really impact resistance and should not be dropped in favor of Micro Deval. Each test tells you something different and valuable.
- Cleanliness, size/gradation, resistance to abrasion and polishing are all critical with chip seal aggregates.
- From 35 years of dealing with chip seals cleanliness of the aggregates is extremely important, particularly in areas where the aggregates are not washed. Wetting the chips before delivery to the chip box helps to clean them. When paving-grade asphalt ce
- Gradation and hardenss (LA or Micro D).
- I believe the tests are available now to give good numbers for the aggregates used in the pavement preservation techniques used. THe only suggestion I would have is something like a tolerance for the sand equivalent test to make sure some measure of cons
- I think all apply. If you are trying to develop and place high performance seals then good high quality aggregates are needed. All the above tests help in that regard. Maximum and minimum test values would have to be established to ensure high quality. A

- I think that it is a very good idea to evaluate chip seal aggregates. In Wisconsin, the only requirements are in size/gradation along with fractured surfaces. In my opinion, there are many other factors besides size that make up a good chip sealing aggr

- Mainly Micro Deval and Cleanliness.

- Only tests that are applicable to the interaction between the emulsion and aggregate should be included in an emulsion spec. Other test such as polishing and LA Abrasion relate to pavement performance and not necessarily to the quality of the emulsion.

- Particle shape is one of the most important characteristics in chip seal design. This should be controlled.

- Quality aggregate is a necessity. While pme will allow you to obtain satisfactory results with marginal aggregate, the objective should be the best performance possible and a way to reduce the risk. Identify the test that are applicable to the application

- SEQ spec for micro needs to be enhanced by adding a tolerance to mix design sample for example min of 70 with +/- 5 tolerance

- The most critical would be cleanliness and gradation followed by some type of durability.

- The sand equivalent test indicates the presence of clays whereas the Methylene blue indicated the presence of clay as well as reactive fines. I believe the presence of clay materials is more detrimental.

- These tests are instrumental to determine the quality of the aggregate as well as sizing and adhesion qualities (cleanliness).

- We have aggregate specifications that are adequate that cover our sealing chip quality in New Zealand.

Sweep Test Comments

- Will vary to much based on the various aggregates and weather conditions.

- We like to let the traffic on the new seal as soon as possible and conditions on the day are likely to be different than lab conditions normally faster and specs developed around a lab test could create more issues than they solve. A contractor will not

- Too many field variables which cannot be duplicated in the laboratory. This test is only relevant when compared to a control sample and then only at the test conditions of temperature and humidity. Softer asphalts will not perform as well on the sweep test

- This test is specifically designed to simulate field performance.

- The sweep test was one of the first performance test for Chip Seal. Still needs work.

- The sweep test is interesting from a comparitive standpoint (single rock source / multiple formulas) when formulating . However I am reluctant to have this become an acceptance or penalty test - because of the uncertainty of obtaining representative emulsion

- The sweep test is an excellent tool for formulating emulsions. But as a QA test it has several deficiencies; it takes too long to perform, it is only relevant with job aggregates.

- The agencies do not vary the time for curing. The job is chip sealed, rolled and traffic is opened immediately.

- Sweep test is a good replicate of teh soundness of the finished pavement, especially is loose aggregates leading to windshield damage is a concern.

- So many factors contribute to successful performance that could not be manipulated/controlled in the lab sweep test that I think this approach is impractical.

- Seems to be a good indicator of performance.

- Refinement of the test is necessary. The test can be highly variable and a slight change can induce large errors.

- Once performance criteria can be established I think the sweep test or an improved version would be a solid tool to help

- Not as written. It needs the Takamura modifications to be more consistent. It also needs the aggregate to be graded to a specified single size for the test to have meaning. If a multi sized aggregate is used, the loss can be considerable even if the e

- No additional comments.

- Maybe for certain high traffice situations

- Lab conditions would not necessarily indicate field performance due to varying conditions of temperature, sunlight, moisture, etc.

- Knowing the necessary cure time or specifying a maximum cure time or even a pass fail cure time addresses the fundamental problem of chips adhereing to the road surface. Cure time is different than the break time and is dependent upon temperature and other

- It is my opinion that the sweep test is too user depended. I worked on devolping the test and it is a good test to make sure the asphalt and rock will work togather but not sure about time to sweep. There are easier ways to determine in the field which

- I would support using the sweep test--perhaps in its modified version--to certify a job aggregate/emulsion formula. I would not be in favor of using the sweep test as part of emulsion specification verification testing.

- I have never done the test so I can't give an informed response.
- I believe the two level spec helps keep the cost down on lower volume while increasing the performance on higher ADT
- I believe it would be good tool to determine aggregate and emulsion compatibility. After that it would help in performance.
- Field tests would be helpful to identify application issues in support of visual observations.
- Extensive testing is being done under NCHRP 14-17 and a specification is to be developed.
- But, as discussed before, this needs to be verified with multiple labs before implementing any sort of specifications.
- Again, if the appropriate curing times and temperatures are used for the climate and performance expectations.

Chip Loss Comments

- Are these test indicators of long-term chip loss? Validate. We need help.
- Frosted Marble
- frosted marble to take aggregate variations out of play
- Frrosted Marble
- I believe as time passes, chip loss is less of a problem.
- I dont know if these truly represent what happens in the field.
- I prefer the Vialit Plate Shock Test but this test is only relevant on the fresh emulsion and is no indicator of longer term performance.
- I think this question assumes that these tests actually do differentiate the long term chip loss and I do not think they may.
- It is a chip seal that adds no structural value to the pavement and is for maitenance purposes mainly.
- Long-term chip loss is more often caused by inappropriate binder application rate or poor surfacing design rather than the quality of the binder or emulsion.

- Methods selected should have some relation to field performance.
- Not applicable. Not currently familiar with these tests.
- Not familiar with the quoted tests, but something to address long-term performance would be benefial.
- Not that familar with these tests. However, have heard good things about Vialit Plate Shock test.
- The finished product of a chip seal is the contractor's responsibility, not that of the emulsion supplier. Emulsion suppliers have no control over the quality of the aggregate used and should not be subjected to the responsibility.
- These seem to be a bit subjective. May not be good indicators of performance.
- These tests do not consider the traffic.
- They should be evaluated and compared to field performance. I would support this type of test if there is a correlation to actual performance. Field performance would need to be evaluated in multiple climates and conditions, such as snow plowing.
- Vialet may give more meaning to chip loss as it involves the aggregate and emulsion resudue being used on the job. The frosted marble relates more to the binder and one aggregate.
- Vialit cohesion and adhesion tests
- We currently use ASTM D-7000 and the Frosted Marble to evaluate products. We use the Frosted Marble to measure Binder cohseion development and cure rate and ASTM D-7000 to evaluate the system.
- We have found that the Vialit does not indicate the chip loss because we can not get the job aggregate, nor do we have any control of the aggregate.
- We have used the Vialit test to confirm poor aggregate / binder combinations. If these different tests provide a way of identifying different problems with the emulsion, they would help. Otherwise it might be extra testing to identify the same issue.
- With all this testing the chip would be too hard for agencies to use. I support better training of the inspectors and contractors to take care of these issues.

Micro Comments

- Examination of the mix cure behavior at an appropriate depth would an improvement.

- I am not familiar with these tests, but if they can be implemented cost-effectively they will be attractive.
- I said yes on this, but I think a deficiency is in the determination of a true rut filling test on the mixture. A micro-surfacing material can cure quickly, but still not perform in a rut filling application.
- I think these preformance tests are more useful than the emulsion or aggregate tests, as they explore the full mixture, instead of just the components.
- N/A
- Not applicable. Need to fully examine ISSA tests.
- One of the best test is also very easy to run and repeatable and that is one hour night time test strip. In our state it has seperated the polymer modified slurry systems from the true chemical curing Micro systems. It is easy for the inspector to run
- Should be reviewed by a task group before just accepting it.
- The ISSA tests are a good starting place
- the test method used in the mix design were mostly if not completely the work of the late Ben Bennidict. great man , great pioneering work but still there are a lot of loop hole in the methods and a lot of unfinished work as to confirmation of design the
- Tighter specs, higher minimum polymer loading , tighter lateral and vertical values. A refinement of the loaded wheel tester should be done. A different way of how the weighted material is mover across the sample strip. Tends to put high strees on ends
- Too many contractors are putting down a rapid cure slurry and calling it a micro-surface. We need to hold micro whcich is a higher price product to a higher standard so that the tax payers get what they paid for.
- Validate.
- We have had good performance from micro-surfacing designed to ISSA standards.
- What is the benefit? Ruts have nothing to do with agencies that is chip sealing as it is usually done with maintenance.

Rut Comments

- Although the same equipment is used, rut filling is a very different application than a general surfacing.

- An excellent tool for rutfilling.
- Don't know.
- I believe that PG grading would be more important. Using the same PG grade for southern Texas as for Minnesota makes no sense. We have filled ruts using PG asphlat that graded as 48-34 with no rutting. This grade of asphalt cracked slow and less.
- If states have had success with non-PME grades, there is no reason to force them to change.
- If the application is limited, testing for only the application performance criteria is logical.
- Most CQS emulsions will pass the regular test for Microsurfacing PME until rut filling comes into the equation.
- N/A
- need polymer/mineral filler structure
- Not applicable, see above.
- not sure
- Performance. Blind to the system.
- pg 76-28
- polymer should be included for rut filling, either latex or polymer in the asphalt.
- Rut fill is a separate area and specific requirements should be established. Possibly tighter numbers on laterals and verticals and higher minimum polymer loading.
- Rut-filling to any extent pushes the capability of aggregate and binder, necessitating the need for a very tough binder.
- See above response in question four. We are using too many latex modified slurry seals that are called micro-surfacing. When not used for rut filling they are usually appropriate, but for filling ruts some formualtions are inadequate.
- The performance tests on the on the system (aggregate, additives, emulsion) such as ISSA displacement, are adequate. There are too many variables in a micro system to dictate specific grades of PME.

- The reality of a multistone depth rut fill introduces a higher expectation of performance of the emulsion / aggregate system. As such a short term strength of cure test should be adopted.

- The same emulsion can be used for rut-filling and surfacing. Any required difference in performance can be achieved through aggregate gradation and construction or mix design.

- There are many rut-filling mixes that are working great currently and do not need the added expense of PME grading. We have our own product that works and developed testing in house to maintain quality. Easy street is a good example of another product. Th

- We would use a PME grade for all micro-surfacing applications. A lot is expected from micro-surfacing and the PME is worth the additional cost.

- why specifically for rutfilling if the mix needs the modified binder then use it otherwise don't.

Low Traffic Comments

- They should be upgraded but low traffic does not apply in most places. I would like to see more specifications on the product.

- There are some issues with the equipment that merit some changes.

- The technical representative from ISSA have a wealth of practical field experince.

- Possible refinement of WTAT maximums to a lower number.

- Performance Testing.

- Our experience has shown that the current design works well for low traffic areas.

- Not sure what this question means. If it means that a latex modified slurry seal should be differentiated for low volume areas, while a true micro-surfacing spec can be in place for low and high volume areas I would agree.

- not sure

- Not applicable, no comment.

- No opinion. We have not used a slurry system for many years.

- N/A

- Lower traffic areas generally require higher levels of asphalt. High levels of asphalt almost always pass the wet track abrasion test.

- If they can be implemented cost-effectively they will be attractive.

- If the wet track is being used for control then it is being used incorrectly. The wet track is run to determine the minimum amount of asphalt needed for a given system. It does not demonstrate too much asphalt or even the correct amount of asphalt...jut t

- Any improvement to the system is a good thing.

- Again, if agencies have had success with their current proceudres, there is no reason to force them to change.

- 1 day soak WTAT's should be allowed. All should be 6 day soak. I would not recommend any other changes.

Fugro Microsurfacing Study Comments

- Conflicting answers so Yes I am familiar but no i do not support the changes wholesale at least as I currently understand them. Two examples, There is a single spec for both micro and slurry...this is not practical. There is no effort to determine minim

- Again, micro is a more expensive product, not just a fast setting slurry. We need better standards to distinguish micro from regular slurry.

- I agree with some of the changes, but not others that they are recommending

- I am aware and very limited to the knowledge.

- I am not aware of the details of this. I would like to know more.

- I am not aware of this research and would wish to review proposed changes before offering support.

- I am on the TAP for the pooled fund and I am very disappointed with the work. They missed what the States where asking for in my opinion.

- I support the research only if the automated testing is not going to be put into specification without a lot of test date. Many times automated equipment comes in and it can only work on one type of material or the results are all over the map depending o

- N/A

- Need to review. Validate. Performance changes.

- Not aware of their specific recommendations.

- Not up with the play on this

- Two questions here. I am not aware of the research, so I can not support the changes one way or the other.

- We have not had the equipment to compare and evaluate. Once that equipment is in stock and comparable testing can be made we can make better judgements on this.

- Yes, better tests or improvements in the current tests are needed to better reflect what is going on in the real world.

Certification Comments

- all are equally important.

- All participants in the supply chain (including consultant) should be certified for minimum levels of competency, training and quality management.

- As you know, there are many factors that can cause chip seal failures. Some of these failures are linked to emulsion quality. But many others are caused by construction practices, weather, existing road conditions, etc. As an emulsion supplier, we feel

- Certification is a good way to educate, but unless jobs are inspected by knowledgeable inspectors with the authority and willingness to shut down a project for non-compliance, the certification will be a waste of time.

- Contractors could be certified but not individual workers. Labs and technicians could be certified. Superpave lab tests on emulsion residues would need additional time (after shipment) to be tested, possibly at a central or outside lab.

- For our binder ASC program we only accept certification testing from laboratories AMRL accredited in the applicable tests. Contractors generally do not have the facilities to become accredited.

- I believe all parties should be certified.

- I think a contractor could be ISO certified which would cover him. I believe the technicians doing the testing should be certified as is the case with HMA and concrete. The suppliers should be certified be it through ISO or some other type of certificat

- If the owner's representative is one of the Individuals that is as it should be because too often it is the on-site inspector or engineer who is not experienced enough to make the necessary field decisions. For instance cure time is a time versus condition

- Individual certification for contractors would be difficult to accomplish. I feel a contractor certification in general would be adequate if the certification includes them having a good training practice for employees.
- Material Supplier
- Material supplier certification similiar to the Combined States Binder Group certification program.
- Most material suppliers have trained staff and equipment to perform certification tests , some more than others.
- Only those with a reasonable liability or contractual obligation need to be certified.
- Should be certification to ISO 9000 Quality systems with standard prescribed requirements from TCCC specific for each portion. eg Contractor requirements, Materials Suppliers requirements etc.
- The total system should be certified to insure the opportunities for success.
- Until it is determined which of the four is most useful, all four should be run in parallel.
- We have found that the contractor is very knowledgable about the emulsion and chip seal. We found that often times it is the agency who knows nothing and trys to control the job thus causing the problem themselves.

Interest Comments

- Again, we would welcome the opportunity to be involved.
- As a supplier would be willing to contribute and comment on any program
- I believe I'm out of this game and my ideas are probably dated.
- I believe the certification process at all levels should include folks from all industries and agencies.
- I do not know if I would be able to be directly involved in the development of the program.
- I think this a great start to continue to improve our products and certification processes.
- I would like to kept updated on the status of this.

- I would like to see more RELEVANT testing of asphalt emulsions for different applications in place. However I would not like to see certification criteria put in place which are detrimental to the smaller operations in our industry who make a quality prod

- Interested in training. I think the roads would improve a great deal by just a little training. I feel we are a long way off for certification. In the states where we supply there are only a few contractors who do chip seals or slurry seals. They are very

- Might be interested in providing some training through our Tech Transfer program.

- These programs should have wide participation to get the best results.

- This is a USA initiative and may not be adopted internationally

- we are suppliers to the industry...what helps industry grow helps us grow.

- We have a limited research budget and since we have had overall success with micro-surfacing we would probalby not contribute to the development of certification criteria. We would evaluate any developments to determine if it would be beneficial for us t

- Would be willing to participate in establishing certification criteria as well technicians and labs.

- Would participate in developing the criteria and resulting specifications.

APPENDIX B - SPECIFICATIONS USED FOR FIELD PROJECTS

FHWA Technology Study: *Using Polymer Modified Asphalt Emulsions in Surface Treatments*

Task 3: Laboratory evaluation of strawman testing protocol.

Project Specifications

409 – Chip Seal Specification (standard)

410 – Micro-Surfacing Specification (Utah projects only)

702 – Asphalt Emulsion & Aggregate Specification – Utah Projects

702 – Asphalt Emulsion & Aggregate Specification – Crater Lake (CRLA)

702 – Asphalt Emulsion & Aggregate Specification – Death Valley (DEVA)

702 – Asphalt Emulsion & Aggregate Specification – Dinosaur (DINO) Standard Specification (some project-by-project modification)

B.1 Section 409. - ASPHALT SURFACE TREATMENT

Standard Specification (some project-by-project modification)

Description

409.01 This work consists of constructing a single or multiple asphalt surface treatment with aggregate or precoated aggregate. This work also includes constructing an asphalt fog seal without aggregate.

Surface treatment aggregate designation is designated as shown in Tables 409-1, 409-2, and 409-3.

Provide emulsified asphalt grade CRS-2P or equivalent meeting the requirements of Table 702-4.

Material

409.02 Conform to the following Subsections:

Aggregate 703.10

Asphalt binder 702.01

Blotter 703.13

Emulsified asphalt 702.03

Construction Requirements

409.03 Qualifications. Submit the following information for approval at least 28 days before placement.

Companies and individuals involved with the placement of asphalt surface treatments must conform to the:

(a) Demonstrate satisfactory completion of at least 10 comparable projects.

(b) Provide Superintendent or Foremen experience in surface treatment construction on at least 10 comparable projects.

409.04 Composition. Submit the following information and samples for approval at least 21 days before placement:

(a) Aggregate samples. 80 pounds from each stockpile produced and the gradation range represented by each.

(b) Aggregate gradation target values. The proposed percentage of each stockpile to be used and the proposed target value for each sieve size. Standard Specification (some project-by-project modification)

(c) Asphalt samples. 2 1-quart samples of asphalt binder or emulsified asphalt from the same source and of the type to be used for the surface treatment.

(d) Asphalt temperature. Apply asphalt at temperatures according to Table 702-1.

(e) Spread rates. The proposed spread rate for the aggregate and asphalt material.

409.05 Equipment. Furnish equipment as follows:

(a) Asphalt distributor.

(1) Capable of heating asphalt evenly.

(2) Adjustable full circulation spray bar to 15-foot width.

(3) Positive controls including tachometer, pressure gauge, volume measuring device, or calibrated tank to uniformly deposit asphalt over the full width within 0.02 gallons per square yard of the required rate.

(4) Thermometer for measuring the asphalt temperature in the tank.

(b) Vacuum Sweeper. Furnish a minimum of two vacuum sweepers both with the following capabilities:

(1) Self-propelled.

(2) Capable of controlling the vertical broom pressure.

(3) Capable of removing excess aggregate particles.

(c) Pneumatic-tire rollers. Furnish a minimum of two pneumatic-tire rollers both with the following capabilities:

(1) Self-propelled.

(2) Minimum compacting width - 5 feet.

(3) Gross weight adjustable within the range of 200 to 360 pounds per inch of compaction width.

(d) Aggregate spreader.

(1) Self-propelled.

(2) Minimum of 4 pneumatic tires on 2 axles. Standard Specification (some project-by-project modification)

(3) Positive controls to uniformly deposit the aggregate over the full width of asphalt within 10 percent by mass of the required rates.

(e) Other equipment. Other equipment of proven performance may be used in addition to or in lieu of the specified equipment when approved by the CO. Provide two-way communication between the asphalt distributor and the aggregate spreader if the roadway alignment does not permit visual contact.

409.06 Surface Preparation. On existing asphalt surfaces, ensure that the surface is dry. Immediately before placing the layer of chips, remove loose dirt and other objectionable material from the surface by approved methods. Fog seal patches using a slow setting emulsion diluted with an equal part water. Apply the diluted emulsion at a rate of 0.15 gallons per square yard.

409.07 Weather Limitations. Apply surface treatment or fog seal according to the following:

(a) Apply single or multiple asphalt surface treatments when:

(1) Between June 15th and September 4th unless other dates are approved by the CO.

(2) Ambient air temperature is above 68°F and rising and surface temperatures are between 80°F and 140°F

(3) Weather is not foggy or rainy, and when rain or temperatures below 40°F are not anticipated for at least 24 hours after application.

(4) Winds are less than or equal to 10 miles per hour.

(5) Complete surface treatment application at least 2 hours before sunset.

(b) Apply fog seal when:

(1) Ambient air and surface temperatures are above 50°F and rising.

(2) Weather is not foggy or rainy, and when rain or temperatures below 40°F are not anticipated for at least 24 hours after application.

(3) Complete fog seal applications at least 2 hours before sunset.

409.08 Production Start-Up Procedures for Surface Treatments. At least 10 days before the start of constructing all surface treatments containing aggregate, arrange for a pre-surface treatment conference. Coordinate attendance with the CO and any applicable subcontractors. Be prepared to discuss or submit the following:

(a) Proposed schedule of operations. Standard Specification (some project-by-project modification)

(b) List of all personnel involved in the production and construction of the work including equipment calibration, sampling, and testing.

(c) List of equipment, quantity, and description to be used in the production and construction of the work.

(d) Proposed traffic control plan.

(e) Discuss Section 153, minimum frequency schedule for process control sampling and testing (to be performed by the Contractor).

(f) Discuss Subsections 409.08; 409.09, and 409.10.

(g) Discuss spill prevention and safety contingency plan.

Provide 7 days advance notice before constructing all asphalt surface treatments containing aggregate. Also use these start-up procedures when resuming production after termination due to nonconforming work.

On the first day of placement of each surface treatment layer, or whenever there is a change in the surface texture or aggregate gradation, construct a minimum of three 50-foot control strips that are one-lane wide. Each control strip will have different application rates of emulsion and/or surface aggregate. The CO will indicate which strip of the will serve as the approved project control strip. Coordinate location of the control strips with the CO.

Construct the control strip using material, lay-down, and compaction procedures intended for the remainder of the surface treatment. Cease production after construction of the control strip until the material and the control strip are evaluated and accepted.

Acceptable control strips may remain in place and will be accepted as a part of the completed surface treatment.

Repeat the control strip process until an acceptable control strip is produced.

409.09 Asphalt Application. Calibrate the asphalt distributor spray bar height, nozzle angle, pump pressure and check the longitudinal and transverse spread rates daily, before start up, and as directed by the CO according to ASTM D 2995. If different asphalt distributors are used, calibrate each before use on the project. Ensure that the length of the spread is no more than can be covered with aggregate immediately after application. Document all calibration and application rates and provide to the CO at the end of each days production.

Protect the surfaces of nearby objects, such as stone curbing, to prevent spattering or marring. Spread building paper on the surface for a sufficient distance from the Standard Specification (some project-by-project modification)

beginning and end of each application so the flow through the distributor nozzles may be started and stopped on the paper.

Apply the asphalt uniformly with an asphalt distributor at the optimum application rate determined from the test strip. Move distributor forward at the proper application speed at the time the spray bar is opened. Stop application if any nozzles are plugged or if triple nozzle spray coverage is not occurring. Use care not to apply excess asphalt at the junction of spreads.

Correct skipped areas or deficiencies. Remove and dispose of paper or other material used.

409.10 Aggregate Application. When using asphalt binder, the aggregate surface should be dry. When using emulsified asphalt, the aggregate surface should be moist. Verify aggregate stockpiles moisture daily during production with visual inspection.

Apply the aggregate uniformly with an aggregate spreader immediately after the asphalt is applied at the optimum application rate determined from the test strip. Check and record spread rate daily, before start up, and as directed by the CO. Operate aggregate spreader so the asphalt is covered with the aggregate before wheels pass over it. During part-width construction, leave uncovered a strip of sprayed asphalt approximately 6 inches wide to permit an overlap of asphalt material.

Immediately correct excesses and deficiencies by brooming or by the addition or removal of aggregate until a uniform texture is achieved. Use hand methods in areas not accessible to power equipment.

When precoated aggregates are used, they may be mixed on the job or at a central mixing plant. Uniformly coat the aggregate with 1.0 to 2.0 percent residual asphalt, by weight of aggregate. Maintain the flow qualities of the precoated aggregate, so it is satisfactorily spread with an aggregate spreader.

Operate rollers at a maximum speed of 5 miles per hour. Do not permit the aggregate to be displaced by pickup or sticking of material to the tire surface. Roll the surface to uniformly and thoroughly bond the aggregate over the full width. Complete rolling within 1 hour after asphalt is applied to the surface.

409.11 Fog Seal. A fog seal consists of applying slow-setting emulsified asphalt diluted with water onto an existing asphalt surface. Unless otherwise noted on the plans, dilute the specified emulsion one part water to one part emulsified asphalt. Apply the diluted emulsified asphalt according to Subsection 409.09 at a rate of 0.10 to 0.15 gallons per square yard depending on the condition of the existing surface. Allow the fog seal to penetrate undisturbed for at least 2 hours or until the emulsified asphalt breaks and is substantially absorbed into the existing surface. Then lightly cover remaining spots of excess asphalt with blotter according to Section 411 before opening the surface to traffic. Standard Specification (some project-by-project modification)

409.12 Single-Course Surface Treatment. A single-course surface treatment consists of applying asphalt material onto an existing surface immediately followed by a single, uniform application of aggregate. Apply the asphalt and aggregate according to Subsections 409.09 and 409.10 at the approximate rates shown in Table 409-1. Application rates shown in Table 409-1 should be used for estimating purposes only. The contractor shall determine aggregate and asphalt application rates that may fall outside the ranges shown in Table 409-1. Before curing, the emulsion should rise just below the top of the aggregate. After curing, embedment depth of the aggregate in the residual asphalt should be approximately 60% of the nominal maximum size. Determine the exact rates based on approved control strips.

Use a pilot car according to Section 635 to limit traffic speeds. During the initial 45 minutes after completion of rolling, limit the traffic speeds to 10 miles per hour. Limit traffic speeds to 20 miles per hour for 24 hours.

Lightly broom the aggregate surface on the morning after construction. Maintain the surface for 4 days by distributing blotter according to Section 411 to absorb any free asphalt and by repairing areas deficient in aggregate. Remove excess material from the surface using a rotary broom. Do not displace embedded material. Do not broom the surface where the air temperature is above 90°F.

Table 409-1

Approximate Quantities of Material for Single Course Surface Treatments

Single-Course Surface Treatment Designation	Nominal Maximum Size of Aggregate	Aggregate Gradation	Estimated Quantity of Aggregate pounds/yd^2	Estimated Quantity of Emulsified Asphalt gallons/yd^2	Estimated Quantity of Asphalt Binder gallons/yd^2
1A	¾ inch	B	44 – 53	0.48 – 0.65	0.31 – 0.43
1B	½ inch	C	29 – 33	0.39 – 0.53	0.27 – 0.36
1C	⅜ inch	D	24 – 28	0.27 – 0.43	0.17 – 0.29
1D	No. 4	E	18 – 24	0.22 – 0.29	0.14 – 0.19
1E	Sand	F	13 – 18	0.17 – 0.24	0.12 – 0.17

(1) See Table 703-7 for aggregate gradations.

(2) Aggregate masses are for aggregates having a bulk specific gravity of 2.65, as determined by AASHTO T 84 and AASHTO T 85. Make proportionate corrections when the aggregate furnished has a bulk specific gravity above 2.75 or below 2.55.

Standard Specification (some project-by-project modification)

409.13 Acceptance. Asphalt binder, and emulsified asphalt, will be evaluated under Subsections 106.03, 106.04 and 702.09. Furnish a production certification for the grade of emulsified asphalt specified in Subsection 409.01.

Aggregate gradation for asphalt surface treatment will be evaluated under Subsection 106.05.

See Table 409-4 for sampling and testing requirements.

The upper and lower specification limits are equal to the calculated mean of all test results plus or minus the allowable deviations shown in Table 703-7, except as follows:

(a) If the calculated mean value for any tested sieve exceeds the maximum gradation value shown in Table 703-7, the upper specification is equal to the maximum gradation value plus the allowable deviation, and the lower specification is equal to the maximum gradation value minus the allowable deviation.

(b) If the calculated mean value for any tested sieve is less than the minimum gradation value shown in Table 703-7, the upper specification is equal to the minimum gradation value plus the allowable deviation and the lower specification is equal to the minimum gradation value minus the allowable deviation.

Construction of asphalt surface treatment course will be evaluated under Subsections 106.02 and 106.04.

Prime coat and blotter will be evaluated under Section 411.

Measurement

409.14 Measure the Section 409 items listed in the bid schedule according to Subsection 109.02 for each day's production and the following as applicable.

Measure and provide temperature volume corrections for emulsified asphalt and asphalt binder to 60°F.

Measure surface treatment aggregate in the hauling vehicle prior to stockpiling or prior to placement if not stockpiled.

Measure fog seal including water added for dilution.

Indicate a breakdown of total emulsion and water added on the load invoices supplied to the CO for payment.

Measure blotter under Section 411. Standard Specification (some project-by-project modification)

Payment

409.15 The accepted quantities will be paid at the contract price per unit of measurement for the Section 409 pay items listed in the bid schedule except the aggregate surface treatment contract price unit bid price will be adjusted according to Subsection 106.05. Payment will be full compensation for the work prescribed in this Section. See Subsection 109.05.

Table 409-4
Sampling, Testing and Acceptance Requirements

Material or Property	Type of Acceptance (subsection)	Characteristic	Category: Test Methods Specifications	Sampling Frequency	Point of Sampling	Split Sample	Reporting Time
Aggregate Surface treatment aggregate [1] (703.10)	Measured and tested for conformance (106.04 & 105)	LA abrasion	--- AASHTO T 96	1 per type & source of material	Source of Material	Yes, when requested	Before using in work
		Sodium sulfate soundness loss (coarse & fine)	---	AASHTO T 104	“	“	“
		Fractured faces	---	ASTM D 5821	“	“	“
		Flat & elongated particles	---	ASTM D 4791	“	“	“
		Adherent coating	---	ASTM D 5711	“	“	“
		Clay lumps & friable particles	---	AASHTO T 112	“	“	“
Aggregate surface treatment aggregate [1]	Statistical (106.05)	Gradation. See Table 703-7 for applicable sieves	I AASHTO T 27 & T 11	1 per 750 tons	Production belt or spreader discharge	Yes	24 hours
	Measured and tested for conformance (106.04 & 106.05)	Fractured faces	ASTM D 5821	1 per 750 tons	Production belt or spreader discharge	Yes	24 hours
		Liquid limit [2]	AASHTO T 89 “	“	“		“
Asphalt binder [3] (702.01) or emulsified asphalt [3] (702.03)	Measured and tested for conformance (106.04)	Quality	Subsection 409.13	1 per tanker truck including trailer	Point of shipment delivery	2 1-quart samples	---

1) Applies to each aggregate grade furnished.
(2) For blotter material only.
(3) Applied to each asphalt material furnished.

B.2 Section 410. - MICRO-SURFACING

Description

410.01 This work consists of applying a polymer modified micro-surfacing mix on an existing pavement surface.

Micro-surfacing Type III as shown in Table 703-8 is to be used on this project. The residual asphalt content specified is 7.5 ± 2 percent by dry total weight of aggregate.

410.02 Conform to the following Subsections:

Aggregate 703.11

Emulsified asphalt, polymer modified 702.03(d)

Mineral Filler 725.05

Water 725.01(c)

Construction Requirements

410.03 Composition of Mix (Job-Mix Formula). Furnish a micro-surfacing mixture of aggregate, water, polymer modified emulsified asphalt and additives according to ASTM 6372-05. Conform to the Type III aggregate gradation in Table 703-8 and the residual asphalt content in Subsection 410.01.

Submit a written job-mix formula for approval at least 14 days before production that meets the mix design requirements in ISSA A143 for micro-surfacing. Submit the following:

(a) Aggregate gradation values. The representative value for each sieve size for the aggregate blend.

(b) Emulsified asphalt content. The residual asphalt content, as a percent by mass of dry aggregate.

(c) Polymer modifier. Type and amount of polymer modifier solids based on the residual asphalt content.

(c) Aggregate samples. 100-pound sample of each aggregate.

(d) Emulsified asphalt sample. Source of and 5-gallon sample of the emulsified asphalt to be used in the mix.

(e) Mineral filler samples. 50-pound sample of each proposed mineral filler.

(f) Qualifications. Demonstrate satisfactory completion of at least 5 comparable projects. Provide Superintendent or Foreman experience in micro-surfacing on at least 5 comparable projects.

The job-mix formula will be evaluated for approval.

410.04 Equipment. Furnish with the following capabilities.

(a) Mixing equipment.

(1) Self-propelled;

(2) Continuous-flow mixing,

(3) Calibrated controls;

(4) Easily readable metering devices that accurately measure all raw material before entering the pugmill;

(5) Automated system for sequencing in all raw material to ensure constant slurry mix;

(6) Mixing chamber to thoroughly blend all ingredients together;

(7) Fines feeders with an accurate metering devices for introducing additives into the mixer;

(8) A pressurized water system with a fog-type spray bar capable of fogging the surface immediately ahead of the spreading equipment at a rate of 0.03 to 0.06 gallons per square yard;

(9) Proportioning system that is accurate for measuring all material independent of engine speed;

(10) Minimum speed of 60 feet per minute and maximum speed of 180 feet per minute;

(11) Minimum storage capacity of 7 tons; and

(12) Capable according to ISSA Performance Guidelines A143.

(b) Mechanical-type single squeegee spreader box.

(1) Attaches to the slurry seal mixer; CO IMR-PRES-1(08) Utah Parks

(2) Flexible squeegee in contact with the surface to prevent loss of slurry;

(3) Adjustable to ensure a uniform spread over varying grades and crowns;

(4) Adjustable in width with a flexible strike-off; and

(5) Augers for uniform flow to edges.

(c) Auxiliary equipment. Furnish hand squeegees, shovels, and other equipment necessary to perform the work. Provide cleaning equipment including, but not limited to, power brooms, air compressors, water flushing equipment, and hand brooms for surface preparation.

410.05 Surface Preparation. Clean the existing surface of all loose material, dirt, or other deleterious substances by approved methods. Protect all service entrances such as manholes, valve boxes and drop inlets from the micro-surfacing by a method suitable to the CO. Protect all concrete work, rock walls and other objects from the micro-surfacing with a method suitable to the CO.

410.06 Weather Limitations. Apply the mixture when the air temperature in the shade and the surface temperature are at least 45°F and rising and when the weather is not foggy, rainy, or overcast. Do not apply when there is a danger that the finished product will freeze within 24 hours.

410.07 Production Start-Up Procedures for Surface Treatments. At least 10 days before the start of constructing the micro-surfacing, arrange for a premicro-surfacing conference. Coordinate attendance with the CO and any applicable subcontractors. Be prepared to discuss or submit the following:

(a) Proposed schedule of operations.

(b) List of all personnel and equipment involved in the production and construction of the work including equipment calibration, sampling, and testing.

(c) Proposed traffic control plan.

(e) Discuss Section 153, minimum frequency schedule for process control sampling and testing (to be performed by the Contractor).

(f) Discuss Subsections 410.05, 410.06, 410.07 and 410.08.

(g) Discuss spill prevention and safety contingency plan.

Provide 7 days advance notice before constructing all micro-surfacing. Also use these start-up procedures when resuming production after termination due to nonconforming work.

On the first day of placement, construct a 300-foot test strip, one lane wide. The CO will approve the test strip before production begins. Coordinate location of the control strips with the CO.

Construct the control strip using material and lay-down procedures intended for the remainder of the micro-surfacing. Cease production after construction of the control strip until the material and the control strip are evaluated and accepted.

Acceptable control strips may remain in place and will be accepted as a part of the completed surface treatment.

Repeat the control strip process until an acceptable control strip is produced.

410.08 Application. Mix the materials using a slurry seal mixer and according to ISSA Performance Guideline A 143. Fog the surface with water immediately preceding the spreader.

Blend the additives with the aggregate using the fines feeders. Pre-wet the aggregate in the pugmill immediately before mixing with the polymer-modified emulsified asphalt.

Mix the surfacing materials a maximum of 4 minutes. Ensure the mix is of the desired consistency as it leaves the mixer and conforms to the approved job-mix formula. Adjustment of the mineral filler and the emulsified asphalt content during construction may be approved to adjust for variations in field conditions.

Carry sufficient mix in the spreader to completely cover the surface. Spread the mix with a mechanical-type squeegee spreader box. In areas not accessible to the spreader box, use hand squeegees to work the mix.

Allow treated areas to completely cure before opening to traffic. Cure is complete when clear water can be pressed out of the mix with a piece of paper without discoloring the paper.

Prior to starting application of micro-surfacing, calibrate each mixing unit to be used on the project in accordance with ASTM D 6372 and in the presence of the CO or designated representative. Clean spreader box prior to start of each work shift.

Transverse joints: Use a butt joint. Use building paper placed over previously placed slurry seal or other suitable method to avoid double placement of slurry seal. Ridges or bumps in the finished surface are not permitted.

Longitudinal joints: Place longitudinal joints on lane lines. Half passes and odd-width passes can be used only in turnouts and parking areas. When half passes are used, they shall not be the last pass of any paved area. Overlap longitudinal joints no more than 3 inches. Keep elevation difference at joints less than ¼ inch.

Roll parking areas and turnouts with a self-propelled, 10-ton pneumatic roller with a tire pressure of 50 psi and equipped with a water spray system. Subject surfaced areas to a minimum of 2 full-coverage passes with the roller. Do not commence rolling until micro-surfacing has cured to the point where it will not pick up on the tires of the roller.

No streaks or transverse ripples as defined by ISSA Performance Guidelines A 143 are allowed in the finished surface. Ensure straight lines along curb and shoulders. No runoff on these areas is permitted. Mask off surface areas at the project start, end, and as directed by the CO to provide straight and neat starting and ending joints.

Clean up all material spills; remove from the park, and dispose of in accordance with all local, state, and federal regulations. On a daily bases remove all debris associated with the performance of the work from the park, and dispose of in accordance with all local, state, and federal regulations.

410.09 Acceptance. See Table 410-1 for sampling and testing requirements.

Polymer modified emulsified asphalt will be evaluated under Subsections 106.03 and 702.09.

Aggregate for surfacing mixture will be evaluated under Subsections 106.02 and 106.04.

Construction of surfacing will be evaluated under Subsections 106.02 and 106.04.

Construction of asphalt surface treatment course will be evaluated under Subsections 106.02 and 106.04.

Measurement

410.10 Measure the Section 410 items listed in the bid schedule according to Subsection 109.02 for each day's production.

Payment

409.11 The accepted quantities will be paid at the contract price per unit of measurement for the Section 410 pay items listed in the bid schedule. Payment will be full compensation for the work prescribed in this Section. See Subsection 109.05.

Table 410-1
Sampling, Testing and Acceptance Requirements

Material or Property	Type of Acceptance (subsection)	Characteristic	Category	Test Methods Specifications	Sampling Frequency	Point of Sampling	Split Sample	Reporting Time
Aggregates for surfacing mixture (703.11)	Measured and tested for conformance (106.04)	Gradation	---	AASHTO T 27 & T 11	1 per 500 tons	Stockpile	Yes, when requested	Before using in work
		LA abrasion	---	AASHTO T 96	1 per aggregate	Aggregate source		"
		Soundness	---	AASHTO T 104	"	"		"
		Sand equivalent	---	AASHTO T 176, alternate method no.2, reference method	"	Stockpile		"

B.3 Section 702. - ASPHALT MATERIAL – Utah Parks

Section 702. - ASPHALT MATERIAL

702.03 Emulsified Asphalt. Add the following:

702.03(d) Polymer modified emulsions. Delete the title and text of this subsection and substitute the following:

(d) Polymer modified emulsions. Mill or blend the polymer material into the asphalt or emulsifier solution prior to or during the emulsification process. AASHTO T 59 will be followed for all test methods, except as noted.

For chip seal application, use a polymer cationic rapid set emulsified asphalt (CRS-2P) conforming to Table 702-4. Latex latex modified cationic rapid setting emulsified asphalt (LMCRS-2) conforming to Table 702-6 may be used in lieu of CRS-2P for this project.

Table 702-4
Chip Seal Emulsion Specification

Emulsion Grade	**CRS-2P** [1]	
Tests on emulsion:	**Minimum**	**Maximum**
Viscosity, Saybolt Furol at 140°F, Sfs	100	400
Settlement, 5 days, %	---	5.0
Storage stability test, 24-hour, % [2]	---	1.0
Sieve test, %	---	0.10
Particle charge test	Positive	
Demulsibility ,%	40	---
Residue by distillation, % [3]	68	---
Tests on residue from distillation test:		
Penetration, 77°F, 100 g, 5 sec	80	150
Ductility, 77°F, 5 cm/min, cm	125	---

[1] CRS-2P will be an emulsion blend of polymerized asphalt, water, and emulsifiers. The asphalt cement will be polymerized prior to emulsification and will contain a minimum of 2½ percent polymer by weight of asphalt cement.

[2] This test requirement on representative samples is waved if successful application of the material has been achieved

[3] The standard distillation procedure will be modified as follows: The temperature on the lower thermometer will be brought slowly to 400 ±8°F and maintained at this point for 20 minutes. Complete the total distillation in 60 ±15 minutes from the first application of heat.

For micro-surfacing application use a quick-set polymer modified asphalt emulsion conforming to AASHTO M 208 CSS-1h; delete the cement mixing test requirements. The polymer modifier to be added at a minimum of 2.5 percent solids based on the residual asphalt content. The asphalt/polymer emulsion must parallel the standard from an established infrared spectrum characterizing the asphalt/polymer emulsion. The modified emulsion residue must meet Table 702-5.

Table 702-5
Modified Emulsion Residue

Emulsion Grade	CSS-1h quick set	
Tests on emulsion:	**Description**	**Specification**
AASTHO T 49	Penetration @ 77°F	40-90
AASHTO T 53	Softening Point	135°F minimum
AASHTO T 59-modified (a)	Residue by Distillation	62% minimum
AASHTO 316	Rotational Viscosity 275°F	650 CPS minimum
(a) Modified distillation procedure: Heat emulsion residue to 270 ± 10 degrees F and maintain that temperature for 20 minutes. Perform the distillation within 60 ± 15 minutes.		

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Table 702-6
Latex Modified Cationic Rapid Setting Emulsified Asphalt (LMCRS-2)

Tests	AASHTO Test Method	Min.	Max.
Emulsion			
Viscosity, SF, 122ºF (50ºC), s (Project Site Acceptance/Rejection Limits)	T59	140	400
Settlement (a) 5 days, percent	T 59		5
Storage Stability Test (b) 1 d, 24 h, percent	T 59		1
Demulsibility (c) 35 ml, 0.8% sodium dioctyl Sulfosucinate, percent	T 59		40
Particle Charge Test	T 59		Positive
Sieve Test, percent	T 59		0.3
Distillation			
Oil distillate, by volume of emulsion, percent			0
Residue (d), percent			65
Residue from Distillation Test			
Penetration, 77ºF (25ºC), 100 g, 5 s, dmm	T 49	40	200
Torsional Recovery (e)			18
(a) The test requirement for settlement may be waived when the emulsified asphalt is used in less than a five-day time; or the purchaser may require that the settlement test be run from the time the sample is received until it is used, if the elapsed time is less than 5 days. (b) May use the 24-hour (1-day) storage stability test instead of the five-day settlement test. (c) Make the demulsibility test within 30 days from date of shipment. (d) Determine distillation by AASHTO T 59, with modifications to include a 350 ± 5ºF (177±3ºC) maximum temperature to be held for 15 minutes. (e) CA 332 (California Test Method) Co-mill latex and asphalt during emulsification			

Section 703. – AGGREGATE

703.05 Subbase, Base, and Surface Course Aggregate.

(a) General. Delete lines (3), (4), (5), and (6) and substitute the following:

(3) Fractured faces, one or more, ASTM D 5821	50% min.
(4) Free from organic matter and lumps or balls of clay	

(b) Subgrade or Base aggregate.

Table 703-2 Target Value Ranges for Subbase and Base Gradation. Delete reference to the "436-74(6)" percent by mass passing the 4.75 millimeter sieve for grading E (base) and substitute "36-74 (6)".

(c) Surface Course Aggregate. Delete the text including Table 703-3 and substitute the following:

Furnish hard, durable particles or fragments of crushed stone, crushed slag, or crushed gravel conforming to the following:

(1) Los Angeles abrasion, AASHTO T 96	50% max.
(2) Fractured faces, one or more, ASTM D 5821	50% min.
(3) Free from organic matter and lumps or balls of clay	
(4) Liquid Limit, AASHTO T 89	35 max.
(5) Dust ratio: $\frac{\text{\% passing \#200}}{\text{\% passing \#40}}$	2/3 max.
(6) Gradation and plasticity index, AASHTO T 90	Table 703-3

Do not use material that breaks up when alternately frozen and thawed or wetted and dried.

Obtain the aggregate gradation by crushing, screening, and blending processes as necessary. Fine aggregate, material passing the No. 4 sieve, will consist of natural or crushed sand and fine mineral particles.

Do not furnish material that contains asbestos fibers.

Table 703-3
Target Value Ranges for Surface Course Gradation and Plasticity Index

Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27 and T 11)
¾ inch	100 [(1)]
No. 4	41-71 (7)
No. 40	*(5)
No. 200	9-16 (4)
Plasticity Index (PI)	8 (4)

[(1)] Statistical procedures do not apply.
(*) Submit target values for applicable sieves
( ) Allowable deviations (+/-) from the target values

703.06 Crushed Aggregate. Add the following to the end of the paragraph:

When aggregate is used as a surface course, furnish an aggregate with a Plasticity Index conforming to Table 703-3a.

Table 703-3a
Surface Course Gradation and Plasticity Index

Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27 and T 11)
¾ inch	100
No. 4	41-71
No. 40	*
No. 200	5-20
Plasticity Index (PI)	4-12
Plasticity Index (PI)	4-12

(*) Submit target values for applicable sieves

703.10 Asphalt Surface Treatment Aggregate.

Delete lines (d), (e), (f), (g), and (h) and substitute the following:

(d) Fractured faces, one or more, ASTM D 5821 — 90% min.

(e) Flat and elongated particles, 1:3 ratio. +⅜ inch sieve, calculated by mass, weighted average, ASTM D4791 — 10% max.

(f) Clay lumps and friable particles, AASHTO T 112 — 1.0% max.

(g) Adherent coating, ASTM D 5711 — 0.5% max.

Table 703-7. Delete Table 703-7 and substitute the following:

Table 703-7
Target Value Ranges for
Single and Multiple Course Surface Treatment Aggregate Gradation

Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27 & T 11)					
	Grading Designation					
	A	B	C	D	E	F
1½ inch	100 [1]					
1 inch	90 – 100 (3)	100 [1]				
¾ inch	0 – 35 (5)	90 – 100 (3)	100 [1]			
½ inch	0 – 8 (3)	0 – 35 (5)	90 – 100 (3)	100 [1]		
⅜ inch	---	0 – 12 (3)	0 – 35 (5)	70-90 (3)	100 [1]	100 [1]
No. 4	---		0 – 12 (3)	0-10 (5)	85 – 100 (3)	85 – 100 [1]
No. 8	---			0-5 (3)	0 – 23 (4)	---
No. 200	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 - 10 [1]

[1] Statistical procedures do not apply.
( ) The value in parentheses is the allowable deviation (±) from the target value.

703.11 Micro-Surfacing Aggregate.

Table 703-8. Delete Table 703-8 and substitute the following:

Table 703-8
Micro-Surfacing Aggregate
Gradation and Tolerance

SIEVE SIZE		TYPE II PERCENT PASSING	TYPE III PERCENT PASSING	STOCKPILE TOLERANCE
3/8	(9.5 mm)	100	100	
# 4	(4.75 mm)	90 - 100	70 – 90	± 5%
# 8	(2.36 mm)	65 - 90	45 – 70	± 5%
# 16	(1.18 mm)	45 – 70	28 – 50	± 5%
# 30	(600 um)	30 – 50	19 – 34	± 5%
# 50	(330 um)	18 - 30	12 – 25	± 4%
#100	(150 um)	10 - 21	7 – 18	± 3%
#200	(75 um)	5 - 15	5 – 15	± 2%

B.4 Section 702. – ASPHALT MATERIAL - Crater Lake National Park

Section 702. - ASPHALT MATERIAL

702.03 Emulsified Asphalt. Add the following:

702.03(d) Polymer modified emulsions. Delete the title and text of this subsection and substitute the following:

(d) Polymer modified emulsions. Mill or blend the polymer material into the asphalt or emulsifier solution prior to or during the emulsification process. AASHTO T 59 will be followed for all test methods, except as noted.

For chip seal application use a polymer cationic rapid set emulsified asphalt conforming to Table 702-4.

Table 702-4
Chip Seal Emulsion Specification

Emulsion Grade	**CRS-2P / HFRS-P2** (1)	
Tests on emulsion:	**Minimum**	**Maximum**
Viscosity, Saybolt Furol at 140°F, Sfs	100	400
Settlement, 5 days, %	---	5.0
Storage stability test, 24-hour, % (2)	---	1.0
Sieve test, %	---	0.10
Particle charge test	Positive	
Demulsibility ,%	40	---
Residue by distillation, % (3)	65	---
Tests on residue from distillation test:		
Penetration, 77°F, 100 g, 5 sec	90	200
Ductility, 77°F, 5 cm/min, cm	125	---

(1) CRS-2P or HFRS-P2 will be an emulsion blend of polymerized asphalt, water, and emulsifiers. The asphalt cement will be polymerized prior to emulsification and will contain a minimum of 2½ percent polymer by weight of asphalt cement.

(2) This test requirement on representative samples is waved if successful application of the material has been achieved

(3) The standard distillation procedure will be modified as follows: The temperature on the lower thermometer will be brought slowly to 400 ±8°F and maintained at this point for 20 minutes. Complete the total distillation in 60 ±15 minutes from the first application of heat.

For micro-surfacing application use a quick-set polymer modified asphalt emulsion conforming to AASHTO M 208 CSS-1h; delete the cement mixing test requirements. The polymer modifier to be added at a minimum of 2.5 percent solids based on the residual asphalt content. The asphalt/polymer emulsion must parallel the standard from an established infrared spectrum characterizing the asphalt/polymer emulsion. The modified emulsion residue must meet Table 702-5.

Table 702-5
Modified Emulsion Residue

Emulsion Grade	**CSS-1h quick set**	
Tests on emulsion:	**Description**	**Specification**
AASTHO T 49	Penetration @ 77°F	40-90
AASHTO T 53	Softening Point	135°F minimum
AASHTO T 59-modified (a)	Residue by Distillation	62% minimum
AASHTO 316	Rotational Viscosity 275°F	650 CPS minimum
(a) Modified distillation procedure: Heat emulsion residue to 270 ± 10 degrees F and maintain that temperature for 20 minutes. Perform the distillation within 60 ± 15 minutes.		

Section 703. – AGGREGATE

703.10 Asphalt Surface Treatment Aggregate.

Delete lines (d), (e), (f), (g), and (h) and substitute the following:

(d) Fractured faces, one or more, ASTM D 5821 90% min.

(e) Flat and elongated particles, 1:3 ratio. +⅜ inch sieve, calculated by mass, weighted average, ASTM D4791 10% max.

(f) Clay lumps and friable particles, AASHTO T 112 1.0% max.

(g) Adherent coating, ASTM D 5711 0.5% max.

Table 703-7. Delete Table 703-7 and substitute the following:

Table 703-7
Target Value Ranges for Single and Multiple Course Surface Treatment Aggregate Gradation

Sieve Size	**Percent by Mass Passing Designated Sieve (AASHTO T 27 & T 11)**					
	Grading Designation					
	A	**B**	**C**	**D**	**E**	**F**
1½ inch	100 (1)					
1 inch	90 – 100 (3)	100 (1)				
¾ inch	0 – 35 (5)	90 – 100 (3)	100 (1)			
½ inch	0 – 8 (3)	0 – 35 (5)	90 – 100 (3)	100 (1)		
⅜ inch	---	0 – 12 (3)	0 – 35 (5)	85-100 (3)	100 (1)	100 (1)
¼ inch	---			0 – 15 (3)		
No. 4	---		0 – 12 (3)		85 – 100 (3)	85 – 100 (1)
No. 30	---			0-2	0 – 23 (4)	---
No. 200	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 - 10 (1)

(1) Statistical procedures do not apply.
( ) The value in parentheses is the allowable deviation (±) from the target value.

B.5. Section 702. - ASPHALT MATERIAL – Death Valley National Park

Section 702. - ASPHALT MATERIAL

702.03 Emulsified Asphalt. Add the following:

702.03(d) Polymer modified emulsions. Delete the title and text of this subsection and substitute the following:

(d) Polymer modified emulsions. Mill or blend the polymer material into the asphalt or emulsifier solution prior to or during the emulsification process. AASHTO T 59 will be followed for all test methods, except as noted.

For chip seal application use a polymer cationic rapid set emulsified asphalt conforming to Table 702-4.

Table 702-4
Chip Seal Emulsion Specification

Emulsion Grade	**CRS-2P (1)**	
Tests on emulsion:	**Minimum**	**Maximum**
Viscosity, Saybolt Furol at 140°F, Sfs	75	300
Settlement, 5 days, %	---	5.0
Storage stability test, 24-hour, % (2)	---	1.0
Sieve test, %	---	0.10
Particle charge test	Positive	
Demulsibility ,%	60	95
Residue by distillation, % (3)	65	---
Tests on residue from distillation test:		
Penetration, 77°F, 100 g, 5 sec	40	90
Ductility, 77°F, 5 cm/min, cm	125	---

(1) CRS-2P will be an emulsion blend of polymerized asphalt, water, and emulsifiers. The asphalt cement will be polymerized prior to emulsification and will contain a minimum of 2½ percent polymer by weight of asphalt cement.

(2) This test requirement on representative samples is waved if successful application of the material has been achieved

(3) The standard distillation procedure will be modified as follows: The temperature on the lower thermometer will be brought slowly to 400 ±8°F and maintained at this point for 20 minutes. Complete the total distillation in 60 ±15 minutes from the first application of heat.

703.10 Asphalt Surface Treatment Aggregate.

Delete lines (d), (e), (f), (g), and (h) and substitute the following:

(d) Fractured faces, one or more, ASTM D 5821 — 90% min.

(e) Flat and elongated particles, 1:3 ratio. +⅜ inch sieve, calculated by mass, weighted average, ASTM D4791 — 10% max.

(f) Clay lumps and friable particles, AASHTO T 112 — 1.0% max.

(g) Adherent coating, ASTM D 5711 — 0.5% max.

Table 703-7. Delete Table 703-7 and substitute the following:

Table 703-7
Target Value Ranges for Single and Multiple Course Surface Treatment Aggregate Gradation

Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27 & T 11)					
	Grading Designation					
	A	**B**	**C**	**D**	**E**	**F**
1½ inch	100[1]					
1 inch	90 – 100 (3)	100[1]				
¾ inch	0 – 35 (5)	90 – 100 (3)	100[1]			
½ inch	0 – 8 (3)	0 – 35 (5)	90 – 100 (3)	100[1]		
⅜ inch	---	0 – 12 (3)	0 – 35 (5)	70-90 (3)	100[1]	100[1]
No. 4	---		0 – 12 (3)	0-10 (5)	85 – 100 (3)	85 – 100[1]
No. 8	---			0-5 (3)	0 – 23 (4)	---
No. 200	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 - 10[1]

[1] Statistical procedures do not apply.
( ) The value in parentheses is the allowable deviation (±) from the target value.

B.6 Section 702. - ASPHALT MATERIAL – Dinosaur National Monument

702.03 Emulsified Asphalt. Add the following:

702.03(d) Polymer modified emulsions. Delete the title and text of this subsection and substitute the following:

(d) Polymer modified emulsions. Mill or blend the polymer material into the asphalt or emulsifier solution prior to or during the emulsification process. AASHTO T 59 will be followed for all test methods, except as noted.

For chip seal application, use a polymer cationic rapid set emulsified asphalt (CRS-2P) conforming to Table 702-4. Latex latex modified cationic rapid setting emulsified asphalt (LMCRS-2) conforming to Table 702-6 may be used in lieu of CRS-2P for this project.

Table 702-4
Chip Seal Emulsion Specification

Emulsion Grade	**CRS-2P** [1]	
Tests on emulsion:	**Minimum**	**Maximum**
Viscosity, Saybolt Furol at 77°F, Sfs	50	350
Settlement, 5 days, %	---	5.0
Storage stability test, 24-hour, % [2]	---	1.0
Sieve test, %	---	0.10
Particle charge test	Positive	
Demulsibility ,%	40	---
Residue by distillation, % [3]	65	---
Tests on residue from distillation test:		
Penetration, 39.2°F, 100 g, 5 sec	40	---
Ductility, 77°F, 5 cm/min, cm	125	---

[1] CRS-2P will be an emulsion blend of polymerized asphalt, water, and emulsifiers. The asphalt cement will be polymerized prior to emulsification and will contain a minimum of 2½ percent polymer by weight of asphalt cement.

[2] This test requirement on representative samples is waved if successful application of the material has been achieved

[3] The standard distillation procedure will be modified as follows: The temperature on the lower thermometer will be brought slowly to 400 ±8°F and maintained at this point for 20 minutes. Complete the total distillation in 60 ±15 minutes from the first application of heat.

Section 703. – AGGREGATE

703.06 Crushed Aggregate. Add the following to the end of the paragraph:

When aggregate is used as a surface course, furnish an aggregate with a Plasticity Index conforming to Table 703-3a.

Table 703-3a
Surface Course Gradation and Plasticity Index

Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27 and T 11)
¾ inch	100
No. 4	41-71
No. 40	*
No. 200	5-20
Plasticity Index (PI)	4-12

(*) Submit target values for applicable sieves

703.10 Asphalt Surface Treatment Aggregate.

Delete lines (d), (e), (f), (g), and (h) and substitute the following:

(d) Fractured faces, one or more, ASTM D 5821 — 90% min.

(e) Flat and elongated particles, 1:3 ratio. +⅜ inch sieve, calculated by mass, weighted average, ASTM D4791 — 10% max.

(f) Clay lumps and friable particles, AASHTO T 112 — 1.0% max.

(g) Adherent coating, ASTM D 5711 — 0.5% max.

Table 703-7. Delete Table 703-7 and substitute the following:

Table 703-7
Target Value Ranges for Single and Multiple Course Surface Treatment Aggregate Gradation

Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27 & T 11)					
	Grading Designation					
	A	B	C	D	E	F
1½ inch	100[1]					
1 inch	90 – 100 (3)	100[1]				
¾ inch	0 – 35 (5)	90 – 100 (3)	100[1]			
½ inch	0 – 8 (3)	0 – 35 (5)	90 – 100 (3)	100[1]		
⅜ inch	---	0 – 12 (3)	0 – 35 (5)	70-90 (3)	100 (	1) 100[1]
No. 4	---		0 – 12 (3)	0-10 (5)	85 – 100 (3)	85 – 100[1]
No. 8	---			0-5 (3)	0 – 23 (4)	---
No. 200	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 – ½ (½)	0 - 10[1]

[1] Statistical procedures do not apply.
( ) The value in parentheses is the allowable deviation (±) from the target value.

ACKNOWLEDGEMENTS
This work is being accomplished with the help of many entities. The authors would especially like to thank the support, guidance and patience from the staff of the Central Federal Lands Highway division and the Federal Highway Administration's Office of Construction and System Preservation who greatly encouraged and facilitated the information sharing among researchers, suppliers and specifiers. We also thank the many, many material suppliers, agency users, academic and State Highway researchers and independent testing labs who gave their support, expertise, testing and materials (much of which was donated), to further the goals of this project. Members of the Binder Expert Task Group, the newly formed Emulsion Task Force, various Transportation Research Board committees, and the Asphalt Emulsion Manufacturers Association International Technical Committee have also reviewed proposed methods and shared their knowledge. The authors are excited about the prospects for the future as many researchers and field practitioners are now sharing information and working together to develop polymer modified emulsion specifications that will provide higher quality, longer lasting and lower cost surfaces for Public Lands and highway users.

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FLH Field Study

Defining a path forward for emulsion performance specs

FHWA

FLH Polymer Modified Emulsion (PME) Study

Acknowledgements

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    - Industry volunteers
      - PRI, BASF, Paragon, SemMaterials
    - Collaboration
      - Colorado State, TX A&M, UW, NC State, WRI

FHWA

The Project

- Literature review & knowledge gathering sessions
  - Industry, academic, federal & local government
  - On-line user/ producer survey
  - Presentations & input: AEMA/ARRA/ISSA, TRB, ETGs, AASHTO
    - Led to creation of the Emulsion Task Force
- Draft performance spec
- Laboratory and/or Field trials
- Field guide – Guidelines for Use of PME Emulsions in Surface Treatments

FHWA

Sample Proposed Performance Tests

Purpose	Test	Conditions	Report
Residue Recovery	Forced Draft Oven	24 hrs @ambient + 24 hrs @60°C	✓% Residue
Tests on Residue from Forced Draft Oven			
High Temperature (Rutting/Bleeding)	DSR-MSCR DSR freq sweep	T_h T_h	✓J_{nr} ✓G* & phase angle
Polymer Identifier (Elasticity/Durability)	DSR-MSCR	T_h @3200 Pa	✓% Recoverable Strain
High Float Identifier (Bleeding)	DSR – non-linearity	T_h	✓Test to be developed
Tests on PAV after Forced Draft Oven Residue			
Low Temperature (Aged Brittleness)	DSR freq sweep	10 & 20° C Model low T	✓G* ✓Phase Angle
Polymer Degradation (Before/After PAV)	DSR-MSCR	T_h @3200 Pa	✓Recoverable Strain Ratio

T_h = high pavement temp; DSR = dynamic shear rheometer
MSCR = multiple stress creep recovery

Emulsion & residue tests

- ✓ Conventional Properties (ASTM D 244)
- ✓ Low Temperature Recovery of Asphalt Emulsion Residue
  - > Force Draft Oven – 24 hours @ ambient & 24 hours @ 60°C
- ✓ Use of Dynamic Shear Rheometer
  - >Residue Properties – Frequency Sweep, MSCR
    - Freq. Sweep & MSCR (58°C, 64°C & 70°C)
  - >PAV Residue Properties – Frequency Sweep, Strain Sweep, MSCR
    - Freq. Sweep & MSCR (58°C, 64°C & 70°C)
    - Low Temp Freq. Sweep (10°C & 20°C)
    - Strain Sweep (25°C)

Develop specifications using the Dynamic Shear Rheometer

FHWA

TESTING COST

- ✓ Field Study
  - Emulsion/residue tests with extensive DSR evaluation
  - Sweep Test for cure time of chip seals
  - ISSA performance tests for microsurfacing
  - Cost: approximately $4000-5000/sample
- ✓ Specification compliance
  - Target cost: $1000 per sample

FHWA

FLH – NCPP PME Study.ppt
May, 2009 GHK, Inc.

Utah Parks - Construction

- ✓ **90 miles total 9/6/08 – 10/17/08**
  - Arches & Canyonlands Nat'l Parks,
  - Natural Bridges & Hovenweep Nat'l Monuments
- ✓ **Chip Seal – 1,140,000 sy (fogged)**
  - SBR latex modified CRS-2LM
- ✓ **Microsurfacing - 60,000 sy**
  - Natural latex modified Ralumac®



Utah Parks – Testing Plan

- ✓ **PRI:** Testing both chip & micro emulsion & aggregates
- ✓ **Paragon:** chip emulsion & aggregates
- ✓ **BASF:** chip emulsion & aggregates
- ✓ **SemMaterials:** micro emulsion
- ✓ **NCHRP study (Shuler):** chip emulsion & aggregates
- ✓ **CFLHD Lab:** acceptance testing only


Death Valley National Park

- ✓ **13 miles – 11/11/08 – 11/14/08**
- ✓ **Chip seal – 161,400 sy**
  - SBR latex modified CRS-LM
- ✓ **Test plan:**
  - PRI: emulsion & aggregates
  - Paragon: emulsion & aggregates
  - BASF: emulsion & aggregates
  - CFLHD Lab: acceptance testing only

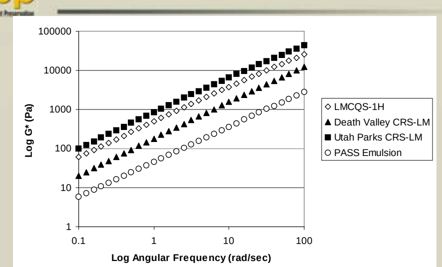
Dinosaur National Monument

- ✓ **11.4 miles – 9/23/08 – 9/30/08**
- ✓ **Chip seal – 135,000 sy**
  - Neoprene modified PASS®
- ✓ **Test plan:**
  - PRI: emulsion & aggregates
  - CFLHD Lab: acceptance testing only


Crater Lake National Park

- ✓ **23 miles chip seal**
  - Planned for late spring 2009
  - 367,000 sy
- ✓ **Hope: SBS modified CRS-2P**
- ✓ **Testing to be determined**

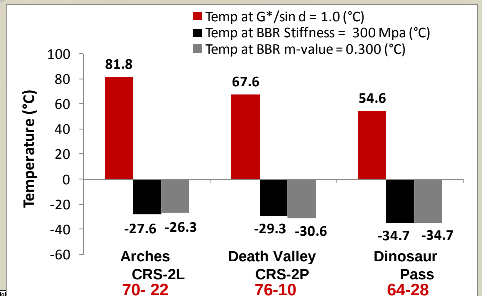
Frequency Sweep - Four Residues



G* is a function of the base binder and level of polymer modification

FLH – NCPP PME Study.ppt
May, 2009 GHK, Inc.

Chip Seal Emulsion Residue
Temperature Grading (SHRP Parameters)



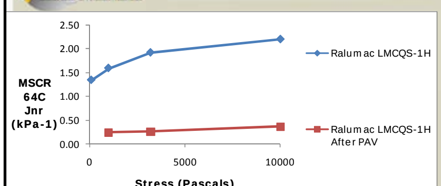
MSCR

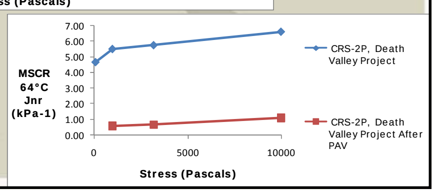
PAV Aging

Forced Draft Oven cure in PAV pan

Standard PAV (20 hr, 100C)

Effect of Aging on 64°C J_{nr} for Ralumac & CRS-2P

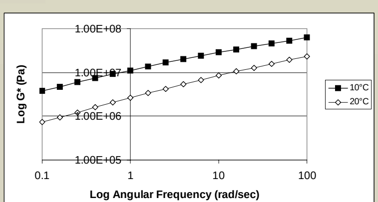




Low temperature grading

Predict low temperature physical properties using intermediate temperature frequency sweeps and the CAM model for asphalt rheology.

Low Temperature Frequency Sweep - PAV Residue

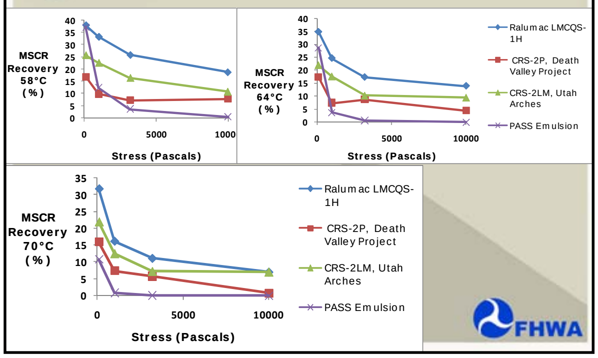


Correlate to BBR m-value and Creep Stiffness

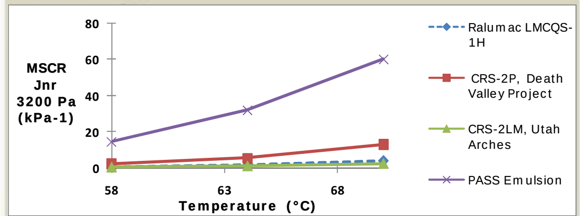
MSCR

Influence of Experimental Conditions

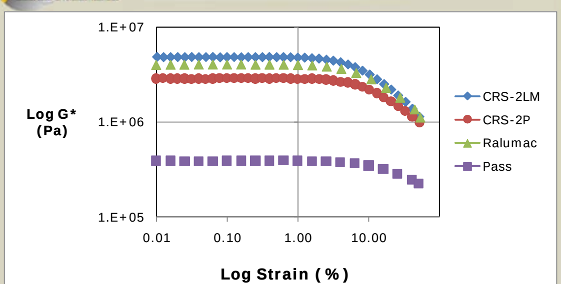
MSCR - % Recovery vs. Stress



MSCR
Effect of Temperature on J_{nr} @ 3200 Pa



Strain Sweeps on PME Residues



PME Project Status

- ✓ **Preliminary report under review**
  - Final report after results of 2009 project
  - Will be posted on NCPP website
- ✓ **Field Guide written, published soon**
- ✓ **Project Database available at www.pavementpreservation.org**

PME Project Status

- ✓ **Envisioned next steps:**
  - May 14-15, 2009: **ETG/ETF Meeting**
  - August 3-7, 2009: **AASHTO SOM** – Study results discussed with emulsion subsection
  - September, 2009: **Testing completed**
  - October, 2009: **Report finalized**
  - November, 2009: Begin **study to develop specification** for AASHTO provisional adoption
  - August, 2010: **Provisional specification** presented to AASHTO SOM for adoption
  - 2009, 2010, 2012, 2014, 2016: **ongoing performance monitoring** of 4 project sites

Recommendations for Further Study

- ✓ **Continue development work on performance specs for emulsions**
- ✓ **Include testing of unmodified emulsion**
- ✓ **Continue knowledge sharing of related projects**
  - Coordinated by Emulsion Task Force (Pavement Preservation Expert Task Group)
- ✓ **Use lessons-learned and identified data gaps to refine research needs as proposed in the PP roadmap**

Next Steps:

- ✓ **Gain support from AASHTO & Industry**
  - AASHTO agency field trials
    - Identify AASHTO lead states (one per region)
  - Create agency-industry task groups
    - User-Producer Groups
    - Pavement Preservation Partnerships
  - Planned presentations
    - PPP meetings
    - AEMA/ARRA/ISSA
    - ASTM, TRB, International PP symposium

ETF Action Item: Residue Recovery Method

- ✓ **Write provisional AASHTO standard for residue recovery in Forced Draft Oven. Kadrmas/Hazlett**
  - 48 hour method per ASTM proposal
  - 6 hour thin-film accelerated method
- ✓ **Develop Research Needs Statement to improve reproducibility of FDO recovery method(s), and to provide needed statistics for test reliability. Lewandowski/Epps**

ETF Action Items: Initiate Pooled Fund Study

- ✓ **Present findings to AASHTO Materials**
- ✓ **Develop a plan and solicit support from 10-20 agencies to join a pooled fund study to continue emulsion field trials. NCPP/CFLH**
- ✓ **Purpose:**
  - Collect data to link test procedures to performance
  - Identify material failure properties so specification limits can be set.

2010 NCHRP Program

4 Projects related to Preservation

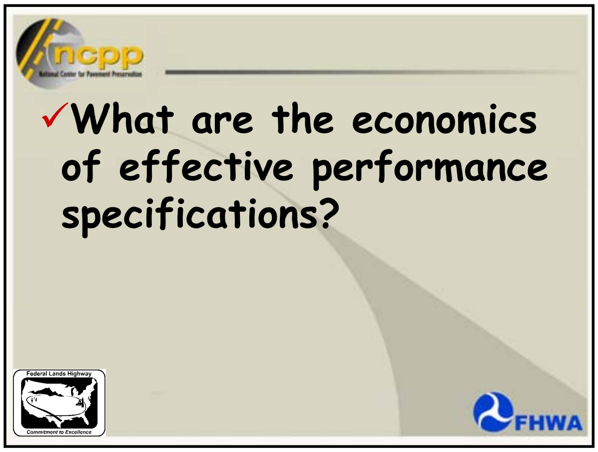
- ✓ **Chip Seal Performance Specs**
  - Knowledgeable Panel Members Needed

Related Projects

- ✓ ASTM – Committee D 4.42,
  - Low temperature recovery of emulsion residue & emulsion viscosity.
- ✓ Manual for Emulsion-Based Chip Seals for Pavement Preservation (NCHRP 14-17)
  - Scott Shuler, Colorado State University, and Amy Epps Martin, Texas A&M University.
- ✓ Emulsion Cold Mix (Asphalt Research Consortium)
  - Husain Bahia, University of Wisconsin, and Peter Sebaaly, University of Nevada at Reno.
- ✓ "Chip Seal Design and Performance" North Carolina DOT Project HWY 2004-04
  - Richard Kim, North Carolina State University.
- ✓ "Using DSR and Rheological Modeling to Characterize Binders at Low Temp"
  - Fred Turner and Mike Harnsberger, Western Research Institute.
- ✓ "Slurry/Micro-Surface Mix Design Procedure" Caltrans Contract 65A0151
  - Jim Moulthrop, Fugro, and Gary Hicks.

On-going research needed

- ✓ **Update D-244, including field viscosity test**
- ✓ **Refine recovery method and provide statistics for AASHTO/ASTM spec needs**
- ✓ **Refine DSR rheological tests**
  - Polymer and High Float identification
  - Low temperature specifications
- ✓ **Define failure parameters and tie to physical properties**
- ✓ **Develop SPG Grading System with spec limits**


ncpp
✓What are the economics of effective performance specifications?
FHWA


ncpp
Thank You.
FHWA

Attachment 14

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: **INFORMATION:** Presentation of "A Quick Check of Your Highway Network Health"

Date: APR 10 2009

From: Butch Wlaschin
Director, Office of Asset Management

In Reply Refer To:
HIAM-20

To: Directors of Field Services
Federal Lands Highway Division Engineers
Director of Technical Services
Division Administrators

We are facing a daunting challenge to determine the best ways to maintain and preserve the highway investments to date. With numerous issues placing more demands on transportation department resources, it is ever more critical to have a focus on asset management principles and the use of this enhanced business decision process in your State's decisionmaking process. Balancing the available funds to meet the States network needs is a challenge we must all work towards.

The Office of Asset Management is dedicated to providing the division offices with expertise and examples to assist you and you're State in managing their highway assets. Over the past several years we have offered technical support, case studies and various tools to assist the States and divisions with this task. One area of applied asset management is the States use of effective pavement preservation treatments to extend the service life of reasonably sound pavements. We consider Transportation System Preservation (TSP) a cornerstone for a sound Asset Management approach. TSP focuses on "Keeping good roads in good condition" instead of deferring maintenance until a more expensive rehabilitation or reconstruction is required. Managing pavement assets is a critical consideration for any state or local agency. Unfortunately, we have found many agencies focus on historical funding distribution or on addressing their "worst roads first." During the past three years we have visited many of the State DOT's and our division offices.

During these field evaluations it became evident we needed a mechanism to demonstrate the benefit of re-evaluating how we manage our highway network and what can be done to better protect our highway pavement investments. We developed a short, 8-page brochure to explain the concept of Remaining Service Life and to demonstrate the effectiveness of applying Asset Management principles to State funding decisions. The brochure, "A Quick Check of Your Highway Network Health" was published by FHWA Office of Asset Management and the National Center for Pavement Preservation in 2007 and has been widely distributed and used in the pavement preservation evaluations conducted to date. State response and acceptance of this tool has been very positive.

For your continued use, we have developed a PowerPoint to support this brochure. This is a very simple exercise that will help an agency determine if their pavement network condition is getting better or worse based on the projects that are currently programmed for any given year. We have found that the minimal data and network information is readily available in DOT and local agencies. The data used and the results calculated are presented in a way that is easily understood by everyone. This PowerPoint and notes will readily assist you or your staff in discussing the merits of pavement preservation with highway administrators and managers. We recommend that this exercise be considered for key decisionmakers in the central office and the districts. States may want to similarly use this tool to demonstrate funding needs to the public and elected officials. The same concept can be applied to other highway assets such as bridges, roadway safety and signing assets and the like.

A copy of this memorandum, the PowerPoint file and an Excel worksheet are being transmitted along with two copies of the pamphlet, "A Quick Check on Your Highway Network Health" and worksheet in the official mail. Additional copies are available at your request.

We encourage each of you to share this with your State and local counterparts and work with them to maintain and preserve their pavement assets. If you have any questions, please contact Joe Gregory or Chris Newman. If your State wishes additional technical support in this area, they can also request similar support from the AASHTO TSP·2 Program at 517-432-8220 or by visiting www.tsp2.org.

Attachments

"Remaining Service Life"(RSL) is the tool we need to apply. RSL generally uses data already being collected though the agency's pavement management system (PMS). Construction and rehabilitation costs and performance can generally be pulled from existing databases. Maintenance and preservation data can be estimated until the agency gains actual experience with preservation treatments and integrates maintenance and preservation costs into their PMS.

For more information, please contact the National Center for Pavement Preservation.
www.pavementpreservation.org
(517) 432-8220

If you would like to view the electronic version of this exercise along with the worksheet, please visit the FHWA System Preservation Web site at http://www.fhwa.dot.gov/preservation/library.cfm.

Publication No. FHWA-IF-07-006

U.S. Department of Transportation
Federal Highway Administration

ncpp
National Center for Pavement Preservation

A Quick Check of Your Highway Network Health

by Larry Galehouse, Director,
National Center for Pavement Preservation
and
Jim Sorenson, Team Leader,
FHWA Office of Asset Management

Historically, many highway agency managers and administrators have tended to view their highway systems as simply a collection of projects. By viewing the network in this manner, there is a certain comfort derived from the ability to match pavement actions with their physical/functional needs. However, by only focusing on projects, opportunities for strategically managing entire road networks and asset needs are overlooked. Although the "bottom up" approach is analytically possible, managing networks this way can be a daunting prospect. Instead, road agency administrators have tackled the network problem from the "top down" by allocating budgets and resources based on historic estimates of need. Implicit in this approach is a belief that the allocated resources will be wisely used and will prove adequate to achieve desirable network service levels.

By using a quick checkup tool, road agency managers and administrators can assess the needs of their network and other highway assets and determine the adequacy of their resource allocation effort. A quick checkup is readily available and can be usefully applied with minimum calculations.

It is essential to know whether present and planned program actions (reconstruction, rehabilitation, and preservation) will produce a net improvement in the condition of the network. However, before the effects of any planned actions to the highway network can be analyzed, some basic concepts should be considered.

Assume that every lane-mile segment of road in the network was rated by the number of years remaining until the end of life (terminal condition). Remember that terminal condition does not mean a failed road; rather, it is the level of deterioration that management has set as a minimum operating condition for that road or network. Consider the rated result of the current network condition, shown in Figure 1.

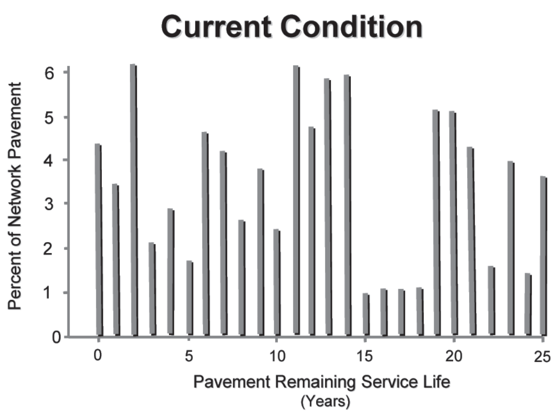


Figure 1. Current condition.

If no improvements are made for 1 year, then the number of years remaining until the end of life will decrease by 1 year for each road segment, except for those stacked at zero. The zero stack will increase significantly because it maintains its previous balance and also becomes the recipient of those roads having previously been stacked with 1 year remaining. Thus, the entire network will age 1 year to the condition shown in Figure 2, with the net lane-miles in the zero stack raised from 4% to 8% of the network.

Some highway agencies still subscribe to the old practice of assigning their highest priorities to the reconstruction or rehabilitation of the worst roads. This practice of "worst first," that is, continually addressing only those roads in the zero stack, is a proven death spiral strategy because reconstruction and rehabilitation are the most expensive ways to maintain or restore serviceability. Rarely does sufficient funding exist to sustain such a strategy.

The measurable loss of pavement life can be thought of as the network's total lane-miles multiplied by 1 year, that is, *lane-mile-*

mile-years added to the network. A palette of pavement preservation treatments, or mix of fixes, is available to address the network needs at a much lower cost than traditional methods.

Preservation treatments are only suitable if the *right* treatment is used on the *right* road at the *right* time. In Figure 9, the added treatments used include concrete joint resealing, thin hot-mix asphalt (HMA) overlay (≤1.5 in.), microsurfacing, chip seal, and crack seal. By knowing the cost per lane-mile and the treatment life extension, it is possible to create a new strategy (costing $36,781,144) that satisfies the network need. In this example, the agency saved in excess of $500,000 from traditional methods (costing $37,323,132) while erasing the 1,654 lane-mile-year deficit produced by the initial program tally.

Network Strategy

Programmed Activity		Lane-Mile-Years	Total Cost
Reconstruction	(31 lane-miles)	820	$15,200,340
Rehabilitation	(77 lane-miles)	1,125	$14,545,002
Pavement Preservation	(84 lane-miles)	412	$1,475,850
Concrete Resealing	(4 yrs x 31 lane-miles)	124	$ 979,600
Thin HMA Overlay	(10 yrs x 16 lane-miles)	160	$ 870,560
Microsurfacing	(7 yrs x 44 lane-miles)	308	$1,309,000
Chip Seal	(5 yrs x 79 lane-miles)	395	$1,104,420
Crack Seal	(2 yrs x 506 lane-miles)	1,012	$1,296,372
		4,356	**$36,781,144**

Figure 9. New program tally.

In a real-world situation, the highway agency would program its budget to achieve the greatest impact on its network condition. Funds allocated for reconstruction and rehabilitation projects must be viewed as investments in the infrastructure. Conversely, funds directed for preservation projects must be regarded as protecting and preserving past infrastructure investments. Integrating reconstruction, rehabilitation, and preservation in the proper proportions will substantially improve network conditions for the taxpayer while safeguarding the highway investment.

knowing the only two components for reconstruction and rehabilitation projects: lane miles and design life of each project fix. Figure 4 shows the agency's programmed activities for reconstruction, and Figure 5 displays it for rehabilitation.

Reconstruction Evaluation

Projects This Year = 2

Project	Design Life	Lane-Miles	Lane-Mile-Years	Lane-Mile Cost	Total Cost
No. 1	25 yrs	22	550	$463,425	$10,195,350
No. 2	30 yrs	18	540	$556,110	$10,009,980
	Total	=	**1,090**		$20,205,330

Figure 4. Reconstruction evaluation.

When evaluating pavement preservations treatments in this analysis, it is appropriate to think in terms of "extended life" rather than design life. The term *design life*, as used in the reconstruction and rehabilitation tables, relates better to the new pavement's structural adequacy to handle repetitive loadings and environmental factors. This is not the goal of pavement preservation. Each type of treatment/repair has unique benefits that should be targeted to the specific mode of pavement deterioration. This means that life extension depends on factors such as type and severity of distress, traffic volume, environment, and so forth. Figure 6 exhibits the agency's programmed activities for preservation.

Rehabilitation Evaluation

Projects This Year = 3

Project	Design Life	Lane-Miles	Lane-Mile-Years	Lane-Mile Cost	Total Cost
No. 10	18 yrs	22	396	$263,268	$5,791,896
No. 11	15 yrs	28	420	$219,390	$6,142,920
No. 12	12 yrs	32	384	$115,848	$3,707,136
	Total	=	**1,200**		$15,641,952

Figure 5. Rehabilitation evaluation.

Projects This Year = 5

Project	Life Extension	Lane-Miles	Lane-Mile-Years	Lane-Mile Cost	Total Cost
No. 101	2 yrs	12	24	$2,562	$30,744
No. 102	3 yrs	22	66	$7,743	$170,346
No. 103	5 yrs	26	130	$13,980	$363,480
No. 104	7 yrs	16	112	$29,750	$476,000
No. 105	10 yrs	8	80	$54,410	$435,280
	Total	=	**412**		$1,475,850

Figure 6. Preservation evaluation.

To satisfy the needs of its highway network, the agency must accomplish 4,356 lane-mile-years of work per year. The agency's program will derive 1,090 lane-mile-years from reconstruction, 1,200 lane-mile-years from rehabilitation, and 412 lane-mile-years from pavement preservation for a total of 2,702 lane-mile-years. Thus, these programmed activities fall short of the minimum required to maintain the status quo and hence would contribute to a net loss in network pavement condition of 1,654 lane-mile-years. The agency's programmed tally is shown in Figure 7.

Network Trend

Programmed Activity	Lane-Mile-Years	Total Cost
Reconstruction	1,090	$20,205,330
Rehabilitation	1,200	$15,641,952
Preservation	412	$1,475,850
Total	**2,702**	**$37,323,132**
Network Needs (Loss)	(—) 4,356	
Deficit =	**—1,654**	

Figure 7. Programmed tally.

This exercise can be performed for any pavement network to benchmark its current trend. By using this approach, it is possible to see how various long-term strategies could be devised and evaluated against a policy objective related to total-network condition.

Once the pavement network is benchmarked, an opportunity exists to correct any shortcomings in the programmed tally. A decision must first be made as to whether to improve the network condition or to just maintain the status quo. This is a management decision and system goal. Continuing with the previous example, a strategy will be proposed to prevent further network deterioration until additional funding is secured.

The first step is to modify the reconstruction and rehabilitation (R&R) programs. An agonizing decision must be made about which projects to defer, eliminate, or phase differently with multiyear activity. In Figure 8, deductions are made in the R&R programs to recover funds for less costly treatments in the pavement preservation program. The result of this decision recovered slightly over $6 million.

Program Modification

Programmed Activity	Lane-Mile-Years	Cost Savings
Reconstruction ***31 lane-miles*** (40 lane miles)	***820*** ~~(1,090)~~	$5,004,990
Rehabilitation ***77 lane-miles*** ~~(82 lane-miles)~~	***1,125*** ~~(1,200)~~	$1,096,950
Pavement Preservation (84 lane-miles)	(412)	0
Total =	***2,357*** ~~(2,702)~~	**$6,101,940**

Figure 8. Revised R&R programs.

Modifying the reconstruction and rehabilitation programs has reduced the number of lane-mile-years added to the network through reconstruction and rehabilitation from 2,702 to 2,357. However, using less costly treatments elsewhere in the network to address roads in better condition will increase the number of lane-

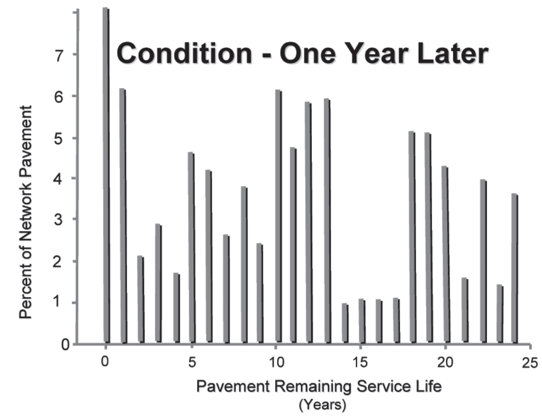


Figure 2. Condition 1 year later.

years. Consider the following quantitative illustration: Suppose your agency's highway network consisted of 4,356 lane-miles. Figure 3 shows that without intervention, it will lose 4,356 lane-mile-years per year.

Agency Highway Network = 4,356 lane-miles

Each year the network will lose

4,356 lane-mile-years

Figure 3. Network lane-miles.

To offset this amount of deterioration over the entire network, the agency would need to annually perform a quantity of work equal to the total number of lane-mile-years lost just to maintain the status quo. Performing a quantity of work that produces fewer than 4,356 new lane-mile-years would lessen the natural decline of the overall network but still fall short of maintaining the status quo. However, if the agency produces more than 4,356 lane-mile-years, it will improve the network.

In the following example, an agency can easily identify the effect of an annual program that consists of reconstruction, rehabilitation, and preservation projects on its network. This assessment involves

PAVEMENT NETWORK EVALUATION WORKSHEET

Total Network Lane Miles = ____________

RECONSTRUCTION

Project	Design Life	Lane Miles	Lane Mile Years	Lane Mile Cost	Total Cost
	×	=			
	×	=			
	×	=			
		Total	=		

REHABILITATION

Project	Design Life	Lane Miles	Lane Mile Years	Lane Mile Cost	Total Cost
	×	=			
	×	=			
	×	=			
	×	=			
		Total	=		

PAVEMENT PRESERVATION

Project	Design Life	Lane Miles	Lane Mile Years	Lane Mile Cost	Total Cost
	×	=			
	×	=			
	×	=			
	×	=			
	×	=			
		Total	=		

NETWORK TREND

Project	Lane Mile Years	Total Cost
Reconstruction		
Rehabilitation		
Preservation		
Total		

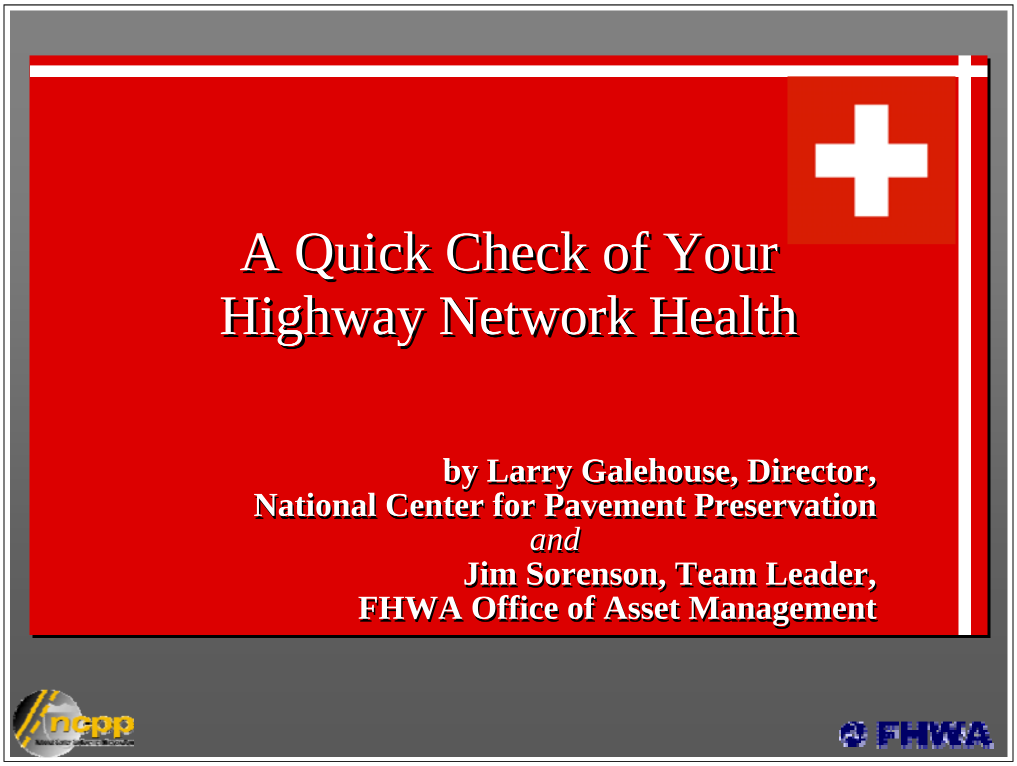
SUMMARY

Programmed Activity (Lane Mile Years)	=
Total Network (Lane Mile Years)	=
Gain (+) / Deficit (-)	=

If you would like to view the electronic version of this exercise along with the worksheet, please visit the FHWA System Preservation Web site at **http://www.fhwa.dot.gov/preservation/library.cfm**.

A Quick Highway Network Evaluation Worksheet

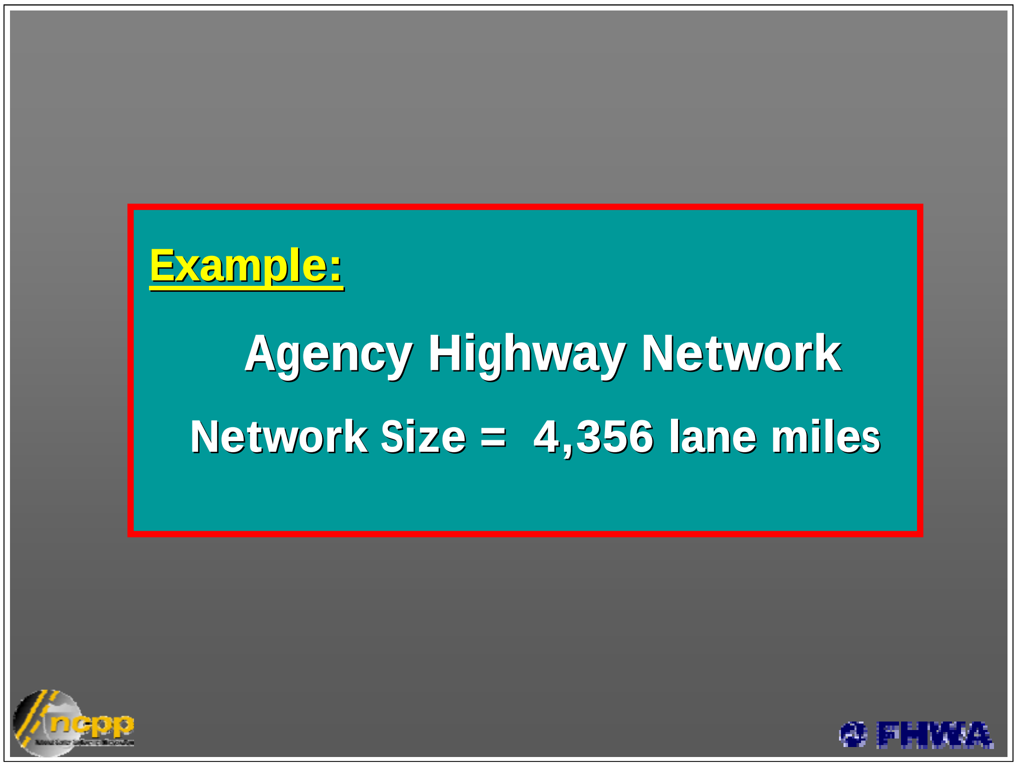
A Removable Worksheet to Assess the Needs of the Highway Network and the Adequacy of Resource Allocations



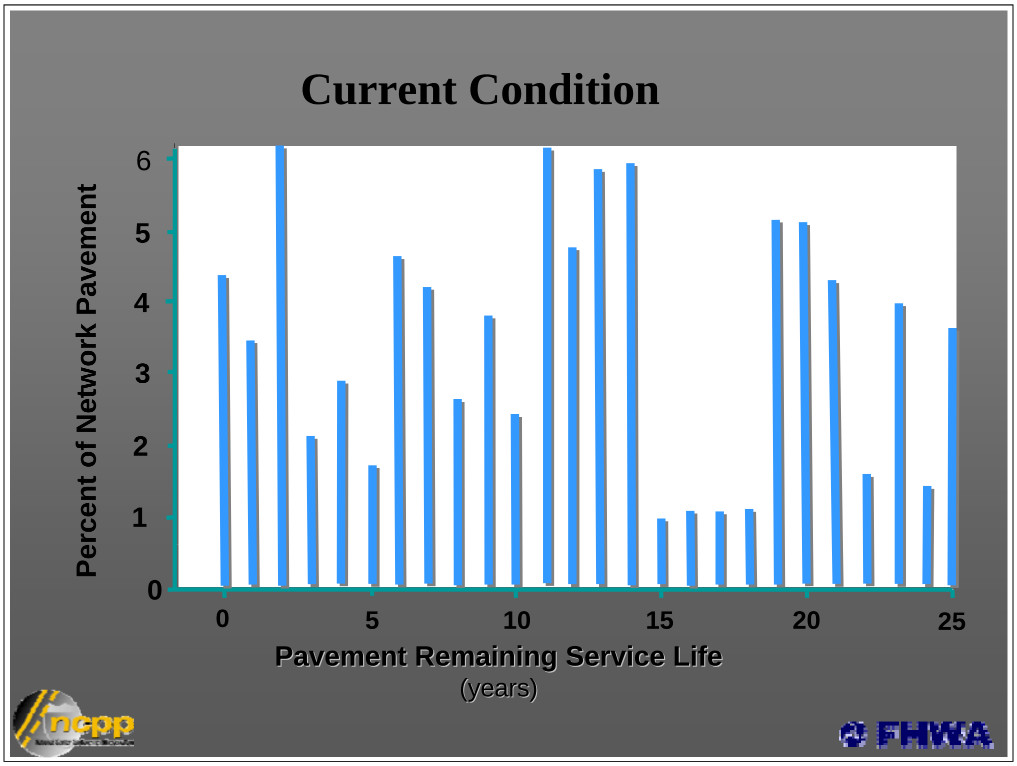
Today, highway agency managers and administrators face the daunting assignment of allocating resources to address the needs of their highway network. This "Quick Check" will determine the adequacy of their resource allocation effort with minimal calculations. All that is required is the following information:

1. The total number of "lane-miles" in the agency's network,

2. The present or planned program actions within a given year. Program actions include reconstruction, rehabilitation and various pavement preservation (e.g., resurfacing, crack sealing, chip seals, etc.).



Let's assume we have a highway network of 4,356 lane-miles. This network could be representative of a county, city, or a state DOT district, region or division. However, the "Quick Check" always works regardless of the network size.



Assume we could plot the condition of every pavement segment in the highway network. The plot might appear as shown in this graph.

The horizontal-axis represents years of Remaining Service Life (RSL) and the vertical-axis depicts the percentage roads in the network that have that RSL. For example, in above chart pavement segments having an RSL=5 comprise nearly 2% of the entire pavement network.

So what is Remaining Service Life? It represents how many years of life a pavement has until it will reach zero life, or RSL=0.

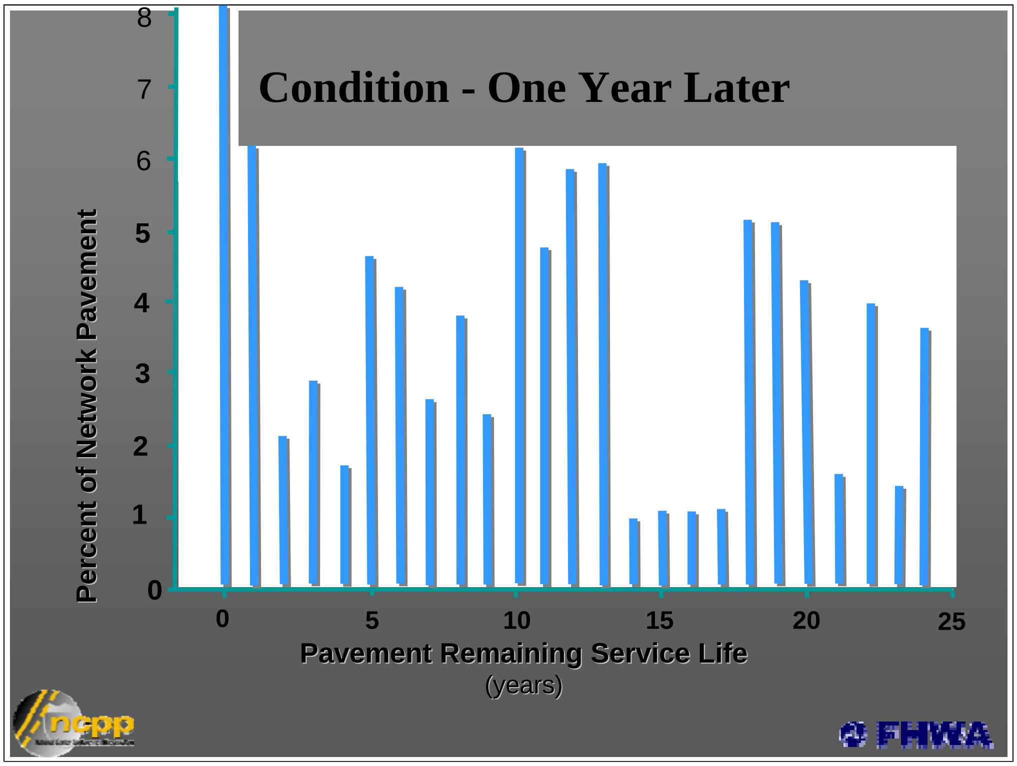
So what is Zero Life (RSL=0)? If the agency has established zero (RSL=0) correctly it represents two things.

First, the public is not satisfied with the condition of the road. Telephones begin to ring and the agency is forced into a reactive mode to maintain the pavement serviceability.

Second, and more importantly the only financially feasible option the agency has, is to reconstruct or rehab the road. Maintenance forces can generally patch and fill potholes, but the expenditure is not gaining pavement life. Preservation is no longer an option. Reactive actions only maintain a limited degree of serviceability.

Let's assume a worst-case scenario – no funds will be available for highway work (of any kind) next year.

What will happen to the network and this graph? Every pavement segment will lose (on average) one year of life. The graph columns will move one year to the left. So pavement segments with an RSL=9 will now have an RSL=8, an RSL=5 will now have an RSL=4, and so on.



Pavements with an RSL=1 now have an RSL=0. Note that these pavements coming from RSL=1, are added to the pavements already in the RSL=0 category. This group represents "dead" pavements that can't be revived without reconstruction or major rehabilitation.

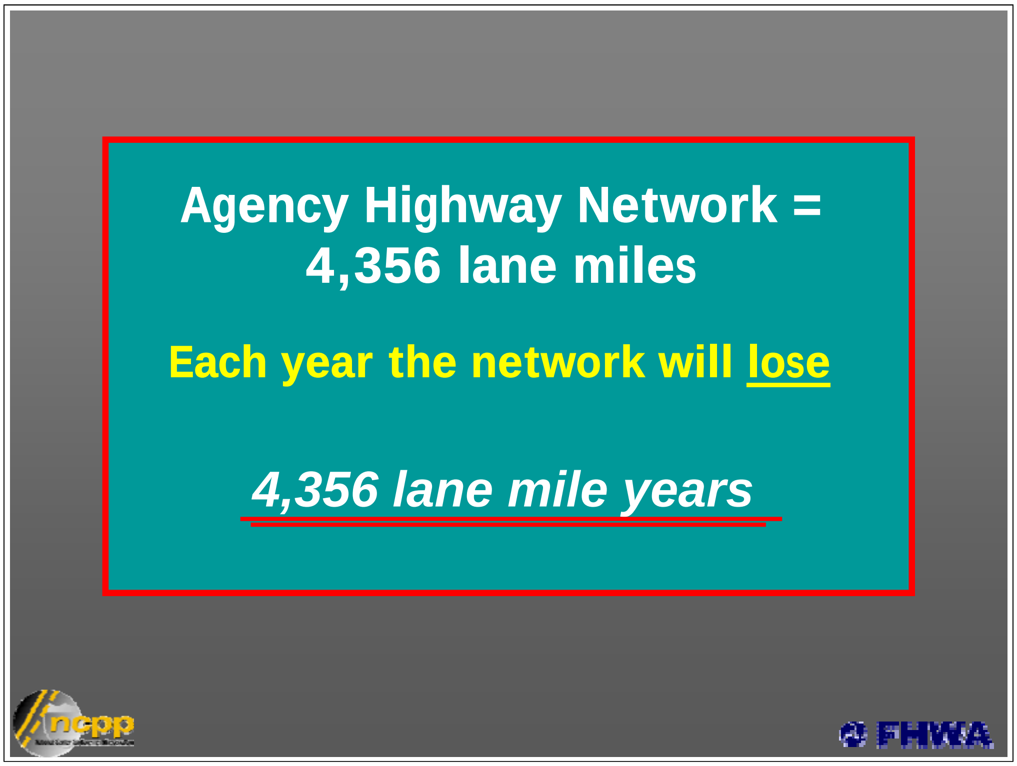
Background Information

Most highway agencies have a limited budget and can only address a small percentage of their highway network through reconstruction or rehabilitation projects. For example, if their entire annual budget were used to reconstruct or rehabilitate the worst pavements, it would amount to somewhere between 2% and 4% of the network.

One year later, another stack of projects fall into the "dead" pavement column, or RSL=0. These columns are similar to ocean waves hitting the beach. Each year another group of pavements move into the RSL=0 column.

This graph illustrates a much larger issue, which is, working only on the network's worst roads is a losing proposition and a bad strategy. Few agencies can ever receive enough funding to stay ahead of the backlog of poor pavements, by only reconstruction and rehabilitation efforts. Pavement preservation is also necessary, by adding life to roads in good condition and moving them to the right by increments of 2 to 10 years, depending on the treatment used.

The goal of pavement preservation is to manage the network by extending pavement life and reducing the backlog of roads in poor condition. This will permit agencies and the contracting industry to accurately predict future resource needs.



Now, let's look at all the miles in the network as a whole.

We can say that every year, the entire network will age one year. Therefore, the entire network of 4,356 lane-miles will lose 4,356 lane-mile years.

The analogy here is similar to an active bank account - the effects of Mother Nature and traffic are continually making withdrawals from the network.

Background Information

In reality pavements age at different rates, but on a *network average*, the entire network will age one year in one year's time.

Reconstruction Evaluation

Project	Lane Miles	Design Life	Lane Mile Years	Lane Mile Costs	Total Cost
#1	22	25 yrs	550	$463,425	$10,195,350
#2	18	30 yrs	540	$556,110	$10,009,980
	Total	=	1,090		$20,205,330

Continuing our bank account analogy, deposits must be made to maintain financial solvency. The reconstruction, rehabilitation, and pavement preservation projects initiated during one year represent deposits to the network, offsetting the losses of lane-mile-years.

In the example, the highway agency plans two reconstruction projects for next year. Project #1 will involve 22 lane-miles of pavement using a 25 year design life. When the 22 lane-miles are multiplied by a design life of 25 years, the result is 550 lane-mile years. The same calculation is performed on Project #2. The sum of the reconstruction efforts represents 1,090 lane-mile years, synonymous with a network deposit for reconstruction.

Rehabilitation Evaluation

Project	Lane Miles	Design Life	Lane Mile Years	Lane Mile Costs	Total Cost
#3	22	18 yrs	396	$263,268	$5,791,896
#4	28	15 yrs	420	$219,390	$6,142,920
#5	32	12 yrs	384	$115,848	$3,707,136
	Total	=	1,200		$15,641,952

The same calculations are performed for the rehabilitation program planned for next year. The example has three projects. Multiply the lane-miles of each project by the design life and the result is lane-mile years.

The sum of the rehabilitation efforts represents 1,200 lane-mile years, synonymous with another network deposit.

Pavement Preservation Evaluation

Project	Lane Miles	Life Ext.	Lane Mile Years	Lane Mile Costs	Total Cost
#101	12	2 yrs	24	$2,562	$30,744
#102	22	3 yrs	66	$7,743	$170,346
#103	26	5 yrs	130	$13,980	$363,480
#104	16	7 yrs	112	$29,750	$476,000
#105	8	10 yrs	80	$54,410	$435,280
	Total	=	412		$1,475,850

The final calculations are performed for the pavement preservation program planned for next year. The same calculation method is used as in the previous examples for reconstruction and rehabilitation, with one notable exception. When evaluating pavement preservation treatments, it is appropriate to think in terms of “extended life” rather than design life. The term design life, as used in the reconstruction and rehabilitation examples, relates to a new pavement’s structural adequacy. The goal of pavement preservation is not to improve the structural adequacy of the pavement, but rather stop or slow various distresses caused by traffic volume, environment, and so forth.

Since pavement preservation treatments are applied to pavements that are in good structural condition, there is some degree of Remaining Service Life (RSL). Pavement preservation treatments must not be applied to projects that have an RSL=0.

In the example, the agency plans five pavement preservation projects for a total of 412 lane-mile years.

Network Trend

Required: 4,356 lane mile years

Programmed Activity	Lane Mile Years	Total Cost
Reconstruction $$$$ *(40 lane miles)*	1,090	$20,205,330
Rehabilitation $$$ *(82 lane miles)*	1,200	$15,641,952
Pavement Preservation $ *(84 lane miles)*	412	$1,475,850
Total =	2,702	$37,323,132

ncpp

FHWA

The proposed programmed activity is summarized by the highway agency. The totals are as follows: 40 lane miles for reconstruction, 82 lane miles for rehabilitation, and 84 lane miles of pavement preservation. This amount of work restores 2,702 lane-mile years to the network at a total cost of $37,323,132.

The automation depicts dollar symbols ($). The dollar symbols represent comparative costs of the action (similar to a “AAA” or “Diners Club” cost rating). Substantial funds are necessary for reconstruction and rehabilitation projects, while pavement preservation projects use significantly smaller funding amounts.

Network Needs Summary

Network Size *(needs)*	4,356 *(lane mile years)*
Programmed Activity	2,702 *(lane mile years)*
Deficit =	**1,654** ***(lane mile years)***

The most important consideration is how the proposed program addresses the system's overall need – in other words, how much are you putting back into the road compared to how much it is losing?

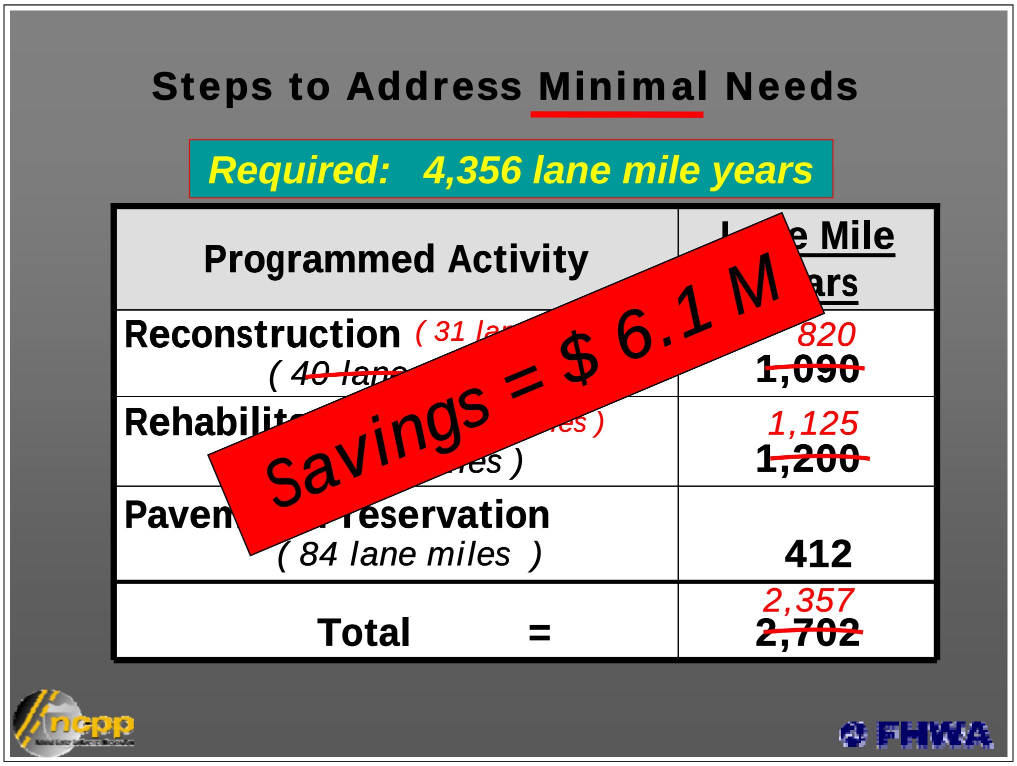
This network loses 4,356 lane-mile years and the proposed program adds 2,702 lane-mile years. As a result the network needs exceed the program size by 1,654 lane-mile years.

Any programmed activity less than the network needs is running a deficit and worsening over time.

A program that meets the network need is maintaining the status quo. The network is not getting better, but not getting worse.

A program that exceeds the network needs is improving the pavement condition.

Too often highway agencies accept the results without adjusting the programmed activity.



This slide contains various animations.

If the system's needs are not being met by a proposed program, we should adjust the program activities to see if the needs can be met within the given budget. When a highway agency readjusts the program activity, tough choices must be made. In this example, the reconstruction program was slightly downsized from 40 lane miles to 31 lane miles (deferring the other 9 lane miles will be deferred until the following year), and the rehabilitation program was also downsized from 82 lane miles to 77 lane miles (deferring the other 5 lane miles until the following year).

This portion of the program costs were reduced by $6.1 million, but note that as the programs were downsized, so was the total of lane-mile years. Next, we will see how that money could be applied elsewhere in the program to restore more lane-mile-years.

Background – reconstruction can be deferred – a year's delay will not significantly affect the scope of a reconstruction project.

Program Modification

Savings = $ 6,101,940   Needs = 1,999 LMY

Preservation Treatment	Life Ext	Lane Miles	Lane Mile Years	Total Cost
Concrete Reseal	**4 yrs**	**31**	**124**	**$979,600**
Thin HMA Overlay	**10 yrs**	**16**	**160**	**$870,560**
Micro-surfacing	**7 yrs**	**44**	**308**	**$1,309,000**
Chip Seal	**5 yrs**	**79**	**395**	**$1,104,420**
Crack Seal	**2 yrs**	**506**	**1,012**	**$1,296,372**
			1,999	**$5,559,952**

The system still needs 1999 of LMY in order to meet the annual deterioration. The $6.1 million redirected from the reconstruction program could be applied to a variety of less expensive pavement preservation projects in such a way that the 1999 LMY can be met.

Note – this presumes that there are suitable candidate roads (still in good condition) for the preservation actions.

Revised Network Strategy

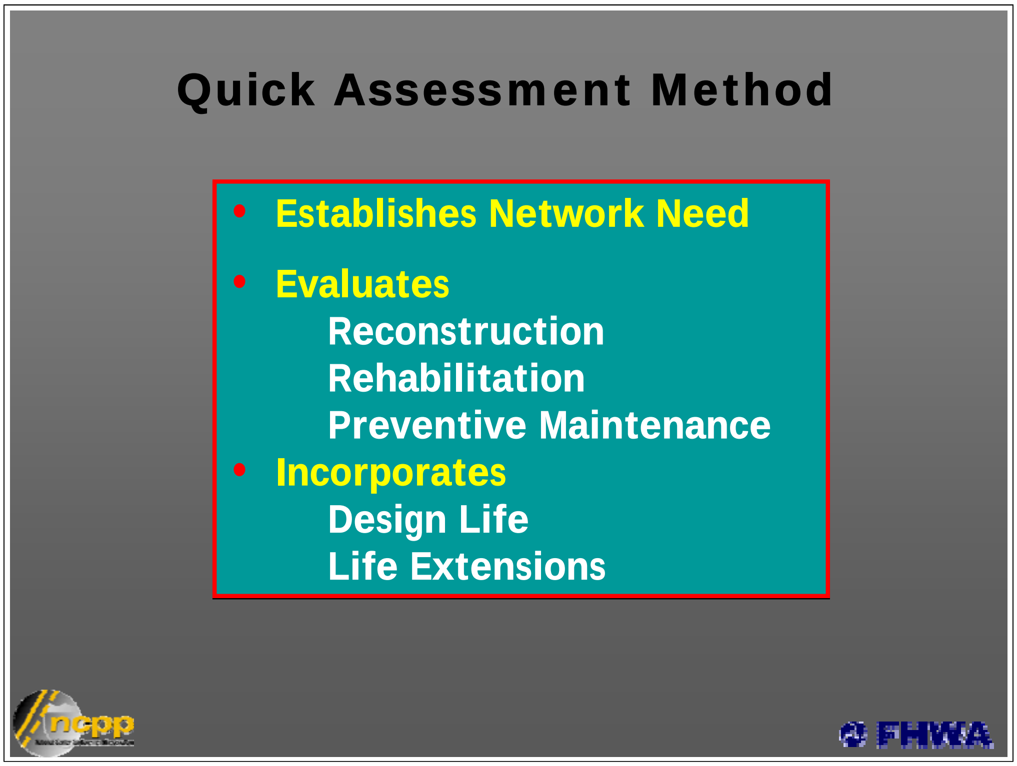
Required: 4,356 lane mile years

Programmed Activity	Lane Mile Years
Reconstruction (*31 lane miles*)	*820*
Rehabilitation (*77 lane miles*)	*1,125*
Pavement Preservation (*2,083 lane miles*)	*2,411*
Total =	**4,356**

Net Savings = $ 541,988

When the highway agency readjusted the program activity, the network need was achieved. In fact, because the $6.1 million was not exhausted, a net savings of $541,988 can be used to perform additional work.

Through careful network analysis there is a potential to actually improve the pavement network condition.



Funds allocated for reconstruction and rehabilitation projects must be viewed as investments in the infrastructure. Conversely, funds directed for preservation projects must be regarded as protecting and preserving past infrastructure investments.

The Quick Assessment Method can be used on any size network with accuracy. It establishes network need, evaluates programmed activity, and incorporates design life and preservation life extensions.

Questions ?

Thank You !

Network Needs Worksheet

Required: ________ lane-mile years

Programmed Activity	Lane-Mile Years
Reconstruction *(_____ lane-mile X ___ yrs)*	______
Rehabilitation *(_____ lane-mile X ___ yrs)*	______
Pavement Preservation *(_____ lane-mile X ___ yrs)*	______
Total =	________

This slide can be used to insert your own data to determine if you are putting as much back into your system as the system is losing.

Use this spreadsheet to insert your agency's data electonically.

Agency

Reviewer

Total Network Lane Miles: 4356

Reconstruction Projects

Project	Design Year (yrs)	Lane Miles	Lane Mile Years	Lane Mile Cost	Total Cost
1	25	22	550	$463,425.00	$10,195,350.00
2	30	18	540	$556,110.00	$10,009,980.00
			0		$0.00
			0		$0.00
			0		$0.00
			0		$0.00
	Total =	40	1090	$1,019,535.00	$20,205,330.00

Rehabilitation Projects

Project	Design Year (yrs)	Lane Miles	Lane Mile Years	Lane Mile Cost	Total Cost
10	18	22	396	$263,268.00	$5,791,896.00
11	15	28	420	$219,390.00	$6,142,920.00
12	12	32	384	$115,848.00	$3,707,136.00
			0		$0.00
			0		$0.00
			0		$0.00
	Total =	82	1200	$598,506.00	$15,641,952.00

Preservation Projects

Project	Life Extension (yrs)	Lane Miles	Lane Mile Years	Lane Mile Cost	Total Cost
101	2	12	24	$2,562.00	$30,744.00
102	3	22	66	$7,743.00	$170,346.00
103	5	26	130	$13,980.00	$363,480.00
104	7	16	112	$29,780.00	$476,480.00
105	10	8	80	$54,410.00	$435,280.00
			0		$0.00
			0		$0.00
	Total =	84	412	$108,475.00	$1,476,330.00

Network Trend

Programmed Activity	Lane Mile Years	Total Cost
Reconstruction	1,090	$20,205,330.00
Rehabilitation	1,200	$15,641,952.00
Preservation	412	$1,476,330.00
Total	**2,702**	**$37,323,612.00**

Summary

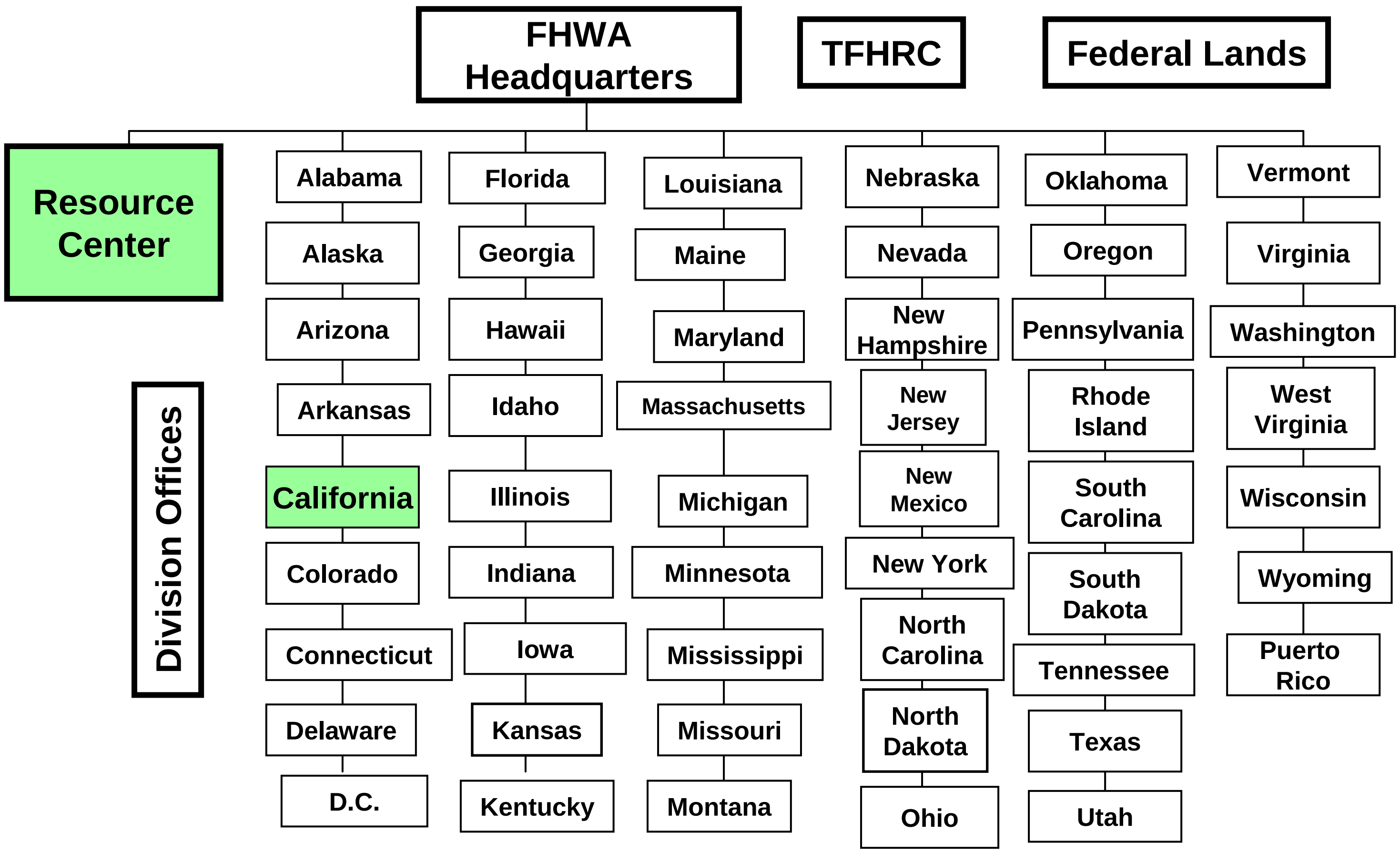
Programmed Activity (Lane Mile Years)	**2,702**
Total Network (Lane Mile Years)	**4,356**
Gain (+) / Deficit (-)	**-1,654**

Sustainable Pavement Preservation Strategies

Stephen R. Mueller, P.E.
Pavement and Materials Engineer
Federal Highway Administration
Steve.Mueller@fhwa.dot.gov

April 8, 2009 California Pavement Preservation Conference

FHWA Organization



FHWA Resource Center
11 Technical Services Teams

Vision –> Mission

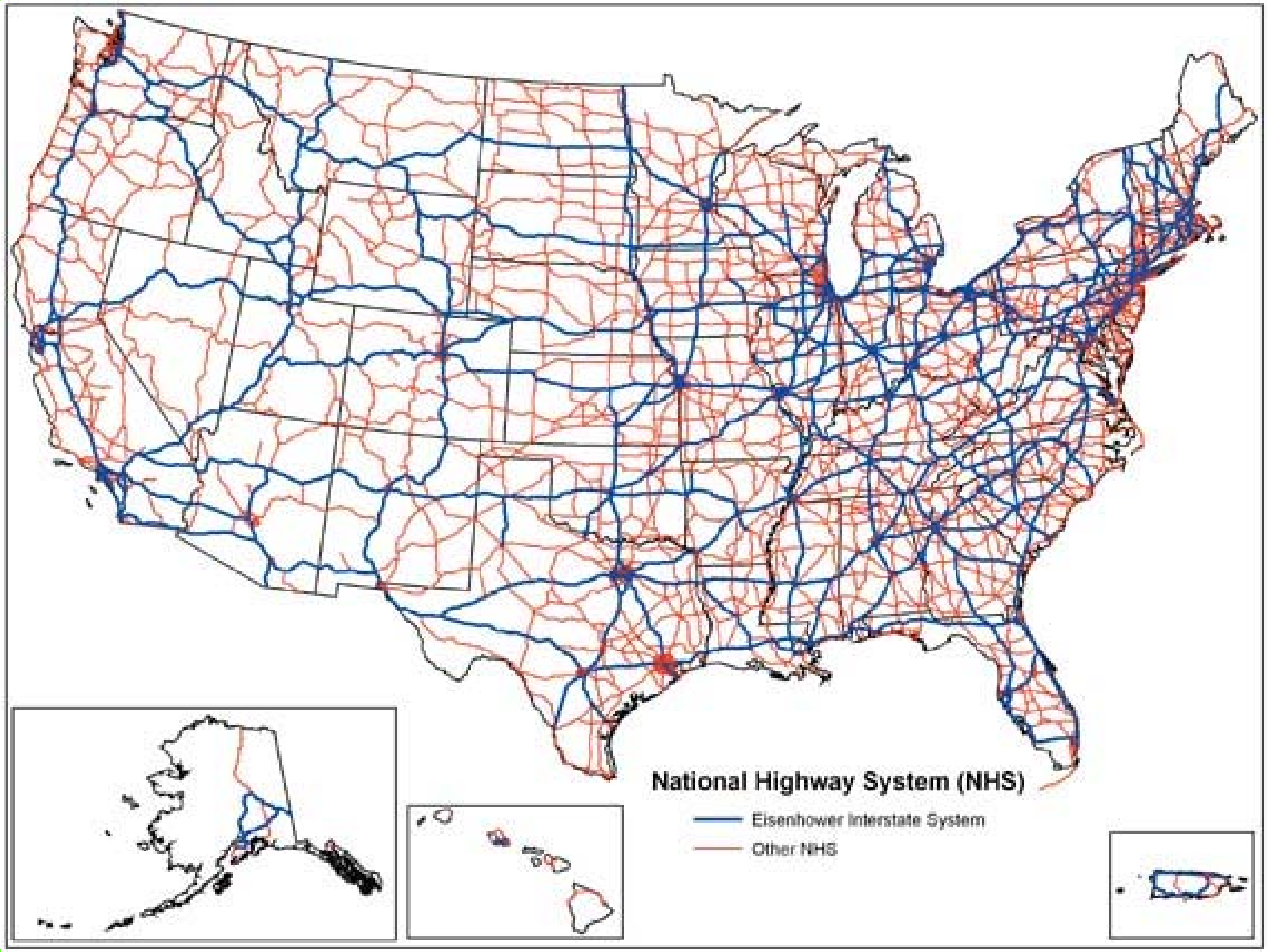
- “Our Agency and our transportation system are the best in the world.”
- “Improve Mobility on our Nation’s Highways through National Leadership, Innovation, and Program Delivery.”

Without Pavement, We Would Be Stuck in the Mud!

Washington-Richmond road, 1919
NMAH, Archives Center, API Collection

4 Million Miles of Roads

600,000 Bridges



Statistics We Should Know:

Federal = 3%

State = 20%

Local = 77%

2/3 are Paved (1/3 Unpaved)

94% of Paved have an Asphalt Surface

Sustainable PP Strategies

- What is “Sustainable” ?
- Sustainable Pavement Technologies:
  - Recycling
  - Reuse
  - Other Technologies
- Resources for Transportation Agencies in Developing Sustainable Pavement Preservation Strategies

What is "Sustainable"

- Definitions / Concepts
- Measurement Tools/Systems
  - Rating Systems
  - Life Cycle Analysis (not LCCA)
- EPA's Mantra / FHWA's Three E's

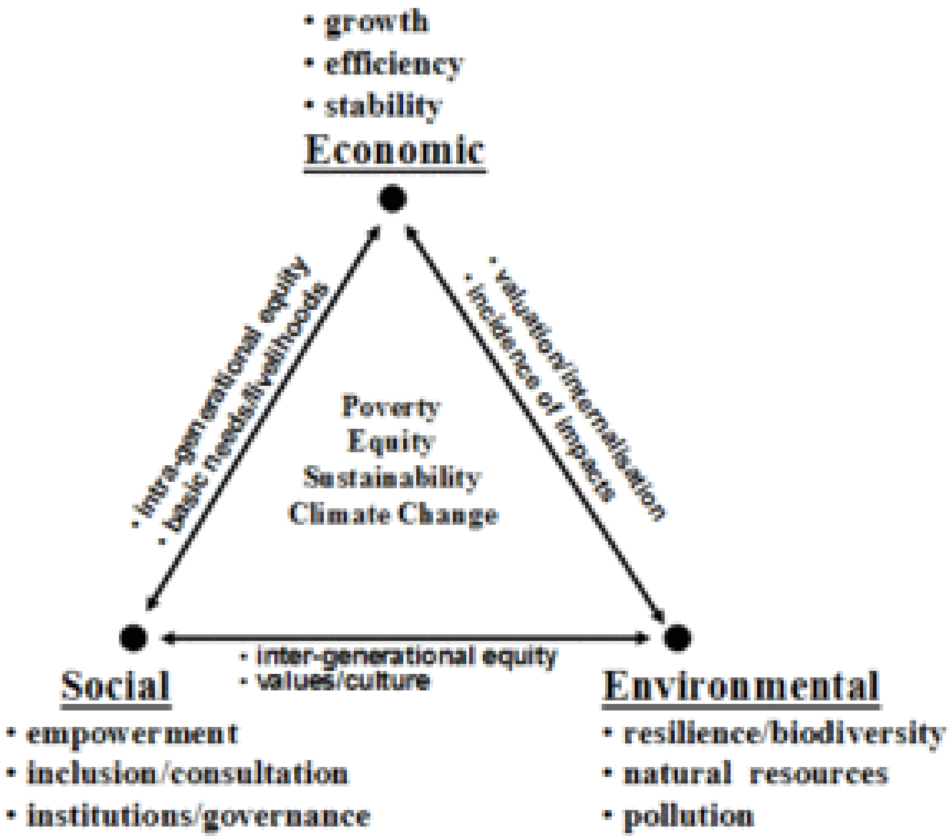
EPA's Sustainability Definition

Sustainability means "meeting the needs of the present without compromising the ability of future generations to meet their own needs."

"Sustainable Development Triangle"

http://www.eoearth.org/article/Basic_concepts_and_principles_of_sustainomics

Environmental, Economic, Social



1999 CIWMB Plan

http://www.ciwmb.ca.gov/greenBuilding

Sustainable Building Implementation Plan

California Integrated Waste Management Board
September 1999

"Green Buildings"
Environmental Rating Systems

- **Siting**
- **Energy Efficiency**
- **Materials Efficiency** Use of Recycled Materials
- **Water Efficiency**
- **Occupant Health and Safety**
- **Building Operations and Maintenance**

Also: Leadership in Energy and Environmental Design (LEED)

"Greenroads" Rating System
www.greenroads.us

Category	Description	points
Project Requirements	Minimum requirements for a Greenroad	11
Environment & Water	Stormwater, habitat, vegetation	14
Access & Equity	Modal access, culture, aesthetics, safety	14
Construction Activities	Construction equipment, quality, use	15
Materials & Resources	Material extraction, processing, transport	12
Pavement Technology	Pavement design, material use, function	11
Custom Credits	Write your own credit for approval	10
	Total	76

Please Note: There are Many Rating Systems, and FHWA does not endorse a national rating system.

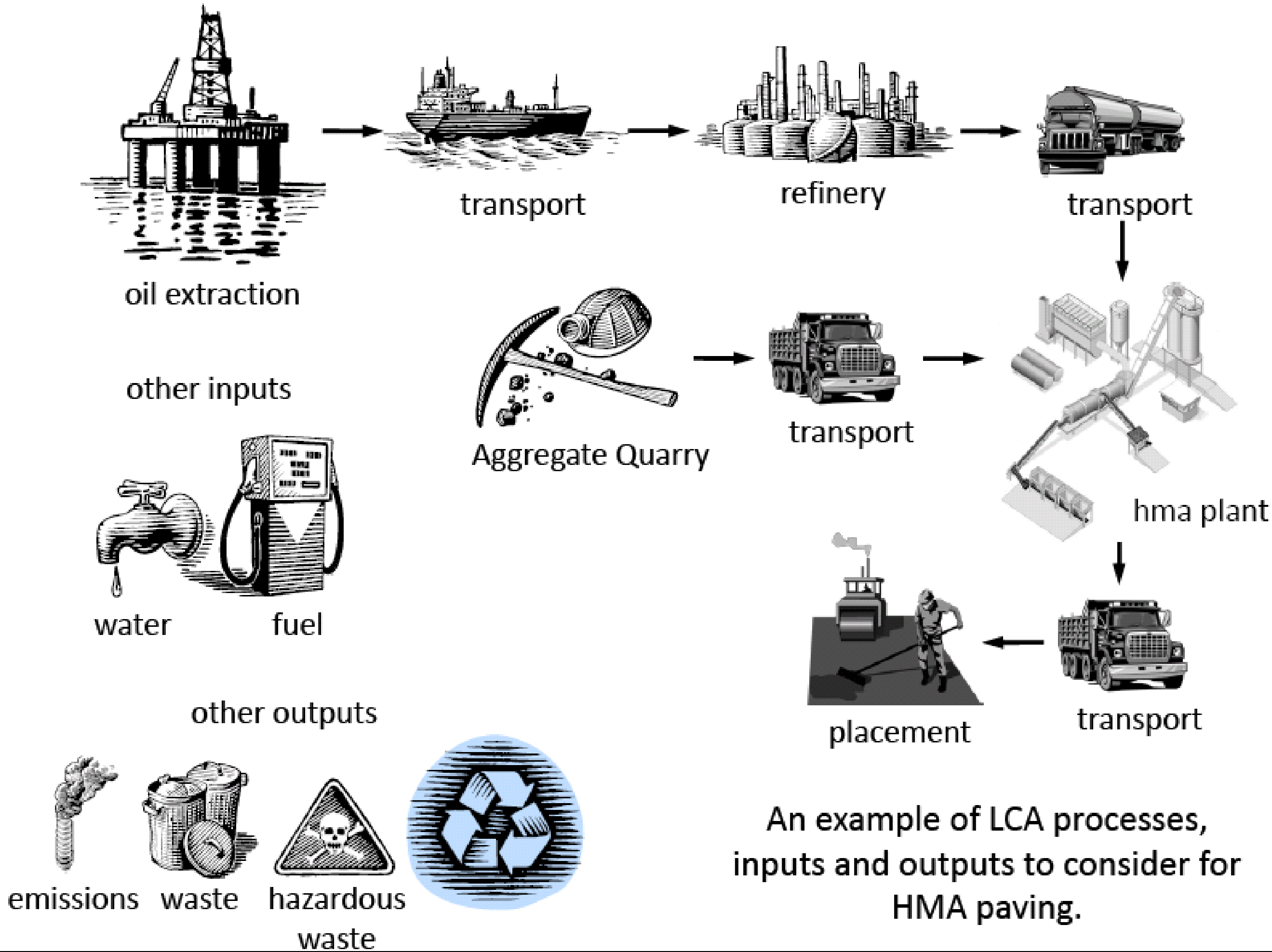
"Greenroads" Certification Levels

Version 0.96: 76 Total Points

Greenroads certified	Greenroads certified SILVER	Greenroads certified GOLD	Greenroads certified EVERGREEN
23-30 points	31-38 points	39-46 points	47+ points
30-40% of total	40-50% of total	50-60% of total	60+% of total

Please Note: There are Many Rating Systems, and FHWA does not endorse a national rating system.

"Greenroads" – LCA in HMA



An example of LCA processes, inputs and outputs to consider for HMA paving.

EPA Mantra

- **REDUCE**
  - **Consume Less If Possible.**
- **RECYCLE**
  - **Reuse Previously Produced Materials.**
- **REUSE**
  - **Incorporate Materials Used in Other Manufacturing Processes Into the Work.**

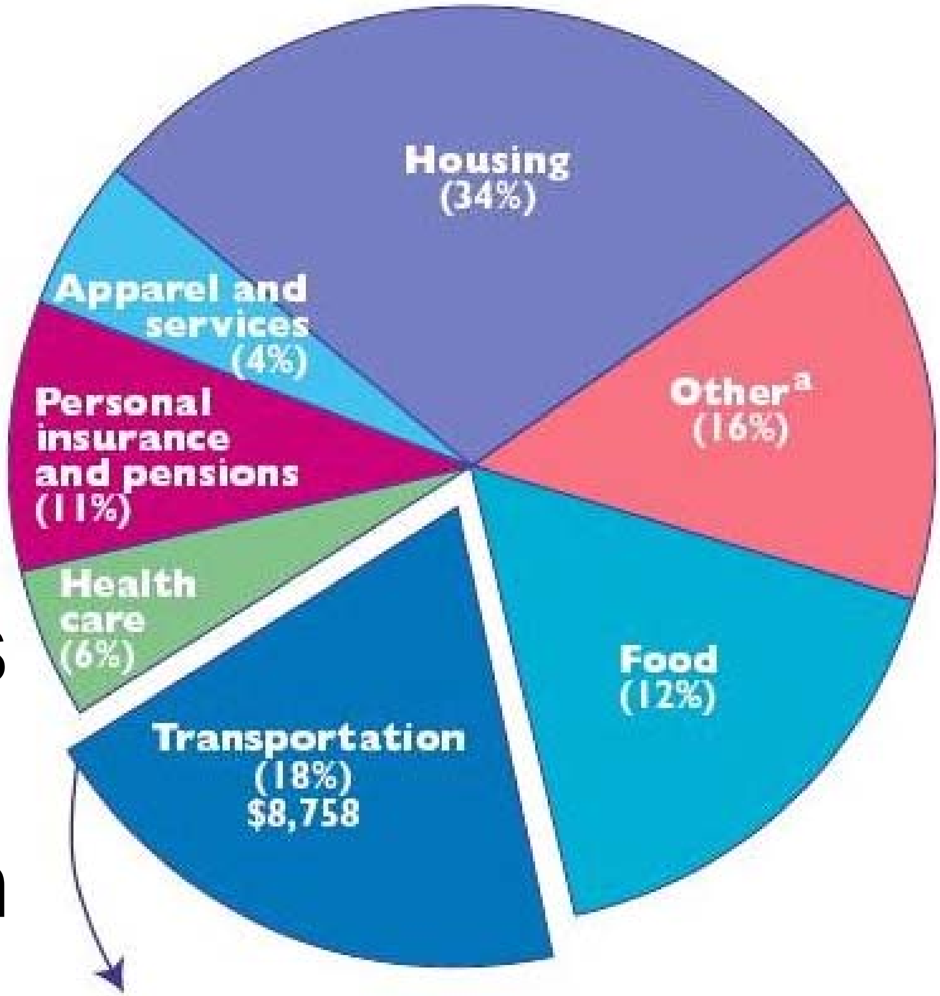
FHWA's "3 E's"

- ENGINEERING
  - Use Good Engineering Design to Assure Long-Life Pavements.
- ECONOMICS
  - Use Life-Cycle Cost Analysis for Project Selection.
- ENVIRONMENT
  - Consider Recycling First
  - Be Good Stewards of the Environment

Components of a "Sustainable" Roadway

National Surface Transportation Financing Commission Report 2009

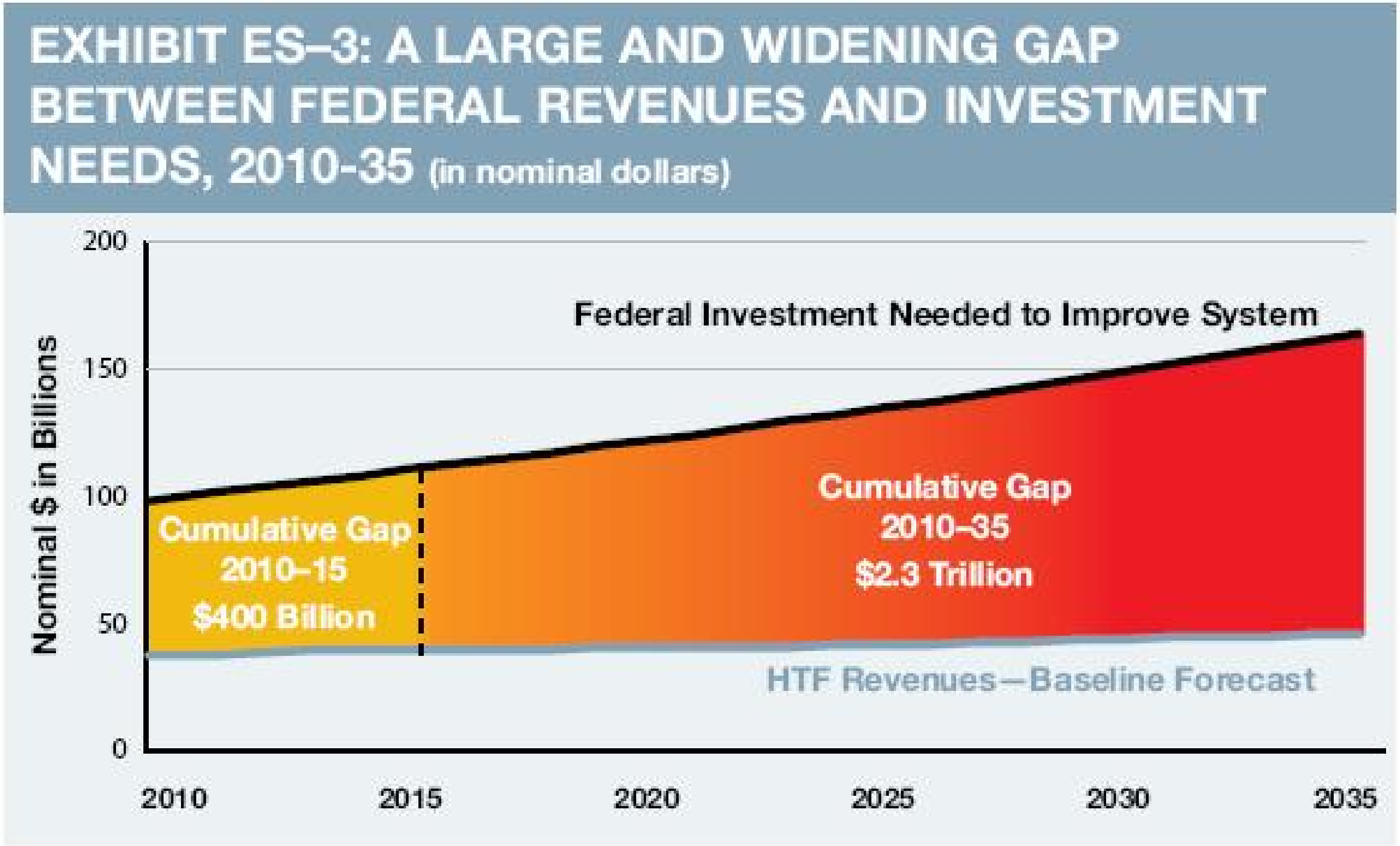
2007 Average US Household Expenditures 18% for Transportation



ARRA FUNDING

- **Provides $48.1 billion for transportation, including:**
  - ***<u>$27.5 billion for highways</u>***
  - **$8.4 billion for transit**
  - **$8.0 billion for high speed rail**
  - **$1.3 billion for Amtrak**
  - **$1.5 billion for National Surface Transportation Discretionary Grants**

National Surface Transportation Financing Commission Report 2009



Environment

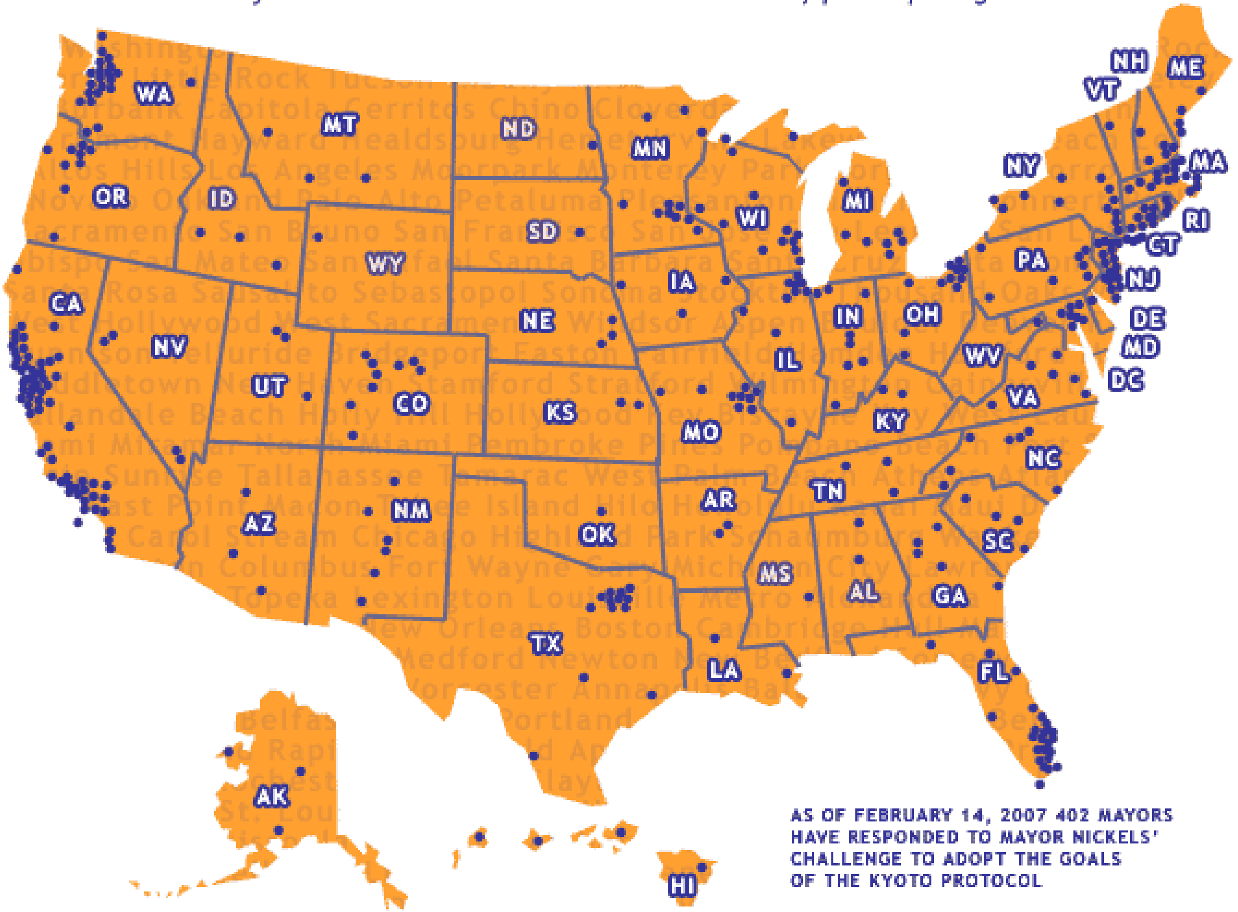
- **Major Citizen Concern**
  - **Kyoto Protocols**
  - **Federal Legislation**

Preservation has most of the "green" cards in the highway industry game. If you're in the game, you should play with <u>all</u> of your cards if you want to win!

402 US Regional Kyoto Commitments

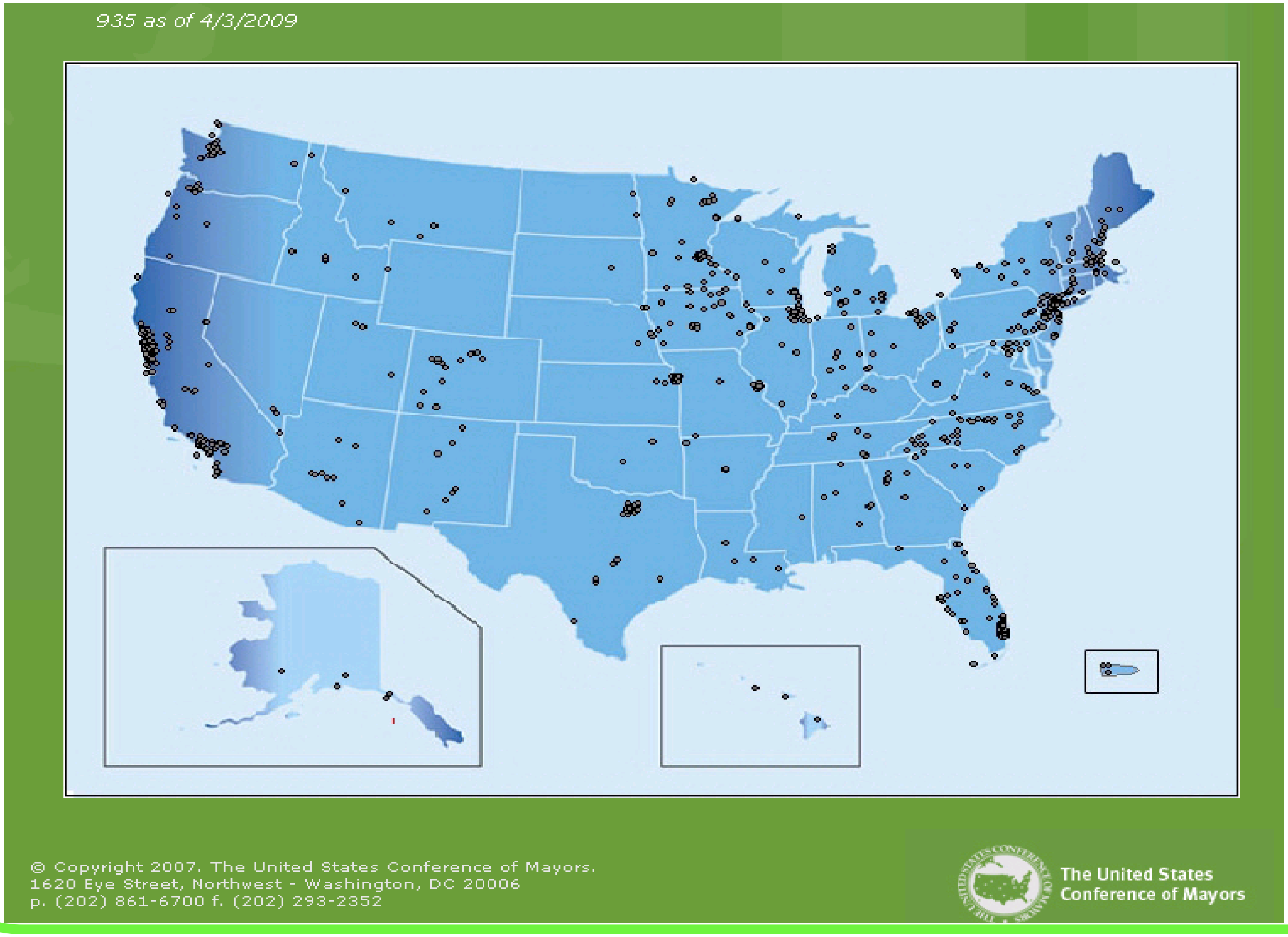
2-14-07

Pass your cursor over each state to view a list of participating cities.



935 US Regional Kyoto Commitments

4-3-09 Source: http://usmayors.org/climateprotection/map.asp



4-3-09 CA Cities Adopting Kyoto

Source: http://usmayors.org/climateprotection/map.asp

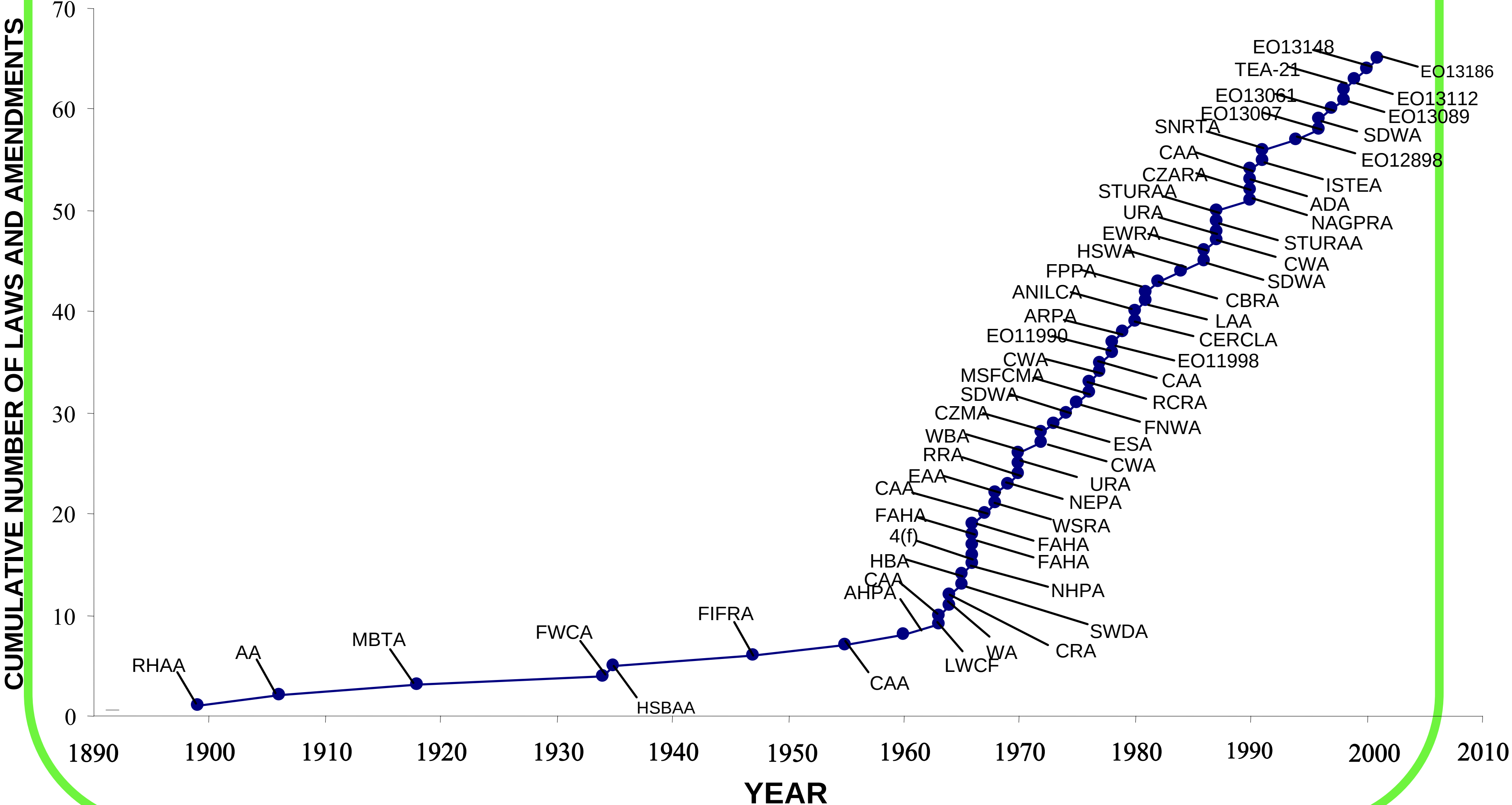
Alameda
Albany
Aliso Viejo
Arcata
Atascadero
Atherton
Avalon
Benicia
Berkeley
Beverly Hills
Burbank
Burlingame
Calistoga
Campbell
Capitola
Chico
Chino
Chula Vista
Claremont
Cloverdale
Concord
Cotati
Culver City
Cupertino
Del Mar
Dublin
El Cajon
El Cerrito
Elk Grove
Fairfax
Fremont
Galt
Glendora

Hayward
Healdsburg
Hemet
Hermosa Beach
Hillsborough
Huntington Beach
Imperial Beach
Irvine
La Mesa
Lafayette
Laguna Beach
Laguna Hills
Laguna Woods
Lakewood
Lemoore
Long Beach
Los Altos
Los Altos Hills
Los Angeles
Los Gatos
Malibu
Mammoth Lakes
Manhattan Beach
Menlo Park
Mill Valley
Millbrae
Monterey
Monterey Park
Moorpark
Moraga
Morgan Hill
Morro Bay
Mountain View

Newark
Novato
Oakland
Pacific Grove
Pacifica
Palm Springs
Palo Alto
Paradise
Pasadena
Petaluma
Pleasanton
Portola Valley
Rancho Palos Verdes
Redlands
Redondo Beach
Redwood City
Rialto
Richmond
Riverside
Rohnert Park
Rolling Hills Estates
Sacramento
Salinas
San Bernardino
San Bruno
San Buenaventura
San Clemente
San Diego
San Fernando
San Francisco
San Jose
San Leandro
San Luis Obispo

San Mateo
San Rafael
Santa Ana
Santa Barbara
Santa Clara
Santa Cruz
Santa Monica
Santa Rosa
Saratoga
Sausalito
Sebastopol
Sierra Madre
Solana Beach
Sonoma
South San Francisco
Stockton
Sunnyvale
Thousand Oaks
Torrance
Tulare
Vallejo
Visalia
Vista
West Hollywood
West Sacramento
Whittier
Windsor
Winters
Yountville
Yucaipa

Federal Environmental Legislation and Executive Orders Affecting Transportation



Prepared by FHWA Office of Natural Environment, May 2001

SUSTAINABLE TECHNOLOGIES

- **RECYCLING APPLICATIONS**
  - **Reclaimed Asphalt Pavement**
  - **Recycled Concrete Aggregate**
  - **In-Place Recycling**
- **REUSE APPLICATIONS**
  - **FLY ASH / COAL ASH**
  - **TIRE RUBBER**
  - **SHINGLES**
  - **SLAG**
  - **FOUNDRY SAND**
- **Warm-Mix Asphalt**

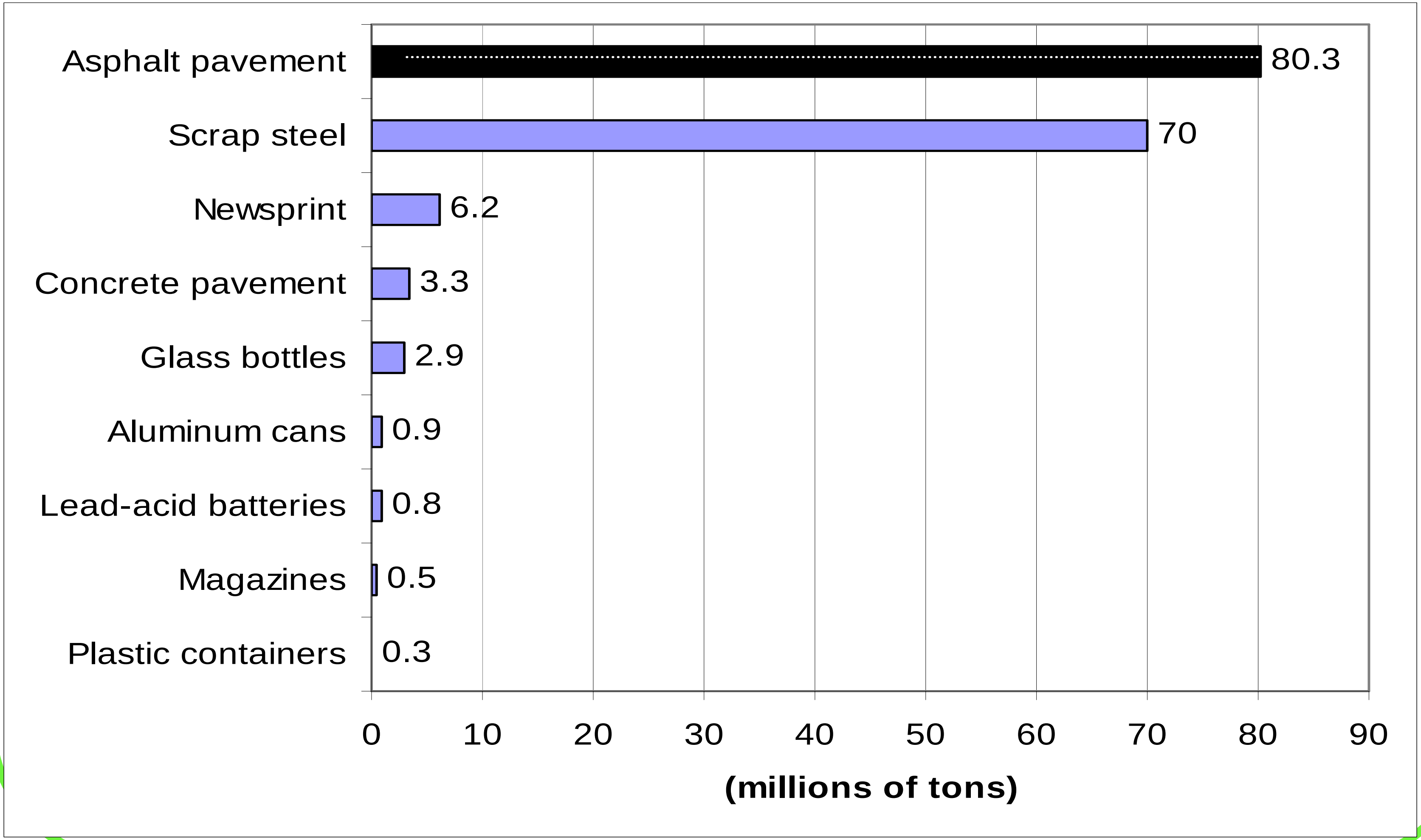
Recycling Applications

- Reclaimed Asphalt Pavement
- Recycled Concrete Aggregate
- In-Place Recycling

Why Recycle?

- Reduce project costs
- Conserve materials
- High quality aggregates unavailable
- Dwindling landfill space
- Increased disposal cost

Materials Recycling – Tons/Year



Source: APA

Asphalt Pavement Recycling

The volume of recycled asphalt pavement is….

- **13 TIMES greater than recycling of newsprint**
- **27 TIMES greater than recycling of glass bottles**
- **89 TIMES greater than recycling of aluminum cans**
- **267 TIMES greater than recycling of plastic containers**

Source: APA

What is RAP?

Aggregate ~ 95%

Asphalt Binder ~ 5%

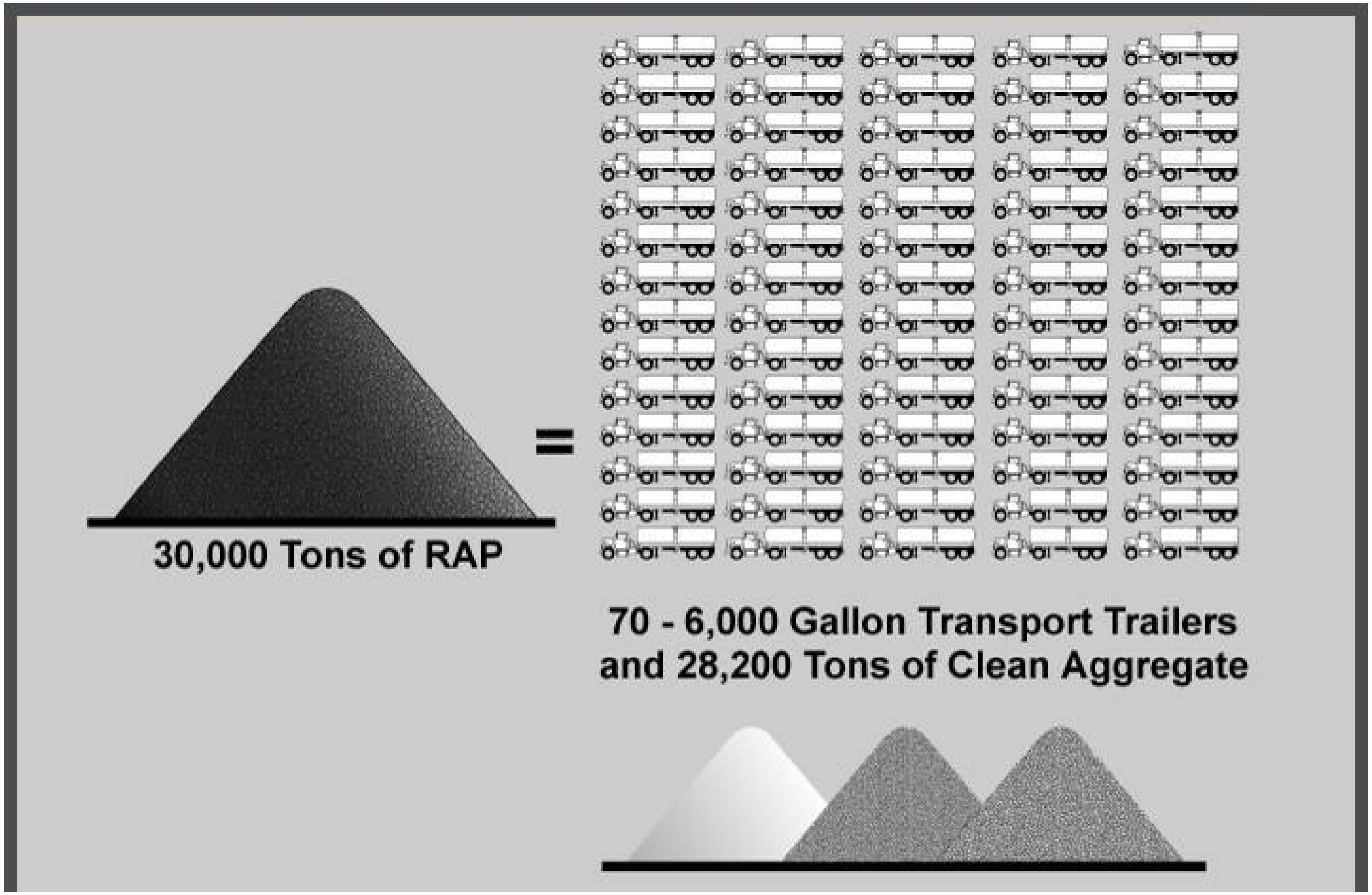
Costs / Values of RAP

- Value = Material it replaces - processing
  - Aggregate - 95% at $10/ton = $9.50
  - Asphalt - 5% at $400/ton = $20
  - Minus the Processing = $5/ton

Total Value = $24.50 per ton

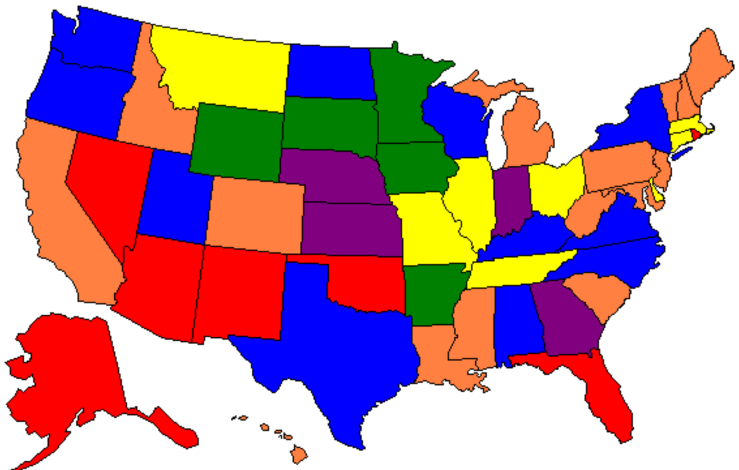
- 10% RAP saves $2.45/ton
- 20% RAP saves $4.90/ton
- 40% RAP saves $9.80/ton

Sustainability Considerations



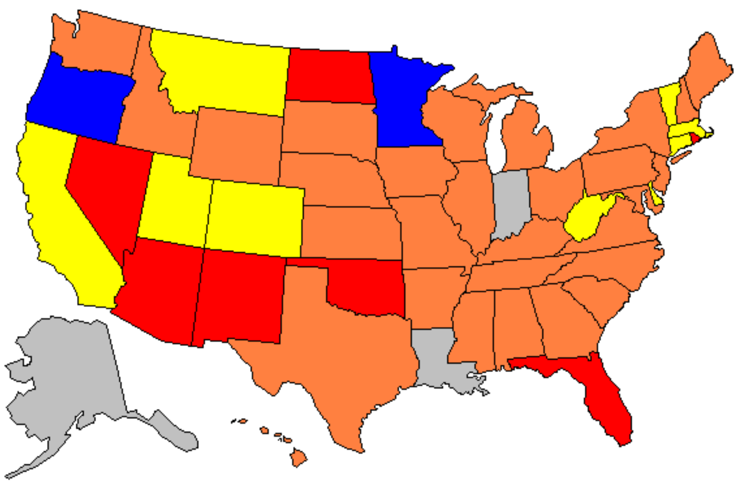
Courtesy: Astec Industries

Surface Mixes -- Specified



0%	
10%	
15%	
20%	
25%	
≥30%	
n/a	

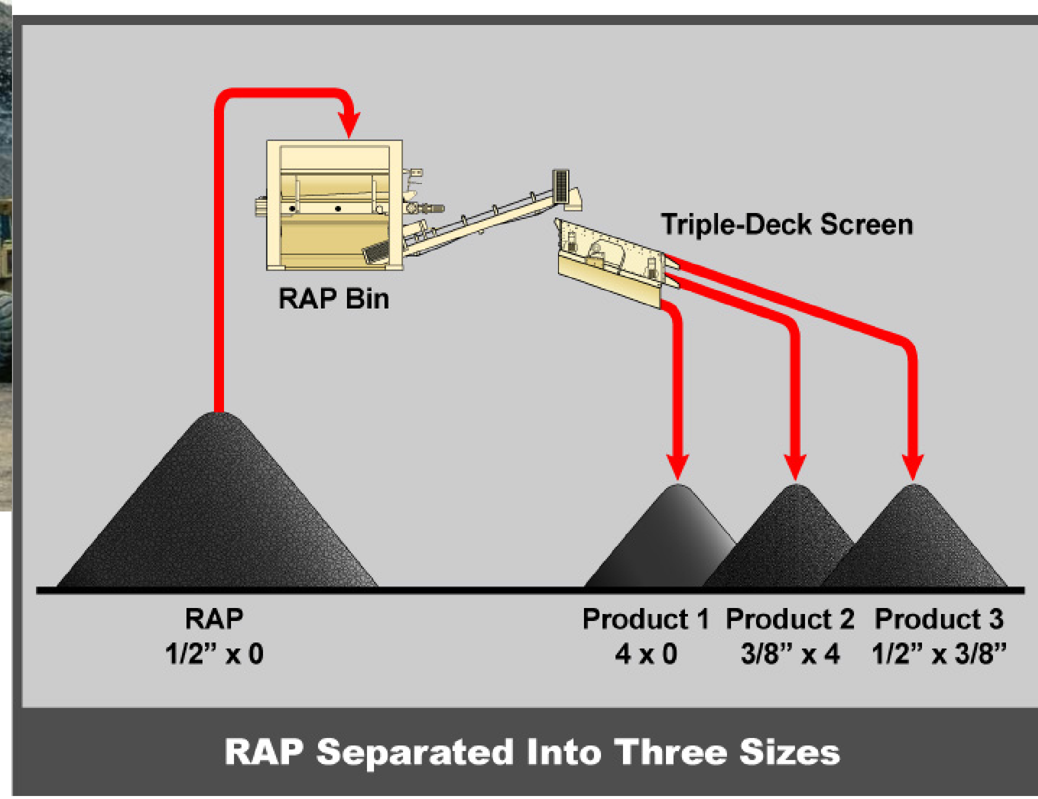
Surface Mixes -- Average Use



0%	
1 - 10%	
10 - 20%	
20 - 30%	
≥30%	
n/a	

Courtesy of NAPA

Reducing Variability



Courtesy of NAPA

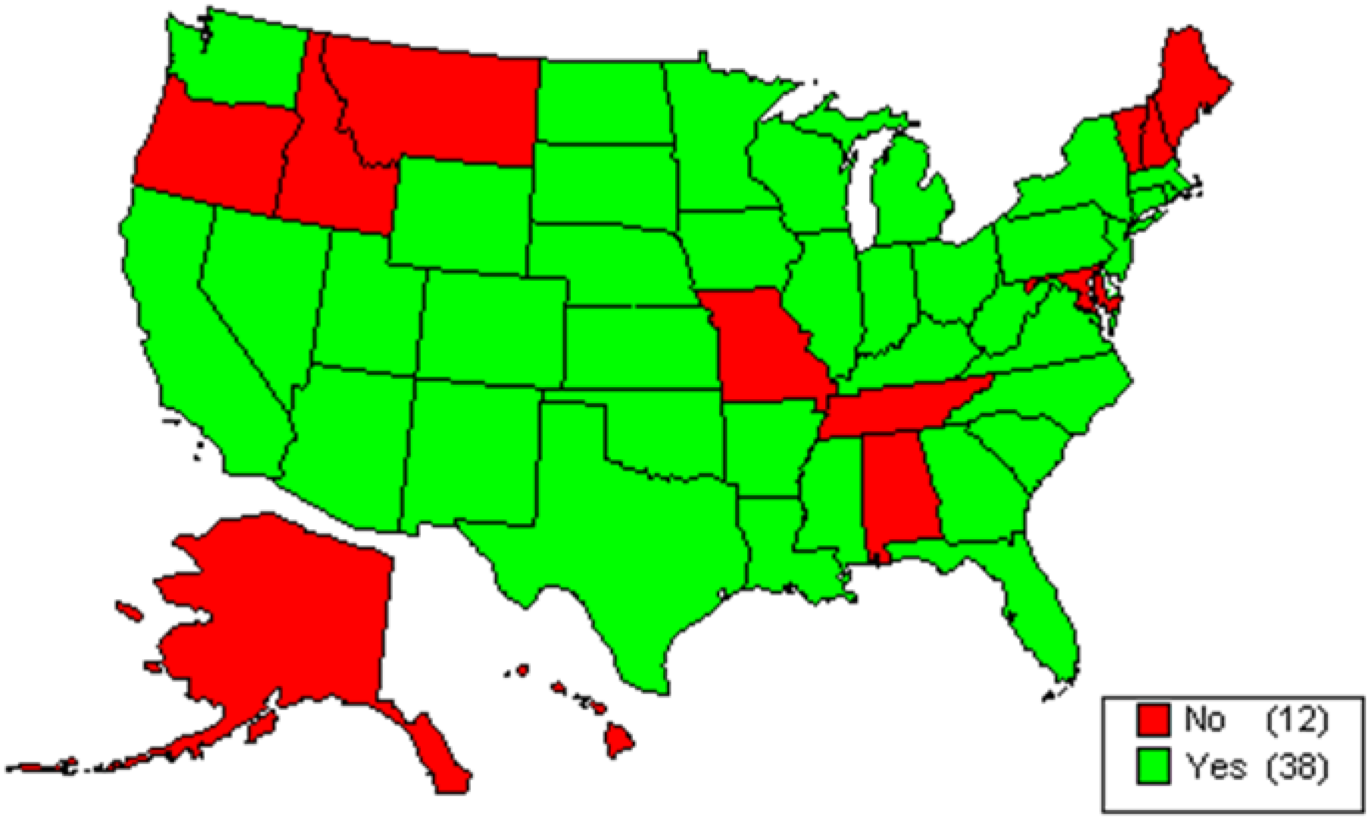
Recycled Concrete Aggregate

2004 FHWA RCA Review

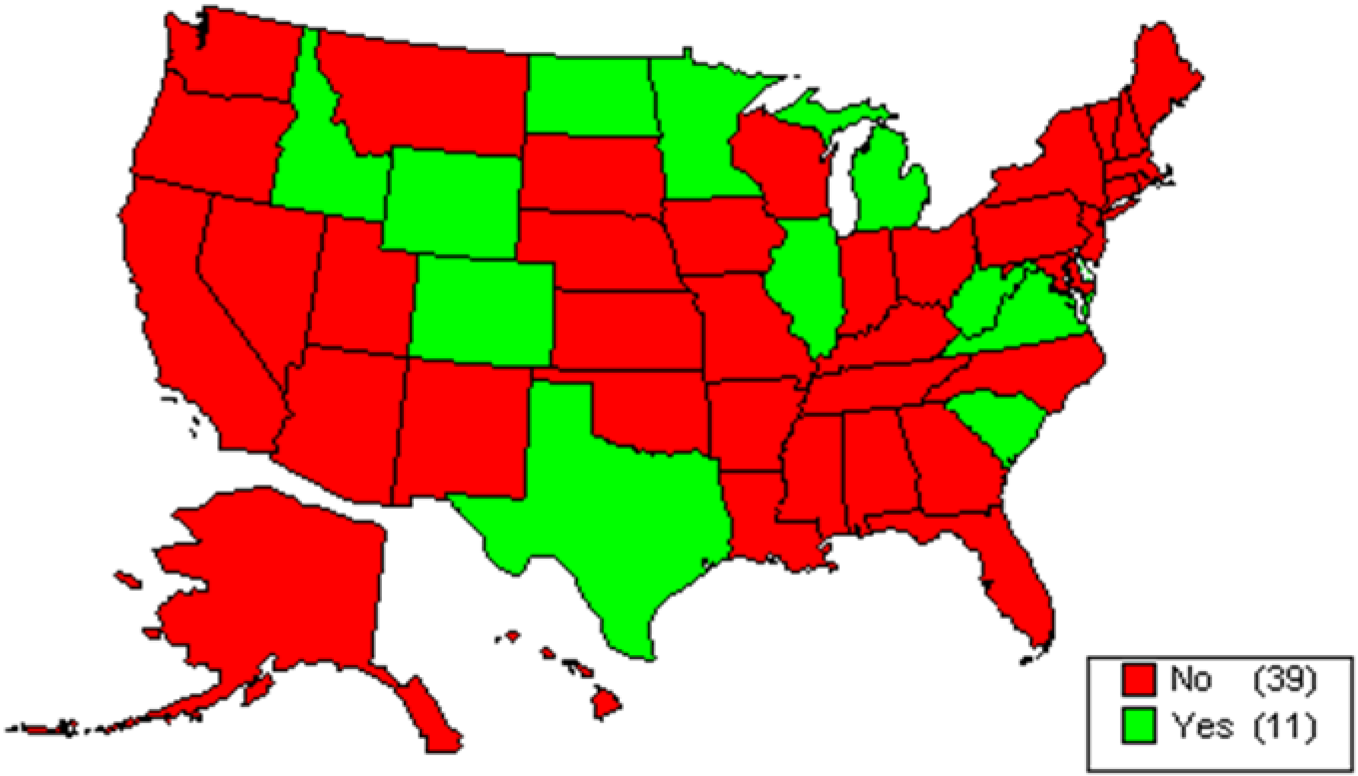
Survey and In-Depth Review of:

- Texas
- Virginia
- Michigan
- Minnesota
- California

States using RCA as Base



States using RCA in PCCP



Worlds Largest "Urban Quarry"

World's 2nd Largest Recycle Project!

El Toro MCAS

Irvine, California

Courtesy of
Recycled Materials, Inc.

Cold In-Place Recycling

Description

Milling, rejuvenating, and replacement of the top portion of the HMA surface (performed without heat)

Purpose

- **Rework HMA to depth of 2 – 4 inches.**
- **Correct surface distresses.**
- **Improve profile, crown, and cross-slope.**

2005 FHWA CIR Review

- 2005 Survey
- 41 / 52 State DOT Responses
- 20 States – No Use
- 21 States Use CIR
  - 9 States use CIR Frequently
  - 4 States have specs but use once/year
  - 3 States use only on County/local roads

Nevada DOT CIR

Lime Slurry

CIR Train

Milling

Milling Teeth

Vibratory Roller

Processed Material

Hot In-Place Recycling

Description

Milling, rejuvenating, and replacement of the top portion of the HMA surface (performed with heat)

Purpose

Rework HMA to depth of 1 to 2 inches.

Correct surface distresses.

Improve profile, crown, and cross-slope.

SUSTAINABLE TECHNOLOGIES

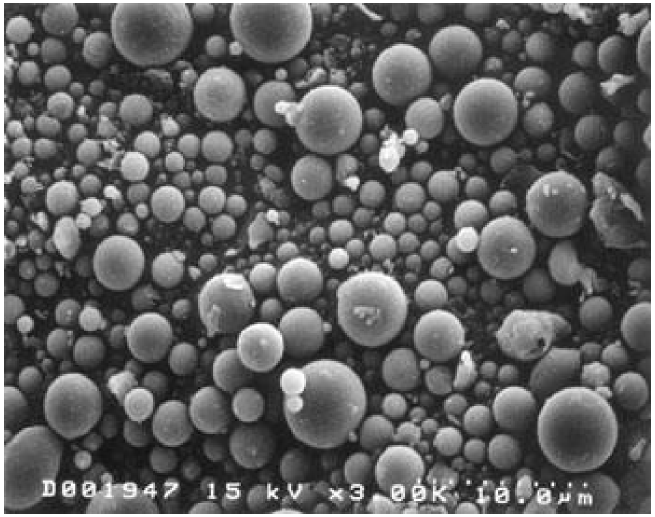
- ✓ Reclaimed Asphalt Pavement
- ✓ Recycled Concrete Aggregate
- ✓ In-Place Recycling
- REUSE APPLICATIONS
  - FLY ASH / COAL ASH
  - TIRE RUBBER
  - SHINGLES
  - SLAG
  - FOUNDRY SAND
- Warm-Mix Asphalt

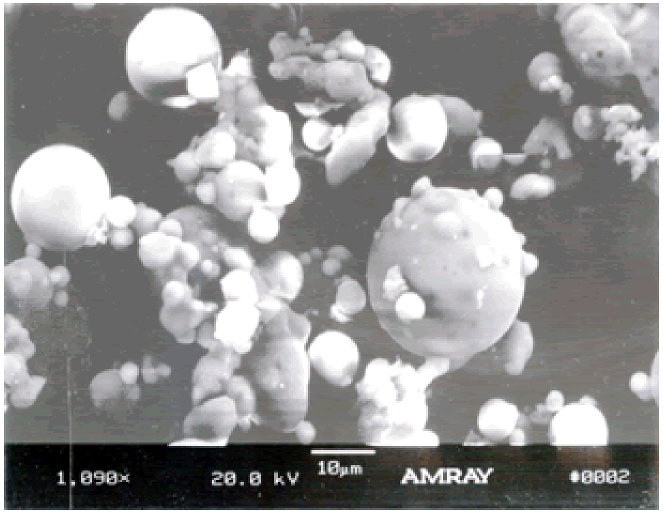
Fly Ash – Substitute for Cement

Essential Component for Durable Concrete

Fly Ash – Substitute for Cement

- Approximately 50% of US electricity is generated by coal-fueled power plants
- In 2003, over 110 million metric tons of CCP were produced
- 38% beneficially used (42 mill. metric tons)





Controlled Low Strength Material

CLSM using Class F fly ash

CLSM using Class C fly ash and sand

Soil Stabilization

Increase the structural capacity of sub-grades and road base.

TIRES

- **What Can We Do With This Mess?**
  - **280 million more are added annually.**

Tire Bales

- Block of Rubber
  - 2.5’x 4.5’x 5’
  - 2000 pounds
- 60% weight reduction over soil
- Permeable
- USES:
  - Embankments.
  - Slope repair and rock fall barriers.

Ground Tire Rubber

- Performance Properties
  - Cost effectiveness
  - Added Benefit
- User Demands
  - Noise abatement
- Sustainability
  - Recyclable in HMA?

Asphalt Roofing Shingles

- Factory rejects are recycled into high-quality pavements.
- Approved for use by North Carolina and Minnesota DOTs.
- Works well for industrial and commercial parking lots.

Slag - Reuse

- A by-product of steel production
  - Works especially well as Aggregate for high-volume roadways and/or
  - High skid-resistance applications
    - Indianapolis Motor Speedway
    - Automobile manufacturers' test tracks
  - Meets requirements for use in Superpave Aggregates

Foundry Sand - Reuse

- **Already screened, blended and ready to use in Hot Mix Asphalt**
  - **Reduces cost of sand by about 40%**
  - **100,000 tons used in HMA per year**

Reuse of Industrial Byproducts

Millions of Tons per Year used in Highway Applications

Byproduct Materials Produced	Production (million metric tons)	Recycled in Highway Applications (million metric tons)	Applications
Blast Furnace Slag	14	12.6	Concrete
Coal Bottom Ash	14.5	4.4	Asphalt, Base
Coal Fly Ash	53.5	14.6	Cement Production, Structural Fill
Foundry Sands	9 to 13.6	?	Flowable Fill, Asphalt
Cement Kiln Dust	12.9	8.3	Stabilizer
Bottom Ash	8	Small Amounts	Asphalt, Base
Nonferrous Slags	8.1	?	Base, Asphalt
Steel Slags	?	7.5	Base, Asphalt, Concrete
Recycled Asphalt Pavement	41	33	Asphalt, Base
Reclaimed Concrete	?	?	Base, Concrete

SUSTAINABLE TECHNOLOGIES

- ✓ Reclaimed Asphalt Pavement
- ✓ Recycled Concrete Aggregate
- ✓ In-Place Recycling
- ✓ REUSE APPLICATIONS
  - ✓ TIRE RUBBER
  - ✓ SHINGLES
  - ✓ SLAG
  - ✓ FOUNDRY SAND
- Warm-Mix Asphalt

Warm Mix Asphalt Benefits

- Savings in energy
- Decreased plant emissions
- Reduced exposure to fumes
- Low/No odor

www.warmmixasphalt.com

- Improved compaction
- Extended haul distances
- Extended paving season
- Higher RAP incorporation
- SAFETY
- Longer binder life?

Sustainable PP Strategies

- ☑ Sustainable Pavement Technologies:
  - ☑ Recycling
  - ☑ Reuse
  - ☑ Other Energy-Savings Technologies
- Use the technologies – help the Environment!!!
- "Keep the Good Roads Good" – Social and Engineering frameworks.
- Manage the System to Maximize Economic Returns and Value – PMS, GASB-34, Asset Management!!!

Resources for Agencies

- ☑ FHWA
- ☑ OTHER ORGANIZATIONS
- ☑ WEBSITES
- ☑ READING RECOMMENDATIONS
- ☑ COLLEAGUES / PARTNERS

FHWA Web-Based Resources

- www.fhwa.dot.gov/pavement/recycle
- www.fhwa.dot.gov/preservation

FHWA Supports Pavement Preservation!

Left to right: Associate Administrator for Infrastructure King Gee; Administrator Tom Madison; James B. Sorenson, Highway Engineer; and Executive Director Jeff Paniati.

FHWA Supports Pavement Recycling!

Other Organizations

- **Recycled Materials Resource Center**

  www.recycledmaterials.org

- **National Center for Pavement Preservation**

  www.pavementpreservation.com

and Industry Associations!

Recommended Websites/Reading

- http://www.recycledmaterials.org/tools/uguidelines/index.asp

User Guidelines for Byproducts and Secondary Use Materials in Pavement Construction

- http://www.dot.state.co.us/Publications/PDFFiles/epagrant.pdf

MATERIALS RECYCLING AND REUSE – FINDING OPPORTUNITIES IN COLORADO HIGHWAYS, October 2007

- www.greenhighways.org

Green Highways Partnership Website/Newsletter

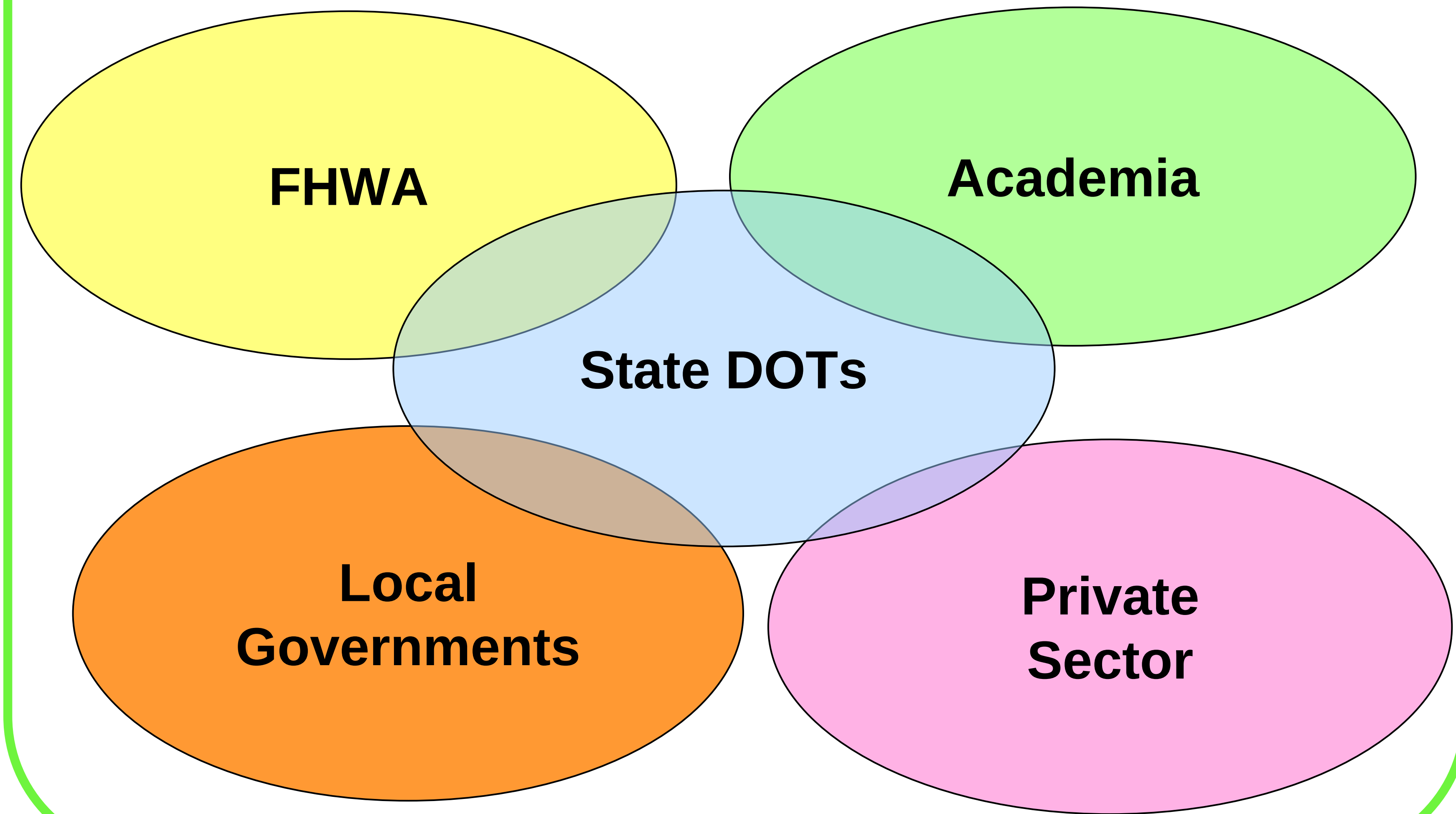
- USEPA Resource Conservation Challenge

Upcoming Opportunities

- Earth Day Webinar – April 22, 2009
  - In-Place Recycling Technologies
- In- Place Recycling Workshop
  - Minneapolis, MN
  - August 25-27, 2009
  - Includes Site Visit to Recycling Project

www.pavementpreservation.org for more info

Partnerships Are Required



THANK YOU!

Steve Mueller

FHWA Resource Center

Pavement and Materials Engineer

(720) 963-3213

Steve.Mueller@dot.gov

FHWA's Environmental Stewardship Highlights

Stephen R. Mueller, P.E.
FHWA Pavement and Materials Engineer
Steve.Mueller@dot.gov

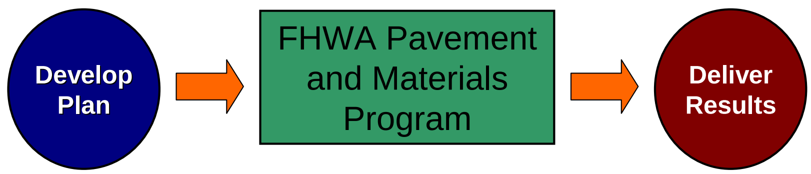
May 15, 2009 PPETG Spring Meeting – New Orleans, LA

This presentation has been prepared for the 2009 California Pavement Preservation Conference.

FHWA Pavement & Materials Program

Process to Deliver National Pavement Network That Is:

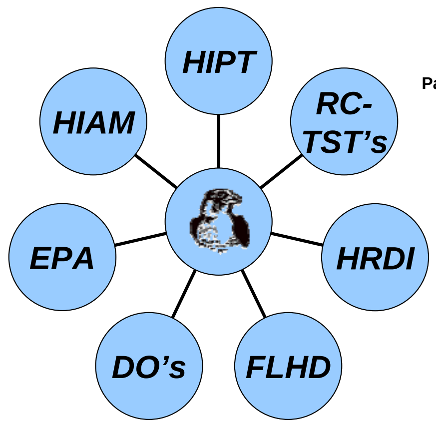
- Safe
- Cost Effective
- Long Lasting
- Effectively Maintained



Focus Area Leadership and Coordination (FALCON) Teams

1. *Pavement Design and Analysis*
2. *Pavement Materials and Construction Technology*
3. *Pavement Management and Preservation*
4. *Pavement Surface Characteristics*
5. *Construction and Materials Quality Assurance*
6. *Environmental Stewardship*

FALCON #6 Team Makeup



Resource Center
Pavement & Materials
Environment
Air Quality

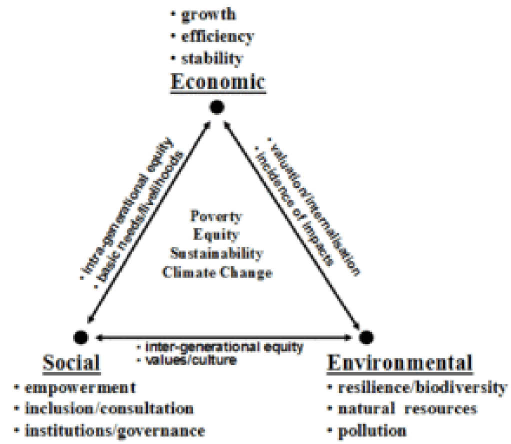
FHWA's "3 E's"

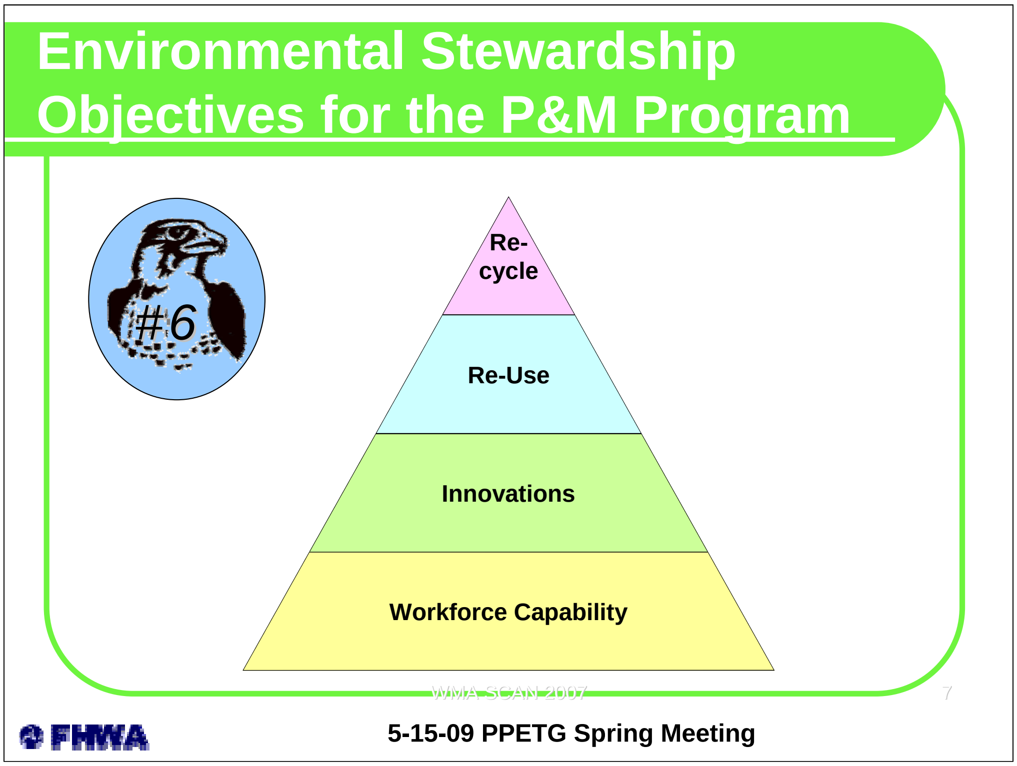
- **ENGINEERING**
  - **Use Good Engineering Design to Assure Long-Life Pavements.**
- **ECONOMICS**
  - **Use Life-Cycle Cost Analysis for Project Selection.**
- **ENVIRONMENT**
  - **Consider Recycling First**
  - **Be Good Stewards of the Environment**

"Sustainable Development Triangle"

http://www.eoearth.org/article/Basic_concepts_and_principles_of_sustainomics

Social, Environmental, Economic





The European Union is adopting standards for roadway material in 2008. The standards address test methods and broad mix descriptions to allow the European nations to "speak the same language." There are both empirical and fundamental standards for HMA. Long term, they would like performance standards that address surface characteristics and eventually user needs.

Key Points of FHWA Recycling Policy
2-7-2002

- Recycled materials should get <u>first</u> consideration in overall materials selection.
- Recycling *(and Reuse)* can offer engineering, economic and environmental benefits.

FHWA **5-15-09 PPETG Spring Meeting**

Engineering – if you ask an civil engineer to fix it, make it work, try it out – you can get some interesting innovations. Many restrictions due to “don’t rock the boat” not in my back yard…

With US EPA/FHWA working to smooth out some of the bumps – offering reviews, technical information, assessment of impacts – all will help

COST - Factor in savings from fuel (digging new materials and their hauling) to re-use of existing industrial by-products. And the impact to landfill for not re-using – life cycle cost and environmental issues need to be factored into material selection process.

These materials are not “first time uses” most are tried and true. Generally asking for a more focused look from owner agencies to review their specs, consider the use of materials in the early stages of project development. Hard to incorporate material as change order.

We ask the DOTs to work with their contractors, industry and DEQ to review specs, permit process -

Key Points of FHWA Recycling Policy

2-7-2002

- **FHWA recognize the need to increase our highway industry's overall use of recycled materials**
- **Forge partnerships among government, industry, and academia**
- **Continue to strengthen the relationship between FHWA, US EPA, and State DOT/DEQ**
- www.fhwa.dot.gov/legsregs/directives/policy/recmatmemo.htm

FHWA 5-15-09 PPETG Spring Meeting

Discuss the need to have partnerships with Federal Govt, -US EPA to help in the environmental assessments

State DOTs and their DEQ/EPA also need to be partners

Recycled Asphalt has been and is a target for FHWA to work on – clean up our house first. RCA is also a target.

2010 Emphasis Topics

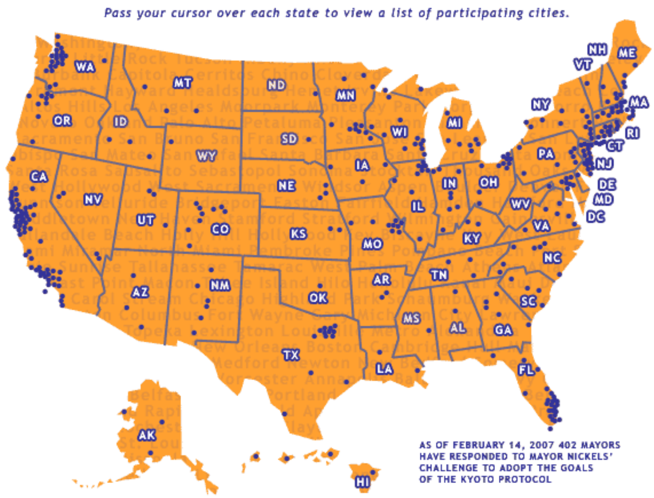
- Recycled/Re-Use Materials
- Warm Mix Asphalt
- Extend Binders and Cement
- Environmental Impacts

Environmental Playing Field
(It's Big – and Growing!)

- Major Citizen Concern
  - Kyoto Protocols
  - Federal Legislation
- Sustainability
  - Measurement Issues

402 US Regional Kyoto Commitments

2-14-07

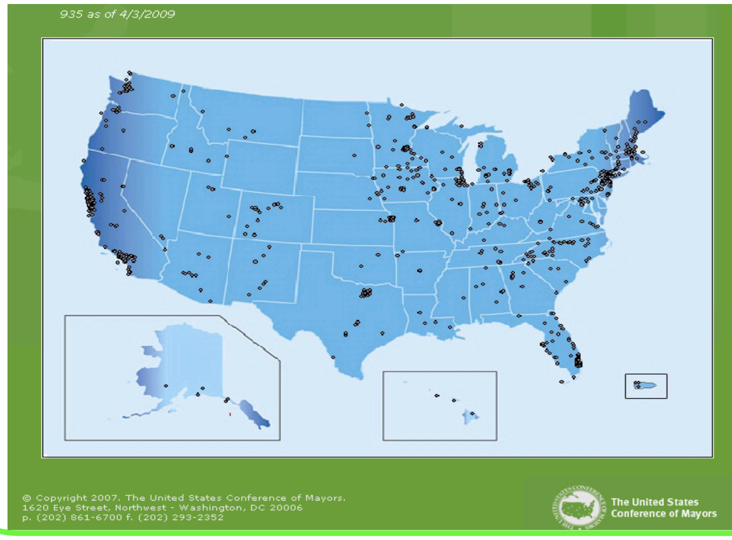


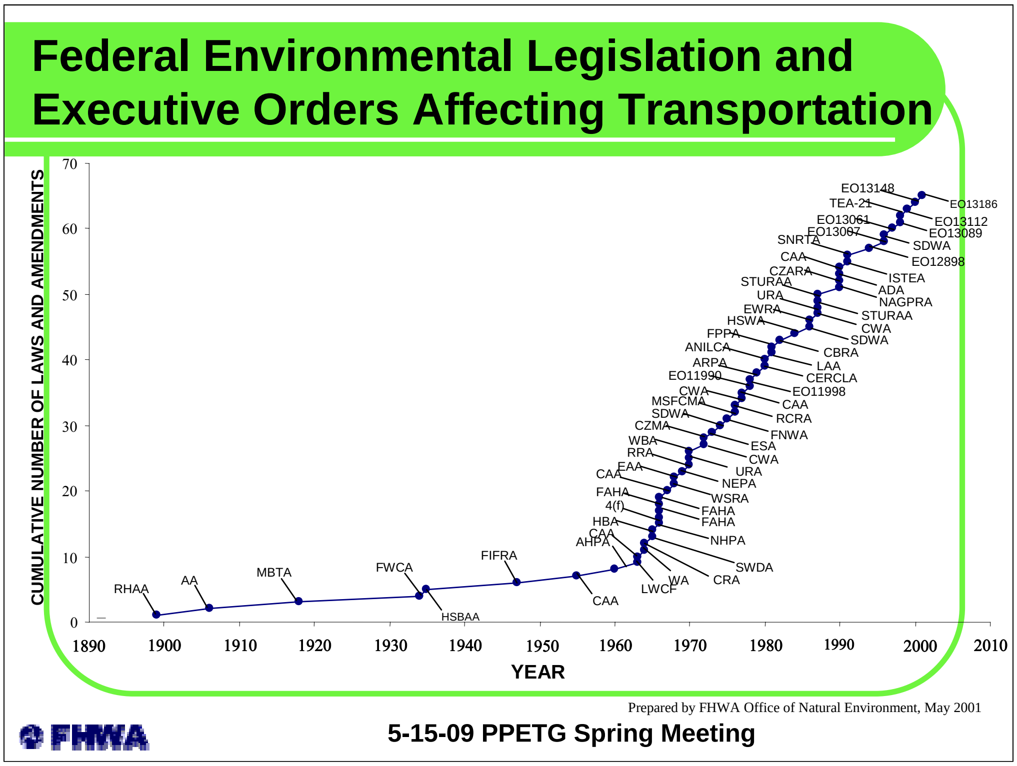
FHWA

5-15-09 PPETG Spring Meeting

935 US Regional Kyoto Commitments

4-3-09 Source: http://usmayors.org/climateprotection/map.asp





One can quickly see that since NEPA – National Environmental Policy Act was made law in 1969, there have been a number of additional federal legislation and Executive Orders pertaining to addressing environmental natural and human resources. Over 70 federal requirements total but with a majority of them being invoked after 1969.

EPA’s Sustainability Definition

Sustainability **means “meeting the needs of the present without compromising the ability of future generations to meet their own needs.”**

"Green Buildings"
Environmental Rating Systems

1999 CIWMB Plan
http://www.ciwmb.ca.gov/greenBuilding

- Siting
- Energy Efficiency
- Materials Efficiency Use of Recycled Materials
- Water Efficiency
- Occupant Health and Safety
- Building Operations and Maintenance

Also: Leadership in Energy and Environmental Design (LEED)

"Greenroads" Rating System
www.greenroads.us

Category	Description	points
Project Requirements	Minimum requirements for a Greenroad	11
Environment & Water	Stormwater, habitat, vegetation	14
Access & Equity	Modal access, culture, aesthetics, safety	14
Construction Activities	Construction equipment, quality, use	15
Materials & Resources	Material extraction, processing, transport	12
Pavement Technology	Pavement design, material use, function	11
Custom Credits	Write your own credit for approval	10
	Total	76

Please Note: There are Many Rating Systems, and FHWA does not endorse a national rating system.

FHWA

5-15-09 PPETG Spring Meeting

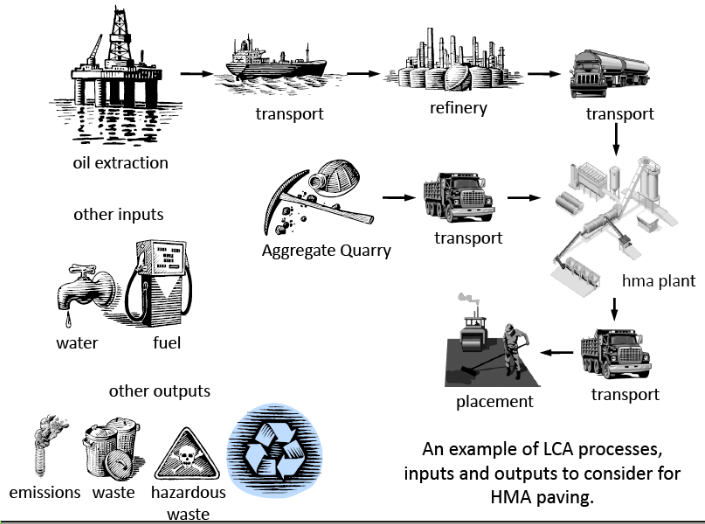
"Greenroads" Certification Levels

Version 0.96: 76 Total Points

Greenroads certified	Greenroads certified SILVER	Greenroads certified GOLD	Greenroads certified EVERGREEN
23-30 points	31-38 points	39-46 points	47+ points
30-40% of total	40-50% of total	50-60% of total	60+% of total

Please Note: There are Many Rating Systems, and FHWA does not endorse a national rating system.

"Greenroads" – LCA in HMA



An example of LCA processes, inputs and outputs to consider for HMA paving.

FHWA

5-15-09 PPETG Spring Meeting

FHWA Supports Pavement Recycling!

FHWA 5-15-09 PPETG Spring Meeting

Jason Harrington, FHWA Recycling Technology Engineer (Center), receives the Closing the Circle Award at the White House in 2008 for the FHWA Recycling Team.

Also present were Pete Stephanos, Director of the Office of Pavement Technology and John D'Angelo, Asphalt Team Leader, and the two White House staff presenting the award.

It's Good to be GREEN!

Note: The source of this image is unknown – do not use in other presentations.

FHWA **5-15-09 PPETG Spring Meeting**

Jason Harrington, FHWA Recycling Technology Engineer (Center), receives the Closing the Circle Award at the White House in 2008 for the FHWA Recycling Team.

Also present were Pete Stephanos, Director of the Office of Pavement Technology and John D'Angelo, Asphalt Team Leader, and the two White House staff presenting the award.

PPETG Emulsion Task Force Meeting Update

May 14-15, 2009

New Orleans, LA

PPETG Emulsion Task Force Update

- New Members since last meeting
  - Jeff Seiders, Darren Hazlett from Texas DOT
  - Fred Mello- BASF
  - Chris Lubbers- Kraton Polymers

ETF Subcommittee Updates

- Emulsion Testing & Residue Recovery Methods – Arlis Kadrmas
  - Identified need for preparation of Research Needs Statement
    - Comparison of various techniques and variability in tests
    - Amy Epps-Martin and Laurand Lewandowski to prepare RNS

ETF Subcommittee Updates

- Emulsion Testing & Residue Recovery Methods (Cont.)
  - Acknowledged ASTM Emulsion Viscosity inter laboratory study that Delmar Salomon is coordinating
    - Cannon Paddle Viscometer vs. Saybolt Furol
  - Darren Hazlett presented TXDOT Test
    - 6 hour recovery on residue vs. 48hr
    - Will gather info to prepare procedure
    - Compared Variability for Distillation, 48Hr, 6Hr data

ETF Subcommittee Updates

- Emulsion Testing & Residue Recovery Methods Cont.)
  - ASTM D7497 Method issued for Evaporative Recovery Method
  - Agreed to submit to AASHTO as a dual procedure method by including TXDOT 6 hour method for Provisional Specification

ETF Subcommittee Updates

- Residue Tests- Gayle King
  - PAV Aging- identified need to move forward on procedural development based upon promising work at PRI
    - Pans need additional development work

ETF Subcommittee Updates

- Residue Tests (Cont.)
  - DSR Strain Sweep- Work by Hussein Bahia at U of Wisconsin, Madison
    - Need to establish connection to Raveling vs. Lab predictive test
    - Promising but Validation now needed
  - MSCR-routine general acceptance for polymer presence identification
  - High Floats being evaluated via Harmonic Method of Rheology Analysis

ETF Subcommittee Updates

- Residue Tests (Cont.)
  - Frequency Sweep- will keep in their study
  - Low Temperature Testing
    - 2 competing methods
      - Uof Wisc. Method
      - WRI- DSR 4mm plate

ETF Subcommittee Updates

- Aggregates, Mix Design, and Performance Tests- Jim Moulthrop
  - Jim Moulthrop has resigned as subcommittee chair
  - Mary Stroup-Gardiner Interim SC
  - Pooled Fund Study on "Microsurfacing Mix Design" not complete

ETF Subcommittee Updates

- Approved Supplier Certification- Roger Hayner
  - Draft Provisional Specification in AASHTO format for review
    - Modeled from R-26 for PG Binders
  - Upon review and comments from ETF members will present to AI TAC and AEMA ITC for review and comment
  - Will require updating with Performance Specifications as developed

ETF Subcommittee Updates

- Inspection & Acceptance- Colin Franco
  - Reviewed Spreadsheet on Needs
    - Material Engineering & Design, Manufacturing of Emulsions, Design & Pre-Construction Engineering, Construction, In-Service Performance & Durability
  - Initial focus on Chipseals and Microsurfacing

ETF Subcommittee Updates

- Inspection & Acceptance (Cont.)
  - Preparing compendium of tests from various groups; ASTM, AASHTO, etc.
  - Will review ongoing research to identify if any fill gaps- New Performance Tests
  - Preparing survey to gather industry and agency input
  - Working to identify critical tests in each step of the process

ETF Research Project Reviews

- Asphalt Research Consortium- Hussein Bahia/Andrew Hanz, Univ. Wisconsin
  - Focusing on Chipseals and Slurry/Microsurfacing on Initial Work
  - Identifying Specific Properties
    - Construction- Storage Stability, Sprayability and Drain out, Breaking and Setting Properties, Early Raveling and Chip Retention

ETF Research Project Reviews

- Asphalt Research Consortium (Cont.)
  - In Service Residue Properties- Bleeding, Raveling, Fatigue Cracking, Thermal Cracking
  - Aggregate Selection- Wear, Soundness, Surface Chemistry Effects
  - Curing Rates- Multiple emulsion types and modifiers, recently added SBR modified

ETF Research Project Reviews

- Asphalt Research Consortium (Cont.)
  - Patti Adhesion Test Developed
    - Adhesion on various Aggregate slabs at different temperature/humidity and cure times (2, 6, and 24 hr)
    - Pneumatic loading to measure pull of strength at failure
    - Promising but further work needed

ETF Research Project Reviews

- Asphalt Research Consortium (Cont.)
  - Evaluating Residue on DSR
    - Strain Sweeps
    - MSCR, % Recovery
  - Work continuing in Conjunction with NCHRP 14-17 Project and Federal Lands Study Researchers

ETF Research Project Reviews

- NCHRP 14-17 Chipseal Evaluation- Amy Epps-Martin
  - Evaluation of 8 emulsions at 3 field sites
  - Comparison of Residue Recovery
    - Stirred Can vs. Hot Oven Evaporation
    - Residue analysis includes Strain Sweep
      - Parameters still need defined- limits

ETF Research Project Reviews

- Chipseal Evaluation NCHRP 14-17 (Cont.)
- Effects of Surface Texture
  - 3 field sites using CT Meter and Sand Patch Test
  - Very Good correlation between CT and Sand Patch Test
  - Evaluating low, medium, and high surface profiles and effects on emulsion requirements

ETF Research Reviews

- Federal Lands Study Update- Gayle King/Laurand Lewandowski
  - Performance Based Testing from FLH Study
    - Addresses current specifications failure to correlate to field performance
    - More Data produced variations calling for further research

ETF Research Reviews

- Federal Lands Study Update (Cont.)
  - Evaluating effects of
    - Aging
    - Rheological Correlations to bleeding, shelling, raveling, chip retention
  - Using data thus far to put together research needs statement for further study

ETF Research Reviews

- Emulsion Training Program Update- Mary Stroup-Gardiner
  - NHI Course Emulsion Training- "Emulsion Course Guideline"
  - Outlined 9 modules
    - Intro to Highway Applications through Emulsions for the 21st Century
    - Needs for Basic Fundamental Best Practices

ETF Research Reviews

- AI/AEMA MS-19 Basic Asphalt Emulsion Handbook Review- Roger Hayner
  - Fourth Edition Reviewed by 5 member panel from ETF
  - Revisions and comments complete
  - Submission by Monday next week

ETF Research Reviews

- Federal Lands Polymer Modified Emulsions Handbook- Helen King
  - Reviewed Handbook from FLH Project
  - 20 page Guide prepared for field training use
    - Project Development & Maintenance Engineer use
    - Agencies as APWA, NACE, LTAP, AASHTO

ETF Action Items

- Prepare Evaporation Recovery Method Provisional Specification for AASHTO submittal with Methods A and B
- Prepare Research Needs Statement for Pooled Funds Study for Performance Criteria on Methods A and B
- Prepare Research Needs Statement for AASHTO/NCHRP Submission continuing work from FLH Study
- Review and Revise Draft of Approved Supplier Certification Provisional Standard for AASHTO submission

ETF Action Items

- Focus Efforts on AASHTO/NCHRP submission of Provisional Methods and Research Needs Statements for funding ongoing research and field study needs through Colin Franco and Jeff Seiders
- Seek ETF Member Panel Review Positions on NCHRP projects involving Asphalt Emulsions and processes

ETF Meeting Notes

Emulsion Task Force Meeting Notes

Meeting Dates:	May 14, 1:00pm – 5:00pm
	May 15, 8:00am – Noon
	May 15, 1:00pm – 3:00pm Join PPETG

Standard Recommended Practice for

Certifying Suppliers of Emulsified Asphalt

AASHTO Designation: x xx-xx (2009)

1. SCOPE

1.1 This standard specifies requirements and procedures for a certification system that shall be applicable to all suppliers of emulsified asphalts (EA). The requirements and procedures shall apply to materials that meet the requirements of either AASHTO M 140 or AASHTO M 208/M 316, (Cationic Emulsions/Polymer Modified Cationic Emulsions), which are manufactured at refineries, terminals, in-line blended, or otherwise produced for use in paving applications.

1.2. This standard may involve hazardous materials, operations, and equipment. It does not purport to address all of the safety problems associated with this use. The user of this standard shall be responsible for appropriate safety and health practices.

2. REFERENCED OR RELATED DOCUMENTS

2.1. *AASHTO Standards:*

- M140, Standard Specification for Emulsified Asphalt
- M208, Standard Specifications for Cationic Emulsified Asphalt
- M316, Standard Specifications for Polymer Modified Cationic Emulsified Asphalt
- T 59, Standard Method of Tests for Emulsified Asphalts
- T 40, Sampling Bituminous Materials

2.2. *ASTM Standards:*

- D244
- D977
- D7497
- D3665

3. TERMINOLOGY

3.1 AAP—AASHTO accreditation program.

3.2. ASC—approved supplier certification.

3.3. AS—approved supplier.

3.4. EA—emulsified asphalt.

3.5. *supplier—a supplier* shall be defined as one who produces the final product or who makes the blend or modification that alters the properties of the EA specified in either M140 or M208/M 316. A supplier shall be a refinery, a terminal, or an paving contractor. If no modification is made to the EA after its initial production at the refinery or terminal, the refinery or terminal shall be the supplier and must provide the certification. If any modifications are made to the EA at the terminal, the terminal shall be the supplier and must provide certification. If any modification, blending, or blending of EA from different sources is made at the job site, the Paving contractor shall be the supplier and must provide the certification.

3.6. *agency*—agency shall be defined as a state highway agency or other agency responsible for the final acceptance of the EA.

3.7. *specification compliance* testing—complete testing in accordance with the either M 140 or M208/M 316 specification requirements. (Update for new specifications as they come available)

3.8. *quality control testing*—the quality control testing shall be described in the supplier's quality control plan. The supplier's quality control plan shall be approved by the agency.

4. SIGNIFICANCE AND USE

4.1. This standard specifies procedures for minimizing disruption of EA shipments. This is accomplished by a certification system which evaluates quality control and specification compliance tests performed by the supplier on samples obtained prior to shipment.

4.2. The number of EA available under M 140 or M 208/M 316 may require construction of additional storage facilities if the procedure of "sample and hold while testing" is followed exclusively. The addition of new storage capacity at a facility may be infeasible at some locations. Standardization of procedures that allow shipment under an approved supplier certification provides the flexibility needed to use existing facilities and to limit the shipment disruptions.

4.3. This standard provides information on the following activities:

4.3.1. General requirements that the supplier shall satisfy to be given approved-supplier status;

4.32. Minimum requirements that shall be included in a supplier's quality control plan;

4.3.3. General requirements that the agency shall satisfy before certification;

4.3.4. Procedure for shipping EA under an ASC system;

4.3.5. Procedure for agency monitoring of an ASC system at the shipping facility; and

4.3.6. Procedure for field sampling and testing of EA shipped under an ASC system.

5. HAZARDS

5.1. The safety requirements of the field and/or laboratory organization and/or OSHA shall be observed.

6. SAMPLING

6.1 All test samples required by this standard shall be obtained in accordance with T 40 and ASTM D 3665. The use of a random sampling procedure is mandatory to the establishment of a valid certification program.

7. TESTING REQUIREMENTS

7.1. Testing required for this standard shall be performed by a laboratory currently accredited by the AAP and be performed according to T 59. Any satellite laboratory of a supplier that performs required testing under this standard shall be identified in the AS quality control plan (Section 9) and shall be approved by the agency.

Note 1—Cost of this inspection shall be borne by the source of data. Satellite laboratories may be inspected by the source's primary AMRL inspected laboratory staff. A copy of the report of the satellite laboratory inspection shall be provided with the test report, if requested.

Note 2- Agencies may elect to allow participation in AMRL Proficiency Sample Program and regional round robin programs in lieu of AMRL accreditation until such time as Supplier may reasonably become AMRL Accredited for AE.

8. SUPPLIER REQUIREMENTS

8.1. The supplier shall submit a written request to the agency for authorization to ship EA under the ASC system and shall list the EA to which the request applies.

8.2. The supplier shall allow the agency to visit the production and/or shipping site to observe the supplier's quality control activities, to inspect the facilities and to obtain samples for test.

8.3. The supplier shall submit to the agency for approval a complete quality control plan, which complies with the requirements of Section 9.

8.4. The supplier shall follow the procedures described in the approved quality control plan.

8.5. The supplier shall establish a continuing test record for each test required on each EA included in the written request prepared to satisfy the requirements of Section 8.1.

8.6. The supplier shall forward to the agency the initial series of test data for each EA included in the written request prepared to satisfy the requirements of Section 8.1. The supplier shall also obtain and provide a split sample for the agency if requested.

8.7. The supplier shall submit to the agency all reports required by this standard in a form approved by the agency.

8.8. The supplier shall have a satisfactory record of compliance with governing specifications. Judgments by the agency concerning this requirement shall be based on the test results furnished by the supplier and satisfactory results when the monitoring and field tests are compared with supplier tests.

9. SUPPLIER QUALITY CONTROL PLAN—MINIMUM REQUIREMENTS

9.1. The supplier's quality control plan shall identify the following:

9.1.1. Facility type (refinery, terminal, manufacturing site);

9.1.2. Facility location;

9.1.3. Name and telephone number of the person responsible for quality control at the facility;

9.1.4. The quality control tests to be performed on each EA; and

9.1.5. Name and location of the laboratory performing quality control tests on the EA that is shipped.

9.2. The supplier's quality control plan shall include a declaration stating that if a test result indicates that a shipment of EA is not in compliance with the purchase specifications, the supplier shall (1) immediately notify the agency of the shipment in question, (2) identify the material, (3) cease shipment until material complies with the specification, (4) notify the agency prior to resuming shipment; and (5) implement any mutually agreed upon procedures for the disposition of the material. In the event a mutual agreement is not obtained, the specifying agency shall have final authority in the decision on specification compliance.

9.3. The supplier's quality control plan shall describe method and frequency for initial testing, quality control testing and specification compliance testing.

9.3.1. Initial Testing - For each grade of EA to be supplied, specification compliance testing (complete testing per either M140 or M208/M 316) shall be performed for at least three consecutive lots. A lot may be a fixed batch of material or a specified quantity in a continuous operation (see Note 3). The supplier and the agency shall agree on a lot size. The agency must approve any change to a lot size.

Note 2 - If a batch operation is used to manufacture the EA, a tank may be defined as a lot. Lot size would be the amount of material batched into the tank. If a continuous process (in-line blending or a shipment from "live" tanks) is used to manufacture the EA, lot size may be obtained at random during the production for continuous operations. Lot size shall depend on the production method used and the quantity of the EA produced.

9.3.2. *Reduced Frequency of Testing for Specification Compliance*—If approved by the agency, the frequency of testing for specification compliance may be decreased if the individual M 140 (or M 208/M 316) test result for every sample of the initial testing is within specification by at least the tolerance of the test method for each of the required test methods. With the approval of the agency, the frequency of testing may be further reduced as long as the individual test results continue to meet the tolerance criterion. If the tolerance criterion is not met, every lot will continue to be tested for the individual M 140 (or M208/M 316) property until three consecutive lots comply with the tolerance criterion.

9.3.3. *Minimum Frequency*—Specification compliance testing shall be run at the minimum frequency required by the agency for each EA that is supplied.

9.4. The supplier's quality control plan shall include a statement that the supplier will prepare monthly summary reports for all quality control and specification compliance tests performed during that period and will submit them to the agency on request.

9.5. The supplier quality control plan shall provide an outline of the procedure to be followed for checking transport vehicles before loading to prevent contamination of shipments. The outline shall include a statement that the transport vehicle inspection report, signed by the responsible inspector, shall be maintained in the supplier's records and will be made available to the agency on request.

10. AGENCY REQUIREMENTS

10.1. The agency shall verify that the supplier's quality control plan is adequate. The agency may visit the shipping site when required.

10.2. The agency shall notify the supplier that the supplier's application for AS status has been granted. The notification shall include a list of the EA covered.

10.3. The agency shall verify that the supplier's primary testing laboratory is currently AASHTO accredited or has met suitable requirements until accreditation may be reasonably obtained.

10.4. The agency may perform split sample testing in accordance with Section 12.

10.5. The agency may perform quality assurance sampling and testing in accordance with Section 13.

10.6. The agency shall authorize shipment of each listed EA under the ASC system only after all ASC system requirements have been satisfied.

10.7. The agency shall inspect the operations of the supplier's facility related to the EA shipments when required.

10.8. The agency shall notify the supplier when split sample data versus supplier sample data does not compare within the limits established in Sections 12 and 13.

Note 3—The supplier and/or the paving contractor may take a split sample for comparison purposes, If a split sample is taken, a third sample shall be taken as a referee. The referee sample shall be retained either by the agency or by the paving contractor until the test results are available. If the test results are disputed, the agency and supplier shall agree upon a test procedure for the referee sample.

11. REQUIREMENTS FOR SHIPPING EA BY AN APPROVED SUPPLIER

11.1. The supplier's quality control plan as approved by the agency shall be implemented. (See Section 9,) 11.2. The supplier shall make EA shipments covered by the certification as dictated by shipping schedules.

11.3. Each shipment shall be accompanied by two copies of the bill of lading, which shall include (1) the name and location of the supplier, (2) the performance grade of material, (3) the quantity of material shipped, (4) the date of shipment, (5) *a statement certifying the material meets specification requirements, and* (6) a statement certifying that the transport vehicle was inspected before loading and was found acceptable for the material shipped.

Note 4—On any invoice or Bill of Lading, it is recommended that US Gallons or Short Tons be used as the primary units of measurement.

11.4. If the specification compliance test results do not conform to EA specifications, the supplier shall remove the non-compliant material from the shipping queue as outlined in Section 9.2.

11.6. Based on the agency's split sample testing on the referee sample (see Note 7), price adjustment may be made for material that does not comply to the specified EA requirements. The price adjustment shall be determined by the agency. If problems with the EA recur at the job site, the agency may suspend use of the EA until the cause for noncompliance with specifications can be identified and corrected.

12. SPLIT SAMPLE TESTING

12.1. The agency may test split samples that are obtained at random from the supplier's facility.

Note 5—Split samples will be obtained from the same general points in the supplier's shipping process that the supplier's samples are taken, for example, from a storage tank at the refinery, from a holding tank at a terminal, or from a loading line downstream from the blending operation of an in-line blending process.

12.2. The agency shall determine the frequency of split sample testing.

12.3. If the split sample data and the supplier test data are not within the test tolerance specified (see Section 15), an immediate investigation shall be conducted to determine the reason for the difference between the data. Unless available facts indicate otherwise, the investigation shall include a review of sampling and testing procedures of both supplier and agency.

13. FIELD SAMPLING

13.1. The agency or Paving contractor may design the field-sampling plan to accomplish the intended purpose,

Note 6—Field samples may be taken for several different purposes: to determine the type and magnitude of any changes in the properties of theEA during transportation and storage; to determine that the material received in the field is the material ordered; or to verify that the quality control/quality assurance system is performing as intended.

13.2. The agency may obtain samples from the field facility on a random basis for the purpose of quality assurance.

13.3. The agency shall determine a minimum frequency of field sampling that shall be adequate to satisfy the purpose for which the field samples are taken, 13.4. If the field test data are not within tolerance, the agency shall immediately notify the approved supplier and paving contractor. Unless available facts indicate otherwise, an investigation shall be conducted that shall include a review of quality control and sampling and testing procedures for field sampling and split sampling. When the differences are not readily resolved, all facts available to identify the problem shall be used to decide on an appropriate course of corrective action.

13.5 If the EA fails to comply with the specification, the supplier or paving contractor producer shall immediately investigate the possibility of contamination in transport vehicles, field storage tanks, pumps, lines and at handling facilities. If the cause is determined, correction shall be made promptly. If field test data show a serious departure from the specifications, the supplier or paving contractor shall delay the project work pending corrective action.

14. REPORT AND DATA SHEETS

14. 1. *Supplier Reports*—The supplier shall prepare the reports described in Sections 8.1, 8.3, 8,6, 8.7, 9.2, 9.4, 9.5, 11.2, and 11.3.

14.2, *Agency Reports*—The supplier may request copies of the split sample test results and field test data.

15. KEYWORDS

15. 1 Approved Supplier (AS); Approved Supplier Certification (ASC); certification system; certified shipments; Emulsified Asphalt (EA) Certification.

Combined State Binder Group

Method of Acceptance for Asphalt Binders

March 2007

Iowa Department of Transportation
Minnesota Department of Transportation
Nebraska Department of Roads
North Dakota Department of Transportation
South Dakota Department of Transportation
Wisconsin Department of Transportation

Revisions to the 2007 edition of the Combined State Binder Group Method of Acceptance for Asphalt Binders are listed below.

Page 1. Michigan DOT removed.
Section VI. Page 6. Michigan DOT removed.
Section VI. Page 6. North Dakota DOT email address update.
Section VII. Page 7. Iowa DOT requirement for project number on bill of lading.
Section VIII.A. Page 7. Iowa DOT sampling rate.
Section VIII.A. Page 8. Michigan DOT removed.
Section VIII.A. Page 8. Nebraska DOR sampling rate.
Section VIII.A. Page 9. Wisconsin DOT sampling method.
Page 13. Michigan DOT removed.
Page 13. Minnesota DOT phone number.
Page 14. North Dakota DOT email address updated.

COMBINED STATE BINDER GROUP
CERTIFICATION METHOD OF ACCEPTANCE
FOR ASPHALT BINDERS

Acceptance of asphalt binder by the **Certification Method** provides for acceptance of these materials for use on Iowa, Minnesota, Nebraska, North Dakota, South Dakota, and Wisconsin Department of Transportation/Roads (Department) projects upon the producer's or supplier's certification that the product as furnished to the contractor (or purchasing agency) complies with the pertinent specification and/or contract requirements.

Department projects include: state, county and municipal federal aid and authorized county and municipal state aid projects. In order to provide asphaltic material to Department projects under the **Certification Method**, a supplier[1], as defined below, shall comply with the following procedures and requirements.

I. GENERAL REQUIREMENTS

The supplier shall have laboratory facilities and qualified personnel available to perform all specification tests and maintain an acceptable quality control program. The supplier shall maintain records of all its control testing done in the production of asphaltic materials. These test records shall be available at all times for examination by the Departments' designated representative[2] and for a period of five (5) years after use on a project.

The supplier shall inspect each transport tank prior to loading to insure suitability for loading and freedom from contaminants.

Continuing acceptance of materials under this process is contingent upon satisfactory compliance with procedures and conformance of materials to requirements as determined by test results for source samples and field samples taken by project personnel.

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[1] Supplier-A Supplier shall be defined as one who produces or supplies the final product or makes a blend or modification that alters the properties of the PGAB specified in M320, prior to final shipment to Department projects. A Supplier shall be a refinery, a terminal, secondary storage facility or an HMA producer. If any modification, blending, or blending of PGAB from different sources is made at the HMA plant, the HMA producer shall be the supplier and must conform to the requirements of this document. If one certified supplier sells to another certified supplier and material is delivered directly to a project, the supplier selling the material to the second supplier is responsible for submitting the daily and bi-weekly QC data to the DOT/DOR's as required by the CSBG document section V.B & V.C.

[2]Hereinafter in this document, the usual designated Department Representatives (contact persons) are listed on pages 13 and 14 of this document.

The tolerances as shown in the "Performance Graded Binder and Test Method Tolerances" Table on page 16 are for use by the Department when comparing to supplier data. All data received from the supplier is expected to meet the base specification values shown in the Table, unless it is agreed upon that a bias exists, based on the results of the Combined State Binder Group Quarterly Round Robins.

If an acid modification process or a modifier (as defined in AASHTO M320, Section 5.2), not including additives such as silicone, is used, the supplier shall assign the modifying process with a unique name and type of modification to be provided to the department for tracking and monitoring purposes. If an anti-strip agent is added at the plant, the HMA producer is considered a supplier (see footnote 1, page 1) and must conform to this document's requirements. Full test results with and without anti-strip in the asphalt binder at the required dosage will be required before production begins.

The department shall be notified of PG grade and/or supplier changes.

II. QUALIFYING FOR CERTIFICATION

Suppliers requesting certified status for supplying material from their individual facilities shall make application in writing to the Department's representative, who will arrange for and authorize the use of the **Certification Method of Acceptance.** This request should present complete information regarding the supplier's quality control program (control tests, testing frequencies, laboratory facilities, programs for maintaining test and shipment records, etc.).

A supplier's certification will remain in effect until denied by the certification program authority or until subsequent re-approval following another inspection. A yearly application in writing need not be made.

Department records will be used to provide a quality history of suppliers. If no quality history exists, one may be established by a cooperative, comprehensive sampling and testing program to ensure that quality control practices are effective.

It is intended that facility inspections will be made each spring by the Department. The inspections will include reviewing sampling and testing procedures, quality control, and facility changes. Also, at this time, the identification and inspection of tanks will be done. Suppliers shall designate and identify tanks that will be used for supplying each grade of asphaltic material for Department projects. The Department inspector will verify that the storage and sampling procedures will be adhered to.

Suppliers will have their requests for certification approved by the Department.

The Departments' Districts/Regions will be notified when suppliers become certified.

The Department inspector shall be permitted to visit asphalt facilities any time during working hours and in the company of appropriate supplier personnel.

Certification of a supplier by one of the Combined State Binder Group members will be accepted by all the member states.

III. LOSS OF CERTIFICATION

Certification will be withdrawn from suppliers when one or more of the following conditions exist.

A. Inability to consistently supply material meeting specifications as measured by non-compliance for three (3) consecutive job site samples according to Department test results for a specific grade.

B. Failure to participate in four (4) Combined State Binder Group "Round-Robins" during any one year. Exceptions will be made for equipment failure. Labs will be required to respond with resolution of equipment failure(s), as detailed in Subsection V.D.6.

C. Failure to respond to notification of outlying labs in writing within the given timeframe, as detailed in Subsection V.D.5.

D. Lack of maintenance of required records.

E. Improper documentation of shipments as defined in Section VII.

F. Failure to maintain a acceptable quality control program.

Decertification of suppliers will be by the Department. Notification will be in writing.

If a supplier loses certification, materials may be accepted, for a 3-month period as defined in section IV qualifying for recertification, according to specific procedures agreed to by the Department and supplier. Procedures may require pre-testing and approval of materials before use and/or increasing the frequency of sampling and testing at the job site (refer to Section VIII.B. of this procedure). The Department's costs for pre-testing and increasing sampling and testing of materials will be paid by the supplier/contractor or their agent unless other arrangements are agreed upon by the Department.

IV. QUALIFYING FOR RECERTIFICATION

If a supplier has lost certification and seeks to be recertified the following is required:

- Fulfill the requirements of Section II, "Qualifying for Certification", of this procedure.

- Submit documentation to the Department's Representative explaining why decertification occurred and the actions the supplier has taken to correct the problems identified by the Department.

A maximum of three-months (of normal production) will be allowed for a supplier to regain certified status under this procedure. If, after that time, the Department determines that the supplier has not attained satisfactory status for certification, material from that source will not be accepted for use on Department projects. The Departments' district/regions will be notified of this action. Decisions regarding the future qualification for certification of a supplier, affected by the above process, shall be at the Department's discretion.

V. SAMPLING AND TESTING BY SUPPLIER

A. Minimum Annual Requirements

1. Prior to the start of the shipping season, adequate testing shall be performed to identify characteristics of tank materials on-hand. Before or at the start of shipping, bi-weekly sample testing (see sub-section V.C.3) shall be completed on a minimum of one sample for each grade of asphaltic material anticipated to be shipped to Department projects.

2. It is intended that facility annual inspections would be made at this time.

3. Participation in Combined State Binder Group "Round Robin" Program will be a requirement, as detailed in Subsection V.D.

This testing will constitute the minimum annual requirements by the Certification method of Acceptance Program for continuation of a supplier certification.

B. Daily Requirements

1. <u>Sampling</u>. One sample from the tank or blender representing each grade of material shipped to state work. For material shipped from tanks, the sample may be taken from the tank, from the line during loading, or from the loaded transport. Material produced from a blender may be sampled from the line during loading or from the loaded transport.

2. <u>Test required</u>. **Performance graded binder**: penetration, any viscosity measurement or dynamic shear. The dynamic shear will be required if material is modified.

3. <u>Report</u>. Send a record of daily quality control results to the Department central laboratory on an approximate weekly basis.

C. Bi-Weekly Requirements

1. <u>Sampling</u>. Sample as for B.1.

2. <u>Tests required</u>. All of the tests listed in the attached schedule of tests for **performance graded binder** material.

3. <u>Report</u>. Send report of test results to the Department central laboratory when completed.

D. Combined State Binder Group Quarterly "Round-Robins"

1. General. WisDOT will send a Combined State Binder Group "Round Robin" PG-Binder sample to each supplier, approximately every three (3) months, with a maximum of four (4) samples annually.

2. Purpose. To provide data about the repeatability and reproducibility of the applicable PG binder test methods.

3. Report. Send a report of test results to the designated WisDOT Representative when completed.

4. Summary. WisDOT will compile a summary report and distribute to all participants. Each supplier's data will remain confidential.

5. Notification of Outliers. WisDOT will notify "Round-Robin" participants of any tests for which their data was determined to be a statistical outlier. An outlier is defined as that data which is outside of three standard deviations from the average. The determination of outliers is an iterative process. The notification will be contained in a letter attached to the summary report. The participant shall have 30 days to provide WisDOT with a response as to the apparent cause of the outlier. This information will be shared with the other Departments.

6. Equipment Failures. Labs will be required to respond to WisDOT in writing with resolution to equipment failures. This information will be shared with the other Departments.

VI. TEST REPORTS (required by Section V)

The supplier chief chemist (or other representative) shall certify test reports for samples and submit them to the Department's Representative. This test information will be evaluated and filed for possible future reference. The reports shall be sent to:

IOWA:

Iowa Department of Transportation
Office of Materials
Attn: John Hinrichsen
800 Lincoln Way
Ames, IA 50010-6915
E-Mail: john.hinrichsen@dot.iowa.gov

MINNESOTA:	Minnesota Department of Transportation Office of Materials and Road Research 1400 Gervais Avenue Maplewood, MN 55109 Attn: Paul Lohmann, Asphalt Certification Specialist E-Mail: paul.lohmann@dot.state.mn.us
NEBRASKA:	Nebraska Department of Roads Materials and Tests Division 1400 NE Hwy 2 Lincoln, NE 68509-4759 ATTN: Dale Byre E-Mail: dalebyre@dor.state.ne.us
NORTH DAKOTA:	North Dakota Department of Transportation Materials and Research Division 300 Airport Road Bismarck, ND 58504 ATTN: Jeff Herman E-Mail: jherman@nd.gov
SOUTH DAKOTA:	South Dakota Department of Transportation Materials Laboratory 104 S. Garfield, Bldg B Pierre, SD 57501 ATTN: Rick Rowen E-Mail: rick.rown@state.sd.us
WISCONSIN:	Wisconsin Department of Transportation Truax Center ATTN: Asphalt Certification Specialist 3502 Kinsman Boulevard Madison, WI 53704 E-Mail: Richard.barden@dot.state.wi.us

VII. CERTIFICATION OF SHIPMENTS AND DOCUMENTATION

For each truck shipment a shipping ticket shall be prepared showing the supplier, location, grade of asphaltic material, unique name (as referenced in Section I, paragraph 5, page 2), additives (silicone or anti-strip), truck number, supplier's tank number from which the truck was loaded, average unit weight, quantity, and date and time of loading. In addition, Iowa DOT requires contract or project number on the shipping ticket. A statement certifying that the material complies with Combined State Binder Group requirements and Department Specifications shall be on or accompany the shipping ticket. The company invoice or manifest form may be used for this purpose.

In addition to the usual contractor's copy of the shipping ticket, a copy (South Dakota DOT to receive two copies) of the shipping ticket containing the certification language for each truck shipment also shall be made available to the project engineer at the job site.

The Department's Representative will furnish a list of certified suppliers to the districts/regions.

Only material shipped from a certified supplier directly to the job site will be accepted as certified material. Material shipped to, and unloaded into, a secondary storage facility and subsequently shipped to state work will **not** be accepted as certified material unless that secondary facility has been certified and is operating in full compliance with these procedures. Modification at HMA plant will not be accepted unless plant is certified as a supplier.

VIII. SAMPLES OBTAINED BY THE STATE

A. Refinery/Terminal Samples

The Department shall have the option to obtain random samples at the source of supply. Samples shall be taken by supplier personnel at the request and under observation of an authorized Department representative. The supplier shall have equipment and facilities available to obtain samples safely.

B. Verification Field Samples

IOWA:
The supplier or contractor personnel will obtain samples, under the observation of a Department representative, of material at the job site. The sampling rate will be one per day. For contracts with less than approximately 40 Mg (45 tons) of asphalt, sampling may be waived.

Sampling shall be accomplished in accordance with Iowa Instructional Memorandum (I.M.) 323, " Method of Sampling Asphaltic Materials."

In addition, project personnel will obtain samples as directed by the project engineer to adequately monitor material quality at the plant for alterations made to the site storage, HMA plant handling process, or if modification is occurring at the HMA plant.

MINNESOTA:
The supplier or contractor personnel will obtain samples, under the observation of a Department representative, by random selection from shipments of material at the job site. The samples shall be taken from the first load and subsequently one sample per 900 Mg (1000 tons) for each supplier and grade of asphalt binder per contract. For contracts with less than approximately 23 Mg (25 tons) (one truck transport) of asphalt, sampling may be waived.

Sampling shall be accomplished by taking a one-liter (one-quart) sample of material from a transport in accordance with AASHTO Designation T40.

In addition, project personnel will obtain samples as directed by the project engineer to adequately monitor material quality at the plant for alterations made to the site storage, HMA plant handling process, or if modification is occurring at the HMA plant.

NEBRASKA:
The Contractors Certified Sampling Technician will obtain samples, under the observation or assistance of the Department representative, of material at the job site. The sampling rate will be a minimum of one (1) per 3750 tons of Hot Mix Asphalt produced for each supplier and grade of Performance Graded Binder per contract. A minimum of two (2) samples will be taken per project.

One Sample will consist of two (2) one-liter (one-quart) cans of material taken from the line between the storage tank and mixer or from the tank supplying material to the line, at a location at which material sampled is representative of the material in the line to the mixer. Sampling shall be accomplished in accordance with NDR T40.

In addition, project personnel will obtain samples as directed by the project engineer to adequately monitor material quality whenever blending of binders of different grades or binders from different suppliers is taking place. These samples will be taken at the start of production following the blending at locations defined above.

NORTH DAKOTA:
NDDOT project personnel will observe the contractor obtain samples from material delivered to the job site. The sampling rate will be a minimum of one sample for every 250 tons (225 Mg) for each supplier and grade of asphalt cement, or fraction thereof. The sample shall be taken randomly within each 250 tons (225 Mg) of material.

A sample will consist of taking two 1-liter (one-quart) samples from the designated transport. The first sample will be used for testing, the second sample will be a check. Both samples will be sent to the NDDOT Central Lab.

Samples will be identified with the following information written on the can:

-Project Number-Field Sample Number

-Manifest Number-PG Grade

-Asphalt Supplier-Date

-Original or Check

Project personnel will also obtain samples as directed by the project engineer at any time extra samples are determined to be necessary.

SOUTH DAKOTA:

The supplier or contractor personnel will obtain samples, under the observation of a Department representative, of material at the job site. The sampling rate will be in accordance to the South Dakota Department of Transportation Materials Manual, "Minimum Sample and Test Requirements", section 1.1C.(3).

The sampling method will be in accordance to SD 301 section 3.2C in the South Dakota Department of Transportation Materials Manual.

In addition, project personnel will obtain samples as directed by the project engineer to adequately monitor material quality at the plant for alterations made to the site storage, HMA plant handling process, or if modification is occurring at the HMA plant.

WISCONSIN:

The supplier or contractor personnel will obtain samples, under the observation of a Department representative, by random selection from shipments of material at the job site. The sampling rate will be a minimum of one (1) per 800 Mg (900 tons) for each supplier and grade of asphalt binder, or fraction thereof, per contract. For contracts with less than approximately 23 Mg (25 tons) (one truck transport) of asphalt, sampling may be waived.

Sampling shall be accomplished by taking a one-liter (one-quart) sample of material from a transport in accordance with AASHTO Designation T40.

Sampling method shall be accomplished by taking a one-liter (one-quart) sample of material representing the middle third of the load from a sample valve attached to the transport in accordance with AASHTO Designation T40 section 10 paragraph 10.1 or other department approved supplier method as outlined in the QC plan.

In addition, project personnel will obtain samples as directed by the project engineer to adequately monitor material quality at the plant for alterations made to the site storage, HMA plant handling process, or if modification is occurring at the HMA plant.

IX. ACCEPTANCE OF ASPHALT BINDER NOT ON THE APPROVED LIST

It is the intention of the Departments to encourage suppliers to become certified according to this procedure. However, if situations occur where a supplier is not on the Departments' approved list, materials may be accepted for a designated interim period according to specific procedures agreed to by the Department and supplier. Procedures may require pretesting and approval of materials before use and/or increasing the frequency of sampling and testing at the job site (refer to Section VIII.B. of this procedure). The Department's costs for pretesting and increased sampling and testing of materials will be paid by the supplier/contractor or their agent unless other arrangements are agreed upon by the Department.

X. SAMPLES TESTED BY THE STATE WITH NON-COMPLYING RESULTS

Should a sample tested by the Department show noncompliance, actions will be taken to investigate the sample failure. The purpose of the investigation(s) will be to quickly obtain information to either substantiate the failure data or to provide conclusive evidence that the reported failure is unreliable. There are two types of samples to be considered: 1) refinery/terminal random samples taken by the supplier under observation of a authorized Department representative at the shipping refinery or terminal, and 2) verification field samples taken under the direction of the Department's project personnel at the job site. The processes to resolve sample failures for each of the two types of samples are as follows:

A. Refinery/Terminal Samples

If a sample obtained by a authorized Department representative at a supplier plant shows test results out of specification limits, the process of resolving the sample failure will include the following actions as appropriate:

1) The Department will notify the supplier.

2) The Department and supplier together will determine the quantity and location(s) of the material in question.

3) The Department will retest the sample as determined necessary to confirm or disaffirm the original test result(s).

4) If material is in transit to or at Department projects, the district/region(s) will be notified.

5) The Department will increase the frequency of sampling at the project site(s) involved.

6) The Department will investigate and review all pertinent test data.

7) The Department's Representative will collect and compile all information, including any from the supplier and district/region(s), and prepare a report

with explanations to resolve the sample problem. A copy of the report will be distributed to the district/region, contractor, and supplier.

8) The supplier shall take corrective action, as warranted, and submit an explanation to the Department.

9) The Department will determine when the sample is adequately investigated and resolved and the supplier is consistently furnishing specification material.

B. Verification Field Samples

If a sample obtained by the Department at a project site shows test results out of specification limits, the process of resolving the sample failure will include the following actions as appropriate:

1) The Department will notify the district/region and determine that the information sent with the sample is correct and the sample does indeed fail. The district/region will notify the contractor. The district/region will arrange for project personnel to investigate all aspects of procuring, handling and submitting the sample for testing. The quantity and location of material in question will be determined. The district/region will report findings to the Department's Representative.

2) The Department will conduct retesting of the sample as determined necessary to confirm or disaffirm the original test result(s).

3) The Department will notify the supplier who will arrange to investigate all aspects of loading, handling and delivery of the material in question. The supplier shall report findings to the Department's Representative.

4) The Department will increase the frequency of sampling at the project site.

5) The Department's Representative will collect and compile all information from the district/region and supplier investigations and prepare a report. The Department will determine when the sample has been adequately investigated. The report will contain data with an analysis of information and recommendations for the district/region to resolve the sample problem. A copy of the report will be distributed to the district/region, contractor, and supplier.

6) The Department will issue the standard report of tests for the sample showing the failing test result(s).

7) The district/region will make the final decision for resolving the sample problem. Generally, the district/region will accomplish this with input from the Department Representative, and supplier. The Department's report of investigations (from step 5 above) will be used in the decision making

process. The district/region will notify the contractor. Should the decision involve reduced payment for material(s) in question, standard Department practices will be followed and administered by the district/region. The contractor will be notified in writing of reduced payments.

8) The supplier shall implement corrective measures suggested by the investigation work and notify the Department of actions taken.

9) The Department will implement changes in this procedure determined to be warranted by the investigation work.

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Copies of this document can be obtained from:

North Central Superpave Center Home Page: http://bridge.ecn.purdue.edu/~spave/

WISDOT's ftp site: ftp://ftp.dot.state.wi.us/dtid/bhc/quality/general/

Any Combined State Binder Group representative.

SCHEDULE OF TESTS
AS REQUIRED BY AASHTO – M320

TEST	TEST METHOD
PERFORMANCE GRADED BINDER	
Solubility	AASHTO - T44
Flash Point	AASHTO - T48
Brookfield Viscosity	AASHTO – T316
Dynamic Shear	AASHTO – T315
Rolling Thin Film Oven Test:	AASHTO - T240
a. Mass Loss	
b. Dynamic Shear	AASHTO – T315
Accelerated Aging (PAV)	AASHTO – R28
a. Dynamic Shear	AASHTO – T315
b. Creep Stiffness	AASHTO – T313
Direct Tension	AASHTO – T314
Elastic Recovery	AASHTO – T301

NOTES:

1. All testing shall be in accordance with the applicable standard methods of the American Association of State Highway and Transportation Officials (AASHTO) or American Society of Testing and Materials (ASTM).

Combined State Binder Group
Method of Acceptance for Asphalt Binders

PERFORMANCE GRADED BINDER SPECIFICATIONS & TEST METHOD TOLERANCES
(IOWA, MINNESOTA, NORTH DAKOTA, SOUTH DAKOTA & WISCONSIN DOT'S & NEBRASKA DOR)

EFFECTIVE MARCH 2007

PERFORMANCE GRADE	SPEC BASE	SPEC W/TOL	TOL
TESTS ORIGINAL BINDER			
AASHTO T44 SOLUBILITY % MINIMUM	99.00	98.87	0.13
AASHTO T48 FLASH POINT TEMP, °C, MINIMUM	230	221	9
AASHTO T316 VISCOSITY[b] Pa.s MAXIMUM	3.0	3.2	7.3%
AASHTO T315 DYNAMIC SHEAR[c] G*/SINδ[d], kPa, MINIMUM	1.00	0.93	7%
TESTS RTFO RESIDUE			
AASHTO T240 MASS CHANGE[e], % MAX	1.00	1.20	20%
AASHTO T315 DYNAMIC SHEAR G*/SINδ[d], kPa, MINIMUM	2.20	1.98	10%
TESTS PAV RESIDUE			
AASHTO T315 DYNAMIC SHEAR G*SINδ[d], kPa, MAXIMUM	5000	5600	12%
AASHTO T313 CREEP STIFFNESS[g] [S] MAXIMUM, MPa	300	324	8%
m-VALUE, MINIMUM	0.300	0.285	5%
AASHTO T314 DIRECT TENSION[g] FAILURE STRAIN, MINIMUM %	1.0	0.8	20%

PG 46-	34	40	46
AVERAGE 7 DAY MAXIMUM PAVEMENT DESIGN TEMP, °C	46		
MINIMUM PAVEMENT DESIGN TEMPERATURE, °C[a]	-34	-40	-46
AASHTO T48 FLASH POINT TEMP, °C, MINIMUM	230		
AASHTO T316 TEST TEMP, °C			
AASHTO T315 (ORIGINAL) TEST TEMP @ 10 rad/sec, °C	46		
AASHTO T315 (RTFO) TEST TEMP @ 10 rad/sec, °C	46		
AASHTO R28 PAV AGING TEMP, C[f]	90		
AASHTO T315 (PAV) TEST TEMP @ 10 rad/sec, °C	10	7	4
AASHTO T313 TEST TEMP @ 60s, °C	-24	-30	-36
AASHTO T314 TEST TEMP @ 1.0 mm/min, °C	-24	-30	-36

PG 52-	10	16	22	28	34	40	46
AVERAGE 7 DAY MAXIMUM PAVEMENT DESIGN TEMP, °C	52						
MINIMUM PAVEMENT DESIGN TEMPERATURE, °C[a]	-10	-16	-22	-28	-34	-40	-46
AASHTO T48 FLASH POINT TEMP, °C, MINIMUM	230						
AASHTO T316 TEST TEMP, °C	135						
AASHTO T315 (ORIGINAL) TEST TEMP @ 10 rad/sec, °C	52						
AASHTO T315 (RTFO) TEST TEMP @ 10 rad/sec, °C	52						
AASHTO R28 PAV AGING TEMP, C[f]	90						
AASHTO T315 (PAV) TEST TEMP @ 10 rad/sec, °C	25	22	19	16	13	10	7
AASHTO T313 TEST TEMP @ 60s, °C	0	-6	-12	-18	-24	-30	-36
AASHTO T314 TEST TEMP @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	-36

PG 58-	16	22	28	34	40
AVERAGE 7 DAY MAXIMUM PAVEMENT DESIGN TEMP, °C	58				
MINIMUM PAVEMENT DESIGN TEMPERATURE, °C[a]	-16	-22	-28	-34	-40
AASHTO T48 FLASH POINT TEMP, °C, MINIMUM	230				
AASHTO T316 TEST TEMP, °C	135				
AASHTO T315 (ORIGINAL) TEST TEMP @ 10 rad/sec, °C	58				
AASHTO T315 (RTFO) TEST TEMP @ 10 rad/sec, °C	58				
AASHTO R28 PAV AGING TEMP, C[f]	100				
AASHTO T315 (PAV) TEST TEMP @ 10 rad/sec, °C	25	22	19	16	13
AASHTO T313 TEST TEMP @ 60s, °C	-6	-12	-18	-24	-30
AASHTO T314 TEST TEMP @ 1.0 mm/min, °C	-6	-12	-18	-24	-30

PG 64-	10	16	22	28	34	40
AVERAGE 7 DAY MAXIMUM PAVEMENT DESIGN TEMP, °C	64					
MINIMUM PAVEMENT DESIGN TEMPERATURE, °C[a]	-10	-16	-22	-28	-34	-40
AASHTO T48 FLASH POINT TEMP, °C, MINIMUM	230					
AASHTO T316 TEST TEMP, °C	135					
AASHTO T315 (ORIGINAL) TEST TEMP @ 10 rad/sec, °C	64					
AASHTO T315 (RTFO) TEST TEMP @ 10 rad/sec, °C	64					
AASHTO R28 PAV AGING TEMP, C[f]	100					
AASHTO T315 (PAV) TEST TEMP @ 10 rad/sec, °C	31	28	25	22	19	16
AASHTO T313 TEST TEMP @ 60s, °C	0	-6	-12	-18	-24	-30
AASHTO T314 TEST TEMP @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30

PG 70-	10	16	22	28	34	40
AVERAGE 7 DAY MAXIMUM PAVEMENT DESIGN TEMP, °C	70					
MINIMUM PAVEMENT DESIGN TEMPERATURE, °C[a]	-10	-16	-22	-28	-34	-40
AASHTO T48 FLASH POINT TEMP, °C, MINIMUM	230					
AASHTO T316 TEST TEMP, °C	135					
AASHTO T315 (ORIGINAL) TEST TEMP @ 10 rad/sec, °C	70					
AASHTO T315 (RTFO) TEST TEMP @ 10 rad/sec, °C	70					
AASHTO R28 PAV AGING TEMP, C[f]	100 (110)					
AASHTO T315 (PAV) TEST TEMP @ 10 rad/sec, °C	34	31	28	25	22	19
AASHTO T313 TEST TEMP @ 60s, °C	0	-6	-12	-18	-24	-30
AASHTO T314 TEST TEMP @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30

PG 76-	10	16	22	28	34
AVERAGE 7 DAY MAXIMUM PAVEMENT DESIGN TEMP, °C	76				
MINIMUM PAVEMENT DESIGN TEMPERATURE, °C[a]	-10	-16	-22	-28	-34
AASHTO T48 FLASH POINT TEMP, °C, MINIMUM	230				
AASHTO T316 TEST TEMP, °C	135				
AASHTO T315 (ORIGINAL) TEST TEMP @ 10 rad/sec, °C	76				
AASHTO T315 (RTFO) TEST TEMP @ 10 rad/sec, °C	76				
AASHTO R28 PAV AGING TEMP, C[f]	100 (110)				
AASHTO T315 (PAV) TEST TEMP @ 10 rad/sec, °C	37	34	31	28	25
AASHTO T313 TEST TEMP @ 60s, °C	0	-6	-12	-18	-24
AASHTO T314 TEST TEMP @ 1.0 mm/min, °C	0	-6	-12	-18	-24

PG 82-	10	16	22	28	34
AVERAGE 7 DAY MAXIMUM PAVEMENT DESIGN TEMP, °C	82				
MINIMUM PAVEMENT DESIGN TEMPERATURE, °C[a]	-10	-16	-22	-28	-34
AASHTO T48 FLASH POINT TEMP, °C, MINIMUM	230				
AASHTO T316 TEST TEMP, °C	135				
AASHTO T315 (ORIGINAL) TEST TEMP @ 10 rad/sec, °C	82				
AASHTO T315 (RTFO) TEST TEMP @ 10 rad/sec, °C	82				
AASHTO R28 PAV AGING TEMP, C[f]	100 (110)				
AASHTO T315 (PAV) TEST TEMP @ 10 rad/sec, °C	40	37	34	31	28
AASHTO T313 TEST TEMP @ 60s, °C	0	-6	-12	-18	-24
AASHTO T314 TEST TEMP @ 1.0 mm/min, °C	0	-6	-12	-18	-24

PG+ Specifications[h]

	Spec Base			Spec w/tol			Tol
Temperature Spread[i]	92	98	104	92	98	104	
Elastic Recovery; AASHTO T301 tested at 77°F (RTFO Aged AASHTO T301)	Minimum 65%			Minimum 60%			7.7%
Phase Angle; degrees (Maximum) (Original Binder)	77.0	75.0	73.0	79.0	77.0	75.0	2.6%

a PAVEMENT TEMPERATURES ARE ESTIMATED FROM AIR TEMPERATURES USING AN ALGORITHM CONTAINED IN THE LTPP BIND PROGRAM, MAY BE PROVIDED BY THE SPECIFYING AGENCY, OR BY FOLLOWING THE PROCEDURES AS OUTLINED IN MP2 AND PP28.

b THIS REQUIREMENT MAY BE WAIVED AT THE DISCRETION OF THE SPECIFYING AGENCY IF THE SUPPLIER WARRANTS THAT THE ASPHALT BINDER CAN BE ADEQUATELY PUMPED AND MIXED AT TEMPERATURES THAT MEET ALL APPLICABLE SAFETY STANDARDS.

c FOR QUALITY CONTROL OF UNMODIFIED ASPHALT CEMENT PRODUCTION, MEASUREMENT OF THE VISCOSITY OF THE ORIGINAL ASPHALT CEMENT MAY BE USED TO SUPPLEMENT DYNAMIC SHEAR MEASUREMENTS OF G*/sinδ AT TEST TEMPERATURES WHERE THE ASPHALT IS A NEWTONIAN FLUID.

d G*/sinδ = HIGH TEMPERATURE STIFFNESS AND G*sinδ = INTERMEDIATE TEMPRATURE STIFFNESS.

e THE MASS CHANGE SHALL BE LESS THAN 1.00 PERCENT FOR EITHER A POSITIVE (MASS GAIN) OR A NEGATIVE (MASS LOSS) CHANGE

f THE PAV AGING TEMPERATURE IS BASED ON SIMULATED CLIMATIC CONDITIONS AND IS ONE OF THREE TEMPERATURES 90°C, 100°C, OR 110°C. NORMALLY THE PAV AGING TEMPERATURE IS 100°C FOR PG 58-XX AND ABOVE. HOWEVER, IN DESERT CLIMATES THE PAV AGING TEMPERATURE FOR PG 70-XX AND ABOVE MAY BE SPECIFIED AS 110°C.

g IF THE CREEP STIFFNESS IS BELOW 300 MPa, THE DIRECT TENSION TEST IS NOT REQUIRED. IF THE CREEP STIFFNESS IS BETWEEN 300 AND 600 MPa, THE DIRECT TENSION FAILURE STRAIN REQUIREMENT CAN BE USED IN LIEU OF THE CREEP STIFFNESS REQUIREMENT. THE m-VALUE REQUIREMENT MUST BE SATISFIED IN BOTH CASES.

h BINDERS SIGNIFIED BY PG XX-XXP SHALL BE REQUIRED TO MEET OR EXCEED THE PG+ SPECIFICATIONS IN ADDITION TO M320 SPECIFICATIONS

i TEMPERATURE SPREAD IS DETERMINED BY SUBTRACTING LOW TEMPERATURE FROM THE HIGH TEMPERATURE (PG 64-28: 64 - (-28) = 92).

Kentucky Method 64-445-~~04~~05
Dated ~~3/11/04~~12/28/04
Supersedes KM 64-445-~~03~~04
Dated ~~1/8/03~~3/11/04

KENTUCKY TRANSPORTATION CABINET (KYTC) APPROVED SUPPLIER CERTIFICATION PROGRAM FOR EMULSIFIED ASPHALTS (EASC)

1. SCOPE:

1.1. According to the Department's *Standard Specifications for Road and Bridge Construction*, Section 806, emulsified asphalts (including polymer asphalt emulsions and PrimerL) may only be furnished by suppliers included in the KYTC EASC program. The KYTC EASC program permits the manufacture and shipment of emulsified asphalts within the framework of a quality-control plan (QCP) without complete pretesting of the emulsified asphalt by the Department's Division of Materials. This EASC program follows the requirements outlined in AASHTO R26, *Standard Recommended Practice for Certifying Suppliers of Performance-Graded Asphalt Binders*, with modifications as found in this Kentucky Method (KM).

1.2 Supplier qualification: All suppliers that: 1) create or substantially change the properties of an emulsified asphalt or 2) supply material to multiple contractors are required to comply with the guidelines of the EASC program.

1.3 EASC approval will be specific for the supplier/terminal and will not be transferable to other sites.

1.4 For specialty applications, other emulsified asphalts specified for use on Department projects will be tested and approved by the Division of Materials. Maintain responsibility for full compliance testing at a frequency to be determined by the Division of Materials.

2. QUALIFICATION:

2.1. Laboratory requirements: In addition to AASHTO R26, AASHTO Materials Reference Laboratory (AMRL) proficiency-sample ratings of three or better on all AASHTO T59, *Testing Emulsified Asphalts*, properties are required. If a pattern of low ratings is noted by the Division of Materials, the laboratory will be unable to provide test data for supplier certification. Forward copies of the AMRL inspection report and AMRL proficiency results to the Division of Materials within one month of receipt. Satellite laboratories at terminals may be inspected by a primary laboratory that meets the above requirements. Satellite laboratories are not required to participate in the AMRL proficiency-sample-testing program.

2.2. QCP:

2.2.1 If any changes that affect the QCP occur, the supplier shall notify the Division of Materials immediately, and subsequently update the QCP to reflect these changes.

2.2.2 In addition to AASHTO R26, include instructions for the proper storage and handling of each emulsion in the plan. If the handling instructions change for a particular emulsion, notify the Division of Materials, and update the QCP.

2.2.3 The minimum testing frequency for emulsions is one complete AASHTO T59 analysis, including the modifications found in Section 806, every 30 days or when new material produced.

2.3. Quality-Control Testing:

2.3.1. Perform the minimum quality-control tests according to the Department's *Standard Specifications*, Section 806, for each type of emulsified asphalt included in the attached table.

2.3.2. A supplier will be qualified to ship a specific emulsified asphalt after a minimum of three production lots/batches are tested by the Division of Materials and full compliance with test specifications is indicated for each sample. If no new lot/batch is manufactured within one week of taking the first sample, the existing lot/batch may be re-tested. Do not sample these production lots/batches closer than one week apart. If two separate production lots/batches are produced within the same week, sample these lots/batches, and submit them for testing provided documentation of the new production is attached.

2.3.3. While a supplier is in the process of having the three production lots/batches tested, the Department will allow material to be shipped from the lot(s) in question if it has been tested and approved by the Division of Materials. In this case, the Division of Materials will supply a lot/batch number for the material.

2.3.4. The Department will not require compliance testing for winter months when production ceases. For suppliers continuing on the EASC program, furnish a complete set of compliance test data and one sample of material for each type of emulsified asphalt produced to the Division of Materials when shipping resumes. If the test data and sample are not received by the Division of Materials within two weeks of the resumption of shipping, the supplier shall be removed from the EASC program and will be ineligible to ship material to Department projects.

3. DOCUMENTATION:

3.1 Quality control tests: Submit a summary for all quality-control tests performed to the Division of Materials every two months. Maintain detailed records of all quality-control tests, inspections, and shipments for at least two years.

3.2 Tank certification: For each tank of material, submit to the Division of Materials a completed KYTC Materials form, TC 64-439Y, containing complete test data furnished by the supplier. The type of inspection shall be listed as "CER," and any quality-control tests performed at the terminal shall be listed under "REMARKS." Submit this form every 30 days or when new material is added to a tank.

3.3. Bills-of-lading: In addition to the requirements of AASHTO R26, Subsection 11.3, also include the following three items: the lot/batch number; the project number/destination of the material; and the signature of the producer's representative.

4. QUALITY ASSURANCE SAMPLING AND TESTING: Furnish split samples obtained at random, as directed by the Division of Materials, from the supplier's terminal. Ensure that the emulsified asphalt samples are representative of the material being shipped from that particular location. Submit two gallons of emulsified asphalt to the Division of Materials for each split sample. All appropriate test results, as required by Section 806, must follow from the supplier within five working days of sampling the material. Division of Materials representatives may visit any of the supplier's facilities for the purpose of obtaining split samples and/or observation of sample collection and quality-control testing.

5. NON-COMPLIANT MATERIALS:

5.1. Notification of non-compliance: If any information is obtained by the supplier that indicates a quantity of emulsified asphalt is not in compliance with the specification(s): 1) notify the Division of Materials immediately, by telephone and in writing, of the emulsified asphalt in question; 2) identify the material by lot number; and 3) cease shipment/use of the material. A full analysis of the material in question, as required by Section 806, indicating full compliance is required before shipping may resume. Notify the Division of Materials before resuming shipment.

5.2. Probation and removal from the KYTC EASC program: The Department will consider unsatisfactory material to be production lots/batches of a particular emulsified asphalt, tested either through split samples or field samples, that are not in compliance with the specification(s). Two unsatisfactory lots/batches will place a supplier on probation. Two out of three unsatisfactory production lots/batches will constitute removal from the EASC program for that emulsified asphalt. The Division of Materials will investigate the cause of non-compliance. If the non-compliance is found to be terminal-related, only the terminal will be restricted from shipping instead of the supplier.

5.3. Reinstatement: If a supplier is removed from the EASC program, reinstatement will be contingent upon three consecutive production lots/batches being tested by the Division of Materials and meeting the compliance specifications.

6. CONTINUED APPROVAL - Continued approval under the EASC program will be contingent upon a record of satisfactory performance as determined by the Division of Materials and documented by inclusion on the Department's List of Approved Materials.

APPROVED ______________________________
Director
Division of Materials

DATE________~~3/11/04~~12/28/04______________________

Kentucky Method 64-445-~~04~~05
Dated ~~3/11/04~~12/28/04
Supersedes KM 64-445-04~~2~~
Dated ~~1/8/03~~3/11/04

~~km44504~~km44505.doc

Attachment

QUALITY-CONTROL TESTS FOR EMULSIFIED ASPHALTS

Tests	SS-1H	SS-1	RS-1	RS-2	CRS-2	AE-200	HFMS-2	HFRS-2	CRS-2P	Primer L
Viscosity @ 77 °F	X	X	X			X	X			X
Viscosity @ 122 °F				X	X			X	X	
Residue by Distillation	X	X	X	X	X	X	X	X	X	
Oil Distillates					X	X			X	
Demulsibility			X	X	X			X	X	
Residue Penetration	X	X	X	X	X		X	X	X	
Particle Charge					X				X	
Float Test @ 140 60 °F						X	X	X		
Coating Test						X	X			X
Sieve	X	X	X	X	X			X	X	
Float Test @ 122 °F										X
Asphalt Content										X
Water Content										X
Storage Stability	X	X	X	X	X	X		X	X	
Ductility @ 77 °F	X	X	X	X	X		X	X		
Solubility	X	X	X	X	X	X		X	X	X
Softening Point									X	
Ductility @ 39 °F									X	

The minimum testing frequency as required by Section 806, is every 30 days, or when new material is produced, and at least once when the asphalt source changes.

September 1, 2004
Rev: September 16, 2004

Tennessee Department of Transportation
Division of Materials and Tests

Emulsified Asphalt Certified Supplier Requirements (SOP 3-2)

Purpose- The purpose of this document is to establish the minimum requirements for an emulsified asphalt supplier to become certified in Tennessee, and therefore provide emulsified asphalt on TDOT projects.

Policy- All Emulsified Asphalt supplied to a TDOT project must come from a certified emulsified asphalt supplier and be in compliance with TDOT Specifications (Section 904). To become certified, the supplier or manufacturer must submit a quality control plan (QCP) in resemblance with AASHTO R-26, and as modified or required in this procedure, to TDOT for approval. The supplier must also demonstrate a history of quality control data and proof of full QCP implementation. New emulsified asphalt suppliers must submit three (3) consecutive split samples for each emulsion being shipped for TDOT verification testing.

Procedure- Definitions- The *manufacturer*, as further referenced in this procedure will be the last source to produce or modify the final product. The *supplier* will be the last source to handle the product before being shipped, and the supplier will provide the emulsion certification report with each shipment. In many instances, the manufacturer and the supplier will be one in the same. When using a live batching system, a *lot* will be defined as once every two weeks.

Laboratory-Each manufacturer and supplier must have a designated laboratory to either certify the emulsion or to conduct quality control testing. Laboratories used to certify emulsified asphalts must be AMRL Accredited in the following tests or they must be under the direct jurisdiction of an accredited laboratory and participate in AMRL proficiency testing:

1. AASHTO T-59, Residue by Evaporation
2. AASHTO T-59, Residue by Distillation
3. AASHTO T-59, Sieve Test
4. AASHTO T-59, 24 Hour Storage Stability Test
5. AASHTO T-59, Saybolt Viscosity
6. AASHTO T-51, Ductility Test
7. AASHTO T-49, Penetration Test

Supplier laboratories that conduct quality control testing must have equipment to perform a Saybolt-Furol viscosity test, a Sieve test, and Percent Residue by evaporation or Percent Distillate. Quality Control testing may be defined by the supplier for rapid testing. However, all Specification Compliance testing shall follow AASHTO current test methods. Personnel conducting quality control testing must be qualified; either by training from the equipment manufacturer, trained under the direct supervision of an individual who routinely completes AMRL demonstrations and proficiency testing, or trained by other highly proficient and competent individuals with emulsified asphalt testing experience.

Quality Control Plan (QCP)- Each manufacturer and supplier must submit a QCP for approval. The QCP shall contain the information required in resemblance with Section 9.1 and 9.2 (or as revised below) of AASHTO R-26. In addition the plan shall include the following:

- A plan view of the facility and description of storage tanks,

- A narrative description on how each emulsified asphalt will be blended and handled to assure a consistent product.

Testing for tanks

As a minimum *quality control* testing shall be completed on every batch added to a tank. Quality Control shall be completed before the material is shipped. *Quality Control* shall consist of the tests in Table I.

As a minimum, TDOT will require specification compliance testing on every lot. Testing shall include all tests in *Table 1* for each emulsified asphalt.

If a tank remains idle for more than 2 weeks, it shall be tested for quality control before shipment and every 2 weeks thereafter until another batch is added to it.

Asphalt Emulsions that are used for Microsurfacing, Slurry Seals, and other Specialty Applications shall meet specification compliance testing before they may be shipped.

Each manufacturer and supplier shall keep a record of all specification compliance and quality control test results on file for immediate review by the TDOT. All records shall be retained for a minimum of 5 years.

If test results indicate a lot is not in compliance with TDOT Specifications, in addition to the resemblance of Section 9.2 of AASHTO R 26, the supplier must provide a list of all shipments (date, quantity, contract number) to which the questionable material was shipped.

Quality Assurance (Split samples, random sampling and Round Robin testing) - The manufacturer shall split samples for specification compliance testing and for quality control testing. Half of each sample shall be retained at the supplier's facility for a minimum of 30 days to act as a referee sample. The producer shall obtain samples for TDOT verification testing when requested and in the presence of a TDOT inspector (every 2 weeks).

The TDOT, at any time, may request additional quality control samples to be taken and tested by the supplier or by TDOT, for assurance purposes.

The TDOT, at any time, may request the manufacture or supplier to participate in round robin proficiency testing. TDOT will provide a reasonable time period for the test results to be submitted.

The TDOT will have the right to visit each approved supplier to review quality control activities and records, to obtain random check samples, or to inspect production.

Shipment- All shipments from the supplier must be accompanied with a completed Form DT-0293Emulsion.

REV: Added Appendix A Rapid Testing Protocol for Quality Control

Table 1, Certified Emulsified Asphalt Suppliers Tests for Quality Control & Compliance

	SS-1		AEP		CRS-2		AE3		CRS-2P		RS-2		RS-1	
	QC[A]	Compliance	QC[A]	Compliance	QC[A]	Compliance	QC[A]	Compliance	QC[A]	Compliance	QC[A]	Compliance	QC[A]	Compliance
*Saybolt-Furol Viscosity @ 77°F (seconds)	20 - 100	20 - 100	10 - 50	10 - 50	NA	NA	NA	NA	NA	NA	NA	NA	20 - 100	20 - 100
*Saybolt-Furol Viscosity @ 122°F (seconds)	NA	NA	NA	NA	100 - 400	100 - 400	50 Minimum	50 Minimum	100 - 400	100 - 400	75 - 400	75 - 400	NA	NA
*Storage Stability Test, 24-h, %	NA	1	NA	NA	NA	1	NA	NA	NA	1	NA	1	NA	1
[E]*Settlement, 5 days, % 24 hours	NA	NA	NA	5	NA	NA	NA	5	NA	NA	NA	NA	NA	NA
*Particle Charge	NA	NA	NA	NA	NA	Positive	NA	NA	NA	Positive	NA	NA	NA	NA
Sodium Diocyl Sulfosucinate, %	NA	NA	NA	NA	NA	40 Minimum	NA	NA	NA	40 Minimum	NA	NA	NA	NA
*Distillate @ 500°F, %	NA	NA	55 Maximum	55 Maximum	NA	NA	30 Maximum	30 Maximum	NA	NA	NA	NA	NA	NA
*Oil Test, % (by Volume)	NA	NA	NA	12.0 Maximum	NA	3.0 Maximum	NA	6.0 Maximum	NA	3.0 Maximum	NA	NA	NA	NA
*Demulsibilty min. %	NA	NA	NA	NA	NA	40 Minimum	NA	NA	NA	NA	NA	60 Minimum		60 minimum
[D]*Stone Coating	NA	NA	NA	NA	NA	NA	NA	90 Minimum.	NA	NA	NA	NA	NA	NA
[C]*Cement Mix	NA	2.0 Maximum	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
*Sieve Test, %	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	NA	NA	NA	NA	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum
*Residue, %	57 Minimum	57 Minimum	NA	NA	NA	65 Minimum	NA	NA	65 Minimum	65 Minimum	63 Minimum	63 Minimum	55	55
*Float Test, (seconds)	NA	NA	NA	20 Minimum	NA	NA	NA	20 Minimum	NA	NA	NA	NA	NA	NA
[B]Solubility in Trichloroethylene, %	97.5 Minimum	NA	97.5 Minimum	NA	97.5 Minimum	NA	97.5 Minimum	NA	97.5 Minimum	NA	97.5 Minimum	NA	NA	NA
Penetration	NA	100 - 200	NA	NA	NA	100 - 250	NA	NA	NA	NA	NA	100 - 200	NA	100 - 200
Elastic Recovery @ 50°F, % (AASHTO T301-99)	NA	NA	NA	NA	NA	NA	NA	NA	NA	65 Minimum	NA	NA	NA	NA
Ductility @ 77°F, cm (AASHTO T 51)	NA	40 minimum	NA	40 minimum	NA	40 minimum	NA	40 minimum	NA	125 minimum	NA	40 minimum	NA	40 minimum
Ductility @ 40°F, cm (AASHTO T 51)	NA	NA	NA	NA	NA	NA	NA	NA	NA	30 Minimum	NA	NA	NA	NA
Softening Point, °C (AASHTO T 53-96)	NA	NA	NA	NA	NA	NA	NA	NA	NA	100 - 126	NA	NA	NA	NA

* Tests shall be performed in accordance with AASHTO T 59-97

[A] For Quality Control Testing, the supplier may use a defined raped testing protocol.

[B] Solubility in Trichloroethylene will be required on the base asphalt as QC every 3 months.

[C] Cement Mixing Test will not be required if emulsions are used <u>only</u> for tack coats.

[D] Stone Coating - Individual project aggregates shall be verified annually or as directed by the engineer.

[E] 5 Day Settlement Test - The producer may conduct a 24 hour (1% Max) storage stability test in lieu of the 5 Day Settlement Test, if the emulsions are to be used within 5 days.

Table 1, Certified Emulsified Asphalt Suppliers Tests for Quality Control & Compliance (Continued)

	CAE-P		CSS-1		CSS-1H		SS-1H		TST-1P		CQS-1H (Slurry Seals)	CQS-1H-P (Mircosurface)
	QC[A]	Compliance	QC[A]	Compliance	QC[A]	Compliance	QC[A]	Compliance	QC[A]	Compliance	Compliance	Compliance
*Saybolt-Furol Viscosity @ 77°F (seconds)	10 - 50	10 - 50	20 - 100	20 - 100	20 - 100	20 - 100	20 - 100	20 - 100	10 - 75	10 - 75	20 - 100	20 - 100
*Saybolt-Furol Viscosity @ 122°F (seconds)	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
*Storage Stability Test, 24-h, %	NA	1	NA	1	NA	1	NA	1	NA	NA	NA	NA
[E]*Settlement, 5 days, % 24 hours	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
*Particle Charge	NA	Positive	NA	Positive	NA	Positive	NA	NA	NA	NA	Positive	Positive
Sodium Diocyl Sulfosucinate, %	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
*Distillate, % (by volume)	55 Maximum	55 Maximum	NA	NA	NA	NA	NA	43	NA	NA	NA	NA
*Oil Test, % (by Volume)	NA	12.0 Maximum	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
*Demulsibilty min. %	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
*Stone Coating	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
[C]*Cement Mix	NA	NA	NA	2.0 Maximum	NA	2.0 Maximum	NA	2.0 Maximum	NA	NA	NA	NA
*Sieve Test, %	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum	0.1 Maximum
*Residue, %	NA	NA	57 Minimum	57 Minimum	57 Minimum	57 Minimum	57	57	55 - 60	55 - 60	62 Minimum	62 Minimum
*Float Test, (seconds)	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
[B]Solubility in Trichloroethylene, %	97.5 Minimum	NA	97.5 Minimum	NA	97.5 Minimum	NA	97.5 Minimum	NA	97.5 Minimum	NA	97.5 Minimum	97.5 Minimum
Penetration	NA	300 Minimum	NA	100 - 250	NA	40 - 90	NA	40 - 90	NA	75 - 150	40 - 90	40 - 90
Elastic Recovery @ 50°F, % (AASHTO T301-99)	NA	NA	NA	NA	NA	NA	NA	NA	NA	25 Minimum	NA	NA
Ductility @ 77°F, cm (AASHTO T 51)	NA	40 minimum	NA	40 minimum	NA	40 minimum	NA	40 minimum	NA	NA	40 minimum	70 minimum
Ductility @ 40°F, cm (AASHTO T 51)	NA	NA	NA	NA	NA	NA	NA	NA	NA	10 - 35	NA	NA
Softening Point, °C (AASHTO T 53-96)	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	135 minimum

* *Tests shall be performed in accordance with AASHTO T 59-97*

[A] *For Quality Control Testing, the supplier may use a defined raped testing protocol.*

[B] *Solubility in Trichloroethylene will be required on the base asphalt as QC every 3 months.*

[C] *Cement Mixing Test will not be required if emulsions are used only for tack coats.*

[D] *Stone Coating - Individual project aggregates shall be verified annually or as directed by the engineer.*

[E] *5 Day Settlement Test - The producer may conduct a 24 hour (1% Max) storage stability test in lieu of the 5 Day Settlement Test, if the emulsions are to be used within 5 days.*

Appendix A: Rapid Testing Protocol for Quality Control Testing

Method for Rapid Sieve Test

Purpose: A rapid method for sieve test of emulsified asphalt for quality control purposes.

Scope: This method identifies asphalt particles or other discreet solids larger than 20 mesh.

Safety: Utilize safety precautions and personal protective equipment according to facility procedures. Dispose of materials according to facility procedures.

Apparatus

- #20 mesh sieve having a 76.2-mm frame or a piece of #20 mesh wire cloth approximately 3" by 3"
- Sieve pan or container to retain residue during drying period
- Balance capable of weighing 1000 +/- 1 g.
- Balance capable of weighing 500 +/- 0.1 g

Procedure:

1. Obtain sample using proper sampling procedure

2. The temperature of the emulsion sample is related to the emulsion viscosity.

   For emulsions less than 100 SSF @ 25°C perform test at room temperature. Condition sample to room temperature in a closed container using an oven, water bath or allow to cool on counter followed by stirring to achieve homogeneity.

   For all other emulsions perform test at 50 ± 3°C. Condition sample in a closed container using an oven, water bath or allow to cool on counter followed by stirring to achieve homogeneity.

3. Once the sample is conditioned remove any skin that may have formed during the conditioning period.

4. Weigh sieve or screen assembly to nearest 0.1g.

5. Pour 1000g +/- 1g of emulsion through sieve or screen

6. Gently rinse sieve or screen with distilled water.

7. If no material is retained on the sieve or screen the test is considered passing.

8. If material is retained, remove free water from the bottom and sides of sieve. Care should be used to not disturb particles. Weigh wet assembly. If the difference between wet and dry assembly weights, "wet weight", is less than 1.0g (failure threshold) the test is considered passing.

9. If the wet weight exceeds 1.0g the sieve assembly will be placed in an oven to dry. During the drying period the assembly can be periodically weighed. At anytime, if the weight gain is less than 1 g (failure threshold) the test will be considered passing.

Method for Rapid Residue by Evaporation Test

Purpose: A rapid residue content test method for emulsified asphalt quality control purposes.

Scope: This method determines residue content of an asphalt emulsion.

Safety: Utilize safety precautions and personal protective equipment according to facility procedures. Dispose of materials according to facility procedures.

Apparatus

- Suitable metal container for evaporation. Capacity should be large enough to retain splatters.
- Balance capable of 1000 +/- 0.1 g.

Procedure:

10. Obtain sample using proper sampling procedure

11. Tare a metal container and weigh into it 50 +/- 0.1 g of emulsion.

12. Heat container using a direct flame or hot plate. Splattering can be controlled by adjusting the heat source to prevent localized overheating and by constantly moving container.

13. Continue heating until sample is smooth or has reached a constant weight. Record weight of container and residue.

14. Determine residue. Divide remaining net residue by the original sample weight and multiply by 100.

    For CAE-P, AE-P or AE-3 emulsions reported as distillate percentage.

    - Distillate % = 100 - % residue

    For TST-1P

    - % residue range, 56 - 59

15. If results fail to meet material specifications the standard distillation or evaporation will be performed.

Method for Rapid Saybolt-Furol Viscosity Test

Purpose: A rapid viscosity test method for emulsified asphalt quality control purposes.

Scope: This method determines saybolt-furol viscosity of an asphalt emulsion.

Safety: Utilize safety precautions and personal protective equipment according to facility procedures. Dispose of materials according to facility procedures.

Apparatus

- Saybolt-furol viscometer
- #20 mesh sieve or a piece of #20 mesh wire cloth
- Suitable transfer containers
- Suitable timing device

Procedure:

16. Obtain sample using proper sampling procedure

17. Inspect viscometer, making sure it's clean, at the proper test temperature and outlet stopper is in place. Place an approved receiving flask under viscometer outlet.

18. For emulsions tested at 77° F - condition approximately 300g of emulsion to 78 – 80° F in a closed container using an oven, water bath or allow to cool on counter followed by stirring to achieve homogeneity.

    For emulsions tested at 122° F - condition approximately 300g of emulsion to 125° F in a closed container using an oven, water bath or allow to cool on counter followed by stirring to achieve homogeneity.

19. Once the sample is conditioned transfer emulsion through a #20 mesh sieve into the viscometer so that a portion begins to overflows into the outer rim.

20. Without further conditioning time, simultaneously remove stopper and start the timer. Emulsion should flow into the receiving flask. Stop timing when flask is filled to fill line.

21. Determine viscosity by multiplying fill time by tube correction factor.

22. If the following limits are exceeded the standard method will be performed

    77 F Viscosity 10% of Lower Spec Limit < Vis < 10% of Upper Spec Limit

    122 F Viscosity 5% of Lower Spec Limit < Vis < 5% of Upper Spec Limit

Standard Recommended Practice for

Certifying Suppliers of Emulsified Asphalt

AASHTO Designation: x xx-xx (2009)

1. SCOPE

1.1 This standard specifies requirements and procedures for a certification system that shall be applicable to all suppliers of emulsified asphalts (EA). The requirements and procedures shall apply to materials that meet the requirements of either AASHTO M 140 or AASHTO M 208/M 316, (Cationic Emulsions/Polymer Modified Cationic Emulsions), which are manufactured at refineries, terminals, in-line blended, or otherwise produced for use in paving applications.

1.2. This standard may involve hazardous materials, operations, and equipment. It does not purport to address all of the safety problems associated with this use. The user of this standard shall be responsible for appropriate safety and health practices.

2. REFERENCED OR RELATED DOCUMENTS

2.1. *AASHTO Standards:*

- M140, Standard Specification for Emulsified Asphalt
- M208, Standard Specifications for Cationic Emulsified Asphalt
- M316, Standard Specifications for Polymer Modified Cationic Emulsified Asphalt
- T 59, Standard Method of Tests for Emulsified Asphalts
- T 40, Sampling Bituminous Materials

2.2. *ASTM Standards:*

- D244
- D977
- D7497
- D3665

3. TERMINOLOGY

3.1 AAP—AASHTO accreditation program.

3.2. ASC—approved supplier certification.

3.3. AS—approved supplier.

3.4. EA—emulsified asphalt.

3.5. *supplier—a supplier* shall be defined as one who produces the final product or who makes the blend or modification that alters the properties of the EA specified in either M140 or M208/M 316. A supplier shall be a refinery, a terminal, or a paving contractor. If no modification is made to the EA after its initial production at the refinery or terminal, the refinery or terminal shall be the supplier and must provide the certification. If any modifications are made to the EA at the terminal, the terminal shall be a manufacturing site and considered the supplier. As a manufacturing site, the terminal must provide certification. If any modification, blending, or blending of EA from different sources is made at the job site, the Paving contractor shall be the supplier and must provide the certification.

3.6. *agency*—agency shall be defined as a state highway agency or other agency responsible for the final acceptance of the EA.

3.7. *specification compliance* testing—complete testing in accordance with the either M 140 or M208/M 316 specification requirements. (Update for new specifications as they come available)

3.8. *quality control testing*—the quality control testing shall be described in the supplier's quality control plan. The supplier's quality control plan shall be approved by the agency.

4. SIGNIFICANCE AND USE

4.1. This standard specifies procedures for minimizing disruption of EA shipments. This is accomplished by a certification system which evaluates quality control and specification compliance tests performed by the supplier on samples obtained prior to shipment.

4.2. The number of EA available under M 140 or M 208/M 316 may require construction of additional storage facilities if the procedure of "sample and hold while testing" is followed exclusively. The addition of new storage capacity at a facility may be infeasible at some locations. Standardization of procedures that allow shipment under an approved supplier certification provides the flexibility needed to use existing facilities and to limit the shipment disruptions.

4.3. This standard provides information on the following activities:

4.3.1. General requirements that the supplier shall satisfy to be given approved-supplier status;

4.32. Minimum requirements that shall be included in a supplier's quality control plan;

4.3.3. General requirements that the agency shall satisfy before certification;

4.3.4. Procedure for shipping EA under an ASC system;

4.3.5. Procedure for agency monitoring of an ASC system at the shipping facility; and

4.3.6. Procedure for field sampling and testing of EA shipped under an ASC system.

5. HAZARDS

5.1. The safety requirements of the field and/or laboratory organization and/or OSHA shall be observed.

6. SAMPLING

6.1 All test samples required by this standard shall be obtained in accordance with T 40 and ASTM D 3665. The use of a random sampling procedure is mandatory to the establishment of a valid certification program.

7. TESTING REQUIREMENTS

7.1. Testing required for this standard shall be performed by a laboratory currently accredited by the AAP and be performed according to T 59. Agencies may require that the manufacturing site have a quality control laboratory on site with sufficient testing equipment to guide quality control in day to day manufacturing. Any satellite laboratory of a supplier that performs required testing under this standard shall be identified in the AS quality control plan (Section 9) and shall be approved by the agency.

Note 1—Cost of this inspection shall be borne by the source of data. Satellite laboratories may be inspected by the source's primary AMRL inspected laboratory staff. A copy of the report of the satellite laboratory inspection shall be provided with the test report, if requested.

Note 2- Agencies may elect to allow participation in AMRL Proficiency Sample Program and regional round robin programs in lieu of AMRL accreditation until such time as Supplier may reasonably become AMRL Accredited for AE.

8. SUPPLIER REQUIREMENTS

8.1. The supplier shall submit a written request to the agency for authorization to ship EA under the ASC system and shall list the EA to which the request applies.

8.2. The supplier shall allow the agency to visit the production and/or shipping site to observe the supplier's quality control activities, to inspect the facilities and to obtain samples for test.

8.3. The supplier shall submit to the agency for approval a complete quality control plan, which complies with the requirements of Section 9.

8.4. The supplier shall follow the procedures described in the approved quality control plan.

8.5. The supplier shall establish a continuing test record for each test required on each EA included in the written request prepared to satisfy the requirements of Section 8.1.

8.6. The supplier shall forward to the agency the initial series of test data for each EA included in the written request prepared to satisfy the requirements of Section 8.1. The supplier shall also obtain and provide a split sample for the agency if requested.

8.7. The supplier shall submit to the agency all reports required by this standard in a form approved by the agency.

8.8. The supplier shall have a satisfactory record of compliance with governing specifications. Judgments by the agency concerning this requirement shall be based on the test results furnished by the supplier and satisfactory results when the monitoring and field tests are compared with supplier tests.

9. SUPPLIER QUALITY CONTROL PLAN—MINIMUM REQUIREMENTS

9.1. The supplier's quality control plan shall identify the following:

9.1.1. Facility type (refinery, terminal, manufacturing site);

9.1.2. Facility location;

9.1.3. Name and telephone number of the person responsible for quality control at the facility;

9.1.4. The quality control tests to be performed on each EA; and

9.1.5. Name and location of the laboratory performing quality control tests on the EA that is shipped.

9.2. The supplier's quality control plan shall include a declaration stating that if a test result indicates that a shipment of EA is not in compliance with the purchase specifications, the supplier shall (1) immediately notify the agency of the shipment in question, (2) identify the material, (3) cease shipment until material complies with the specification, (4) notify the agency prior to resuming shipment; and (5) implement any mutually agreed upon procedures for the disposition of the material. In the event a mutual agreement is not obtained, the specifying agency shall have final authority in the decision on specification compliance.

9.3. The supplier's quality control plan shall describe method and frequency for initial testing, quality control testing and specification compliance testing.

9.3.1. Initial Testing - For each grade of EA to be supplied, specification compliance testing (complete testing per either M140 or M208/M 316) shall be performed for at least three consecutive lots. A lot may be a fixed batch of material or a specified quantity in a continuous operation (see Note 3). The supplier and the agency shall agree on a lot size. The agency must approve any change to a lot size.

Note 1 - If a batch operation is used to manufacture the EA, a tank may be defined as a lot. Lot size would be the amount of material batched into the tank. If a continuous process (in-line blending or a shipment from "live" tanks) is used to manufacture the EA, lot size may be obtained at random during the production for continuous operations. Lot size shall depend on the production method used and the quantity of the EA produced.

9.3.2. *Reduced Frequency of Testing for Specification Compliance*—If approved by the agency, the frequency of testing for specification compliance may be decreased if the individual M 140 (or M 208/M 316) test result for every sample of the initial testing is within specification by at least the tolerance of the test method for each of the required test methods. With the approval of the agency, the frequency of testing may be further reduced as long as the individual test results continue to meet the tolerance criterion. If the tolerance criterion is not met, every lot will continue to be tested for the individual M 140 (or M208/M 316) property until three consecutive lots comply with the tolerance criterion.

9.3.3. *Minimum Frequency*—Specification compliance testing shall be run at the minimum frequency required by the agency for each EA that is supplied.

9.4. The supplier's quality control plan shall include a statement that the supplier will prepare monthly summary reports for all quality control and specification compliance tests performed during that period and will submit them to the agency on request.

9.5. The supplier quality control plan shall provide an outline of the procedure to be followed for checking transport vehicles before loading to prevent contamination of shipments. The outline shall include a statement that the transport vehicle inspection report, signed by the responsible inspector, shall be maintained in the supplier's records and will be made available to the agency on request.

10. AGENCY REQUIREMENTS

10.1. The agency shall verify that the supplier's quality control plan is adequate. The agency may visit the shipping site when required.

10.2. The agency shall notify the supplier that the supplier's application for AS status has been granted. The notification shall include a list of the EA covered.

10.3. The agency shall verify that the supplier's primary testing laboratory is currently AASHTO accredited or has met suitable requirements until accreditation may be reasonably obtained.

10.4. The agency may perform split sample testing in accordance with Section 12.

10.5. The agency may perform quality assurance sampling and testing in accordance with Section 13.

10.6. The agency shall authorize shipment of each listed EA under the ASC system only after all ASC system requirements have been satisfied.

10.7. The agency shall inspect the operations of the supplier's facility related to the EA shipments when required.

10.8. The agency shall notify the supplier when split sample data versus supplier sample data does not compare within the limits established in Sections 12 and 13.

Note 1—The supplier and/or the paving contractor may take a split sample for comparison purposes, If a split sample is taken, a third sample shall be taken as a referee. The referee sample shall be retained either by the agency or by the paving contractor until the test results are available. If the test results are disputed, the agency and supplier shall agree upon a test procedure for the referee sample.

11. REQUIREMENTS FOR SHIPPING EA BY AN APPROVED SUPPLIER

11.1. The supplier's quality control plan as approved by the agency shall be implemented. (See Section 9,) 11.2. The supplier shall make EA shipments covered by the certification as dictated by shipping schedules.

11.3. Each shipment shall be accompanied by two copies of the bill of lading, which shall include (1) the name and location of the supplier, (2) the grade or type of emulsion material, (3) the quantity of material shipped, (4) the date of shipment, (5) *a statement certifying the material meets specification requirements, and* (6) a statement certifying that the transport vehicle was inspected before loading and was found acceptable for the material shipped.

Note 4—On any invoice or Bill of Lading, it is recommended that US Gallons or Short Tons be used as the primary units of measurement.

11.4. If the specification compliance test results do not conform to EA specifications, the supplier shall remove the non-compliant material from the shipping queue as outlined in Section 9.2.

11.6. Based on the agency's split sample testing on the referee sample (see Note 7), price adjustment may be made for material that does not comply to the specified EA requirements. The price adjustment shall be determined by the agency. If problems with the EA recur at the job site, the agency may suspend use of the EA until the cause for noncompliance with specifications can be identified and corrected.

12. SPLIT SAMPLE TESTING

12.1. The agency may test split samples that are obtained at random from the supplier's facility.

Note 1—Split samples will be obtained from the same general points in the supplier's shipping process that the supplier's samples are taken, for example, from a storage tank at the refinery, from a holding tank at a terminal, or from a loading line downstream from the blending operation of an in-line blending process.

12.2. The agency shall determine the frequency of split sample testing.

12.3. If the split sample data and the supplier test data are not within the test tolerance specified (see Section 15), an immediate investigation shall be conducted to determine the reason for the difference between the data. Unless available facts indicate otherwise, the investigation shall include a review of sampling and testing procedures of both supplier and agency.

12.4 Samples shall be tested within a reasonable time to ensure that the integrity of the sample taken is fairly representative of actual production materials. For Rapid Set emulsions, care should be taken to evaluate the sample within 48 hours of sampling and to maintain temperatures during the storage and transportation process prior to testing. For Medium and Slow Set emulsions, testing shall occur within 7 days. Samples shall be protected and not allowed to freeze or be exposed to extreme temperatures during transportation to Agency or third party laboratories.

13. FIELD SAMPLING

13.1. The agency or Paving contractor may design the field-sampling plan to accomplish the intended purpose,

Note 6—Field samples may be taken for several different purposes: to determine the type and magnitude of any changes in the properties of theEA during transportation and storage; to determine that the material received in the field is the material ordered; or to verify that the quality control/quality assurance system is performing as intended.

13.2. The agency may obtain samples from the field facility on a random basis for the purpose of quality assurance.

13.3. The agency shall determine a minimum frequency of field sampling that shall be adequate to satisfy the purpose for which the field samples are taken,

13.4. If the field test data are not within tolerance, the agency shall immediately notify the approved supplier and paving contractor. Unless available facts indicate otherwise, an investigation shall be conducted that shall include a review of quality control and sampling and testing procedures for field sampling and split sampling. When the differences are not readily resolved, all facts available to identify the problem shall be used to decide on an appropriate course of corrective action.

13.5 If the EA fails to comply with the specification, the supplier or paving contractor producer shall immediately investigate the possibility of contamination in transport vehicles, field storage tanks, pumps, lines and at handling facilities. If the cause is determined, correction shall be made promptly. If field test data show a serious departure from the specifications, the supplier or paving contractor shall delay the project work pending corrective action.

14. REPORT AND DATA SHEETS

14. 1. *Supplier Reports*—The supplier shall prepare the reports described in Sections 8.1, 8.3, 8,6, 8.7, 9.2, 9.4, 9.5, 11.2, and 11.3.

14.2, *Agency Reports*—The supplier may request copies of the split sample test results and field test data.

15. KEYWORDS

15. 1 Approved Supplier (AS); Approved Supplier Certification (ASC); certification system; certified shipments; Emulsified Asphalt (EA) Certification.

Official Announcement

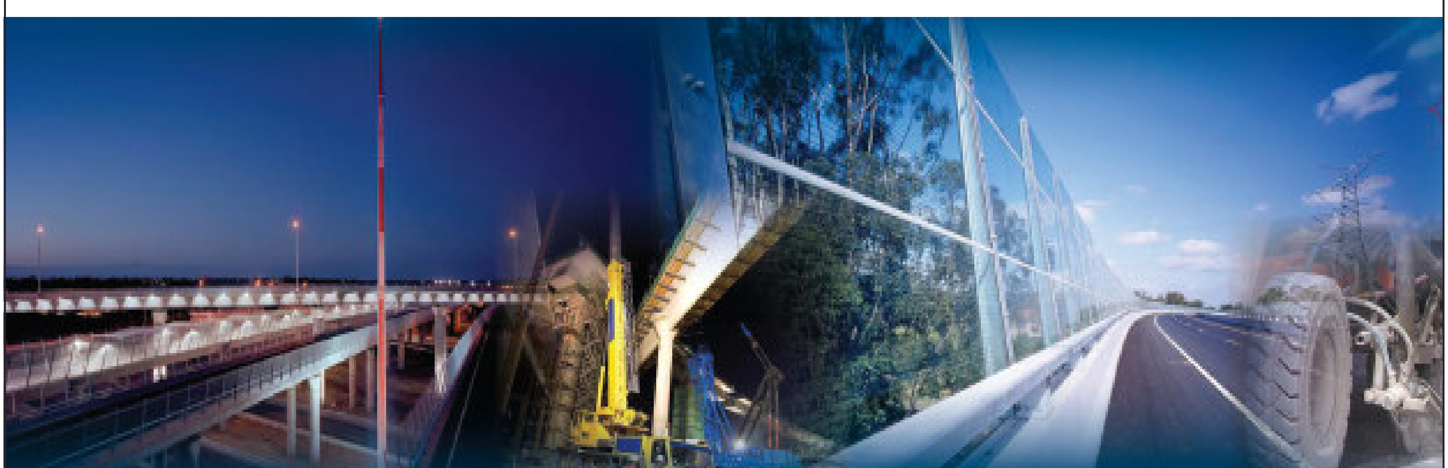
International Road Federation

Preserving our Highway Infrastructure Assets

August 4 - 7, 2009
Orlando, Florida

Preliminary Program

Better Roads. Better World.



Affordable, Safer, and Environmentally Friendly Pavement Preservation Practices

As the aging pavement network condition deteriorates, there is a need to increase investments in maintenance and rehabilitation treatments to maintain or restore the condition at acceptable levels. If pavement preservation treatments are deferred, due to an agency policy or just simply because of lack of funds, the user's costs and agency funds needed to maintain the roads in good condition will increase dramatically.

The deterioration of the pavement condition over time combined with the limitation of funds available for maintenance and rehabilitation creates a challenging problem. Pavement preservation plays a key role in addressing this problem. Affordable, safer, and environmentally friendly pavement preservation practices are needed. Funding agencies as well as communities served by roads will be greatly benefited from these practices. As the global leader in advocating for better, safer and environmentally friendly roads, IRF will bring together public and private sector experts to examine policies and practices affecting pavement preservation and discuss new strategies in moving forward.

The workshop will focus on such key issues as the role of new materials and technologies for pavement preservation, new funding strategies and contracting models to sustain pavement preservation, and latest tools to better managing pavement networks. It will connect international transportation leaders with people at all levels of government and private industry to discuss the challenges and cutting-edge solutions. This workshop will be the first on a venue of international events hosted by the IRF with the aim of fostering affordable, safer, and environmentally friendly roads.

Registration Fees:

EARLY BIRD (PRIOR TO APRIL 30, 2009):

- $ 995 MEMBER RATE
- $1599 NON MEMBER RATE

MAY 1 – JULY 15

- $1595 MEMBER RATE
- $2199 NON MEMBER RATE

AFTER JULY 15

$2495 ALL

To register visit www.irfnews.org or fax the registration form with payment information to Magid Elabyad at +1 703 535 1007.

For more information please contact Magid Elabyad at melabyad@irfnews.org or call +1 703 535 1001

Preliminary Program

Tuesday August 3

- Registration and pick up seminar materials

Wednesday August 4

- What is a Successful Pavement?
- Green Roads for Better Living
- Bitumen Sustainable Development
- Asphalt Modification Processes

LUNCH with keynote

- Evolution of Pavement Maintenance Techniques
- New Asphalt Technologies for Pavement Preservation
- Maintenance Techniques for Preserving Rigid Pavements
- Use of Recycled Materials in Roadway Construction

Welcome Reception

Thursday August 5

- Considerations for Flexible vs. Rigid Pavement Designs
- Technical-Economical Project Evaluations
- Warm Asphalt Mix Technologies
- New Experiences with Rubberized Asphalt Pavement

Networking Lunch

- Highway Development and Management
- Group Project Exercise

Friday August 6

- Group Project Presentations
- Latest Technologies: Pavement Management: Best Practices and Tools *Best Practices for Preserving Low Volume Roads Pavement Maintenance Contracts*
- Executive Summary: “Affordable, Safer, and Environmentally Friendly Pavement Preservation Practices”

Closing Comments and Certificate of Completion Awards

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Alexandria, VA 22314 USA
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E-mail: info@irfnews.org

Preserving our Highway Infrastructure Assets - Registration Form

FAX COMPLETED FORM TO +1 703 535 1007

NAME: ______________________________

TITE: ______________________________

ORGANIZATION: ______________________________

ADDRESS: ______________________________

CITY: ____________________ STATE: ____________________

COUNTRY: ____________________ POSTAL CODE: ____________________

TELEPHONE (+ COUNTRY CODE): ______________________________

EMAIL: ______________________________

EARLY BIRD REGISTRATION FEES—BEFORE APRIL 30, 2009(USD)

Accommodations are all inclusive (2 days / 3 nights). Breakfast and lunch will be served Wednesday - Friday.

□ **$995 – IRF MEMBER RATE** □ **$1,599 – NON MEMBER RATE**

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Name (as it appears on card): ______________________________

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OR, SEND CHECK WITH COMPLETED REGISTRATION FORM TO:

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CP² CENTER NEWS

Newsletter of the California Pavement Preservation Center

No 9 March 2009

Richard Land

Interview with Richard Land, Caltrans Chief Engineer

Rick Land is currently the Chief Engineer and Deputy Director for Project Delivery for the California State Department of Transportation (Caltrans). He is a graduate of California State University, Sacramento, and has been with the Department for over 30 years. Rick has spent most of his career working in various project delivery functions. Prior to his current assignment Rick was Chief of the Department's Structure Design operations.

As the Chief Engineer and Deputy Director, Rick oversees the statewide development and construction of transportation improvement projects on California's State Highway System. The Department's project delivery divisions under his direction include project management, environmental analysis, right-of-way and land surveys, design, engineering services, and construction. The Department currently has over 2500 projects in some stage of development, of which approximately 550 projects worth more than $9.5 billion are under construction.

How will Caltrans deal with the impact of limited funding for pavement preservation projects?

Caltrans will continue to focus its limited funding on those areas around the state where further deterioration will cost us significantly more in the future. The areas where the cost to completely rehabilitate failed pavements and impact on traffic are high should receive more attention than areas where such work is not as costly or disruptive. The prioritization of such work will be done through collaboration between the recently created Division of Pavement Management and each individual district. In the not too distant future, we plan to incorporate a more robust pavement management system into the prioritization and decision making process. By using an enhanced pavement management system, making decisions based on observed surface conditions, known subsurface conditions and performance curves developed through our research efforts, we can improve the effectiveness of our future pavement preservation expenditures.

What are the major differences in delivery for preservation vs. rehabilitation projects?

Rehabilitation projects, including roadway rehabilitation and pavement rehabilitation projects, are projects that focus on pavement conditions which are in need of major repair or replacement. Rehabilitation projects also address drainage facility rehabilitation as well as the need to bring the existing roadway up to the current safety design standards. Other work can include increasing shoulder widths, improving sight distance, increasing vertical clearances and adding roadside safety features to reduce the potential for roadway runoff accidents. These projects follow the full project development process and must be scoped, estimated and programmed into the State Highway Operation Protection Program (SHOPP). Projects are then developed, designed and delivered, generally over a three to four year period after initial project evaluation. These projects typically occur on roadways that have deteriorated beyond the scope of a preventive maintenance effort.

Preventative maintenance and pavement preservation projects focus on keeping the roadways in good condition and postpone the need for major rehabilitation. These projects are generally scoped for the next year's plan, and then developed, designed and delivered in a much shorter time frame than needed for rehabilitation projects. The work of these projects is typically confined to the existing roadway and, therefore, these projects have few environmental and utility or right of way issues that need to be addressed during design and construction. Modifications to the existing roadway are not usually part of preservation projects and require significantly less time to complete the construction phase.

Continued next page

In this issue

What will be Caltrans' emphasis in the next three to five years on preservation vs. rehabilitation projects?

Highway rehabilitation projects for both highways and bridges will continue to be a major focus in the SHOPP over the next few years. But without a significant increase in available funding, rehabilitation needs will continue to outpace projects to address those needs. The 2010 SHOPP programming cycle suggests that approximately $6.2 billion will be needed annually to meet SHOPP needs, but only $1.5 billion will be available.

Pavement that is still in good condition will be evaluated for preventative maintenance needs and Caltrans will continue to advance preservation activities as much as current funding permits. Currently, we have about $206 million per year allocated for preservation projects.

The federal government has passed a significant stimulus package to help rebuild the infrastructure. Is California ready for this, and if so, what will be the priorities addressed?

Caltrans has been working with the Federal Highway Administration and our local partners to prepare for the economic stimulus package. Our focus is on projects that can move quickly to the construction phase to create or sustain jobs. The California Business Roundtable has reported that 18,000 jobs are created for every $1 billion in construction projects and this is significant to California's economy and financial recovery. The portion for highways coming to California is estimated to be about $2.6 billion.

Controlling project costs has been difficult in recent years with the volatile changes in fuel and materials costs. How will Caltrans deal with the cost volatility in the future?

Keeping up with changes in fuel and material costs has been challenging. In addition to constantly monitoring bid prices received for our advertised construction contracts, we continue to monitor economic forecasts for various construction material commodities typically used in transportation improvement projects and adjust our estimates accordingly. The Caltrans Division of Construction also meets regularly with our industry partners to discuss the cost volatility for paving asphalt, fuel and steel products. In response to industry concerns, Caltrans has facilitated joint task teams with the Associated General Contractors of California (AGC), the Engineering and Utilities Contractors Association (EUCA) and the Southern California Contractors Association (SCCA) to discuss price indexing for fuel and steel products. The teams are also developing recommendations for possibly adjusting price indexes currently being used on Caltrans construction projects for asphalt products.

Surface preservation warfare

By: C Bryan Graves. P.E.

Bryan Graves is a graduate of CSU, Chico, and is currently the Maintenance Superintendent for Butte County. A north state leader in the use of pavement preservation treatments, including pavement recycling, he shares some of the issues facing local agencies in California.

An example of some of the other challenges that take time away from pavement surface treatments.

As the person in charge of maintaining the roadways in Butte County located in Northern California, I have to get into the trenches in order to preserve what we have. Many different foes raise their ugly heads to challenge the time, money and resources available to combat pavement preservation issues. Time is always whittling away at the strength and beauty of the roadway surface, turning from black to grey, and sometimes to white or even dirty brown. There are several different innovations in the surfacing and preservation world, the trick is to pick the right treatment, apply it correctly and keep within budget. Most importantly, we have to focus on the budget, as the amount that is made available is just a fraction of what it really costs to keep the roads in the condition the politicians expect and the traveling public demands. So where does that leave the status of surface preservation warfare?

Let us first look at time and how it affects the roadways. We have roads today that are made up of a mixed bag of components. Some began as market, agriculture and logging gravel roadways that were surfaced with oil and rock. Limited and inadequate structural sections usually prove to make the roads a burden, as we see increased traffic and

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Pavement surfaces can vary just from age and required maintenance.

a greater percent of truck traffic. Then there is the engineered and designed roadways using aggregate base and asphalt pavement. History is showing that these roadways were not designed to carry the demanding loads of today and in several cases, what was designed and what is constructed are two different things. Then the twist comes when the concrete slab 'Old State Highway' was gifted to the County. We didn't know much about something that had a white surface, so we made it wider and placed pavement over it to make it black, but now time is letting us know that what was underneath affects the performance of the surface. Furthermore, all these above-mentioned road types already have had some type of repair done on them, making them a patchwork of non-homogeneous asphalt, streaked with crack seal webs, and not so desirable to drive upon.

Since there are so many different road make-ups, designs and demands, I have attempted to arm the County with several options in order to best preserve our road network assets. Butte County has always been aggressive with its 'chip seal' program ranging from thirty to ninety miles of roads constructed annually by contract force account, in-house labor, or a combination of both. The effort made through the years shows success when we start incorporating our Pavement Management System to survey and catalog surface conditions, in order to aid in the selection of a future treatment options. We have used 'single' chip seals, 'double' or 'armor' chip seals, 'scrub seals' using single layer methods, 'cape seals', as well as asphalt pavement overlays using leveling courses or pavement over fabric. We are always looking for the best treatment for the condition, which considers both old and new methods, cost-benefit and reliability.

If the pavement age, conditions and methods did not present enough of a challenge, the County also has to constantly juggle fluctuating budgets and constraints. If one looked at the 1,000 miles of surfaced roads throughout Butte County, divided by 20 years of service life allowed for each surface treatment, we would have to average 50 miles of surface treatment a year to maintain the current condition. The money available to complete pavement preservation in the past has come from the 'Road Fund', which is solely made up of 'Gas Tax' dollars. The gas tax has remained about the same for over 15 years. The cost of materials in many cases has tripled, along with increases in equipment and labor rates. This equates to the same amount of money in the pot divided by higher cost, equals less work done on the roadways. So we are challenged at the County to do more for less. We must take advantage of additional funding sources when they come our way and get more productive with the pavement surfacing that is completed. With the recent financial turmoil facing our nation, the amount of money available for infrastructure is going to be threatened. We have to continue the battle, make progress and arm ourselves to maintain what we have.

Surface preservation can have many different forms. This is a 'scrub seal' being placed by county crews.

A good roadway for the future with the proper preparation, application and finishing touches.

European experiences with use of noise reducing pavements

By Hans Bendtsen, Senior Researcher, Danish Road Institute/Road Directorate and visiting researcher at the University of California Pavement Research Center, Davis

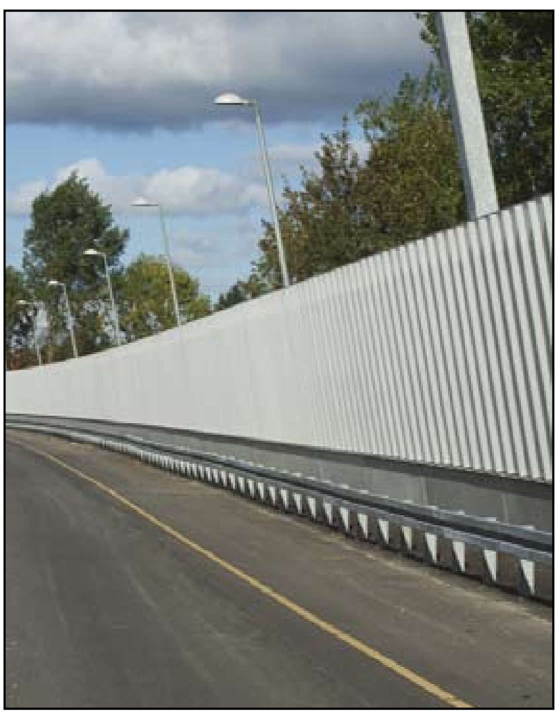
Linus Motumah, Senior Transportation Engineer, Caltrans, Office of Pavement Engineering

Bruce Rymer, Senior Transportation Engineer, Caltrans, Division of Environmental Analysis

There is increasing focus in Europe on applying noise reducing pavements on the road networks. Caltrans contracted with the Danish Road Institute to prepare an overview of the current state of the art. Caltrans plans on using these techncologies to reduce road noise.

Quieter pavement is a new concept for the construction or rehabilitation of roadway surfaces intended to reduce the impacts that tire/pavement noise has on the highway environment. To facilitate better understanding of the acoustic and structural performance of quieter pavements, in 2006, Caltrans implemented its Quieter Pavement Research (QPR) program. The program is designed to develop surface treatment strategies, materials, design specifications, and construction methods that will result in quieter pavements that are also safe, durable, and cost-effective. Caltrans QPR effort involves research on tire/pavement noise characteristics of flexible and rigid pavement surface treatments and textures, including the effects of pavement aging and subsequent surface distresses on tire/pavement noise levels. As a part of this effort, in June, 2008, the Danish Road Institute/Road Directorate (DRI-DK) delivered the report "Use of Noise Reducing Pavements – European Experience" It can be found at www.dot.ca.gov/hq/esc/Translab/ope/DRI-DK-TechNote-69-Report.pdf. The main findings from this report are presented in this article.

The open CPX noise trailer "deciBellA" from the Danish Road Institute.



An express ring-road around Copenhagen in Denmark combines noise reducing pavement, four meters (13 ft.) high noise barriers and façade insulation.

Use of noise reducing pavements is a trend in Europe

There is increasing trend in Europe on using noise reducing pavements as a cost-effective measure to reduce the impacts of traffic noise on the highway environment. In most European countries, noise reducing surface treatments and textures are often used on a case-by-case basis in new construction and pavement rehabilitation projects. Although noise reducing pavements are more often becoming part of the "toolbox," only a few countries have an explicit policy for use of them.

The Netherlands applies noise reducing porous asphalt on all major highways; while in Denmark, where noise-reducing surface courses are frequently used on both new and rehabilitation projects, a policy is still under development by the Danish Road Directorate. As in California and other states in the US, it is the owner/operator who pays for the quieter pavement surface treatments.

Quieter pavements can be used in conjunction with other traffic noise abatement measures like sound walls and building façade insulation, etc. A quieter pavement strategy is an option that addresses the problem at the source (tire-pavement noise) and it is often the most cost-effective measure for noise abatement. The noise reduction obtained by applying quieter pavements depends very much on the type, age and condition of pavement used for comparison or reference. The reference pavements used around Europe are typically chosen from what would have been the most likely pavement type used for major highways.

Danish noise reduction classification system

The introduction of the Danish "so- called" SRS noise reduction classification system for quieter pavements has been an important development towards the use of quieter pavements in Denmark. SRS is the acronym for the Danish wording of Noise Reducing Surfacing. The system is based on the Close Proximity Method (CPX) for noise measurements, which is similar to the California On Board

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Sound Intensity method (OBSI) used in California and other states. In order to ensure reliability and uniformity, the SRS system allows various independent providers of CPX measurements to offer their service as long as they participate in an annual field calibration of the equipment. The SRS system classifies quieter pavements in three classes: A, B and C, where class A surface treatments exhibit the highest noise reducing effect and class C the lowest.

The Danish SRS system provides a process for marketing and contracting quieter pavement surface treatments. When a pavement project is advertised, the bid documents can specify if a quieter pavement surface treatment is required to achieve noise reduction levels specified in one of the three classes:

A: Very good noise reduction: Noise Reduction >7.0 dBA

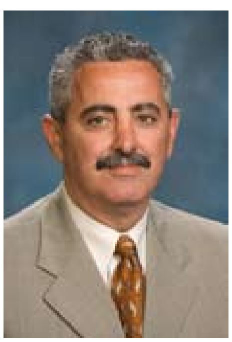
B: Good noise reduction: 5.0 <Noise Reduction <7.0 dBA

C: Noise reduction: 3.0 <Noise Reduction <5.0 dBA

This classification system enables the contractor to produce documentation of the noise reduction of a specific pavement by comparing measured values with a national reference value, such as dense graded hot mix asphalt (HMA). The intent is to certify the noise reduction benefits of pavement surface treatments and textures including new products. The system also enables local agencies — not skilled in noise considerations — to make prudent decisions on the use of proven noise reduction solutions that fit their needs and funding constraints.

Typical On Board Sound Intensity (OBSI) dual probe installation.

Due to more than a decade of research and development carried out in cooperation between the Public Road Research Institute (DRI-DK), transportation agencies and pavement industry, it has been possible to introduce quieter pavements in Denmark. The warranty periods for noise reducing pavements in Denmark are the same as for standard pavements (legally five years) but there is no established practice yet as to how the warranty covers the acoustical performance. This may be changed when more experience is gained.



Basem Muallem

Pavement preservation in District 8

By Basem Muallem, P.E., California Department of Transportation

The California Department of Transportation (Caltrans) is committed to pavement preservation and preventive maintenance as a logical substitute for pavement rehabilitation and/or reconstruction. In order to achieve this mission, a serious effort has to be in place to move from a "worst first" and reactive mode to preservation and maintenance. To keep the investment in our highway assets in good condition, we need first to identify our investment, and then keep a log of the history of our pavement with emphasis on preventive treatments that have already been applied. Caltrans District 8 has the right attitude in endorsing pavement preservation with a plan to account for its current inventory and future treatments.

Background

Caltrans District 8, which is located in San Bernardino, is responsible for about 6850 lane-miles of flexible and rigid pavement and their current condition is:

- 26% of pavement lane miles need rehabilitation
- 1700 lane miles of highways require immediate attention
- 90% of damaged pavement is in outer lane

About 75% of the freeway system was built between 1959-1974. The Maintenance Division in District 8 has had a mission to manage our staff and watch for cost overruns. In addition, we need to emphasize ongoing communication with field staff, and inform the field of status/issues so they can optimize the use of their resources.

Definition of preventive maintenance

From AASHTO's Standing Committee on Highways, Preventive Maintenance is defined as the planned strategy of cost effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of

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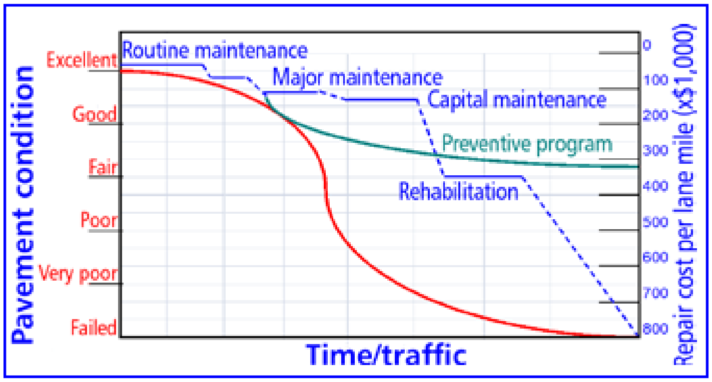


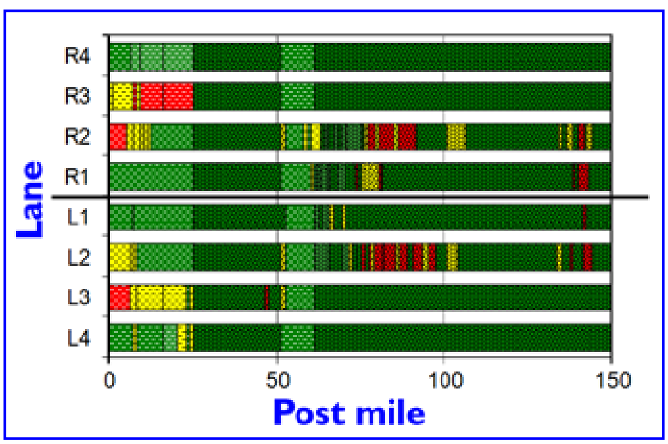
Figure 1. Pavement condition vs. cost to repair

system (without increasing structural capacity). We need to employ the "4R" strategy: right time, right strategy, right location, and the right material. In the United States, the cost to maintain the system at its existing condition level is about $53.6 billion annually. The current spending level is $38.3 billion per year and the cost estimate to bring the entire system up from its current level of "fair to poor" to a "good" level would exceed $200 billion (Figure 1).

Pavement condition survey (PCS) bar chart development

Based on the pavement condition survey, every lane-mile in Caltrans District 8 was color-coded, which reflects the actual level of distress as shown in Figure 2. We used post mile as the x-axis and individual lane as y-axis. Green means the pavement is in good condition and is a preventive maintenance candidate, yellow indicates the pavement has minor structural problems and red indicates a major structural problem. Pavement type was also considered. The dark color reflects flexible pavement, and the light color represents rigid pavement. By reviewing the bar chart in Figure 2, we are able to precisely identify the condition of our freeways.

Figure 2. Pavement condition bar chart



Pavement management chart development

After the pavement condition bar chart is constructed, the next step is to develop the pavement management chart as shown in Figure 4. We use the pavement condition as the x-axis and the 10-year projection as y-axis. According to their construction acceptance dates, existing projects are added to the chart. The projects are color-coded in this figure: highway maintenance (HM) projects in orange, capital rehabilitation projects in violet, and capital preventive maintenance (CAPM) projects in blue.

A decision tree (Figure 3) for the pavement had to be utilized in order to plot the 10-year plan for rigid and flexible pavements. It consisted of the following treatments, including Major Maintenance (HM), Preventive Capital Preservation (PCAP), Corrective Capital Preservation (CCAP), and Rehabilitation (Rehab).

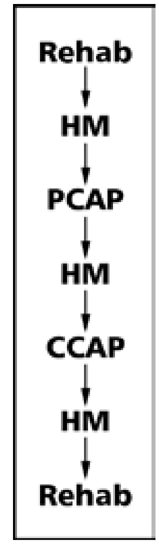


Figure 3. Decision tree for maintenance and rehabilitation

Pavement management

With the decision tree in place, and the existing projects identified, the pavement strategy for the forecasted 10-year plan was plotted for both flexible as well as rigid pavements. The total cost/needs are clearly plotted for the 10-year projection (figure 4) with emphasis on the highway maintenance, capital preventive maintenance and rehabilitation programs.

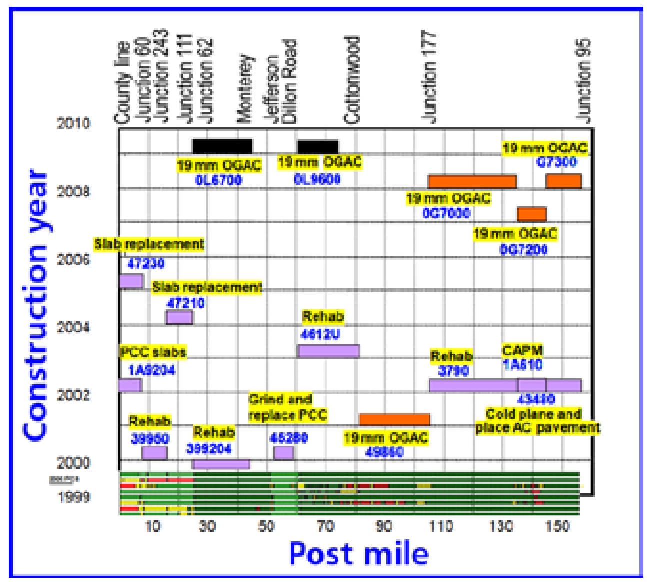


Figure 4. Pavement management chart.

Summary

Pavements are our most expensive investment, and using preventive maintenance is a significant departure from the historical way of doing business and accordingly is definitely the most cost-effective alternative. The adoption of a preventive maintenance program is a shift in philosophy from "worst first" to earlier intervention with emphasis on preventive maintenance. To effectively safeguard our huge investment, a ten-year plan should be available with existing as well as projected projects identified. With our goal identified and management support secured with dedicated funding, pavements can be maintained in good condition for a prolonged period. This is no longer a myth, but a proven reality.

PPTG activities

All Members meeting held in December 2009

Dragos Andrei

Shadi Saadeh

Cathrina Barros

The All Members meeting was held in the offices of Los Angeles County on December 9, 2009. Shakir Shatnawi welcomed over 60 people in attendance at this meeting. He introduced Professors Dragos Andrei of Cal Poly, Pomona, and Shadi Saadeh of CSU, Long Beach, as new additions to the Center in Southern California. These universities are part of the Center's growth plan.

Shakir went on to described the new Division of Pavement Management. The three offices within the new division include Engineering, Preservation, Systems Management and Programming. Currently there is a search for a new state pavements engineer. Major projects of the new division include the development of a new Pavement Management System, Integrating Pavement Preservation into all Caltrans activities, and the development of a new Pavement Design system.

Cathrina Barros discussed Caltrans work on warm mix HMA focusing on the project in District 5, near Morro Bay. She indicated that Caltrans would be doing more warm mixes next year, including the use of warm mixes on an asphalt rubber project in District 11.

Work activities from the 22 sub-groups of the PPTG were discussed. Each co-chair described the activities the groups had been working on as well as future plans.

Mary Stroup-Gardiner summarized the activities of the Center and reported that she became Technical Director of the Center replacing Gary Hicks effective December 1, 2008. Gary will continue to serve the Center and will work on other projects for the CIWMB, NCHRP, and FHWA. Professor Ding Cheng of the Center described the California Innovative Database which is available on-line at www.cp2info.org/center.

Mary Stroup-Gardiner also discussed the upcoming conferences including the Annual Pavement Preservation conference to be held in Oakland on April 8-9, 2009, and the First International Conference on Pavement Preservation to be held in Southern California in 2010.

During the wrap-up, discussions indicated current travel restrictions and the economy will be factors in attendance at upcoming meetings. Therefore, the PPTG will combine the Co-Chairs and All Members meetings so that there are only two All Members meetings a year where the Task Group Co-Chairs provide an update to the other PPTG members. These meetings will provide a mechanism for sharing ongoing Task Group work with all of the PPTG members in a more time-efficient process.

MTAG training and revisions

Due to the state budget crisis, there will be no MTAG training seminars scheduled for this calendar year. Hopefully, we will be able to provide training next year. Also, we will not be working on revisions to the MTAG during this time, except where needed to support revised specifications, such as Crack Treatment and Bonded Wearing Course.

Caltrans pavement preservation specification status

PPTG sub-task groups are busy trying to meet the May 19, 2009 deadline that the Office Engineer (OE) has established for the moratorium on the publication of new and revised specifications. The moratorium has been set to allow work to be concentrated on publication of the 2010 edition of the Standard Specifications.

Current NSSP's that are being upgraded to SSP's are Bonded Wearing Course (39-640, 660 and 680) and Micro-surfacing (37-600). Current SSP's being revised include Crack Treatment (37-400), Modified Binder Chip Seal (37-020) and Asphalt Rubber Chip Seal (37-030). All of these need to go through the Rock Products Committee approval process.

The Bonded Wearing Course specifications will be sent for review very soon. The Micro-surfacing specification was sent for review January 8 and the sub-task group is now working to incorporate the comments that were received. The Crack Treatment specification was sent out for review on February 11, with comments due by March 2. Asphalt Rubber Chip Seal was sent for review on January 18, with comments due February 10. The Modified Binder Chip Seal specification is almost ready for review.

Hans Ho, Casey Holloway, Shakir Shatnawi , Erik Updydke, and Greg Kelley at the PPTG All Members meeting.

For the 2010 edition of the Standard Specifications, the Crack Treatment and Asphalt Rubber Chip Seal SSP's will be incorporated into the Standard Specifications. The Bonded Wearing Course, Micro-

Continued next page

surfacing, and Modified Binder Chip seal specifications will remain as SSP's. The Recycling sub-task group now has NSSP's available for CIR and HIR. These will be used for projects as part of the Recycling Initiative.

PPTG meeting plans for 2009

Tentative dates for PPTG meetings in 2009 are as follows:

- All members meeting will be held in November, 2009, in Southern California. The location has yet to be determined.
- The various committees need to meet or teleconference as needed to deal with the issues in their work plans

New PPTG co-chairs

Several changes to the PPTG leadership have occurred since our last newsletter.

- Craig Hennings replaced Casey Holloway as the concrete industry co-chair. Casey will remain as his backup co-chair
- Dr Ding Cheng of the CP2 Center replaced Rita Leahy as the flexible pavement industry co-chair for the Strategy selection
- Vijay Singha replaced George Bradley as the local agency co-chair in Northern California. We still need a replacement for Phil Demery the other Northern California co-chair. Please contact Shakir Shatnawi if you are interested (Shakir.shatnawi@dot.ca.gov)

Meetings and conferences

TRB meeting held in Washington, D.C.

Center Staff attended the Transportation Research Board (TRB) 88th Annual Meeting held in Washington, D.C., on January 11-15, 2009. The TRB meeting covered all the areas in transportation and attracted more than 10,000 transportation professionals from all over the world. It was an excellent opportunity to promote pavement preservation knowledge. Many of the Center-related documents were exhibited at the TRB meeting, including the CP2 Center brochure, Western Pavement Preservation Partnership (WPPP) brochure, the latest Center Newsletter, and the second announcement for the First International Conference on Pavement Preservation in 2010 and the California Pavement Preservation Conference in Oakland, California, on April 8-9, 2009.

Attending the TRB meeting in January were (left to right) Shadi Saadeh (CSU LB), Wen Huang (Maryland State Highway Administration), Mary Stroup-Gardiner (CP2C), Anne Stonex (MACTEC)

While attending the conference, Technical Director Mary Stroup-Gardiner gave a presentation about the economics of flexible pavement preservation based on MTAG. Professor Ding Cheng also gave a presentation about the curricula development on waste tire applications in civil engineering.

TRB meeting attendees Patte Hahn (NCPP) and Ding Cheng (CP2C)

The Center appreciates the supports of the FHWA, Foundation of Pavement Preservation (FP2), and AASTHO TSP2 for allowing us to display our materials in their exhibit booths and/or hospitality suites. Information on the International conference on Pavement Preservation was presented at various information booths in the major hotels, and at committee meetings including AHD18 Pavement Preservation, AHD20 Pavement Maintenance Committee, and AFD10 Pavement Management Systems Committees.

CCSA holds its annual conference in Ontario

The California Chip Seal Association held its 2009 conference on January 21-22, 2009, at the Doubletree Hotel in Ontario, California. The conference was highlighted by the keynote talk by Will Kempton, Director of Caltrans. He gave a stimulating and honest, but not encouraging, speech on the economic health of the State of California and what the stimulus package might do for the paving industry. "California needs a budget now" was his message, and it needs to find a new way to finance future projects. Without a budget, it is difficult to sell bonds for the infrastructure bond projects which means Caltrans may have to stop work on the projects. The $25.7 billion in the stimulus package would result in a net gain of $2.57 billion for California. This money might be used to back fill the bond issues, which would result in no new paving projects. About 250 people attended the conference and they gave Director

Will Kempton

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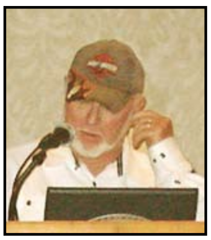
Kempton a rousing hand for his honesty and enthusiasm and for his support of pavement preservation. As he mentioned, the rehabilitation program has shrunk so much that the preservation program now exceeds the rehab program in total dollars.

Scott Dmytrow and Skip Brown discuss the award winning project in Williams.

The rest of the conference consisted of excellent presentations from agencies and industry on the importance of pavement management in supporting preservation programs, the basics for a variety of pavement preservation treatments including chip seals, scrub seals, chip seals over fabrics, and cape seals using a variety of binders. All of the presentations will be posted at the CCSA website www.chipseal.org.

Other presentations were given on the importance of quality control and quality assurance and trouble shooting ideas to ensure successful projects. The consistent message was for agencies and contractors to work together to minimize any chance of early problems. The CP2 Center at Chico State also discussed its resources and its help desk which deals with problems on all types of preservation issues. The website for the Center is located at www.cp2infor.org/center .



Don Milner received the Lifetime Achievement Award from the CCSA board of directors.

Other highlights included the recognition of Don Milner of Graham Contractors who received the association's life time achievement award in 2009. Prior winners of this prestigious award include Jim Towns of Western Emulsions and Murl Butler of ISS.

The Yearly Projects of Excellence awards were given honoring the Contracted Companies/Agencies that had exceptional pavement preservation applications.

Winners of the Yearly Projects of Excellence awards

Company	Agency	Award category
Intermountain Slurry Seal	Caltrans District #5	Slurry-Micro Surface
International Surfacing Systems	City of San Jose	Innovation-Chip Seal
Delta Construction	City of Williams	Innovation-Chip seal
Western Emulsions	Los Angeles County	Chip Seal
Graham Contractors	County of Santa Barbara	Cape Seal

California Pavement Preservation Conference

The 4th Annual California Pavement Preservation conference will be held in Oakland CA on April 8-9-2009. The Keynote speaker will be Director Will Kempton of Caltrans. The conference is presented by the Pavement Preservation Task Group (PPTG) in cooperation with the Center, the California LTAP program and Caltrans. On April 7, pre-conference training sessions will be held on the following topics:

- Pavement management fundamentals
- Asphalt fundamentals
- Concrete pavements
- Pavement preservation concepts
- Asphalt pavement maintenance

For more information on the conference, exhibiting opportunities, and sponsorship, please call 510-665-3628 or email conferences@techtransfer.berkeley.edu.

International Conference on Pavement Preservation (ICPP)

The First International Conference on Pavement Preservation is to be held April 12-16, 2010, in Newport Beach, California, with the purpose of bringing together researchers and experts working in the field of pavement preservation to exchange ideas and discuss critical issues and concerns. The conference will be co-organized by the California Department of Transportation (Caltrans), the Federal Highway Administration (FHWA) and the Foundation for Pavement Preservation (FP2). Others participating in the planning of the conference include the California Pavement Preservation Center, the National Center for Pavement Preservation (NCPP), and the University of California, Berkeley.

The conference venue will be in sunny Southern California close to the John Wayne Airport. Hotel information will be provided on the website soon. The conference will replace the successful California Pavement Preservation Conference for 2010. We will be inviting bids for a location for the second conference, to be held in 2014. If anyone is interested in submitting a bid, please contact Shakir Shatnawi at shakir.shatnawi@dot.ca.gov.

Main topics

The main theme of the conference will be pavement preservation and sustainability. The conference will address an array of issues that are relevant to the pavement preservation community. Presentations were invited on the following topics:

- Benefits of pavement preservation (economic and environmental)
- Integrating pavement preservation into pavement management
- Pavement preservation treatments for flexible pavements (design, materials, constructability, and performance)
- Pavement preservation treatments for rigid pavements (design, materials, constructability, and performance)
- Strategy selection
- Funding pavement preservation
- Promoting pavement preservation to the public and our elected leaders

Preliminary program

The conference program will consist of peer reviewed papers and selected invited presentations. Highlights of the conference are expected to

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include case studies of preservation from US Highway agencies, industry, and international organizations. The presentations are expected to take place on April 13-15, with workshops and/or demonstrations to take place on April 12 and 16, 2010.

Call for abstracts and papers

Over 90 abstracts were received by the extended deadline of February 1, 2009. Authors should hear back from the technical committee by April 15, 2009. Authors of the abstracts selected will be invited to submit full papers to be included in the conference proceedings. Final papers should be submitted electronically using the conference website at www.pavementpreservation.org/icpp/. The submission of full papers and camera-ready copies of papers should be completed by the dates listed below. Full papers will be reviewed by the Technical Committee for selection to be included in the conference proceedings or the Pavement Preservation Journal:

- Submission of full papers: July 1, 2009
- Submission of camera-ready copies of full papers: November 1, 2009

Center news

Update on the CIWMB Continuing Education and University Curricula Project for RAC and CE applications of waste tires

Each year there are about 40 million waste tires generated in California. The California Integrated Waste Management Board (CIWMB) is tasked with diverting these tires from the waste stream to being recycled into useful products. Civil engineering applications are the fastest growing market for waste tire products. These products possess some desirable engineering properties. To promote sustainable and successful waste tire applications in civil engineering, a curriculum development and dissemination project was funded by CIWMB. The primary purpose of this project was to produce and disseminate teaching materials which could be used in undergraduate civil engineering courses.

Professor training workshop participants in Sacramento are, left to right, front row: Darlene Mathias, Hector Estrada, Luke Lee, Albert Johnson, J. Larralde; back row: Ming Xiao, Akthem Al-Manaseer, Kaven Shafizadeh, Justin Reginato, Michael Lepech, Joel Arthur, Gary Hicks, Ali Porbaha, Ding Cheng.

A series of course modules have been developed for a variety of undergraduate Civil Engineering courses including Introduction to Civil Engineering Design, Mechanics of Materials and Materials Testing Lab, Soil Mechanics and Foundations, Contract and Specifications, Environmental Engineering, Solid Waste Management, and Transportation and Pavement Materials. These course materials are available to be integrated into various undergraduate courses in the Civil Engineering curriculum and serve to introduce students to sustainable building practices and "green" construction.

Two training workshops have been given to professors who are teaching classes related to the waste tire application. One professor training workshop was conducted in Sacramento on December 19, 2008, for the universities in the Northern California. Another was held in Pomona on January 5, 2009, for universities in Southern California. About 14 professors from 11 different universities attended these workshops. The workshops were very successful and the professors who attended gave very high rankings for the workshops. They indicated the teaching materials are very useful and can be incorporated into their teaching tasks.

Another workshop is scheduled on April 17, 2009, in San Luis Obispo. For more information on the workshop, please contact Professor Ding Cheng at dxcheng@csuchico.edu. Registration information can be found on the Center website at www.ecst.csuchico.edu/cp2c/ciwmb/SLOProfessorTrainingWorkshop.htm.

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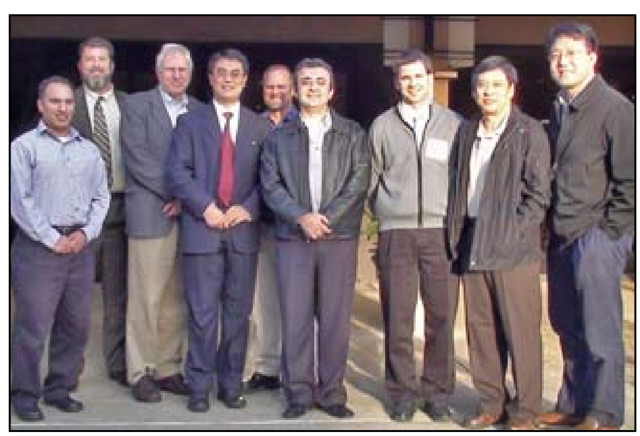


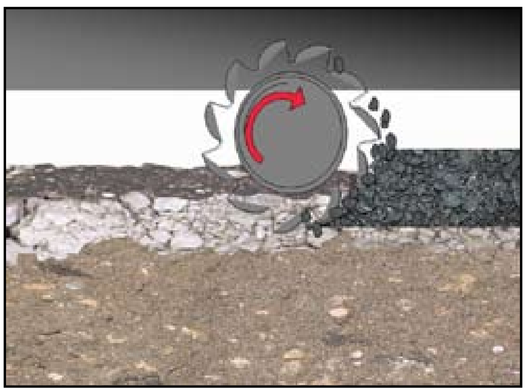
Professor training workshop participants in Pomona, Calif., are, left to right: Julian Ibarra, Joaquin Wright, Gary Hicks, Ding Cheng, Joel Arthur, Shadi Saadeh, Dragos Andrei, Xudong Jia, Uksun Kim.

Update on CIWMB project on terminal blends and warm mixes

Center staff have been working on a project supported by the California Integrated Waste Management Board to accomplish the following goals:

- Investigate the feasibility of including terminal blend asphalt rubber into the grant program of the CIWMB
- Determine the feasibility of using the warm mix technology with asphalt rubber hot mix and with asphalt rubber spray applications

Terminal blends are a form of asphalt rubber binder that is manufactured at the refinery by blending the crumb rubber with asphalt at elevated temperatures. This is a different process than the conventional field blended asphalt rubber that has been used for many years in California. Terminal blends can contain 15-20% or more of crumb rubber so they meet the definition of ASTM for asphalt rubber (min of 15% CRM). We are currently surveying suppliers and users of terminal blends for use in hot mixes and chip seals in California and throughout the United States. If you have experience with the use of terminal blend asphalt rubber binders for the applications, please contact Dr Gary Hicks at rghicks@csuchico.edu. We are interested in the following:



Schematic of a milling machine processing an asphalt pavement.

- Types of uses of terminal blends
- Performance compared to asphalt rubber or polymer modified products
- Potential problems with terminal blends such as the ability to document the amount of rubber in the binder

We are also meeting with suppliers of warm mix technologies to determine the feasibility of using these technologies with field or terminal blended asphalt rubber hot mix or spray applications. The primary benefit of doing this is to reduce energy costs and to reduce emission. We have determined that the AR products can be used with warm mix technology in hot mixes, but have not yet found that they have been used in chip seals. Caltrans will be constructing their first RAC-O project using warm mix technologies this construction season. This project will provide useful information in support of this effort. If you have information regarding warm mix technologies that can help with this endeavor, please contact Dr. Mary Stroup-Gardiner at mstroup-gardiner@csuchico.edu. We are very interested in the following:

- Agencies that have used warm mixes with asphalt rubber in hot mixes and or any pavement preservation treatment
- Potential problems with using warm mix with asphalt rubber application

Dr. Stroup-Gardiner met with Caltrans and a Chinese Delegation on Feb 17, 2009 to discuss the Center's work with warm mixes. Her presentation can be found on the Center's website.

NCHRP Synthesis 40-13: In-place recycling of asphalt pavements

This project deals with the development of a synthesis of information on in-place recycling methods used for asphalt pavements. It will cover the following techniques:

- Surface recycling using both cold and hot in-place recycling techniques
- Full depth reclamation using a variety of additives

The project will consist of a literature search and survey of agencies' practices. The final product will include an update on project selection, design, construction, specifications and more. Best practices and case histories will also be included.

If you have information to share on any of these items, please contact Dr. Stroup-Gardiner at mstroup-gardiner@csuchico.edu. Dr. Stroup-Gardiner made a presentation on this project at the ARRA annual meeting in Palm Springs on Feb. 19, 2009. Her presentation can be found on the Center's website.

NCHRP 40-01: Recycled materials and by-products in highway applications

Recycled materials and industrial byproducts are being used in transportation applications with increasing frequency. While there is a growing body of experience showing that these materials work well in highway applications, the related information and experience are not synthesized in a coherent body. This study will gather the experiences of transportation agencies, both foreign and domestic in determining the relevant properties of recycled materials and industrial by-products and the beneficial use for highway applications. The study will include strengths and weaknesses of material applications.

The synthesis should serve as a guide to states revising the provisions of their materials specifications to incorporate the use of recycled materials and industrial by-products, and should, thereby, assist producers and users in 'leveling the playing field' for a wide range of dissimilar materials.

Information to be gathered for the synthesis will include:

- A comprehensive list of current candidate materials that are readily available or stockpiled for common usage, and their uses, in a matrix format

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- Identify and review available test procedures for assessing physical and chemical characterization, compaction, geomechanical properties, long-term durability, and environmental performance, including suitability and risks
- Summarize best material preparation and quality control techniques (including stockpiling). For more information on this study, please visit our website at www.cp2info.org/center.

Center growth plan

Dr. Mike Ward, Dean of the College of Engineering, Computer Science and Construction Management, attended a Memorandum of Understanding signing on February 18, 2009, in Long Beach to formally establish the growth plan as a partnership between the three universities. Such a partnership will result in a virtual Pavement Preservation Center for the State of California. The other partners of the growth plan include Long Beach State and Cal Poly, Pomona. The faculty at these universities will work as part of the Center to provide more effective delivery of our services in Southern California.

Left to right front row: Ed Hohmann, Dean , Cal Poly Pomona; Mahyar Amouzegar, Associate Dean, CSU, Long Beach; and Mike Ward, Dean, CSU, Chico. Back row: David Dowell, Vice Provost, CSU, Long Beach; and Shakir Shatnawi, Caltrans.

Upcoming pavement preservation events

World of Asphalt, March 9-12, 2009, Orlando Fla., www.worldofasphalt.com

California Pavement Preservation Conference, April 8-9, 2009, Oakland, Calif., www.cp2info.org/conference

National Conference on Preservation, Repair, and Rehabilitation of Concrete Pavements, April 22-24, 2009 St. Louis, Mo., www.fhwa.dot.gov/pavement/concrete/2009cptpconf.cfm

Fourth Rubber Modified Asphalt Conference, May 8-9, 2009, Akron, Ohio, www.rubberdivision.org/meetings/rmac.htm

12th AASHTO/TRB Maintenance Management Conference, July 19-23, 2009, Annapolis, Md., www.marylandroads.com/businesswithsha/aashto/oom/2009aashtoconference.asp

Asphalt Rubber 2009, November 2-4, 2009, Nanjing, China, www.consulpav.com/ar2009/

Developing a Research Agenda for Transportation Infrastructure Preservation and Renewal, November 12-13, 2009, Washington, DC, www.trb.org/news/blurb_detail.asp?id=9834

First International Conference on Pavement Preservation, April 12-16, 2010, Newport Beach, Calif., www.pavementpreservation.org/icpp/

Ferrara discovers that In Italy, *even* potholes are ... artistic

In July 2008, Tom Ferrara traveled to Italy to live in a small town and sit in on a media arts class. Along the way he checked out some pavements. As you can see on the left, Italy has problems similar to those in California, but their distress takes the shape of art. The example on the left can be seen as a rendition of typical Italian subject matter, e.g., Madonna and Child, or as Tom would have it, a mermaid and a peanut. The photo was taken in a minor urban street in Rieti, Italy.

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Pavement Preservation Expert Task Group
Emulsion Task Force

Subcommittee Roster
Updated 5/09

Subcommittee	Members		
Emulsion Testing & Residue Recovery Methods	**Arlis Kadrmas** Paul Morris Gaylon Baumgardner	Laurand Lewandowski Chris Lubbers Roger Hayner	Barry Baughman
Residue Tests	**Gayle King** Hussein Bahia Amy Epps Martin	Paul Morris Gaylon Baumgardner Arlis Kadrmas	Lauren Lewandowski Chris Lubbers Barry Baughman
Aggregates, Mix Design, and Performance Tests	**Mary Stroup Gardiner** Jim Moulthrop Hussein Bahia	Scott Schuler Gayle King Chris Lubbers	Laurand Lewandowski Jack Youtcheff Barry Baughman
Approved Supplier Certification	**Roger Hayner** Arlis Kadrmas Colin Franco	Chris Abadie Kevin Van Frank Jim McGraw	Mike Anderson
Inspection and Acceptance	**Colin Franco** Roger Hayner Delmar Salomon	Chris Abadie Tom Wood	

Note: Names in boldface are Subcommittee Chairs or Co-Chairs